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Solomon Gulch Hydroelectric Facility Pool Raise

Feasibility Report

McMILLEN JACOBS ASSOCIATES

Final

Revision No. 2

December 2020

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Executive Summary

A feasibility study to investigate pool raise alternatives at Solomon Gulch was conducted for Copper Valley Electric Association by McMillen Jacobs during 2019-2020. The study was conducted for two reasons; 1) Capture spill of unused water at the dam, which has historically occurred at Solomon Gulch since it's construction in the early 1980's, and 2) Greater storage needs at Solomon Gulch due to Copper Valley's new Allison Creek project, which has decreased generation requirements at Solomon Gulch during the run-off season. A pool raise would store water for hydro generation later in the year to off-set winter diesel generation.

The study developed three alternatives for pool raise, a 2 ft, a 4 ft, and an 8 ft raise level. For each raise level increase, a greater benefit occurs as more water is stored to off-set winter time diesel generation, but construction costs and permitting risk also increase. Permitting risk increases because factors of safety associated with this dam and dike's stability during an earthquake are already low, and dam stability decreases when water loads increase. To address this issue, future stability evaluations with new increased siesimic loads, as required by FERC, are scheduled for completion in 2021.

McMillen Jacobs' study determined there is no economic benefit for pool raise alternatives greater than 15 ft as this would only be beneficial in rare, very high water years. Pool raise alternatives were limited to 8 ft as the existing dam and dike structures currently have a parapet wall ten ft high. A pool raise of 8 ft was judged the maximum to ensure 2 ft of operational freeboard. A pool raise greater than 8 ft would require re-construction of 750 lineal ft of parapet wall substantially increasing construction costs.

Pool Raise Alternative	Spillway Modification	Annual Generation Benefit (KWh)	Construction Cost 2020 $USD	Annual Diesel Fuel off-set in gallons	FERC Permitting Issues
2 ft	Spillway ogee crest increase	691,000	$913,500	46,600	minimal
4 ft	full length spillway rubber dam	1,382,000	$5,898,600	93,300	potential
8 ft	175 ft rubber dam, 275 ft flashboards	2,759,000	$5,429,400	186,400	higher potential

Each alternative presented can pass the probable maximum flood (PMF) a key design requirement for dam safety considerations. The study used a constant .5% load growth for the period 2020-2050 and assumed a project life of 30 years, but did not account for generation changes at Solomon Gulch resulting from possible power exchanges with Alyeska currently under negotiation.

Key Findings

- Pool raise alternatives generate marginal increases in generation that are off-set by high construction costs.

Executive Summary continued-Key Findings

- The 4 ft pool raise has a high construction cost compared to the 8 ft raise but the 8 ft raise has increased permitting risks and the possibility of flashboard replacement costs if a very high flood event occurs.
- Use of flashboards is required above 4 ft to both pass the PMF and keep construction costs reasonable.
- The Solomon Gulch FERC license expires in 2028, a pool raise would trigger a license amendment process that would have to be repeated during the 2028 re-licensing. Integrating the pool raise into the 2028 re-license process will be much more cost effective.
- Solomon Gulch relicensing efforts will probably start in the 2022 to 2023 time frame. This allows Copper Valley Electric Association time to evaluate dam stability, generation and storage impacts associated with the Alyeska exchange, and allows CVEA to acquire a few more years of inflow and Allison Creek data, all of which will assist with determination of pool raise benefits.

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Revision Log

Revision No.	Date	Revision Description
0	4-30-2020	Draft for Client Review
1	5-11-2020	Final Report
2	12-18-2020	Final with Stamp

1.0 Introduction and Background

1.1 Purpose

This feasibility report presents alternatives considered to increase storage at Copper Valley Electric Association's Solomon Gulch Hydroelectric Project. Additional storage would allow Copper Valley Electric Association (CVEA) to store water that would have passed over the spillway to be used as generation later in the year, thereby offsetting wintertime diesel generation.

1.2 Location

The Project site is the existing Solomon Gulch Hydroelectric Facility. This includes powerhouse, penstocks, rockfill dam, and seasonal storage reservoir located approximately 4 miles south of Valdez, Alaska, as shown in Figure 1-1. The upper works of the hydroelectric power generation project include a dam, valve house, dike, and spillway.

Figure 1-1. Project Location

1.3 Authorization

By letter on January 14, 2019, CVEA authorized McMillen Jacobs Associates (McMillen Jacobs) to complete a feasibility study to investigate the potential for increasing storage at CVEA's Solomon Gulch Hydroelectric Facility, FERC No. 2742.

1.4 Background and Objectives

If spill occurs routinely at a seasonal reservoir such as Solomon Gulch, and if electrical loads are present such that hydroelectric power generation could use the spill at a later time in the hydrologic cycle, then there may be economic benefit to increasing storage. Spill had occurred at Solomon Gulch historically on a seasonal basis, and winter loads continue to require significant diesel generation. Additionally, CVEA commissioned the run-of-river Allison Creek Project late in 2016, which decreased summer and fall generation requirements at Solomon Gulch. The additional storage, in combination with operation of Allison Creek, would extend the wintertime generation of the Solomon Gulch facility, thereby offsetting existing diesel generation.

1.5 Project Feature Scope and Objective

The scope of work and features required to achieve the above stated objectives for the storage increase evaluation are as follows:

1. Collect data on the existing project to gain a clear understanding of the current facility design and operation. CVEA system and Solomon Gulch generation records, reservoir inflow and outflow, project compliance flows, and reservoir levels will be particularly important to determine the available annual runoff that is currently spilled from the reservoir and would be used as dispatchable generation.
2. Complete an assessment of the potential increased storage volume and associated generation that could be captured.
3. If this analysis confirms that increased storage will provide a significant increase in annual production, conduct an analysis of the technical approach for accomplishing the storage increase. This analysis will require development and evaluation of options for raising the existing concrete overflow spillway, building new spillway structures, and potentially increasing the rockfill dam section. If large increases in the reservoir are deemed economical, then the impact on the existing penstock and powerhouse equipment will be evaluated.
4. Develop cost estimates for each of the identified alternatives and incorporate into an overall evaluation of project feasibility considering a wide range of criteria. The primary focus of the evaluation will be to determine the optimum dam raise considering the need for increased generation, operation, and cost.
5. Provide a general overview of the regulatory and permitting requirements to implement a dam raise at Solomon Gulch. The primary focus of this work is to identify the FERC regulatory process impacts for permitting and construction approval.
6. Summarize the analysis and results in a feasibility report.

1.6 Report Organization

This Report is a record of the design effort for the Project. The Report consists of a summary of the design and analysis elements, criteria, methods and approach, engineering calculations, and pertinent references. The major report sections are presented in Table 1-1.

Table 1-1. Major report sections and purpose

Section	Description	Purpose
1	Introduction and Background	Presents the Project background, purpose, location, authorization, objectives, feature scope, and the report organization.
2	Data Collection and Review	Presents a summary of the compilation of Project information used to evaluate the potential for a pool raise.
3	Hydrological and Power Production Analysis	Includes information related to the assessment of increased storage and potential generation.
4	Alternatives Development	Includes information related to the developed options for raising the pool.
5	Cost Estimates	Includes information related to the costs associated with the developed alternatives.
6	Regulatory and Permitting Review	Includes information related to the overview of regulatory and permitting requirements.
7	Summary and Recommendations	Summarizes Reports and suggests next steps.
Appendices		
A	Select STI Spillway and Dam Drawings	Project drawings referenced in text
B	Supporting Calculations	Supplemental calculations referenced in text.
C	Manufacturer’s Data	Rubber Dam example in cold climate, mechanical system manufacturer data.
D	Cost Estimate and Economic Analysis	Present worth benefit/cost analysis for alternative evaluation.

Note: STI = Supporting Technical Information; see CVEA 2018

2.0 Data Collection and Review

2.1 General Description

This section documents the collection and review of the available literature and data related to the Solomon Gulch Hydroelectric Project. Data collected during March through May of 2019 entailed compilation of reservoir structure information, penstock geometry, unit performance data, stream flow gage information, and CVEA system load and diesel generation records. This information was used to evaluate the potential for additional generation provided by various pool raise options.

2.2 Data Collection Categories

The following sections indicate the categories under which data collection falls, along with a summary of the information. See McMillen Jacobs (2019b) for a detailed review of the collected data.

2.2.1 Storage Curve

A plot of the reservoir storage curve was obtained from the CVEA office (see Drawing No. HO1-F-04-2011-R49 in CVEA 2018). The reservoir impounds 31,500 acre-ft when at full pool elevation 685 ft. There is no information or data indicating that the storage-level relationship has been verified by depth soundings or sonar Global Positioning System (GPS) location.

2.2.2 Dam and Dike

The dam and dike (See Figure 2-1 and Figure 2-2) have a parapet wall at the top of the structural sections; the top elevation of the parapet wall is 695 ft. The top ground-level of the hill area between the dam and dike is just slightly above 695 ft; therefore, any pool raise alternative above elevation 695 ft would entail construction of a retaining wall or other large structure between the dam and dike to impound the reservoir. The dam and dike are rockfill structures with an asphaltic concrete covering approximately 12 inches thick on the upstream face. The dam is a zoned, compacted rockfill structure, with a slope of 1.7H to 1V on the upstream face, and 1.4H to 1V on the downstream face. Zone 3 compacted rock was used to construct a berm at the toe of the dam.

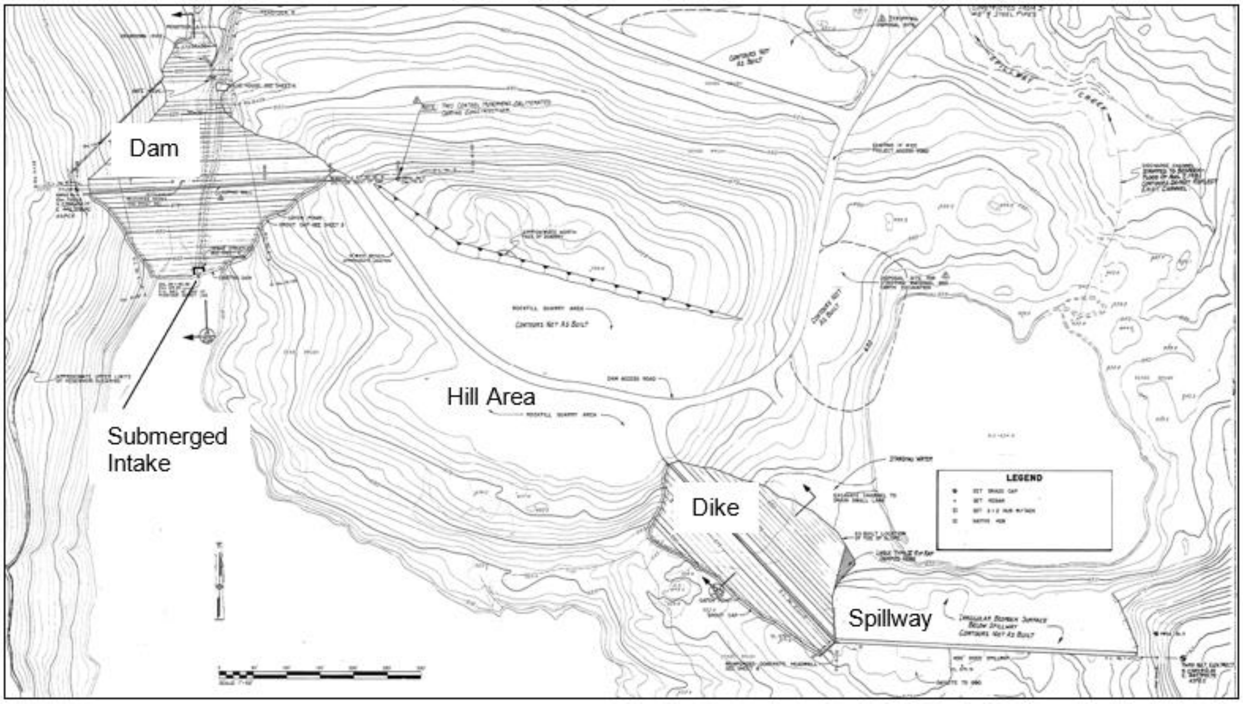


Figure 2-1. Plan view of dam, dike, and spillway

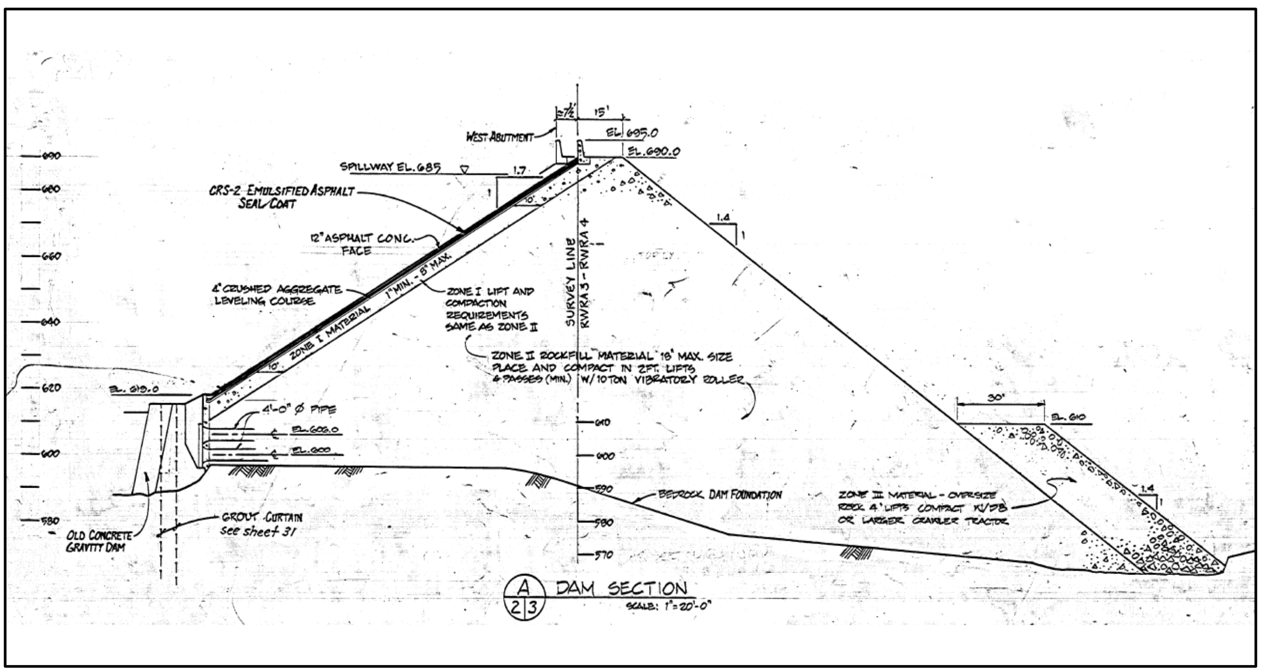
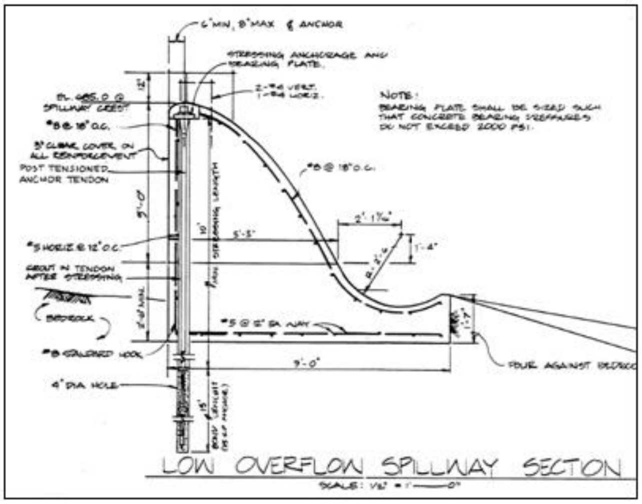


Figure 2-2. Section view of dam

Crest elevation of 685.5 feet



Year	Approximate Maximum Res. El.	Approximate Spillway Discharge (cfs)	Date
1997	686.2	1,727	
1998	686.0	1,283	
1999	685.5	412	
2000	685.7	712	
2001	685.7	712	
2002	685.7	712	
2003	685.9	1,079	
2004	686.0	1,283	
2005	685.9	1,079	
2006	685.9	1,079	
2007	≤ 685.0	0	
2008		238	
2009	685.5	412	8/16/09
2010		387	
2011		335	
2012	687.2	4,541	9/16/12
2013	686.3	1,966	9/6/13
2014	685.8	889	9/14/14
2015	685.0	0	9/30/15
2016	686.8	3,308	9/15/16
2017	686.1	1,499	8/30/17
2018	686.2	1,727	10/16/18

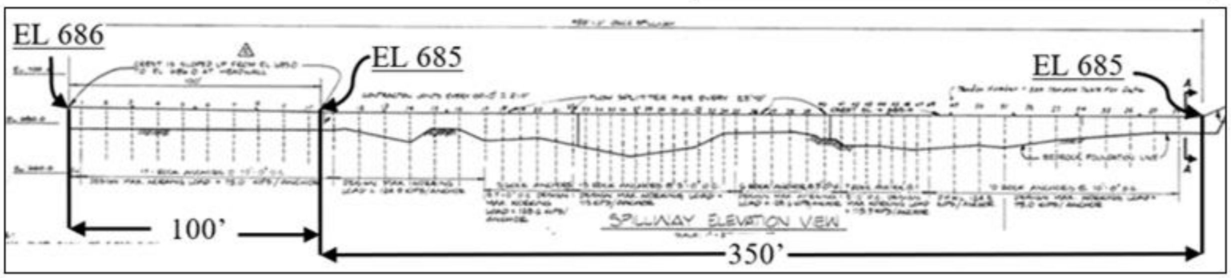
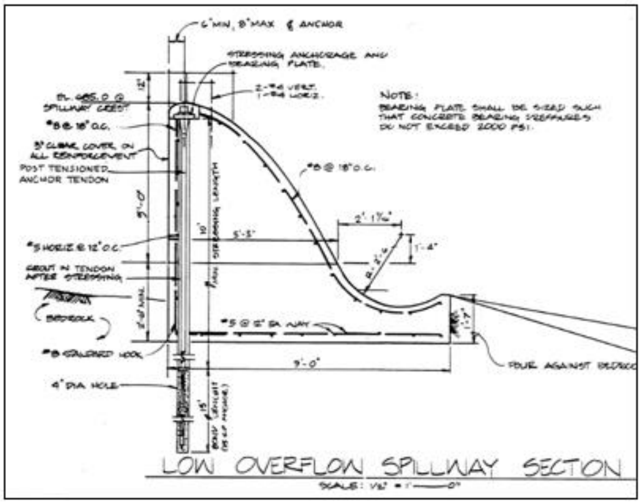


Figure 2-3. Spillway drawings and recent spillway flow data; see larger-scale drawings in Appendix A.

2.2.3 Spillway

Spillway discharge in CVEA (2018) is estimated using the equation Q = CLH^1.5, where C=3.33 and L=450 ft with a 100-ft span of spillway having a sloped crest elevation from 686 ft to 685 ft. Spillway supporting technical information (STI) materials (CVEA 2018) document a revision to this equation. Upon review of the STI for this report, the discharge coefficient was further revised. The revision is documented in McMillen Jacobs (2019a). For the years 1997 to 2018, spill flows recorded as sample day are shown below in

Crest elevation of 685.5 feet



Year	Approximate Maximum Res. El.	Approximate Spillway Discharge (cfs)	Date
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2011		335	
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2013	686.3	1,966	9/6/13
2014	685.8	889	9/14/14
2015	685.0	0	9/30/15
2016	686.8	3,308	9/15/16
2017	686.1	1,499	8/30/17
2018	686.2	1,727	10/16/18

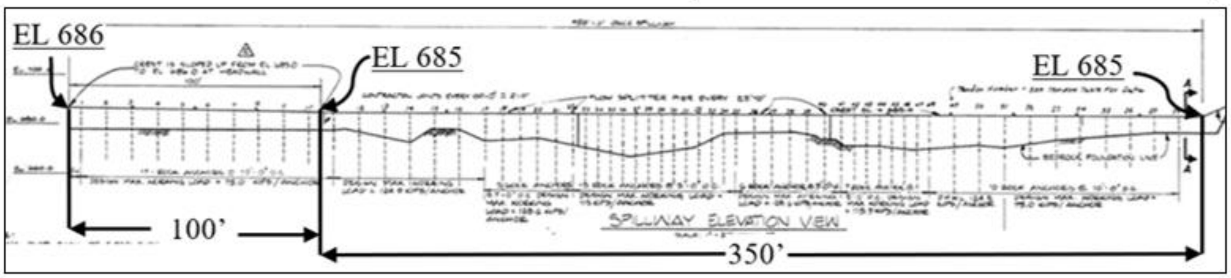


Figure 2-3. There were 2 years (2007 and 2015) without spill flows.

2.2.4 Hydraulic Conveyance

A small concrete outlet structure located at the upstream toe of the dam encloses two 48-inch-diameter penstocks with intake pipe centerline elevations of 600 ft. Each unit has a dedicated penstock, with the total penstock length from intake to turbine isolation valve approximately 3,660 ft. Pipe used for the penstocks was surplus after the Alaska oil pipeline construction project and has been subject to scrutiny by the Federal Energy Regulatory Commission (FERC) dam safety process. Within the last several years, a complete non-destructive test of each weld has been conducted, a hydraulic transient analysis conducted, additional thrust block and saddle anchoring has been added, and an emergency low level outlet structure has been constructed that connects to the upper reach of the penstocks just downstream of the dam for emergency release into Solomon Gulch. From this review, and in consideration that pool raise alternatives would be at most 25 ft (3.7 percent increase), McMillen Jacobs concluded that the conveyance system would not be a limiting factor. Changes to the original penstock and the studies conducted on the penstock system are documented in the STI library for Solomon Gulch (CVEA 2018). Modeling assumptions for the penstock are listed in McMillen Jacobs (2019b).

2.2.5 Unit Characteristics

Solomon Gulch has two Fuji Electric generators, each rated at 7,500 kW, 0.8PF, 4.16 kV, 900 rpm with a 60° C rise for armature and field. The over-excitation limit is 0.6 pu, and the under-excitation limit is 0.5 pu, (1 pu = 7,500 kVA). To our knowledge, the machines have not been rewound. Regarding cooling, there was nothing in either the unit data or from discussions with operators indicating that generator capacity should be reduced from the manufacturer's ratings listed above.

Unit 1 is the original Fuji turbine. Unit 2 has a newer Voith runner, and both units have flow meters and an isolation valve just upstream of the spiral case. Documentation from the commissioning index test was found, but a Fuji turbine curve was not available. A Voith performance curve was obtained and is shown in McMillen Jacobs (2019b). The index test data for Unit No. 1 and the Voith performance curve for Unit No. 2 were used for turbine efficiency in the hydraulic power model. Operating conditions and assumptions are discussed in McMillen Jacobs (2019b).

2.2.6 Reservoir Inflow Estimation – Stream Gage Data Collection

U.S. Geological Survey (USGS) gages 15225997 at Bailey Bridge and USGS 15225996 just downstream of the plant tailrace measure most outflows from the reservoir, plus local inflows between the dam and the powerhouse. Diversions to the Valdez Fisheries Development Association hatchery located across the road and just above the bay are not included in the USGS gage data. Records provided by CVEA for hatchery and Solomon Gulch plant flow were documented in Excel files labeled "hydro meters" (CVEA 2018). McMillen Jacobs (2019b) shows a location diagram of stream measurements for Solomon Gulch.

2.2.7 CVEA Load and Diesel Generation Data Collection

McMillen Jacobs collected generation data from Solomon Gulch plant records and from the CVEA main office records. These data were summarized to understand Solomon Gulch load following demands and is discussed in full in McMillen Jacobs (2019b). Review of plant and system generation verified that additional generation resulting from captured spill could be used in the winter months to displace Glennallen diesel generation while allowing full generation from Allison Creek.

3.0 Hydrological and Power Production Analysis

3.1 General Description

This section describes the development of the hydraulic power model used to evaluate the generation benefit of incremental pool raises. It also includes the results of the analysis performed.

To quantify the generation benefit, a generation model was developed for Solomon Gulch. The model was run with the existing full pool level of 685 ft for different inflow cases, and the resulting generation was tabulated. The model was then run for pool raises in increments of 5 ft, up to 20 ft (at 705 ft full pool) using the same inflow cases as the 685 ft full pool level. The difference in generation between the existing full pool level of 685 ft and a pool raise for low, average, and high inflow cases was then tabulated as the value of the pool raise.

3.2 Model Description and Development

The model is written in visual basic and operates as a Microsoft Excel macro. Model methodology is described in detail in McMillen Jacobs (2019b).

There are three methods of simulation:

1. The model follows a user-specified rule curve to draft and fill the reservoir. The only limits on operation are the capability of the machines. The rule curve is generally based on historical best practices as determined by the utility.

2. By following a generation schedule, the resulting reservoir level is calculated.

3. By following a rule curve, but with generation limits imposed by maintenance or load delivery criteria. An example of this would be Solomon Gulch generation restricted to lower generation levels in spring and summer but would follow a draft schedule during the winter.

The model uses daily average values for inflow, reservoir level, and generation, and runs for 365 days forward from a user-specified start date. User inputs, programmer constraints, and simulation outputs are listed in Table 3-1.

Table 3-1. Generation reservoir model inputs, constraints, and outputs

Model/Simulation Inputs Entered by User	Where Entered in Operations Model by Workbook Sheet	Model Constraints Entered by Programmer	Outputs-Output Sheet Daily Values	Output Unit
Rule Curve	HW Sheet – displays on Control Sheet	Generation Table	HW initial and HW end	Ft msl
Inflow Sequence	Inflow Sheet – displays on Control Sheet	Reservoir Storage Curve	Plant Generation	aMW

Model/Simulation Inputs Entered by User	Where Entered in Operations Model by Workbook Sheet	Model Constraints Entered by Programmer	Outputs-Output Sheet Daily Values	Output Unit
Start Date	Control Sheet	Inflow Tables	Plant Flow	cfs
Initial HW Level	Control Sheet		Release flow – at Reservoir	cfs
Plant Flow Restrictions	Control Sheet		Spill Flow	cfs
Plant Generation Limits	Pgen sheet – not displayed on Control Sheet		Inflow	cfs
			Rule Curve Elevation	Ft msl

The model is governed by continuity on a daily time resolution:

$$Qinflow = Qplant + Qspill + Qrelease + c * \Delta storage \qquad \text{(Storage Increase is Positive)}$$

3.3 Model Results

3.3.1 Graphic Explanation of Simulation

Figure 3-1 shows a plot for a pool raise of 10 ft using an average inflow case. During January through May, the model follows the established rule curve. By adjusting generation starting on May 1, we have assumed that Allison Creek has initiated generation and that diesel generation is sharply reduced. Solomon Gulch generation increases and the model drafts Solomon Gulch Reservoir slightly below elevation 620 ft because, for a very short period, electrical demand increased and inflows decreased. The reservoir starts filling in mid-May and reaches 670 ft by the end of June. At this time, to avoid spill, Solomon Gulch should increase generation. However, because Allison Creak is also experiencing an average inflow year, there is no load to serve, and the reservoir continues to fill. The simulation year is 2029, and a 0.5 percent annual load increase has been added to 2018 loads. Even with this load increase, spill occurs August 1 and continues at varying levels through October (white line in Figure 3-1).

Starting in November, with the beginning of winter loads and lower inflows, Solomon Gulch starts drafting the reservoir. The additional 10 ft of storage is used in November and December to displace Glennallen and/or Valdez diesel, or to delay the start of cogeneration. In the future after 2029, as summer loads continue to grow, more and more of the spill will be converted to generation, but this would also occur without a pool raise. McMillen Jacobs (2019b) discusses these results in further detail.

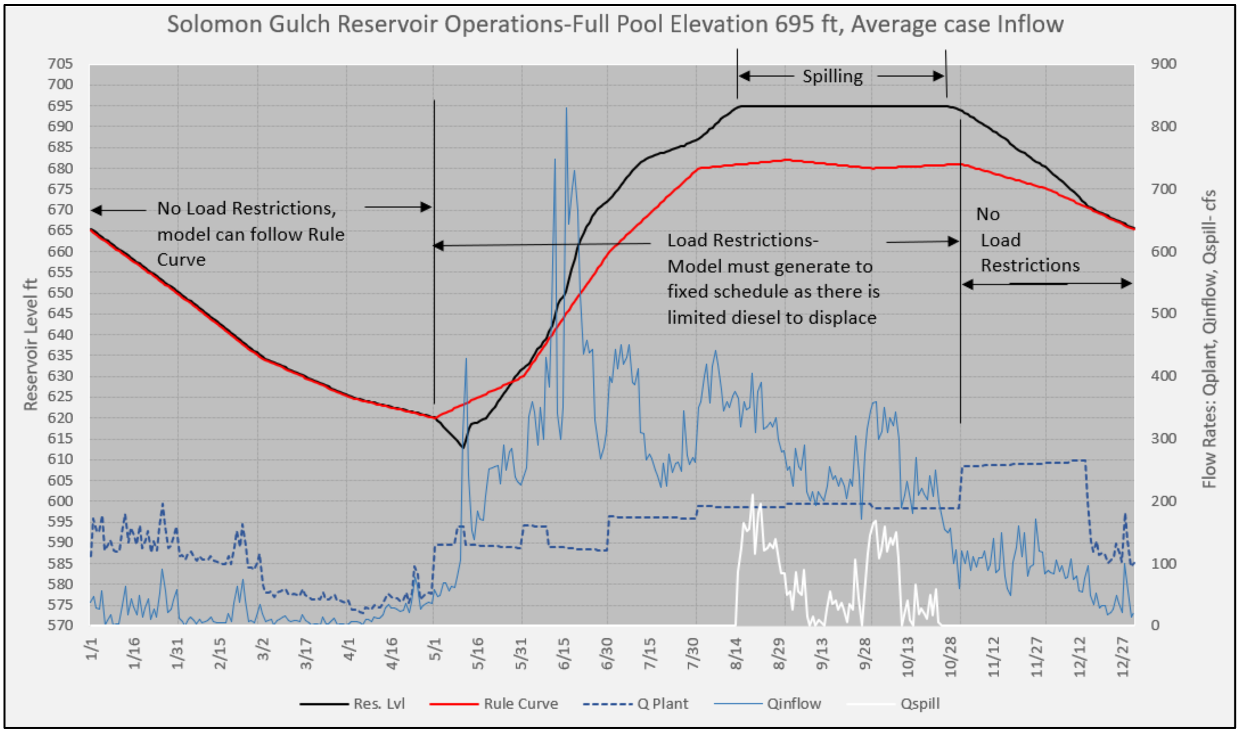


Figure 3-1. Plot of 10-foot pool raise, average inflow simulation

3.3.2 Benefit of Pool Raise

Additional generation for various pool raises is listed in Table 3-2. These results are based on a statistical evaluation of inflows to Solomon Gulch Reservoir. Benefits in terms of additional generation in units of megawatt-hours (1,000 kWh) are tabulated for each inflow case on an annual basis. There are three inflow cases – low, average, and high – for four pool raise elevations in increments of 5 ft. Total inflow volume for the year in units of day-second-ft (dsf) are used; dividing the dsf value by 365 gives the average daily inflow value in cubic ft per second (cfs). Daily average inflow for the average inflow case is 157.5 cfs. This is also the average value of the solid blue line of Figure 3-1 and gives reservoir planners a feel for inflow volatility compared to average. A common rule curve (the red line of Figure 3-1) was used for each simulation, but generation constraints were different for each inflow case. For low inflow cases, generation was assumed to decrease at Allison Creek, so there were fewer generation constraints. As inflows increased over the low case, generation constraints increased at Solomon Gulch as generation was assumed to increase at Allison Creek. For low inflow cases, the modeling results indicate there is no benefit to a pool raise above 10 ft. Each successive raise option above 10 ft yields the same increase in generation. For average years, there is no benefit to a pool raise above 15 ft, as both the 15 ft and 20 ft raise options yield the same increase in generation.

Table 3-2. Generation benefit of pool raise vs. raise elevations

Full Pool Elev. (ft msl)	Pool Raise (ft)	Inflow Case (dsf)			Estimated Annual Benefit (MWh)
		Low	Avg	High	
		μ-σ 46,900 ΔMWh	μ 57,500 ΔMWh	σ+μ 74,378 ΔMWh	
685	0	0	0	0	0
690	5	1,672	2,112	1,994	1,727
695	10	2,677	4,464	4,144	3,447
700	15	2,677	6,539	6,481	4,924
705	20	2,677	6,539	8,756	5,675

Except for a 20 ft raise, the benefit is nearly identical for high and average inflows. At first glance, one would expect benefits to increase with inflow, but because Allison Creek generation increases as inflows increase, there is less load to serve for Solomon Gulch during the summer. This causes the extra water over the average inflow case to be spilled in the summer months, except for carryover storage that occurs into November and December, which is converted to energy and delivered. If a 20 ft raise were constructed, the incremental capital expense would only return a benefit on very high inflow years, and the benefit would be 2,275 MWh over the 15 ft raise (see Table 3-2). The model captured this characteristic on a frequency basis since inflows were quantified in terms of one standard deviation above and below average. A 20 ft raise would return a benefit over the 15 ft raise 16 percent of the time, and importantly, the pool raise has to be above 15 ft to capture a high-inflow-year benefit. Levels of pool raise versus inflow variability can be summarized in a more common-sense manner. There is not much benefit increase for low inflow years because the existing reservoir is large enough. For high inflow years, only a very large pool raise (greater than 15 ft) captures a benefit, but this occurs infrequently (5 out of 30 years). The merit of the model exercise is that it confirms our insight gained from operational experience and provides a quantitative answer for benefits derived from incremental raise efforts.

Table 3-2 shows the annual benefit that would occur if the additional generation from a pool raise was represented by a weighted average of 30 years of operation. The estimated annual benefit is less than the benefit listed under average inflow because pool raise benefits drop off sharply with less-than-average inflow and decreases, except for a 20 ft raise with additional flows above average. The estimated annual benefit is the sum of the product of % of time of occurrence and generation benefit during the time period. Table 3-3 shows the estimated benefit of pool raise vs. percent of time, with the total estimated benefit in the far right column (same as Table 3-2).

Table 3-3. Annual pool raise benefit as a percent of occurrence

Generation Value as a Percent of Occurrence (MWh)						Σ(%time*Generation)
% time	8%	8%	34%	34%	16%	MWh
5 ft raise	0	836	1,892	2,053	1,994	1,727
10 ft raise	0	1,339	3,571	4,304	4,144	3,447
15 ft raise	0	1,339	4,608	6,510	6,481	4,924
20 ft raise	0	1,339	4,608	7,648	8,756	5,675

The technical aspects of constructing pool raise alternatives and the cost and permitting aspects of the feasible alternatives are discussed in the next sections.

4.0 Alternatives Development

4.1 General Description of Alternatives Development

Previous work identified modest generation gains (5,000 MWh) for a pool raise level up to 15 ft, with little additional benefit above 15ft. The existing top of parapet wall elevation is 695 ft, so a pool raise above 10 ft would require new construction of a minimum of 750 ft of parapet wall along the dam and dike. This construction cost, and the ability to demonstrate that the dam is stable with this additional water, makes pool raise levels of less than 10 ft more viable than pool raise levels above 10 ft. It is possible that future stability analysis for pool raise levels above 10 ft would indicate the dam has the required stability. However, recent changes to site seismicity and low factors of safety of previous analyses of the existing dam during an MCE event mean modest increases in pool level (10 ft or less) are more likely to be feasible than options over 10 ft. Dam stability is discussed in Section 6 below. Hydraulic analysis of the existing spillway determined that a pool level of 692.4 ft occurred during the PMF, which leaves 2.6 ft of freeboard to the top of parapet wall. FERC doesn't stipulate freeboard requirements in a quantitative manner; rather, each site is evaluated based on type of dam and environmental conditions. Since the overtopping length is quite large (750 ft) and the dam is of rockfill construction, a small amount of overtopping due to wind and waves present during the PMF would be judged a very small risk. A minimum required freeboard of 0.5 ft during the PMF was selected as criteria for any alternative to be considered feasible.

Each alternative was developed to the point where it could be evaluated as feasible or infeasible, meaning that the alternative is more than possible; that it is practical, workable, would be cost-effective; and if pursued, it would garner FERC acceptance. This last criterion is the most difficult to state in a feasibility study, as the outcome of the FERC design review process is dependent on past experience coupled with a process highly dependent on the make-up of the Board of Consultants and FERC Staff. Determination of feasibility also included an analysis of discharge characteristics necessary to pass the PMF with a minimum of 0.5 ft of freeboard, consideration of design details with an emphasis on constructability, and determination of whether the proposed alternative would provide reliable spillway operation and be cost-effective over the length of the renewed license (through the year 2058).

4.2 Vertical Gates Across Spillway

Vertical roller gates and slide gates could be constructed across the spillway, and this arrangement would pass the PMF but at great cost. This would require 20 gates, each 20 ft wide with 19 2.5-ft to 3-ft-wide concrete piers constructed on top of the existing ogee crest. The piers would rise 10 ft above the crest elevation, as the PMF elevation is 693.92 ft. Structural steel slots would extend above the pier concrete to support the gates in the fully raised position so the bottom of the gate would clear the PMF flow. Structural steel walkways connecting the piers would be required to allow access to the gates. Four or five gates would be roller gates and have electric hoists controlled by a head water controller. To reduce costs, the remaining gates could be slide gates provisioned to allow lifting using a portable generator and drive. The geared drum and cable hoist system would be permanent fixtures on the slide gates. Vertical gate construction, while adding significant weight, would still require anchoring through the piers and crest concrete into foundational rock; the base of the spillway may have to be extended past the current 9 ft length. The 100 ft of sloped ogee crest would require leveling/fill concrete to a flat elevation. This alternative requires significant effort in constructing the piers, and equipment procurement costs are higher than for other alternatives. If the original project construction included vertical gates, 20 ft wide all along the spillway crest, only four of the 20 gates would have operated to date. Winter icing of the seals would require side seal heating elements. Because vertical gates can offer reliable operation, low maintenance, and accurate flow control, 15-ft-wide slide gates are considered on a portion of the spillway later in this report. Use of vertical gates on a portion of the spillway for operational control and flashboards on the remaining section lowers project costs compared to a spillway with roller/slide gates across the entire length. This would not be a good alternative for a river with large tree and root-ball river debris, but the Solomon Gulch drainage does not load the reservoir with this debris, and overall, river debris is very small and of low volume. Vertical gates are workable and may be practical if installation and construction are cost-effective. Thus, this alternative is carried forward until costs are evaluated.

4.3 Concrete Raise with Existing Shape

With the 2.6 ft of freeboard currently predicted at the PMF, raising the existing ogee spillway crest by 2 ft would still pass the PMF without overtopping the dam and would leave about 0.5 ft of freeboard. One option for increasing the spillway capacity is to place new concrete over the existing spillway, matching the characteristic ogee shape to a new uniform crest elevation of 687 ft.

In practice, the new 2-ft raise design would not exactly follow the existing spanwise profile. The 100-ft sloped section of the existing spillway would be brought up to a new, uniform elevation. A 2-ft raise to the main spillway elevation would only be a 1-ft raise to the highest point of the existing sloped section. A uniform elevation across the crest would slightly decrease the head required relative to the flat section of the spillway when passing the PMF. Using the new rating curve, a uniform elevation of 687 ft across the full 450-ft length of the spillway would require 7.25 ft of head, leaving 0.75 ft of freeboard during the PMF.

The specific shape required by this option introduces some complexity to construction. Holes for dowels would need to be drilled into the existing face, then concrete would be placed starting from the bottom of the existing crest and working upward, taking care to closely match the ogee shape. To achieve this raise, concrete would be placed at a depth of 2 ft across the flat 350-ft section of the spillway, and at varying depth across the 100-ft sloped spillway section to achieve a uniform surface across the full length. The cross-section of the new concrete at the lowest and highest points of the existing spillway can be seen in Figure 4-1. Total concrete volume was estimated at 285 cubic yards. These calculations are discussed in more detail in Appendix B, Section B1.

Raising the full pool and modifying the spillway requires verification that the new spillway will not slide or overturn. This stability condition must be shown for both the full pool water level of 687 ft and for the PMF condition, for which the water level would be about 694.5 ft. Using the assumptions established in the STI (CVEA 2018), which negates the post-tensioned anchor contribution for the normal full pool case but allows use of the post-tensioned anchor strength during the PMF, the new spillway would have a factor of safety against over-turning of 1.15 for the full pool condition, and a factor of safety of 1.80 for the PMF condition. Appendix B, Section B1 documents these calculations. This alternative is feasible, and costs and benefits for the concrete raise are summarized later in this section, with costs and economic analysis presented in Section 5 of this report.

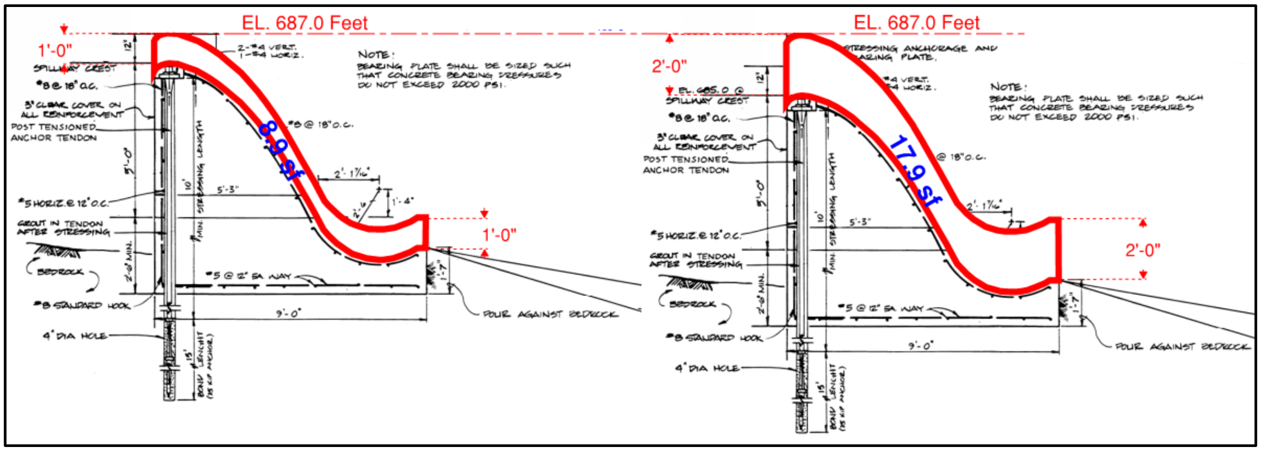


Figure 4-1. New concrete outlines

4.4 Full-length Rubber Dam

A full-length rubber dam was highlighted by HDR Engineering, Inc. (1991) in a feasibility study for the Solomon Gulch Reservoir capacity increase. Following present-day manufacturer recommendations[1] involves modifying the spillway to the point where it can no longer pass the PMF for anything greater than a 4-ft raise when using a rubber dam. This section discusses the design of a rubber dam across the entire length of spillway (450 ft), and the manufacturer recommendations that limit the rubber dam to 4 ft of height.

The advantage of a rubber dam is its ability to spill a range of flows while maintaining the target full-pool elevation. The rubber dam would be sized to the height of the new full pool when fully inflated. If the dam was over-topped significantly, the control system of the rubber dam would sense the rise in water level and automatically begin to deflate the dam to maintain the desired full pool level. When the high-water event tapered off, the control system would also register the drop in water level and would then refill the dam with air.

4.4.1 Design Details

Rubber dam heights from 3 ft to 8 ft were investigated for the alternative of a full-spillway-length dam. The 8 ft option is shown in Figure 4-2, which shows the difficulty of fitting a large rubber dam to an existing thin section spillway. Significant effort to support the rubber dam upstream of the existing ogee crest is required. This supports the rubber structure, provides anchorage, and allows space for deflation without imposing on the ogee crest.

Following manufacturer recommendations, the span of the rubber dam would be divided into three segments, with sloped abutments at either bank, along with sloped center piers separating the individual

[1] Discussions with Mr. Obermeyer during January 2020 included an exchange of drawings and notes of project specifics and current Obermeyer rubber dam support methodology. Obermeyer now makes traditional rubber dams (inflatable dams), as well as the steel-clad-type panel dams. This project is only considering rubber or inflatable dams.

rubber dam bladders. Also matching the requirements from the manufacturer, the crest of the spillway under the rubber dam would be extended upstream to form a broad-crested weir; this is shown as a 2 ft thick platform in red in Figure 4-2. Concrete would also be placed over the face of the spillway to give the downstream side of the crest a constant slope (also in red), which is necessary at the anticipated routine flows to prevent cavitation. The higher full pool along with the new structure in this rubber dam design introduces significant overturning forces. These forces can be opposed by anchoring the support columns into the rock subsurface, by a reinforced connection between the platform and ogee crest, and by installation of grouted anchors (in addition to the existing anchors shown in Figure 4-2) on the upstream side of the spillway crest to provide the additional resistance against overturning and sliding. The upstream anchors could be constructed in conjunction with installation of a concrete seal at the upstream base of the crest to reduce uplift pressures.

4.4.2 Discharge Characteristics

The change in the spillway from an ogee shape to the new crest profile shown in Figure 4-2 is more similar to a broad-created weir and reduces the crest's flow capacity. The discharge characteristics of a broad-crested weir with a downstream slope have been experimentally determined by the U.S. Geological Survey (Tracy 1957). The following equation is used to calculate discharge volume.

$$Q = C * b * H^{3/2}$$

Q – Water flow in cubic ft per second passed by the weir (cfs).

C – Discharge coefficient resulting from the shape of the weir and determined experimentally.

b – Span of the weir section (ft).

H – Total head upstream of the weir relative to the weir crest (ft).

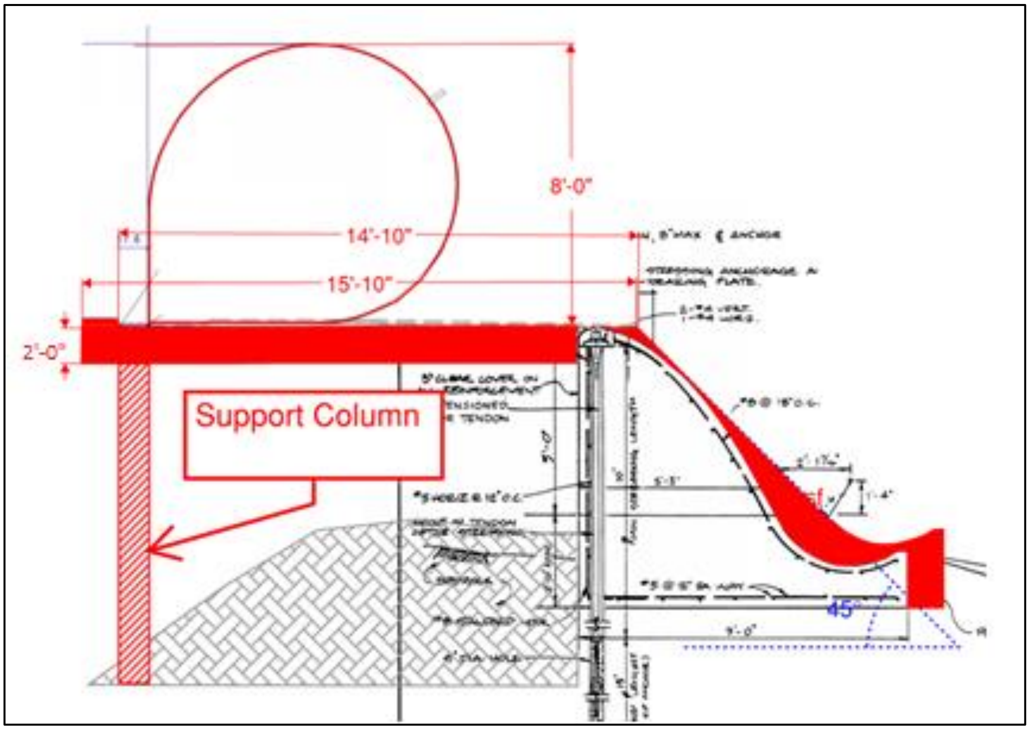


Figure 4-2. Elevation view of 8-ft rubber dam.

The discharge coefficient, C, is a function of both the head, the approach velocity, and L, the length of the platform. Results of an Excel calculation determining the discharge capacity of the rubber dam design can be seen in Table 4-1. Each rubber dam section has a 1.5H:1V sloped invert from the top of the abutment or separation pier down to the ogee elevation of 685 ft. This sloped transition from the top of the abutments and separation piers forms the structure necessary for end attachment but also decreases the discharge characteristic of the spillway. Each sloped section and the flat sections of the ogee crest are listed in Table 4-1 along with the corresponding L and C values. The flow per section using the broad-crested weir formula (Q) is listed in the far-right column.

As shown in Table 4-1, a 4-ft rubber dam with a pool level of 694.5 passes 37,679 cfs, or just over the PMF value of 37,135 cfs. Any rubber dam solution above 4 ft requires larger sloped sections for anchorage, which reduces the discharge capacity.

Table 4-1. Discharge characteristics of sections of 4-ft rubber dam

Spillway Section	Avg. Crest Elevation (ft)	Deflated Thickness	Water Level (ft)	Head (ft)	b (ft)	L (ft)	h/L	C	Q (cfs)
Left Abutment slope	688	0.16	694.5	6.34	6.0	7.915	0.801	2.640	253
1st Flat Section	685.5	0.16	694.5	8.84	94.0	7.915	1.117	3.050	7,535

Spillway Section	Avg. Crest Elevation (ft)	Deflated Thickness	Water Level (ft)	Head (ft)	b (ft)	L (ft)	h/L	C	Q (cfs)
Center Pier	687	0.16	694.5	7.34	12.0	7.915	0.927	2.640	630
2nd Flat Section	685	0.16	694.5	9.34	160.0	7.915	1.180	3.100	14,158
Center Pier	687	0.16	694.5	7.34	12.0	7.915	0.927	2.640	630
3rd Flat Section	685	0.16	694.5	9.34	160.0	7.915	1.180	3.100	14,158
Right Abutment	687	0.16	694.5	7.34	6.0	7.915	0.927	2.640	315
					450				37,679

In summary, the largest pool raise possible using a full-spillway-length rubber dam would be 4 ft in diameter; it will pass the PMF when fully deflated and can be constructed to resist sliding and overturning during the worst-case flood conditions. There is some uncertainty about FERC approval of rock anchors and/or post-tensioned anchors for stabilization of the spillway section. As a minimum to gain FERC approval, new anchors must be tested at the site prior to construction to verify rock strength. In addition, newly installed anchors are pull-tested to design specification, and the design of the new anchors must allow for periodic inspection and testing after completion of construction as the project ages.

Control aspects of this alternative are simplified, as the reservoir level can increase above normal full pool elevation 689 ft. The dam will be fully inflated at the start of run-off. If the reservoir fills and a large inflow event occurs, the dam can be over-topped up to half of the diameter. Since the spillway is 450 ft long, this equates to approximately 5,000 cfs which is six times the peak shown in Figure 3-1 and 90 percent of the 30-year spill maximum. For any conditions when the rate of rise of the reservoir projects above 691 ft (2 ft above full pool), the dam would be deflated; as soon as the water level change rate projects below 689 ft, the dam would be fully re-inflated and over-topping spill would return the reservoir to full pool.

Ice formation is a consideration at Solomon Gulch, but the generation pattern and the wide spillway mitigate ice issues. A cold-weather period necessary for ice formation will also increase electrical load, resulting in a draft of the reservoir. It would be rare but possible for a strong, warm, wet front to surcharge a frozen reservoir near full pool. In this case, passing the flood waters would be more important than maintaining storage and the dam could be deflated. The large unobstructed width of the spillway facilitates ice flow passage. The rubber dam is considered feasible and carried forward to cost evaluation.

4.5 Alternatives Investigated for Emergency Spillway Section

Two alternatives, the rubber dam and an array of vertical gates, have been presented that, if used in conjunction with a flashboard or fuse gate spillway section, would raise the normal full pool level up to 8 ft and pass the PMF, but with an overall project cost decrease compared to the full spillway application of gates or rubber dam. The spillway would be modified to a configuration that would control water levels either by adjustment of gates or by the inflation of a rubber dam in an operating section. The other section of spillway would use flashboards or fuse gates that would release only during very high flood conditions. The released fuse gates or flashboards would require replacement. The combination of an operational section (rubber dam or control gates) and an emergency section (fuse gates or flashboards) reduces project costs but introduces risk of future replacement costs if the emergency system is triggered for release. The following section lists alternatives considered for fuse gates and flashboards that would be used in the emergency spillway section. Operational alternatives are discussed in Section 4.6 as applied to a combined system with the designation of compound spillway.

4.5.1 Emergency Spillway Hydraulic Fuse Gate

Hydraulic fuse gates are prefabricated structures that use predictable overturning to release water. In normal operation, the upstream side of the gate is sealed at the base. When the water level behind the gate reaches a set level, it is free to flow through an inlet into a cavity below the gate bucket. The resulting uplift pressure provides enough momentum to overturn the bucket and allow the flood to pass downstream. Figure 4-3 shows a hydraulic fuse gate manufactured by Hydroplus Corp. This is a proven design with existing implementations. The appeal of this design is the ability to precisely set the point of release at a chosen water level by constructing the inlet to that level. The inlet wells can also be staggered at slightly different elevations, so the individual buckets don’t all tip at once. If the flood could be passed with less than the full emergency spillway span, this would prevent the need to reset or replace all the fuse gates.

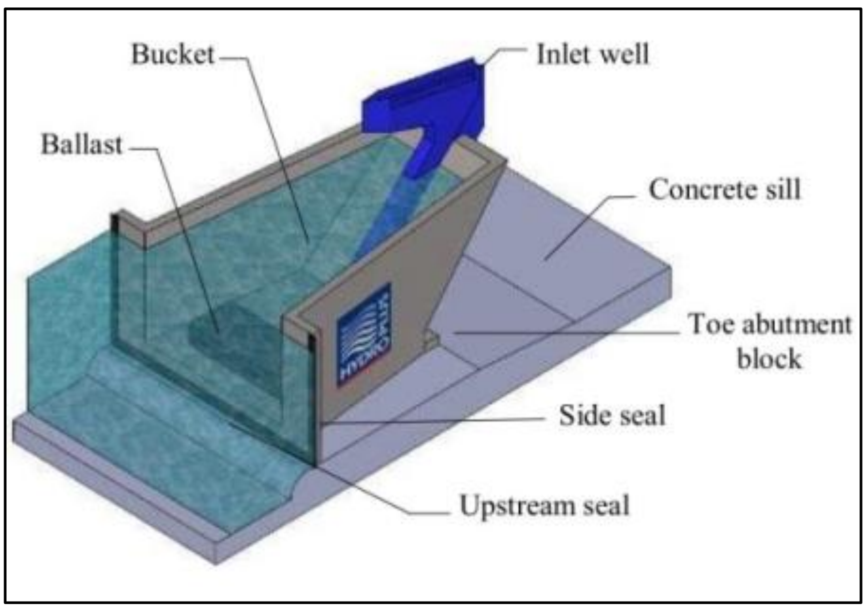


Figure 4-3. Hydraulic fuse gate manufactured by Hydroplus Corp.

This type of fuse gate is designed for very large spillway structures and is not suitable for the small Solomon Gulch spillway. The base of the fuse gate requires a large, flat, concrete sill, which changes the weir to a broad-crested type, reducing the discharge coefficient to a value similar to the 4 ft rubber dam. Since this application would be placed in the emergency spillway where high discharge coefficients are required, pool raise would be limited to 4 ft unless coupled with a control structure such as vertical gates. But that is not a solution, as more vertical gates would be required over a larger portion of the spillway to compensate for the poor discharge coefficient fuse gates. Despite its favorable predictability, this design introduces a few major construction complications at Solomon Gulch. The individual buckets would be large and heavy relative to their height. Installing large prefabricated structures would be challenging due to limited access to the far end of the existing spillway from the dike. Their size would also make it likely that they could damage the ogee crest when they tip, which potentially would require costly repairs. Reliability becomes an issue with the inlet well. If rain or wave water splashed into the inlet channel and froze, it would prevent the trigger mechanism from tipping the bucket during a flood. The advantageous ability to pass water over the top before tipping would turn into a major liability for overtopping the dam without a properly functioning inlet well. Winter ice conditions along the gate interfaces would induce large frictional loads, and this interface is very difficult to heat. This alternative is considered infeasible due to low discharge coefficient, winter freezing, and difficult installation.

4.5.2 Emergency Section Gravel Ballast Fuse Gate

This concept for a fuse plug relies on the water overtopping the emergency spillway flashboard to trigger its release. A panel would be mounted to the crest of the existing spillway to raise the full pool level. The panel and structural mounting by themselves would not be strong enough to support the head of the full pool, but it would be supported on the downstream side by freely placed, clean, graded pea-gravel (see Figure 4-4). During normal operation, the fuse gates would not pass any water and the gravel would remain in place, in contact with the sealed panel. If the capacity of the operational spillway were exceeded, the water level would rise over the top of the panel. The force of the water would wash the gravel downstream, leaving just the panel structural support. Because these alone would be undersized for the full pool head, the panel would break loose and wash downstream.

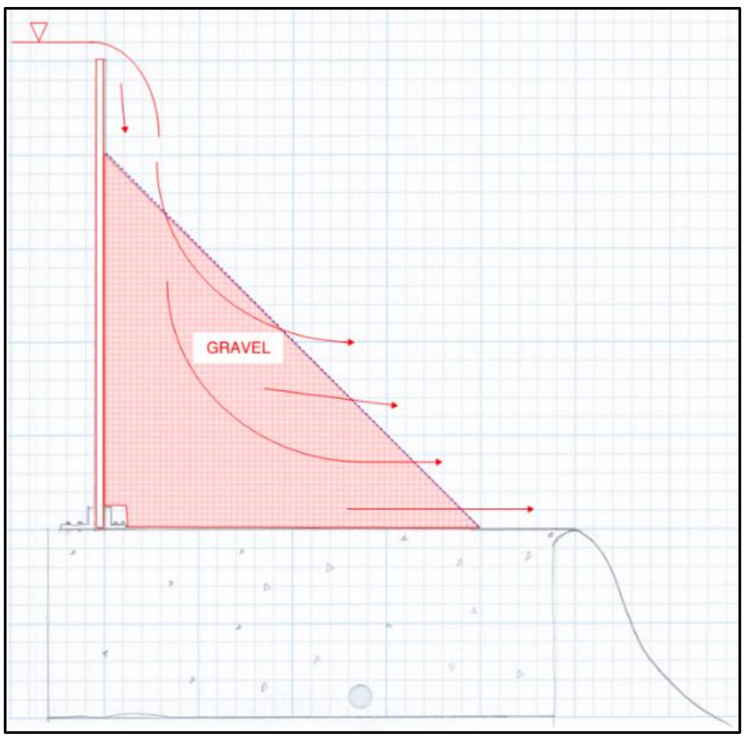


Figure 4-4. Gravel ballast fuse gate

This concept has been approved by FERC and tested and installed at Milner Dam in Idaho. It has the advantage of being an automatic physical release with a predictable water level. It would also be relatively easy to construct as a simple panel with gravel placed behind it. The panel and gravel would be inexpensive dam materials relative to the rubber dam, though the volume of concrete needed for the flat sill under the gravel could reduce or eliminate any cost advantage. The sill presents a larger feasibility issue regarding the PMF. The gravel angle of repose requires such a wide surface that the spillway crest would behave like a broad-crested weir. Even with the benefit of the ogee shape on the downstream side, the discharge coefficient would be below 2.9, and this would not be sufficient to pass the PMF for any combination arrangement. This concept was therefore classified as infeasible.

4.5.3 Emergency Section Buoyancy Release Flashboard

Flashboards present a favorable alternative, as the discharge coefficient of the original ogee crest is available after release of the flashboards. The issue with flashboards is how to support and then safely and reliably release them. Using buoyancy as a trigger mechanism was investigated. A concept was developed using a vertical panel supported horizontally from the upstream direction by a structural truss and a buoyant float designed to lift the panel free of its base only at a specific water level (see Figure 4-5).

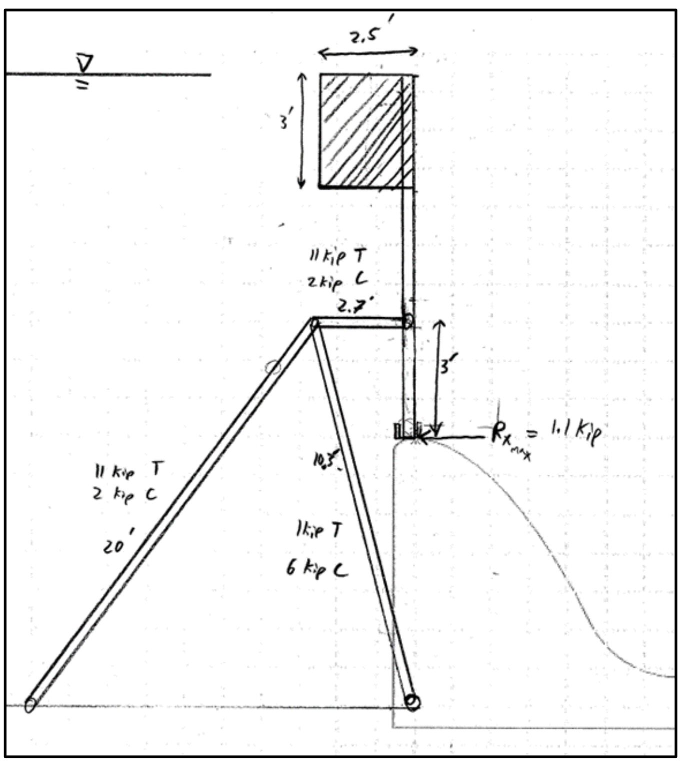


Figure 4-5. Drawing of a vertical panel supported with a structural truss and buoyant float

During normal operation, the panel would be supported against wind and water loads by the horizontal member of the truss and by a channel holding the panel base. The float at the top of the panel would only start to lift the panel when the water level reached it and would only be strong enough to pull the panel from its base channel once fully submerged. At this point, the panel would be free to rotate about the connection point with the truss and water would start releasing. The connection to the truss would be designed to disconnect when the panel would no longer be vertical, letting it wash downstream and leaving the original spillway crest mostly unobstructed. The components to this design are simple and inexpensive and could be constructed on-site relatively easily. This method would automatically release the water and would not require significant change to the existing spillway crest. The trusses and panel bases would not detract much from the effective length of the spillway, so the efficiency of the original ogee shape would safely pass the PMF.

A large-scale bubbler would be required to keep ice loading off the panel. The design has structural members upstream of the spillway where they would often be submerged, and these would remain in the flow or be washed downstream after the spillway release. While horizontal forces are not as crucial as with other designs, friction in the base channel is still dependent on the base horizontal force. Depending on the materials used in the channel, friction could introduce uncertainty in the release point. Snow-loading becomes a significant factor with the top of the float. This force directly opposes the buoyant force used to trigger the release. The potential range of snow load increases the uncertainty in the trigger water level to about ± 0.75 ft. The combination of snow loads resisting the buoyancy force and ice impounding the truss members made the buoyancy-triggered panels infeasible.

4.5.4 Emergency Section Compression Reaction Flashboard

Moving the support structure to the downstream side creates another alternative. The purpose of the truss in Figure 4-6 is to keep the structural support force horizontal to directly counter the horizontal wind and hydrostatic forces without introducing a vertical component. The design in Figure 4-6 intentionally introduces a vertical component to the support reaction with the goal of using that to lift the panel out of a base channel. The red structure represents a way to tune the attachment angle of the downstream compression member, rather than simply using a straight compression beam, as shown in black.

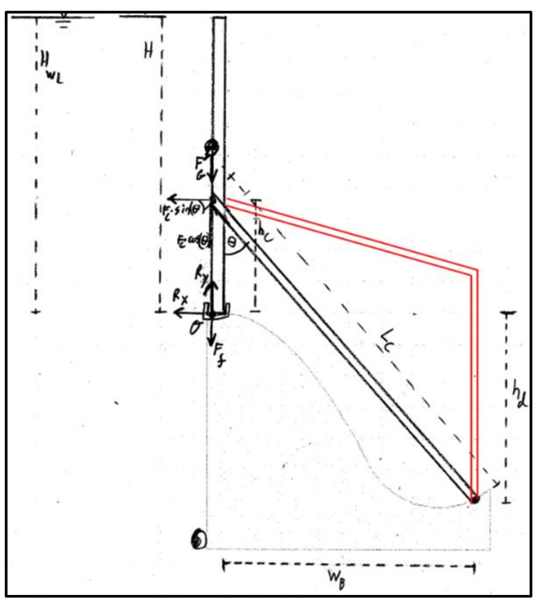


Figure 4-6. Compression reaction flashboard

As the water level rises, the compression force increases to counter the hydrostatic force. Because it attaches at an angle to the panel, the compression member is pushing upwards on the panel as well. The angle and attachment point can be set so that at a chosen water level, the known force of the water produces enough vertical reaction to overcome the weight of the panel and friction in the base channel.

This is a simple design with few components, making it cost-effective. It would also be relatively simple to construct with minimal change to the existing spillway. Both the panel and the compression member would be designed to rotate away and wash downstream, leaving very little obstruction to the flow relative to the original spillway crest. The mechanism would release automatically when the water level gets high enough.

The panels would need a large-scale bubbler to prevent ice loading. This design does not have snow-loading issues like the buoyancy release, but it does have some uncertainty from wind-loading and base friction. The release is dependent on hydrostatic force, so wind countering that force would delay the triggering water level. Knowing the friction coefficient accurately would also be necessary to know the upward force needed to lift the panel from its base. The resulting uncertainty is around ± 0.5 ft for the water level that would trigger a release. This design was tested during the SEAPA Swan Lake spillway hydraulic modeling (1/4 scale) and found to have large uncertainty in the release point without wind. The combination of inherent instability and low resistance to wind makes this alternative infeasible.

4.5.5 Emergency Section Manually Triggered Frangible Nut Flashboard

The previous emergency spillway alternatives do not allow control by means of a manually triggered release. Flashboards need a means of manual triggering to release water should the mechanical system malfunction during a flood. Environmental conditions such as snow load, high winds, and time-based structural friction play a role in the release of mechanical systems, and environmental loads necessary for activation are difficult to design to an exact release point. Designing a release mechanism that includes a manual trigger allows for implementing a factor of safety against inadvertent activation and allows for a factor of safety to be built into the trigger device. This helps reduce the chance of an inadvertent release or a failed release. Manual triggering allows for testing of the release system, a FERC Dam Safety requirement. This section discusses building a flashboard-and-release system for the emergency spillway that can be triggered remotely by an operator during a high flood event.

Water would be held back by a series of 10-ft-high by 7-ft-wide structural steel panels mounted to the top of the existing ogee spillway crest (see Figure 4-7). On the upstream side of the panels, a member would provide compressive support to resist upstream wind loading when water levels were low. On the downstream side, mounted to the existing ogee spillway crest, there would be a structural steel truss designed to provide only horizontal compression force to counter the hydrostatic force. A large-scale bubbler system would be specified to prevent ice formation along the panels. There would still be a small snow and ice load when the water recedes in November, and the steel truss and panels are designed to resist this load. The horizontal compression member would be attached at a joint to a vertical compression member and a tension member internal to the right angle between the two compression members. When the panel is holding back water, the horizontal compression member is supported at the joint by the tension member, and the vertical reaction force this creates at the joint is countered by the vertical compression member.

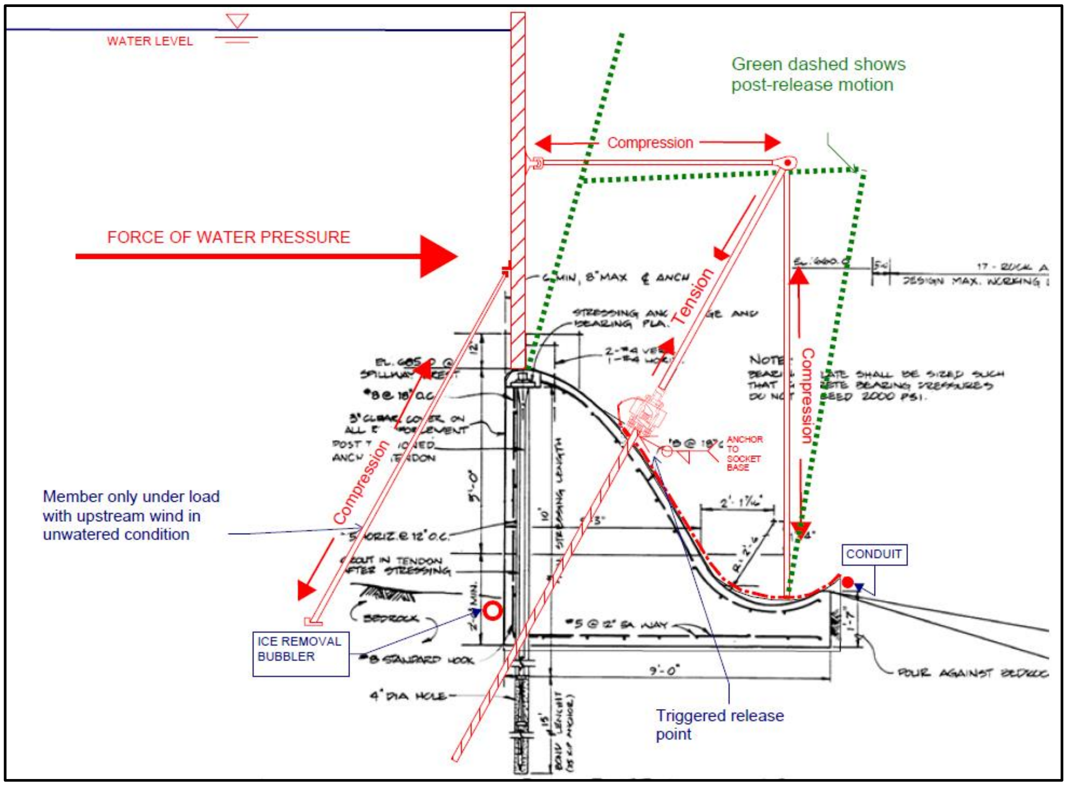


Figure 4-7. Flashboards with frangible nut release on tension member

To release the spillway, the tension member is severed from its anchor in the spillway crest. The base attachment for the panel would be designed to provide horizontal and vertical support but would allow the panel to rotate. Without the tension member, there would be nothing keeping the horizontal compression member in place, so it would no longer support the panel. The panel would be free to rotate around its base. The base would be designed so that as it rotates, the panel becomes detached. The compression member upstream would not be fastened to the panel; it would be held in place during normal operation but would be free to wash away with the panel no longer constraining it. The entire structure – the panel, all compression members, and the tension member – would be free to wash downstream. This would leave the original ogee spillway crest with minimal change from the existing structure to pass very high spill flows.

A device that could support the high-tension loads during normal operation and reliably sever the tension when electrically triggered is a frangible nut, which would provide the link between the tension member and its anchoring into the existing spillway crest. Frangible nuts have a built-in fracture plane and two small explosive boosters inside the body of the nut. When the boosters are actuated, the nut breaks at the fracture plane, severing the mechanical connection to the threaded rod fastened to the nut. Original applications of the frangible nut were developed for the space and military industries, but now the oil industry has created a market for standard stock that has been proven out by testing. McMillen Jacobs contacted three manufacturers; with two of the three companies have frangible nuts that would fit our application. The flashboard cost estimate includes the cost of the nuts based on email quotes, and nut specifications and manufacturing data are found in Appendix C. Notably, the booster technology is common, as these devices that trigger the high-energy burst to the nut are similar to boosters in automobile air bag deployment.

The concrete ogee section would also have significantly more hydrostatic force at full pool, and an analysis shows that it would need additional stabilization. Grouted anchors would be added in-line with the existing post-tensioned anchors, and the tension member would be directly anchored to rock, as shown in Figure 4-7. The anchors would undergo the same FERC review process as described for the rubber dam. Tests of specified anchors in rock near the spillway location would be required to verify rock development length, bonding agent strength, and rock strength. During construction, anchors would be tested and anchor design would provide to ability to inspect and test the anchors later in the project life.

The flashboard system and release mechanism would require a ¼- or ½-scale model to demonstrate reliable release control. Even with a demonstrated design, FERC may require overly conservative measures, which would make the alternative infeasible due to increasing project costs. The tension member flashboard system was the most feasible of the flashboard and fuse gate alternatives considered and was carried forward to Section 5 of this report.

4.6 Compound Spillway

Two alternatives, the rubber dam and an array of vertical gates, have been presented to provide spillway control across the entire 450 ft spillway. If these control alternatives are used in conjunction with a flashboard section, the compound system would raise the normal full pool level 8 ft and pass the PMF, but with an overall project cost decrease compared to the full spillway application of a rubber dam or vertical gates. A compound spillway uses a shorter and more costly operating section for control, and a longer, lower-cost emergency section for large flood events. Lower project cost introduces risk of future replacement costs, and the emergency section, if triggered for release, requires the replacement and reinstallation of structural steel panels and truss work.

Increasing the length of the operational spillway increases initial project cost but reduces the risk of future replacement costs due to a rare release of the emergency section. Decreasing the length of the operational spillway decreases initial project cost but increases the risk of repetitive replacement costs due to replacing the emergency spillway on a more frequent basis. The highest water level ever recorded corresponds to a flow of 5,457 cfs, based on spillway records. The operational spillway should be able to pass this flow plus an additional safety margin. The remainder of the 450-ft spillway would be the emergency spillway, serving the function of flashboards that would release only when the flow exceeded the passable flow through the operational spillway. Figure 4-8 shows conceptually the division between an operational spillway of length X and an emergency spillway over the remaining span.

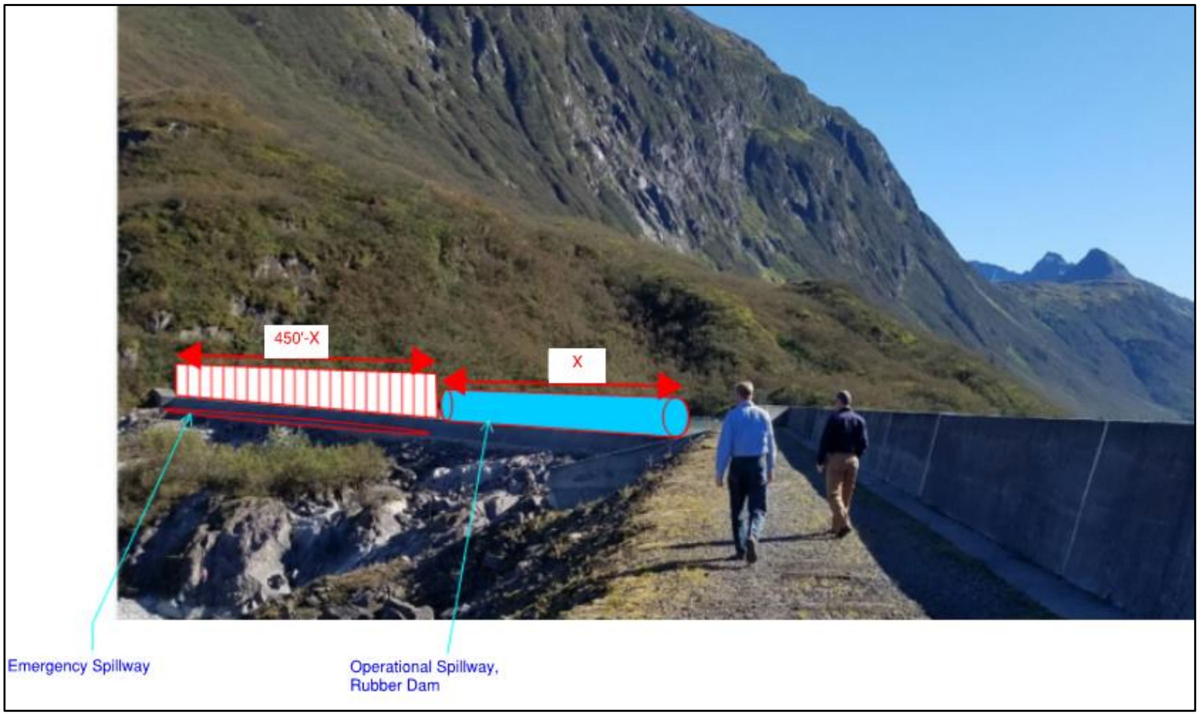


Figure 4-8. Conceptual sketch of a compound spillway

4.6.1 Compound Spillway Rubber Dam Operating Section

A rubber dam provides the ability to decrease the dam elevation to pass typical high seasonal flows without losing the capacity stored at full pool. Rubber dams are not a good choice for accurate flow control but are a good choice if control requirements for the reservoir can accept some degree of tolerance. Rubber dams made today can be over-topped up to one-half of the design diameter. These dams have proven reliable in cold climates, an added benefit for the Solomon Gulch site, where the top surface of the full reservoir can freeze the rapidly in October. If warranted, the bubbler system required for the flashboard system could be sized to include the control section to maintain an ice-free zone in front of the rubber dam. For large floods inundating a frozen reservoir, a rubber dam presents the best control alternative, as it can be completely deflated, opening a large channel for ice flow passage.

A full spillway length rubber dam limits the pool raise to 4 ft. If the length of rubber dam spillway were reduced from the full 450 ft to a smaller operational section capable of passing historical record flows plus a safety margin, the remaining length could be made into an emergency spillway that would only pass water if the capacity of the rubber dam section were exceeded. The emergency spillway would need to have a higher discharge coefficient than the deflated rubber dam, which would provide enough additional capacity to pass the PMF. Determining the length of the operational and emergency sections is a study of cost and risk; the shorter the operational section, the lower the initial project costs but at a greater the risk of replacing the emergency sections. Table 4-2 shows the relationship between operational and emergency lengths in conjunction with normal high inflows and PMF events.

Table 4-2. Combinations of emergency and operational spillway lengths for a compound rubber dam and flashboard spillway, 8 ft pool raise.

Flashboard Emergency Section		Rubber Dam Operating Section			Total Compound Spillway		
Section Length (ft)	PMF Contribution (cfs)	Section Length (ft)	693-ft Full Pool Discharge (cfs)	PMF Contribution (cfs)	PMF Elevation (ft msl)	PMF Discharge (cfs)	Add'l Piers Possible (Hw = 694.2 ft)
300	30,320	150	6,245	6,831	693.47	37,151	10
275	28,535	175	7,716	8,633	693.62	37,168	8
250	26,621	200	9,187	10,525	693.77	37,146	6
225	24,617	225	10,658	12,529	693.93	37,147	4

Four combinations of operational and emergency spillway lengths are listed in Table 4-2. Emergency spillway section lengths are listed in descending order from 300 ft to 225 ft. As this length decreases, the section length of the operating spillway increases and the full pool discharge increases. Cost increases with operating spillway length, but risk of triggering the flashboards decreases (6245 cfs vs. 10,658 cfs). Total project cost increases because on a lineal basis, the rubber dam is more expensive than the flashboard system (costs estimates are discussed in Section 5). The historic maximum spill flow at Solomon Gulch was estimated from recorded water levels as approximately 5,500 cfs. Each length of operating spillway exceeds this value. McMillen Jacobs specified 40 percent over the existing 30-year spill maximum or 7640 cfs that the rubber dam should pass without triggering the flashboard system. This corresponded to an 8 ft rubber dam 175 ft long, and a flashboard system with a total length of 275 ft. The maximum spill value prior to emergency release is a comparison value for other alternatives discussed later and would be modified during the design process as more information became available. Table 4-2 also lists the number of additional piers that could be added to the emergency spillway without exceeding a PMF elevation of 694.2 ft. Each pier is 2.5 ft wide and would allow the installation of partial flap gates, a tiered trigger system to prevent the release of the entire flashboard, or vertical slide gates to increase the flow control function of the spillway system. This is a significant finding that allows greater insurance against a complete flashboard trigger and allows the partial use of simplistic control gates within the flashboard system; these control gates operate once on release and can be reset after the flood event recedes. The rubber dam would spill inflows above outflows at full pool simply by being over-topped. For spill flows above 500 cfs (45 percent above plant capacity), the dam would require deflation.

4.6.2 Compound Spillway Vertical Gate Operating Section

Section 4.2 briefly discusses vertical gates as a means to provide a controlled spillway across the entire length of spillway. The alternative presented here uses vertical slide gates instead of a rubber dam in the operational section, and triggered flashboards in the emergency section. Slide gates, rather than roller gates, are suggested to reduce cost. To ensure that slide gates are appropriate, gate width has been limited to a nominal 15 ft. Table 4-3 lists six combinations of emergency and operational sections. All combinations have excess discharge capacity related to the PMF. PMF elevations range from 692.69 up to 693.24 and are well below the 694.5 free-board criteria. Using the same operating capacity of 7,640 cfs, the minimum operating section length would be 125 ft and the emergency section would be 325 ft. The number of gates is based on a nominal 15-ft width to ensure the use of slide gates but to also maintain lower electrical loads, installation weights, and side friction forces, and to keep piers narrow. If warranted, the bubbler system installed for the flashboard system could be sized to include the control section to maintain an ice-free zone in front of the gates.

Similar to the compound rubber dam, excess discharge capacity allows for flexibility in design of the flashboard system. As an example, the third row of Table 4-3 lists a 325-ft emergency section coupled with a 125-ft operating section. The elevation of the reservoir during the PMF for this system is 692.9 ft. If two additional piers are added to the flashboard system spaced at 14 ft, these bays could contain a flap gate or panels that would trigger earlier than the remaining panels. These additional control or pre-trigger bays allow for the installation of lower-cost control measures than used in the control section and reduce the risk of releasing the entire flashboard system. The addition of the piers decreases effective spillway width but with only a slight increase in PMF elevation.

Table 4-3. Compound spillway using vertical slide gates and flashboards

Emergency Section		Vertical Slide Gate Operating Section						Reservoir	
Section Length (ft)	PMF Contribution (cfs)	Section Length (ft)	No. of Slide Gates	Effective Operation Length (ft)	Gate Width (ft)	693-ft Full Pool Discharge (cfs)	PMF Contribution (cfs)	PMF Elevation (ft msl)	Total PMF Compound (cfs)
375	32,787	75	4	65.0	16.25	4,653	4,367	692.69	37,154
350	31,320	100	6	85.0	14.17	6,084	5,855	692.81	37,175
325	29,587	125	7	107.5	15.36	7,695	7,542	692.9	37,129
300	27,988	150	9	127.5	14.17	9,127	9,181	693.03	37,170
275	26,137	175	10	150.0	15.00	10,737	11,017	693.13	37,154
250	24,245	200	11	172.5	15.68	12,348	12,945	693.24	37,190

Equations used for Table 4-2 and

Table 4-3:

$Q = \eta CLH^{1.5}$

η = Loss coefficient for used only when piers are in flow passage, η=.85

C = Spillway discharge coefficient, C= 4.1 (See Ref 5) for Ogee Spillway sections (Table 4.3)

C = 2.06 used as an average value for the entire rubber dam length of 175 ft (Table 4.2) and Ref 3

L = effective open length of passage or spillway, (ft)

Q = Discharge (cfs)

4.7 Summary of Alternatives Development

Alternatives considered feasible, meaning workable, practical, and appropriate for the estimated benefit of a pool raise, are summarized here. Cost estimates and regulatory review follow in Section 5 and Section 6.

4.7.1 Alternatives Developed by Limiting Raise Level

Hydraulic/power analysis results described Section 3 identified modest generation gains for pool raise levels up to 15 ft, with little additional benefit above 15 ft. The existing top of parapet wall elevation is 695 ft, so a pool raise of 15 ft would require construction of a minimum of 750 ft of new parapet wall along the dam and dike. The new wall would have to be 11 ft high and capable of withstanding 10 ft of hydraulic head. This construction cost, and the ability to demonstrate the dam is stable with this additional water, means pool raise levels of less than 10 ft are more likely to be feasible. Previously, a freeboard of 2 ft between the operating full pool level and the top of parapet wall was selected. Therefore, alternatives considered did not exceed 8 ft.

4.7.2 Design Considerations

Passing river debris is not an issue at Solomon Gulch, so the spillway structures considered in this report did not have a trash accumulation consideration applied to feasibility.

Ice loading and ice flows are a consideration, so ice mitigation was addressed for all alternatives. McMillen Jacobs contacted Canadianpond.ca, a division of Les Etangs PPM, for advice in cold-region spillway ice prevention and mitigation. Canadianpond.ca has designed and installed numerous ice prevention systems for hydroelectric power generation and oil sands applications across northern Canada, the northern United States, and Norway. A company fact sheet and project estimate can be found in Appendix B. All feasible alternatives except raising the ogee crest 2 ft by adding concrete require a bubbler system. The vertical gates would require side-seal heater elements to ensure ice doesn’t form as a result of minor leakage. An example of a large-diameter rubber dam in southeastern Canada found in Appendix B shows rubber dam use in cold climates with significant ice flow.

Control aspects of the alternatives were considered, with an emphasis on simplicity and low cost. For the 2 ft raise alternative, control is not a consideration, as this is merely a raising of the ogee crest. Since rubber dams can sustain over-topping up to ½ of the nominal diameter of the dam, the 4 ft raise alternative can remain inflated at the new full pool level of 689 ft and pass up to 5,000 cfs of spill over the top of the inflated dam. For any conditions in which the rate of rise of the reservoir projects above 691 ft, the dam would be deflated; as soon as the water level rate of change projects below 689 ft, the dam would be fully re-inflated and over-topping spill would slowly return the reservoir to full pool. Over-topping of the compound spillway rubber dam results in less flow, as it has less length and less head when inflated; maximum over-topping flow is approximately 700 cfs. If an 8 ft pool raise demonstrates sufficient dam stability, then rubber dam control would be modeled, and plant operating criteria reviewed. If rubber dam control alone continues to be an issue, one or more vertical slide gates could be added to the dike side of the spillway to improve control deficiencies. Since 700 cfs is two times plant capacity, a rubber dam alone is probably sufficient for control.

4.7.3 Summary of Alternative Hydraulic and Generation Values

A summary of feasible spillway alternatives is listed in Table 4-4. Several flashboard release mechanisms were investigated; the release structure and mechanism listed in Table 4-4 is the manually triggered frangible nut mechanism. Emergency spillway alternatives and flashboard release mechanism evaluation is summarized in Table 4-5. Total evaluation of alternatives is discussed in Section 7, as it requires tabulation of estimated costs (Section 5) and a listing of regulatory and permitting issues (Section 6). Table 4-4 summarizes hydraulic characteristics and benefits by raise level, and Table 4-5 summarizes emergency gate feasibility determinations.

Table 4-4. Summary of alternative hydraulic and generation values

Hydraulic Characteristics and Generation by Raise Level	Raise Level (ft)	Additional Storage (ac-ft) (% incr.)	Generation Benefit (MWh)	Spill Before Release of Emergency FB (cfs)	PMF Elev. (ft msl)	Control Solutions Required
Concrete added to top of existing Spillway	2	1,289 (4.2%)	691	-	694.4	none
4 ft diameter rubber dam across entire spillway	4	2,603 (8.4%)	1,382	-	694.4	none
Compound Spillway, rubber dam 175 ft, Flashboards 275 ft	8	5,309 (17%)	2,759	7,640	693.6	potential
Compound Spillway - seven vert. slide gates 125 ft, flashboards 325 ft	8	5,309 (17%)	2,759	7,690	692.9	none

Storage values from reference drawing No. HO1-F-04-2011-R49
Generation benefit values interpolated from Table 3-2

Table 4-5. Summary of emergency flashboard release alternative evaluation

Alternative	Pros	Cons	Conclusion
Hydraulic Fuse Gate	Known applications, industry developing for use to increase storage at large reservoirs	Winter operation requires large heat source, low discharge coefficient, hard to install	Too many problems, infeasible

Alternative	Pros	Cons	Conclusion
Gravel Ballast Fuse Gate	Known applications previously approved by PRO, inexpensive if base doesn't require construction	Low discharge coefficient due to large base	Can't pass PMF, infeasible
Buoyancy Release Flashboard	Some known applications, low cost	Release system is inexact due to wave action, snow and ice loads above water surface	Too many problems, not practical in harsh winter environment, infeasible
Buoyancy Release Flashboard	Some known applications, low cost	Release system is inexact due to wave action, snow and ice loads above water surface	Too many problems, not practical in harsh winter environment, infeasible
Compression Reaction Flashboard	Some known applications, low cost	Release system is inexact due to wave action, manual trigger doesn't add reliability	too many release problems, infeasible
Manual Triggered Frangible Nut Flashboard	retains high discharge coefficient to pass PMF with 2 ft of freeboard, low installation cost, predicable release,	No known applications, FERC approval more difficult but possible	Feasible

5.0 Cost Estimates

5.1 General Description

An engineer's cost estimate is provided to compare alternatives for the Solomon Gulch Pool Raise Feasibility Study. The conceptual estimate is similar to a Class 5 estimate as defined by the Association for the Advancement of Cost Engineering (AACE).

The formal description of a Class 5 estimate:

> AACE International CLASS 5 Cost Estimate - Class 5 estimates are generally prepared based on very preliminary information, and subsequently have wide accuracy ranges. Typically, engineering is 0% to 2% complete. They are typically prepared for any number of strategic business planning purposes, such as but not limited to market studies, assessment of initial viability, evaluation of alternate schemes, project screening, project location studies, evaluation of resource needs and budgeting, long-range capital planning, etc. Virtually all Class 5 estimates use stochastic estimating methods such as cost/capacity curves and factors, scale of operations factors, Lang factors, Hand factors, Chilton factors, Peters-Timmerhaus factors, Guthrie factors, and other parametric and modeling techniques. Expected accuracy ranges are from -20% to -50% on the low side and +30% to 100% on the high side, depending on the technological complexity of the project, appropriate reference information, and the inclusion of an appropriate contingency determination. Ranges could exceed those shown in unusual circumstances. As little as 1 hour or less to perhaps more than 200 hours may be spent preparing the estimate depending on the project and estimating methodology (AACE International Recommended Practices and Standards).

Solomon Gulch engineering has been completed to a greater degree than 2 percent but is less than 10 percent. McMillen Jacobs suggests a tolerance of -30 percent to +50 percent be used for the cost estimates of Section 5.2.

This Class 5 cost estimate used a scaling of project costs for mobilization and demobilization. No construction schedule was developed, but a single construction season is assumed for each alternative for comparison purposes.

5.2 Cost and Value of Pool Raise Alternatives

Costs for alternatives are listed in Table 5-1 and Table 5-2. The costs for the 4 ft rubber dam and the 8 ft rubber dam are very close, but this is because the 8 ft dam alternative is only installed partially across the spillway – 175 ft vs. 450 ft for the 4 ft rubber dam alternative. Two alternatives for an 8 ft raise using a compound spillway were carried into cost estimating, a 175-ft-long rubber dam section and a 125-ft-long vertical gate section. Since the vertical gate alternative has a significantly higher construction cost and would present more costly annual maintenance than the rubber dam alternative, only the rubber dam alternative is discussed in Section 7. Detailed cost estimates are found in Appendix D.

Table 5-1. Cost estimate for Solomon Gulch Pool Raise Evaluation, 2 ft and 4 ft alternatives

Item	Alternative	
	2 ft Raise Concrete over Ogee Surface	4 ft Raise - 450 ft Rubber Dam
General Requirements/Mobilization/Demobilization	$152,231	$983,098
Site Prep and Access Roads	$38,200	$38,200
Uplift Prevention Measures (Cutoff Wall/Rock Anchors)	-	$211,698
Dam Raise Concrete (2 ft Raise)	$570,722	-
4-Foot Rubber Bladder Dam (450 ft Length)	-	$3,682,493
Project Subtotal (Direct and Indirect)	$761,200	$4,915,488
Contingency (20%)	$152,231	$983,098
Total	**$913,500**	**$5,898,600**

Table 5-2. Cost estimate for Solomon Gulch Pool Raise Evaluation, 8 ft flashboard alternatives

Item	Alternative	
	8 ft Raise - 175 ft Rubber Dam, 275 ft Flashboards	8 ft Raise - 125 ft Vertical Gates, 325 ft Flashboards
General Requirements/Mobilization/Demobilization	$904,889	$1,081,112
Site Prep and Access Roads	$38,200	$38,200
Uplift Prevention Measures (Cutoff Wall/Rock Anchors)	$229,518	$229,518
Rubber Bladder Dam 8 ft Diam., 175 ft Length	$2,297,936	-
Panel System (40 Panels), 275 ft of emergency spillway	$1,053,900	-
Vertical Gates 15 ft 4 in., 125 ft of emergency spillway	-	$2,930,354
Panel System (48 Panels), 325 ft of emergency spillway	-	$1,165,155
Project Subtotal (Direct and Indirect)	$4,524,500	$5,405,600
Contingency (20%)	$904,900	$1,081,200

Item	Alternative	
	8 ft Raise - 175 ft Rubber Dam, 275 ft Flashboards	8 ft Raise - 125 ft Vertical Gates, 325 ft Flashboards
Total	**5,429,400**	**6,486,800**

Notes:

- A contingency of 20% was applied to direct and indirect costs for all alternatives.
- Permitting and engineering are not included.
- Uplift costs include both sealing the upstream rock concrete interface and addition of new spillway anchoring.
- Additional anchoring costs for the vertical gate structure over the rubber dam structure are is included in the vertical gate cost estimate.
- Additional mobilization of a larger RT type crane for vertical gate installation is included in the vertical gate cost and not in the mobilization cost estimate.

5.3 Benefit-to-Cost Analysis

A benefit-to-cost analysis was performed on the three alternatives (2 ft concrete raise, 4 ft raise using a full-length rubber dam, and the 8 ft raise using the rubber dam and flashboard compound spillway). The annual cost of construction was assumed from an issue of debt at 4 percent for 30 years. O&M costs to maintain the new structures are estimated for inspections and materials for air compressors and electrical components. The benefit was assumed to be a savings in diesel fuel consumption. Construction was assumed to start in 2028 with escalation of the 2020 cost estimates to 2028 at 4.5 percent

Results of the analysis are highly dependent on debt terms and the estimated cost of diesel. Table 5-3 states the price of diesel fuel starting in 2028 with a 1 percent escalation that would be necessary to have a benefit-to-cost ratio of unity (i.e., break-even) with a 4 percent bond rate. The 2 ft raise is least sensitive to low-cost fuel, and the compound raise provides the most value in saved fuel consumption, but the fuel price must be greater than $2.16/gal for 30 years for the project to be cost-effective. Additional examples of economic analysis with the 2028 cost of diesel at $3/gal can be found in Appendix D.

Table 5-3. Benefit-to-Cost Analysis results

Alternative	2020 Construction Cost	2028 Construction Cost	Annual Cost of Construction	Annual Benefit (MWh)	Benefit Diesel (gal)	Annual Maintenance Cost	Fuel Price for B/cost=1	Present Value 30-year Analysis
Loan Rate 3.00%								
2 ft Concrete Raise	($913,500)	($1,299,000)	($66,274)	691	46,689	$0	$1.38	$2,163,026
4 ft Rubber Dam	($5,898,600)	($8,388,400)	($427,970)	1,382	93,378	($5,000)	$4.57	($5,182,046)
8 ft Compound Spillway	($5,429,400)	($7,721,100)	($393,925)	2,759	186,419	($10,000)	$2.16	$3,927,860
Loan Rate 4.00%								
2 ft Concrete Raise	($913,500)	($1,299,000)	($75,121)	691	46,689	$0	$1.50	$1,888,761
4 ft Rubber Dam	($5,898,600)	($8,388,400)	($485,102)	1,382	93,378	($5,000)	$4.93	($6,953,139)
8 ft Compound Spillway	($5,429,400)	($7,721,100)	($446,512)	2,759	186,419	($10,000)	$2.33	$2,297,657
Loan Rate 5.00%								
2 ft Concrete Raise	($913,500)	($1,299,000)	($84,502)	691	46,689	$0	$1.62	$1,597,965
4 ft Rubber Dam	($5,898,600)	($8,388,400)	($545,677)	1,382	93,378	($5,000)	$5.32	($8,830,978)
8 ft Compound Spillway	($5,429,400)	($7,721,100)	($502,269)	2,759	186,419	($10,000)	$2.51	$569,201

Note:

- Fuel inflation rate = 1.00%
- Bond term = 30 years
- Construction inflation = 4.50%

- O&M inflation = 3.00%
- Diesel heat rate = 14.8 kW/gal
- Diesel bulk rate = $2.50/gal

6.0 Regulatory and Permitting Review

6.1 General Description of Permitting Requirements

Permitting of the pool raise alternatives discussed in Section 4 would trigger a license amendment process. Per FERC requirements, increases in max pool elevation (storage) for the purposes of additional generation require Agency consultation, verification that impacted lands remain within the original FERC boundary, and that operation of the project would continue to comply with previous operational restrictions (such as withdrawal commitments supplied to the Valdez Fisheries Development Association). The amendment process would be administered by FERC's Division of Hydropower Administration and Compliance in Washington, D.C., and this office would confirm that dam safety requirements administered by the Portland Regional Office (PRO) have been satisfied.

6.2 Design Approval Process

While FERC in Washington, D.C., would play an integral role in the amendment process, ultimate approval of the pool raise alternative and associated infrastructural modifications from a dam safety perspective would be administered by the PRO Division of Dam Safety and Inspections. Construction would not be allowed to start until the PRO had issued a notification to proceed with construction. The notification would be the culmination of the approval process, which would require the formation of a Board of Consultants (BOC) that had reviewed the design and then issued an approval of the design to FERC. Two or three major meetings with FERC and the BOC would be required, at which a joint review of the 30%, 60%, and final design was conducted. From these meetings, the Owner is usually granted the terms under which they can proceed with final design. The Independent Consultant can be on the Board of Consultants, which expedites the process. A key to a quality collaborative start with the BOC and the PRO is to have the existing Probable Failure Modes identified that the design modification would impact.

6.3 History of Solomon Gulch Probable Failure Mode Analysis

The first Solomon Gulch Probable Failure Modes Analysis (PFMA) meeting was held in 2007 with 13 Probable Failure Modes (PFMs) identified. The PFMs were revised with the first update in 2009, and again in 2012. Through this process, six PFMs that were originally designated as *Category III-not enough information*, were re-designated as either *Category II-important* and included as part of the Dam Safety Surveillance and Monitoring Plan (DSSMP), or as *Category IV-ruled out as a PFM*. Table 6-1 lists the seven current Category II PFMs as reported in the 2017 Independent Consultant's report. Of the Category II PFMs, numbers 2, 3, 8, 11, and 12 should not limit pool raise alternatives. Both PFM 4 and PFM 6 may limit or preclude pool raise alternatives above just a few feet. These potentially limiting PFMs are discussed below.

Table 6-1. Solomon Gulch Category II Probable Failure Modes

PFM No.	Failure Mode
2	Spillway discharge up to PMF causing eddy to the left of spillway eroding toe of Saddle Dike embankment.
3	Major Flood Event impacts penstock crossing of spillway channel rupturing penstock.
4	Corrosion of spillway anchor tendons leads to failure of spillway monolith under PMF.
6	Earthquake causing off-set of Main Dam or Saddle Dike asphaltic facing, flow through rockfill embankment, piping within dam, and progressive failure.
8	Earthquake causing failure of penstock directly above powerhouse.
11	Mis-operation of butterfly valves under failure of penstock.
12	Closure of butterfly valves under failure of penstock.

6.4 Discussion of PMF No. 4

PFM No. 4: Corrosion of anchor tendons leading to failure of spillway monolith under PMF. This PFM has also been recorded as sliding failure and/or over-tipping under the PMF load case.

The original classification of PMF No. 4 was *Category III-more information or analysis required*. The 2009 supplement stated that current FERC practice is to require post-tensioned structures to exhibit a Factor of Safety (FS) equal to 1 or greater without the contribution of the post-tensioned anchors under usual loading. After review of the spillway without anchorage, the 2009 PFMA review listed PFM No. 4 as Category II. Section 8.8.2 of the Supporting Technical Information (STI) document (CVEA 2018) contains an analysis summary for the spillway as listed below:

Usual Load Case: reservoir elevation = 685.0, no anchor contribution, friction angle = 45°, tallest section of spillway (15 ft), overturning factor of safety (OFS) = 1.27

Unusual Load Case: PMF reservoir elevation (694 ft) OFS = 2.0, and a sliding factor of safety SFS = 2.07, both with anchor contribution of 23K/ft. For the same case but without anchor contribution OFS < 1.0

Extreme Load Case: maximum credible earthquake, pga = 0.9g, PT anchor force of 23K/ft, Sliding Factor of Safety (SFS) = 1.0, OFS = 1.26

The Independent Consultant (IC) recommendation was to add monitoring of the spillway keys and construction joints both periodically and after large spill events to verify that no movement had occurred. FERC agreed with the proposed monitoring and monitoring note sheets were added to the DSSMP.

6.4.1 Pool Raise Issues

Previous stability analyses of the spillway indicate an FS of 1.0 with the pool level at approximate elevation 686.75 ft, 1.75 ft above the existing full pool level. The future usual load case resulting from a pool raise would use a water level higher than the level where the spillway is stable without contribution of the post-tensioned anchors. Remedies for this load condition include: 1) seal the rock/concrete interface, 2) add drains in the spillway (both of which reduce uplift pressure), and 3) add more anchoring acceptable to FERC.

A request by FERC for a project seismicity review completed by Cornforth 2019 (ref 07) and a pending seismic stability analysis to be competed in 2020 may indicate that the existing structure would require modification just to maintain the current operating level of 685.0 ft. While FERC specifically targeted the dam and saddle dike for review, it is probable that this would later be applied to the spillway.

6.5 Discussion of PMF No. 6

PFM No. 6: An earthquake causes a lateral off-set or a vertical settlement of the Main Dam or Saddle Dike. Damage and separation of the asphaltic upstream face allows flow through the zone 1 rockfill embankment which leads to piping within the dam, and then to progressive failure.

The original PFM was theorized before the first stability analysis was completed, meaning that the failure mode was based more on subjective thought than on modeling results. Subsequent modeling showed that the offset would occur at high elevations of the dam (above 660 ft and therefore limited flow potential). In 2011, a low level outlet works (LLOW) was installed to reduce the consequences of a breach in the upper asphaltic surface. The 2012 IC acknowledged that the LLOW reduced the consequences of PFM No. 6 but continued with a designation of Category II. Recent ICs have theorized that because Zone 1 material is 10 ft wide, the subsequent leakage through Zone 1 and then Zone 2 (a much coarser gradation with very little fine material) could occur without damage to the rock embankment. IC comments state that the Zone 1 cover over the Zone 2 material remaining after offset would slow the flow, and the open porosity of Zone 2 would allow leakage to occur without causing stability degradation. Table 6-2 and Table 6-3 summarize dam stability factors of safety from Section 8 of the STI related to PMF No. 6 (CVEA 2018).

Table 6-2. STI documented stability Factors of Safety for Solomon Gulch Dam

Load Condition	FS
II-Static-Infinite Slope Method	2.13
II-Static Deep Circle Method	2.35

Load Condition	FS
Pseudo Static (k=0.3) Infinite Slope	1.19
Deep Circle	1.45

Table 6-3. STI documented stability Factors of Safety for Solomon Gulch Dam

Load Condition	OBE* (ft)	SEE** (ft)
Maximum Permanent Crest Settlement	0.3-1	0.5-2.5
Max. Permanent Displacement Below Reservoir	0.3-.5	0.5-1
Max. Permanent Displacement	0.5-1	2-9
Maximum Offset-SEE uses 0.5g (2009)	ft	
Elevation 685 (full pool)	0.3-0.9	
Elevation 675 ft	0-0.3	

*OBE = Ordinary Basis Earthquake

**SEE = Safety Evaluation Earthquake

At the conclusion of the 2012 Part 12 process, in a letter dated June 9, 2014, the FERC PRO states that the IC should provide a detailed explanation that specifically addresses the stability and stress analysis and, "the 2017 Part 12 Report should include a review of the currently available seismic information, including more recent ground motion prediction models for subduction events, and a determination if the predicted ground motions and current project Maximum Credible Earthquake are still appropriate for use in stress and stability analysis." During the 2017 PFMA review, it was noted that a number of PFMs would be changed to Category III until a revised seismicity report and stability analysis could be conducted. During 2019, a new seismicity report (Cornforth, rev 01, ref 07) was completed for Solomon Gulch with substantial increase in peak ground acceleration up to 1.1g. Dam stability analysis using the new seismicity data had not been completed by the close of 2019.

6.5.1 Pool Raise Issue

The original stability analysis used a subduction zone earthquake with a 0.5g pga causing a vertical offset from 0.3 to 0.9 ft at elevation 685 ft; dam face offset at elevation 675 ft was estimated between 0 and 0.3 ft. New subduction zone stability analyses using larger peak ground accelerations (1.1g instead of 0.5 g) could indicate that larger vertical displacements are possible and at deeper levels in the dam. If a new full pool raise to elevation to 693 ft were implemented and the offsets occurred at elevations near 680 ft, then more water would leak through the dam face than would have occurred with a full pool elevation of 685 ft.

6.6 Conclusions Regarding FERC Approval and Permitting Process

Future dam stability analyses will be conducted to fulfill recent FERC PRO requests. This analysis should include the determination of the existing full pool elevation of 685 ft stability and determine the limiting value of a future pool raise. This would allow CVEA to prove out the pool raise concept with respect to dam safety before discussing the raise issue with FERC and before funding further engineering and permitting efforts. This process would reduce project risks compared to performing dam stability engineering related to a pool raise later with relicensing efforts.

The Solomon Gulch license will expire in June 2028. Per FERC requirement, relicensing efforts must begin between 5.5 and 5 years prior to license expiration (i.e., no later than June 2023). A change in full pool elevation/storage capacity typically triggers an onerous and potentially expensive license amendment. If proactively and strategically planned, the most cost-effective and implementation-efficient way to make major project infrastructural changes is to combine them with an upcoming relicensing effort. Given the aforementioned uncertainty with the dam stability analysis, the best course of action is likely to solve the stability issues under the existing license and move forward with the best future full pool elevation modifications during the relicensing process.

7.0 Conclusions, Recommendations, and Observations

Solomon Gulch historically spilled water during the summer and fall, and with the recent addition of summer through fall operation of the run of river Allison Creek project, spill is projected to continue. The purpose of this study was to evaluate the economic feasibility of pool raise alternatives that would capture spill in the seasonal Solomon Gulch Reservoir for later use as wintertime generation to offset existing diesel generation.

The methodology McMillen used to determine feasible alternatives includes:

1) Collect operational data from recent and past generation cycles and then model the water to wire system to determine the benefit of additional generation with incremental pool raise levels.
2) Investigate spillway modifications that would become a workable and cost-effective means of affecting pool raise levels. Evaluate modification alternatives as functionally and economically feasible or infeasible.
3) Review FERC dam safety documentation to evaluate dam, dike, and spillway probable failure modes with respect to possible spillway modification alternatives. Identify stability issues and perform dam, dike, and spillway stability analysis.
4) Write a summary report that identifies feasible alternatives and explains the processes used in determining feasible alternatives.

7.1 Conclusion

The energy production model showed the benefits of raising full pool by more than 15 ft only returned additional generation under the highest inflow conditions. This indicates there is no economic benefit to justify major modifications that a pool raise above 15 ft would require. The existing parapet walls have a top elevation of 695 ft; pool raise options above 10 ft would require the construction of 750 ft of new wall, 11 ft tall and capable of resisting 10 ft of hydraulic head. Using the 10 ft limit (elevation 695) and subtracting 2 ft of operational freeboard created a limit to pool raise levels of 8 ft.

7.1.1 Two-foot Raise by Adding Concrete to the Existing Spillway

The 2-ft raise alternative is the practical limit of a simple, ungated or uncontrolled spillway modification and is limited by freeboard during the PMF. We have used 0.5 ft of freeboard as a guide, as this has been accepted by FERC at other projects. Increasing the top elevation of the spillway by 2 ft adds 1,289 ac-ft (4.2 percent) of storage and provides an estimated annual benefit of 691 MWh. Economic analysis of this alternative showed a benefit-to-cost ratio above unity at diesel fuel costs as low as $1.50 per gallon (4% bond rate). This alternative would have the least difficulty gaining FERC approval unless the existing dam and dike are found to be unstable at the existing full pool elevation of 685 ft when future stability analyses are conducted. Construction of this alternative is the least challenging and presents no special circumstances unless a particularly wet spring and fall occur during construction. In this case some lost generation may occur if Solomon Gulch needs to ramp-up (Allison Creek ramps down) to maintain water levels for construction. The modified spillway can pass the PMF with a pool level of 694.4 ft.

7.1.2 Four-foot Raise by Installing a Rubber Dam Across the 450-foot Spillway

Vertical lift spillway gates would be an excellent choice to create additional storage if cost were not an issue. The long spillway length means a large number of gates (at least 20) would be required, with electrical service, motors, and hoist systems necessary to lift the gates. Additionally, structural steel to support the hoist system and access platforms would be required for each gate. Vertical gates were removed from consideration due to cost considerations and replaced with a 4 ft rubber dam. The diameter of the rubber dam is set by the PMF requirement. Building a platform in front of the existing ogee crest to support the rubber dam changes the profile of the crest to a broad-crested weir when the rubber dam is deflated. The shape has a lower discharge capability compared to the ogee weir and requires more head to pass the PMF. If the rubber dam were larger than 4 ft, then the PMF elevation would overtop the parapet walls. A rubber dam would have little problem gaining regulatory approval, as many are in service in the United States. Operationally, a rubber dam at Solomon Gulch is a good choice, as precise flow control is not an objective – the reservoir can range in elevation above the new full pool elevation of 689 ft as excess inflow can spill over the dam.

Economic analysis of the rubber dam alternative shows this to be the least feasible from a cost perspective of the three alternatives considered. The high construction cost relative to the generation benefit requires very high diesel fuel costs ($4.93/gal at 4% finance rate) to be beneficial.

7.1.3 Eight-foot Compound Spillway

The 8-ft compound spillway has a 175 ft operational spillway section using a rubber dam for water level control, and a 275 ft emergency section with a unique flashboard system. The flashboard system is required to maintain a high discharge coefficient over the emergency spillway section for flood control. The combination of the two sections allows for a storage increase of 8 ft for an annual generation benefit of 2,759 MWh while passing the PMF at an elevation of 693.6 ft. The economic analysis of Section 5 shows the 8 ft alternative provides a benefit if diesel fuel costs exceed $2.33/gal (4% finance rate) for the 30-year period following 2028. The flashboard system is considered feasible at this design stage but would require significant effort to gain FERC approval. A ¼-scale to ½-scale modeling effort would be required to verify the release mechanism. Additionally, the flashboard anchoring method and the spillway stability anchors would require FERC approval prior to proceeding with the 30% design. FERC approval of anchoring would, at a minimum, require installation of pre-construction test anchors in rock conditions very close to the spillway to verify the grout or epoxy specification; this test would also verify bond length dimensioning. During installation, anchors would be pull-tested and after construction, the design of the anchors would allow for inspection and pull testing.

The compound spillway uses flashboards to reduce project cost but adds risk of future replacement of the flashboards if a very large flood event occurred. This risk can be mitigated (but with added cost) by installing flap gates in one or two panel bays. The flap gates would release during very high water events and would be reset after high water receded. This would prevent the release of the entire emergency spillway except for exceedingly high inflow events. The design of the flap gates and the determination of the flow to actuate the gates would occur during 30% design efforts.

As with all the proposed alternatives, the dam and dike must demonstrate stability under the new full pool level. The 8 ft alternative is more likely to be a stability issue simply because the higher pool level loads the dam and dike more, and the higher pool level causes over-topping of the spillway or parapet walls with a smaller seismically caused vertical offset. New seismicity data generated during the latter part of 2019 and subsequent stability analyses will determine what level of pool raise can occur.

7.2 Recommendations and Observations

1) Our analysis of pool raise value is based on an assessment of inflows and loads. Major changes to these variables would change our perspective on the need for additional storage. The most significant changes from our expectations could occur with the load forecast. Summer loads are dependent on fish processing and tourism, while winter loads are dependent on winter heating and lighting demand. Superimposed on top of these loads are exchanges and loads related to Alyeska operations. Major changes to load, the expansion of fish processing, Alyeska demand, or the potential of a transmission interconnection where impending spill at Solomon Gulch could be exported by increased generation affect how we view a storage expansion at Solomon Gulch.
2) The power exchange referenced above with Alyeska's Valdez Marine Terminal was under negotiation during the completion of this study. Excess vapors from storing oil at the terminal fuel Alyeska generation and are a function of day-time temperatures and direct sunlight. The power exchange would allow Alyeska to convert the vapor to electricity rather than waste it through a vent burner. CVEA night-time generation would return the Marine Terminal export. Imports to and exports from the CVEA system associated with the power exchange may result in additional load following requirements at Solomon Gulch that would in turn benefit from additional storage and also impact existing storage. As more information becomes available regarding the exchange, the impacts to CVEA's Solomon Gulch plant could be quantified with the power model developed as part of this study.
3) The single most important measured value from CVEA records is the Solomon Gulch Reservoir water level. This value determines spilled energy and allows McMillen Jacobs to calculate inflows. It also will be extremely important in the future as a control input if a pool raise is considered. A new emphasis on the importance of the real-time measure being accurate in SCADA should be conveyed to the operating crew.
4) The pool raise permitting effort, if considered, should be folded into the relicense process because this costs much less than a relicense effort and a license amendment done at separate times. Also, if a pool raise is considered and approved, it would add an improvement credit to the project, which carries value in the relicense process.
5) We did not consult the Alaska DCCED or federal agencies regarding climate change and economic growth forecasts and the subsequent impact on inflows and loads. As these

departments increase this focus and the science continues to mature, this would be one area for future investigation regarding future Solomon Gulch inflows and CVEA load growth.

6) The emergency spillway used a mechanism new to the hydroelectric power industry, the frangible nut. McMillen Jacobs suggested this feature as it provides a means for flashboard release that triggers the structure by manual control, and it triggers the entire flashboard system composed of large structural steel members, simultaneously ensuring that the spillway clears. Getting new elements to gain FERC acceptance is not an easy task and requires additional effort. This review process should proceed if, during the 30% design, frangible nuts are retained as the preferred method over other methods, such as a compressed-air-driven release mechanism.

7) The flashboard system has been carried forward as feasible because at this point in the study process, it offers the only alternative to pool raise efforts above 5 ft in a cost-effective manner relative to diesel fuel costs between $2 and $3 per gallon. Economic conditions change and fuel prices can rise, but the other consideration is that pool raise alternatives return relatively small benefits in terms of additional generation.

8) The next step in pool raise evaluation is the determination of dam and dike stability if the normal full pool is raised. CVEA was in the process of updating seismic hazard evaluation reports as this study was underway. New dam, dike, and spillway stability analysis are scheduled for completion by December 2021. Future stability analysis should include a determination of the maximum pool level the dam and dike can withstand at the required FERC factor of safety.

8.0 References

CVEA. 2018. Supporting Technical Information Document Revision 5, Solomon Gulch Hydroelectric Project (FERC No. 2742-AK). Copper Valley Electric Association.

HDR Engineering Inc., 1991. Solomon Gulch Hydroelectric Project Reservoir Capacity Increase Feasibility Study, prepared for Copper Valley Electric Association. November 1991. Valdez, Alaska.

McMillen Jacobs. 2019a. Technical Memo 01: Evaluation of Solomon Gulch Spillway Capacity. McMillen Jacobs Associates. Boise, Idaho.

_____. 2019b. Technical Memo 02: Hydrologic and Power Production Analysis of Solomon Gulch. McMillen Jacobs Associates. Boise, Idaho.

Tracy, H.J., 1957. Geological Survey Circular 397: Discharge Characteristics of Broad-Crested Weirs. U.S. Geological Survey (USGS). Washington, D.C.

USBR. 1960. Design of Small Dams. A Water Resources Technical Publication. U.S. Bureau of Reclamation.

Cornforth Consultants, 2019. Update to Previously Completed Seismic Hazard Review and Analysis

Appendix A
Select STI Spillway and Dam Drawings

Appendix B
Supporting Calculations

Appendix C
Manufacturer's Data

Appendix D
Cost Estimate and Economic Analysis

McMILLEN JACOBS ASSOCIATES	Job No.	No. 0
Project Solomon Gulch	Computed P. Rader	Date 3/17/2020
Subject Pool Raise Feasibility Study	Checked	Date
Task Cost Estimate	Sheet Cover Sheet	

PURPOSE:

Provide a conceptual engineer's cost estimate to compare the options for the Solomon Gulch Pool Raise Feasibility Study. The conceptual estimate is similar to a Class 5 estimate as defined by the Association for the Advancement of Cost Engineering (AACE).

AACE International CLASS 5 Cost Estimate - Class 5 estimates are generally prepared based on very limited information, and subsequently have wide accuracy ranges. Typically, engineering is 0% to 2% complete. They are typically prepared for any number of strategic business planning purposes, such as but not limited to market studies, assessment of initial viability, evaluation of alternate schemes, project screening, project location studies, evaluation of resource needs and budgeting, long-range capital planning, etc. Virtually all Class 5 estimates use stochastic estimating methods such as cost/capacity curves and factors, scale of operations factors, Lang factors, Hand factors, Chilton factors, Peters-Timmerhaus factors, Guthrie factors, and other parametric and modeling techniques. Expected accuracy ranges are from -20% to -50% on the low side and +30% to 100% on the high side, depending on the technological complexity of the project, appropriate reference information, and the inclusion of an appropriate contingency determination. Ranges could exceed those shown in unusual circumstances. As little as 1 hour or less to perhaps more than 200 hours may be spent preparing the estimate depending on the project and estimating methodology (AACE International Recommended Practices and Standards).

ASSUMPTIONS:

Quantity estimation is based on the information available about the existing structure and from the concepts outlined in the feasibility study technical memo. Unit costs were researched and applied to the material quantities to provide an estimate for the cost of the project. Material costs were obtained from past similar projects as well as past engineering and construction experience building similar projects.

The following items were assumed when preparing the engineer's cost estimate:

1) All costs presented in this cost estimate are in US Dollars
2) No hazardous materials are expected to be encountered on site.
3) Pricing includes applicable overhead, profit and bond.
4) The road to the site will support delivery trucks, loaded concrete trucks, and heavy equipment.
5) There is sufficient laydown area available on site.
6) The reservoir levels will be low enough to work on the upstream side of the dam. No dewatering is included in the estimate.
7) Ready Mix Concrete is readily available and will be sourced from Valdez, AK.
8) Contractor will be allowed to dispose of excavation spoils onsite.
9) A 20% contingency is included in the Project Grand Totals.

OPTION 1 SUMMARY

2' Concrete Dam Raise

Item		Price
General Requirements/Mobilization/Demobilization	$	152,230.56
Site Prep and Access Roads	$	38,200.00
Dam Raise Concrete (2' Raise)	$	570,722.22
Contingency (20%)	$	152,230.56
PROJECT GRAND TOTAL	**$**	**913,383.33**
Accuracy Range +100%	$	1,826,766.67
Accuracy Range -50%	$	456,691.67

OPTION 2 SUMMARY

Install rubber dam, picket system and rock anchors along the upstream dam face.

Item		Price
General Requirements/Mobilization/Demobilization	$	794,172.22
Site Prep and Access Roads	$	38,200.00
Uplift Prevention Measures (Cutoff Wall/Rock Anchors)	$	172,527.51
Rubber Bladder Dam (175' Length)	$	1,992,061.35
Picket System (40 Picket Panels)	$	973,900.00
Contingency (20%)	$	794,172.22
PROJECT GRAND TOTAL	**$**	**4,765,033.30**
Accuracy Range +100%	$	9,530,066.60
Accuracy Range -50%	$	2,382,516.65

OPTION 3 SUMMARY

Concrete cutoff wall along the upstream face, Install rubber dam, picket system and rock anchors along the upstream Dam Face.

Item		Price
General Requirements/Mobilization/Demobilization	$	803,419.92
Site Prep and Access Roads	$	38,200.00
Uplift Prevention Measures (Cutoff Wall/Rock Anchors)	$	209,518.34
Rubber Bladder Dam (175' Length)	$	1,992,061.35
Picket System (40 Picket Panels)	$	973,900.00
Contingency (20%)	$	803,419.92
PROJECT GRAND TOTAL	**$**	**4,820,519.54**
Accuracy Range +100%	$	9,641,039.09
Accuracy Range -50%	$	2,410,259.77

Solomon Gulch Dam Raise
Engineer's Estimate

Option 1	Option 2	Option 3	Item	Quantity	Unit	Unit Price	Option 1 Price	Option 1 Total	Option 2 Price	Option 2 Total	Option 3 Price	Option 3 Total
			General Requirements/Mobilization/Demobilization					**$ 152,230.56**		**$ 794,172.22**		**$ 803,419.92**
x	x	x	General Requirements	15%	%		$ 91,338.33		$ 476,503.33		$ 482,051.95	
x	x	x	Mobilization / Demobilization	10%	%		$ 60,892.22		$ 317,668.89		$ 321,367.97	
			Site Prep and Access Roads					**$ 38,200.00**		**$ 38,200.00**		**$ 38,200.00**
x	x	x	Clear and Grub Laydown Area	5500.00	SY	$ 1.50	$ 8,250.00		$ 8,250.00		$ 8,250.00	
x	x	x	Import Pit Run Fill for Access Road	350.00	TN	$ 50.00	$ 17,500.00		$ 17,500.00		$ 17,500.00	
x	x	x	Import CSBC for Laydown Area and Access Road	50.00	TN	$ 75.00	$ 3,750.00		$ 3,750.00		$ 3,750.00	
x	x	x	Grading Access on Upstream Side of Dam	1000.00	SY	$ 1.50	$ 1,500.00		$ 1,500.00		$ 1,500.00	
x	x	x	Place Imported Fill for Access Rd (over Dam)	360.00	CY	$ 20.00	$ 7,200.00		$ 7,200.00		$ 7,200.00	
			Uplift Prevention Measures (Cutoff Wall/Rock Anchors)					**$ -**		**$ 172,527.51**		**$ 209,518.34**
		x	Rock Excavation	66.67	CY	$ 100.00					$ 6,666.67	
		x	Drill and Dowel Rebar	450.00	EA	$ 10.00					$ 4,500.00	
		x	Form/Rebar/Pour Concrete Seal Strip	66.67	CY	$ 1,250.00					$ 83,333.33	
			Mobilize Drill Rig (Drains)	1.00	LS	$ 15,000.00						
			Drill Drains Through Dam	10000.00	LF	$ 15.00						
	x		Upstream Side - Drill Rock Anchors (No Concrete Seal)	612.00	LF	$ 183.87			$ 112,527.51			
		x	Upstream Side - Drill Rock Anchors (Use Concrete Seal)	408.00	LF	$ 183.87					$ 75,018.34	
	x		Procure & Install Rock Anchor Attachment Plates (No Concrete Seal)	60.00	EA	$ 1,000.00			$ 60,000.00			
		x	Procure & Install Rock Anchor Attachment Plates (USE Concrete Seal)	40.00	EA	$ 1,000.00					$ 40,000.00	
			Dam Raise Concrete (2' Raise)					**$ 570,722.22**		**$ -**		**$ -**
x			Drill and Dowel Rebar	6000.00	EA	$ 15.00	$ 90,000.00					
x			Form/Rebar/Pour Concrete	281.48	CY	$ 1,500.00	$ 422,222.22					
x			Finish Concrete Slope	5850.00	SF	$ 10.00	$ 58,500.00					
			Rubber Bladder Dam (175' Length)					**$ -**		**$ 1,992,061.35**		**$ 1,992,061.35**
	x	x	Rock Excavation for Fdn.	23.70	CY	$60.00			$ 1,422.22		$ 1,422.22	
	x	x	Procure and Install Support Columns	3.85	TN	$10,000.00			$ 38,500.00		$ 38,500.00	
	x	x	Form/Rebar/Pour Concrete Elevated Slab & Fdn.	120.93	CY	$1,500.00			$ 181,388.89		$ 181,388.89	
	x	x	Form/Rebar/Pour Concrete Dam Slope, DS	86.53	CY	$1,500.00			$ 129,791.67		$ 129,791.67	
	x	x	Finish Concrete Slope	1300.00	SF	$5.00			$ 6,500.00		$ 6,500.00	
	x	x	Procure Rubber Bladder Dam	175.00	LF	$3,899.75			$ 682,456.37		$ 682,456.37	
	x	x	Procure Mechanical and Electrical Equipment	175.00	LF	$2,150.52			$ 376,341.29		$ 376,341.29	
	x	x	Mechanical Building	200.00	SF	$200.00			$ 40,000.00		$ 40,000.00	
	x	x	Install Rubber Bladder Dam	175.00	LF	$2,141.38			$ 374,741.38		$ 374,741.38	
	x	x	Install Mechanical and Electrical for Rubber Dam	175.00	LF	$919.54			$ 160,919.54		$ 160,919.54	
			Picket System (40 Picket Panels)					**$ -**		**$ 973,900.00**		**$ 973,900.00**
	x	x	Fabricate Picket System	60.00	TN	$ 7,000.00			$ 420,000.00		$ 420,000.00	
	x	x	Procure Picket Attachment System (Frangible nuts and Anchors)	40.00	EA	$ 5,000.00			$ 200,000.00		$ 200,000.00	
	x	x	Rock Excavation	11.85	CY	$ 100.00			$ 1,185.19		$ 1,185.19	
	x	x	Form/Rebar/Pour Foundations	11.85	CY	$ 1,250.00			$ 14,814.81		$ 14,814.81	
	x	x	Drill and Install Anchoring System	40.00	EA	$ 1,500.00			$ 60,000.00		$ 60,000.00	
	x	x	Install Picket System	40.00	EA	$ 2,500.00			$ 100,000.00		$ 100,000.00	
	x	x	Install Electrical/Control Wiring	1.00	LS	$ 50,000.00			$ 50,000.00		$ 50,000.00	
	x	x	Grout Base of Dam	60.00	CY	$ 1,000.00			$ 60,000.00		$ 60,000.00	

Solomon Gulch Dam Raise
Engineer's Estimate

Option 1	Option 2	Option 3	Item	Quantity	Unit	Unit Price	Option 1 Price	Option 1 Total	Option 2 Price	Option 2 Total	Option 3 Price	Option 3 Total
	x	x	Procure Bubbler System (Quote + Freight Allowance $5000)	1.00	LS	$ 32,900.00			$ 32,900.00		$ 32,900.00	
	x	x	Install Bubbler System (Onsite crews +mfr supervision)	1.00	LS	$ 35,000.00			$ 35,000.00		$ 35,000.00	
								$ -		$ -		$ -
			Project Subtotal (Direct Costs Only) =					$ 608,922.22		$ 3,176,688.87		$ 3,213,679.70
			Project Subtotal (Direct and Indirect) =					$ 761,152.78		$ 3,970,861.08		$ 4,017,099.62
			Contingency (20%)					$ 152,230.56		$ 794,172.22		$ 803,419.92
			Grand Total =					$ 913,383.33		$ 4,765,033.30		$ 4,820,519.54
			Accuracy Range									
			+100%					$ 1,826,766.67		$ 9,530,066.60		$ 9,641,039.09
			-50%					$ 456,691.67		$ 2,382,516.65		$ 2,410,259.77

Appendix A

A1-Dam and Dike Plan View

A2- Dam and Dike Sections

A3- Pool Raise Alternatives

A4- Flashboard System

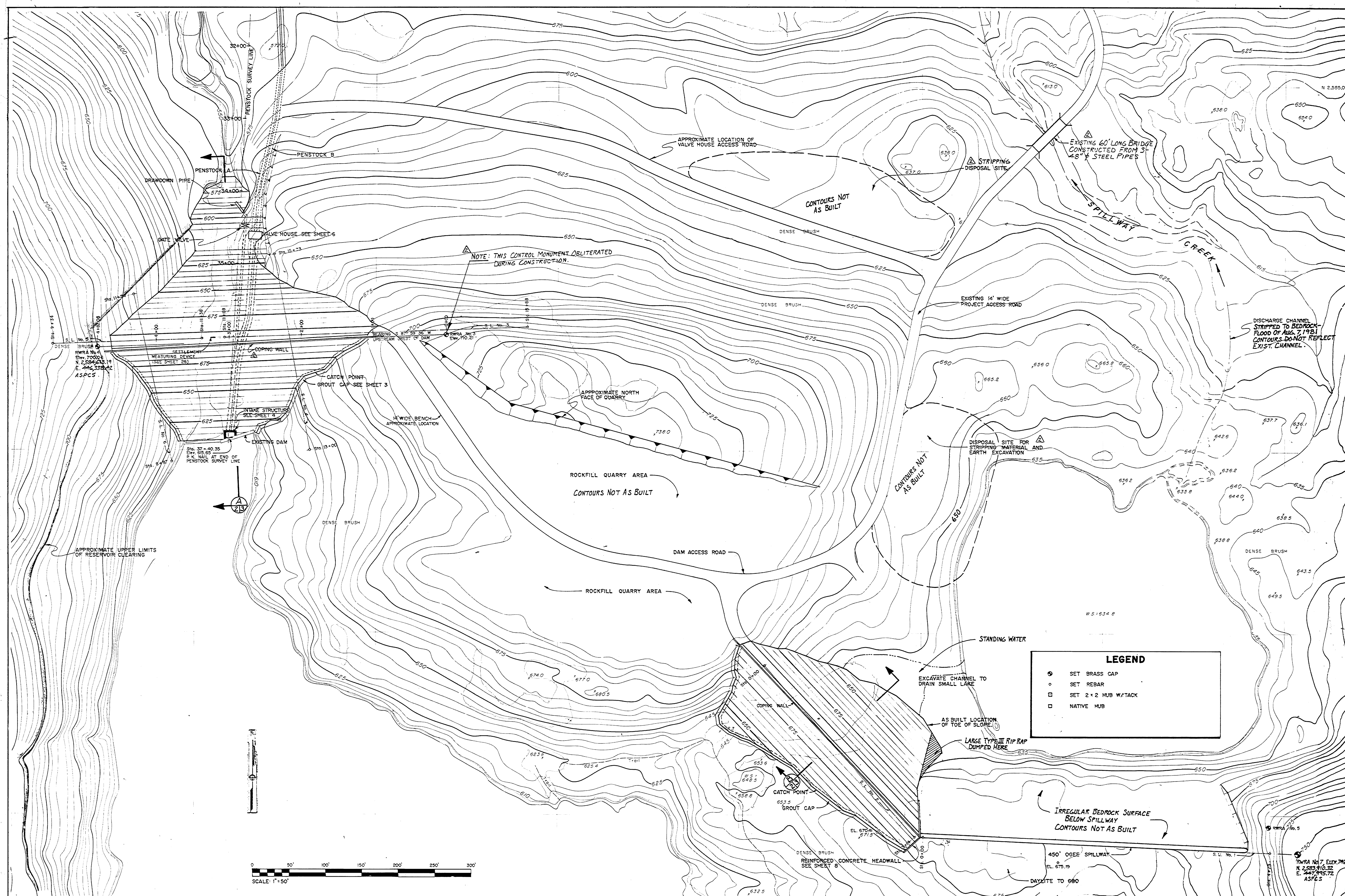
A5- Flashboard Panel Section

A6-Flashboard Panel Detail

A7-Flashboard Anchoring

A8-Flashboard Frangible Nut

A9-Flashboard Frangible Nut Details



ROBERT W. RETHERFORD ASSOCIATES
ANCHORAGE, ALASKA

DESIGN BY: M.D.H.
DRAWN BY: A.W.S.
CHECKED BY: M.D.H.
SUPERVISED BY: C.H.S.

APPROVAL RECOMMENDED BY:
ENGINEER

SUBMITTED BY:
PROJECT ENGINEER
DATE: 4/10/79

NO	DATE	REVISION	BY
1	3/5/80	AS BUILT TOE OF SLOPE, ADD DISCH. CHNL.	MDH
2	4/1/82	As Noted	
3	9/20/83	As Built - Rev.	MDH

COPPER VALLEY ELECTRIC ASSOCIATION INC.
GLENNALLEN, ALASKA

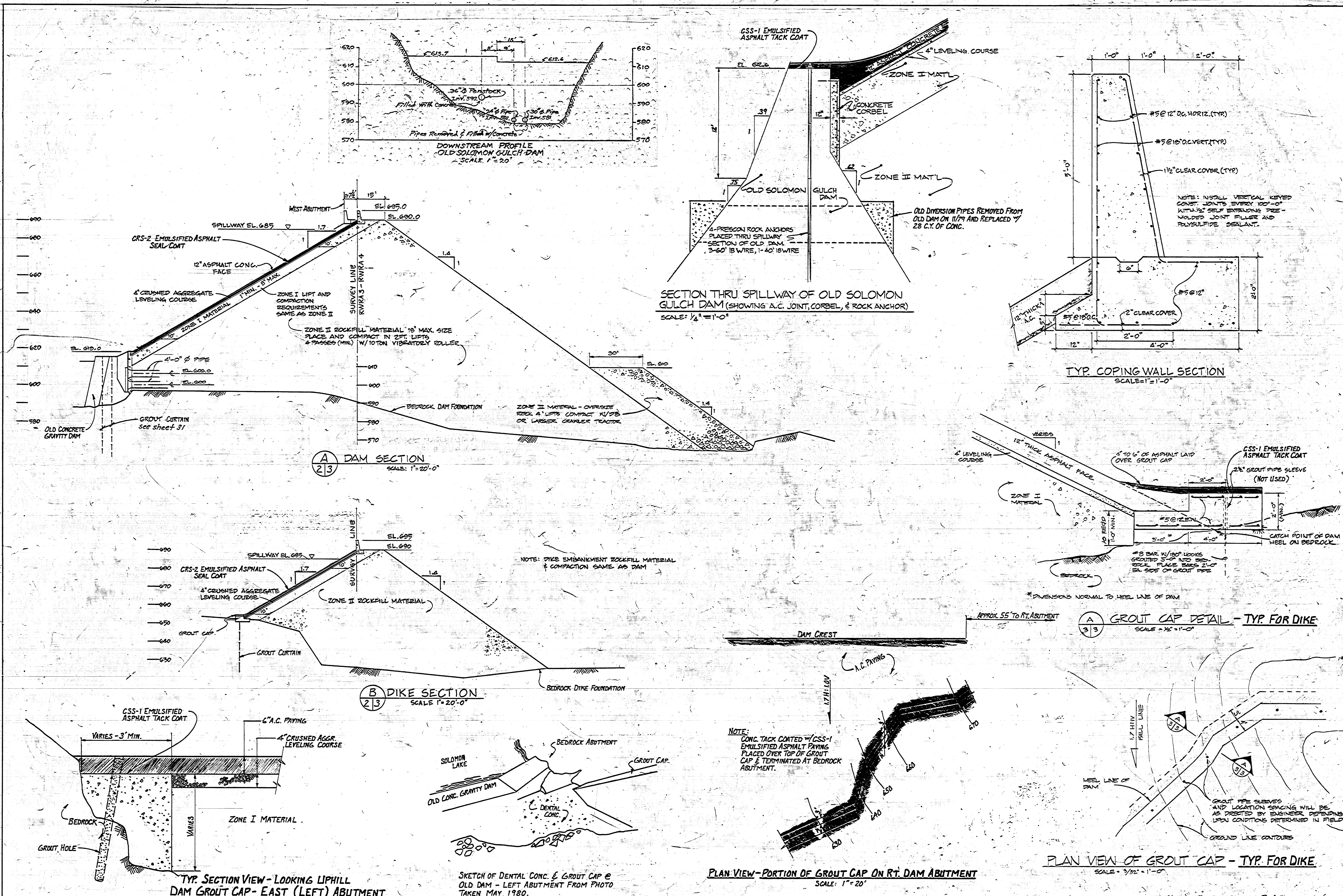
RURAL ELECTRIFICATION ADMINISTRATION
PROJECT: ALASKA 18 U9 COPPER VALLEY

SOLOMON GULCH PROJECT
F.E.R.C. LICENSE NO. 2742

PRODUCTION PLANT
PLAN VIEW
DAM, DIKES, & SPILLWAY

ROBERT W. RETHERFORD ASSOC.
DRAWING FILE: 118-820-2/32
SHEET 2 OF 32 SHEETS

11-1002
H01-F-11-1002-R49



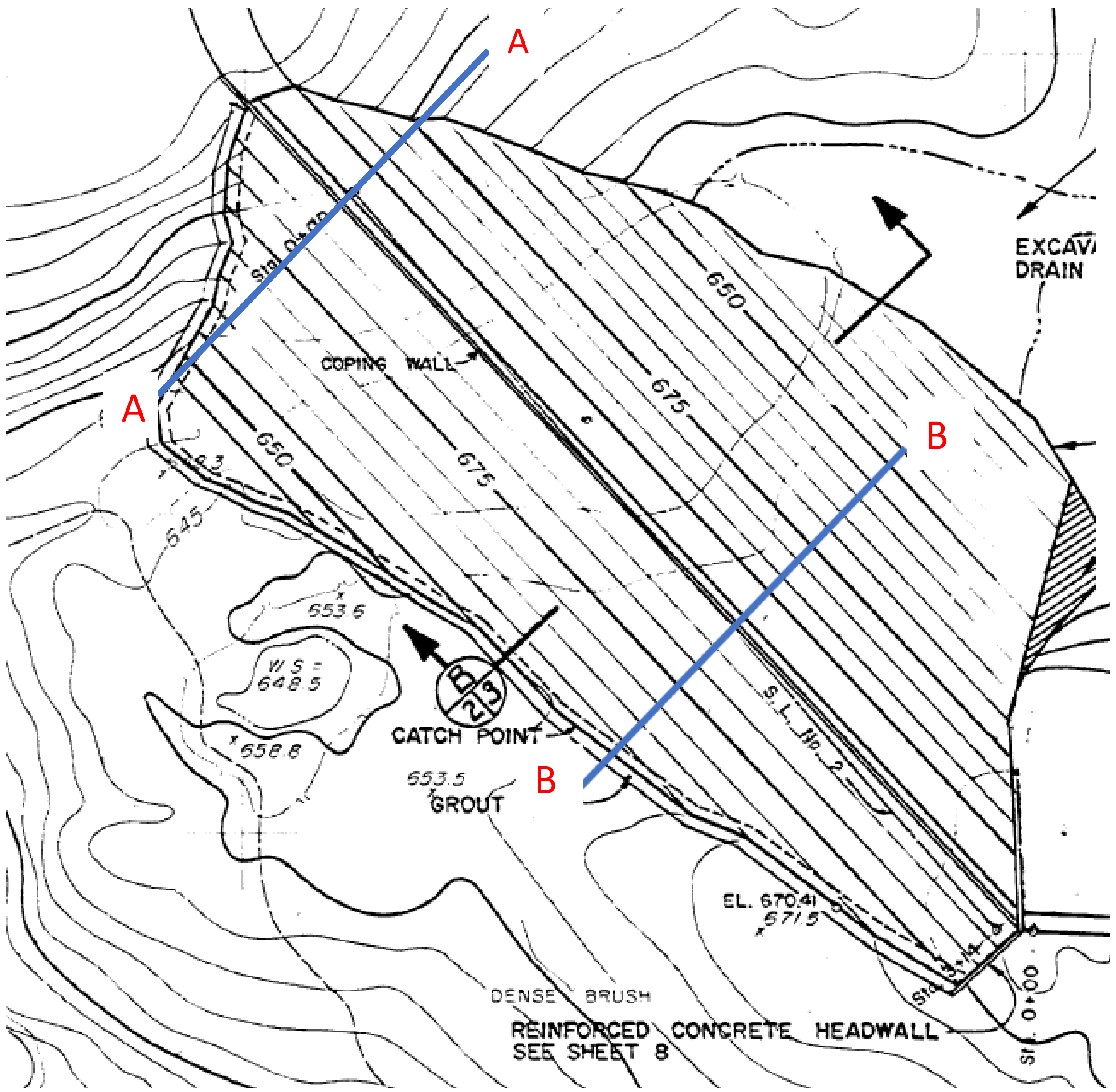
H01-F-11-1003-R49

ROBERT W. RETHERFORD ASSOCIATES, ANCHORAGE, ALASKA	APPROVAL RECOMMENDED BY: ENGINEER	SUBMITTED BY: PROJECT ENGINEER DATE: 4/10/79		COPPER VALLEY ELECTRIC ASSOCIATION INC. GLENNALLEN, ALASKA	SOLOMON GULCH PROJECT F.E.R.C. LICENSE NO. 2742
DESIGN BY: M.D.H.					PRODUCTION PLANT
DRAWN BY: R.N.W.					TYPICAL DAM & DIKE SECTIONS GROUT CAP & COPING WALL
CHECKED BY: M.D.H.				RURAL ELECTRIFICATION ADMINISTRATION PROJECT: ALASKA 18 U9 COPPER VALLEY	ROBERT W. RETHERFORD ASSOC. DRAWING FILE: 118-820-3/32
SUPERVISED BY: C.H.S.					SHEET 3 OF SHEETS 32

NO	DATE	REVISION	BY
1	12-15-80	AS BUILT	MDH
2	9/20/83	AS BUILT - REV.	MDH
3	6/6/85	ADDED PROFILE OLD S.G. DAM	CHS

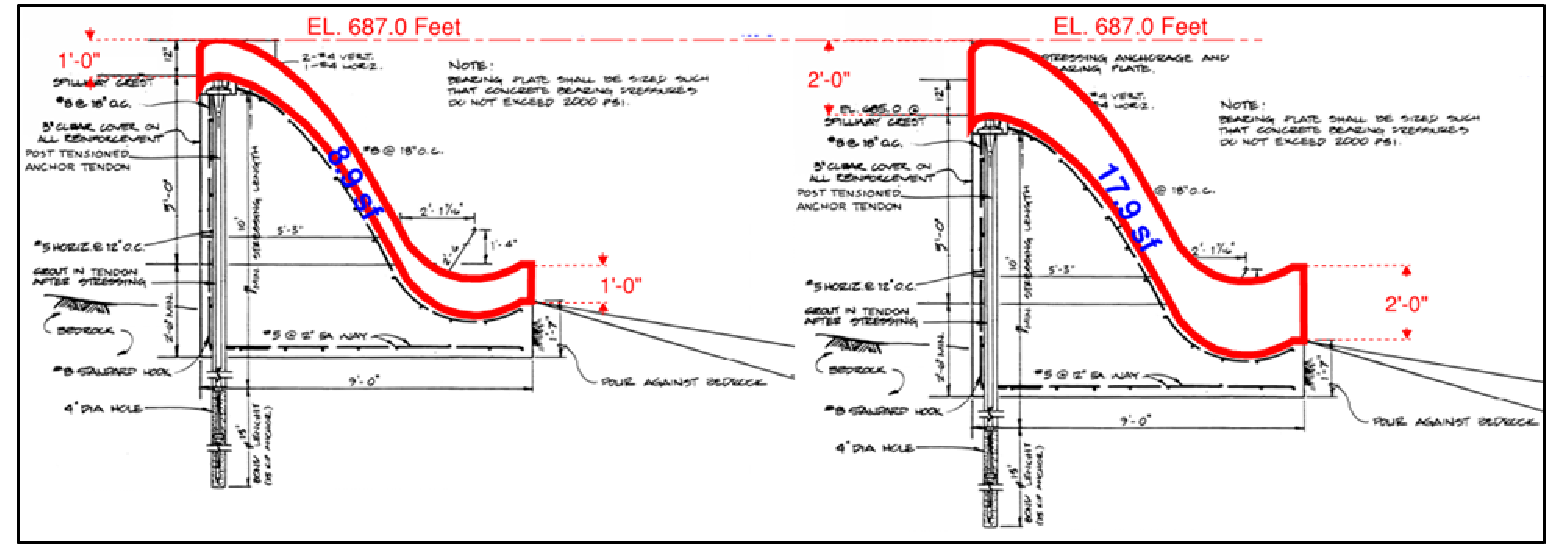
11-1003

Plan and Section Views of Pool Raise Alternatives

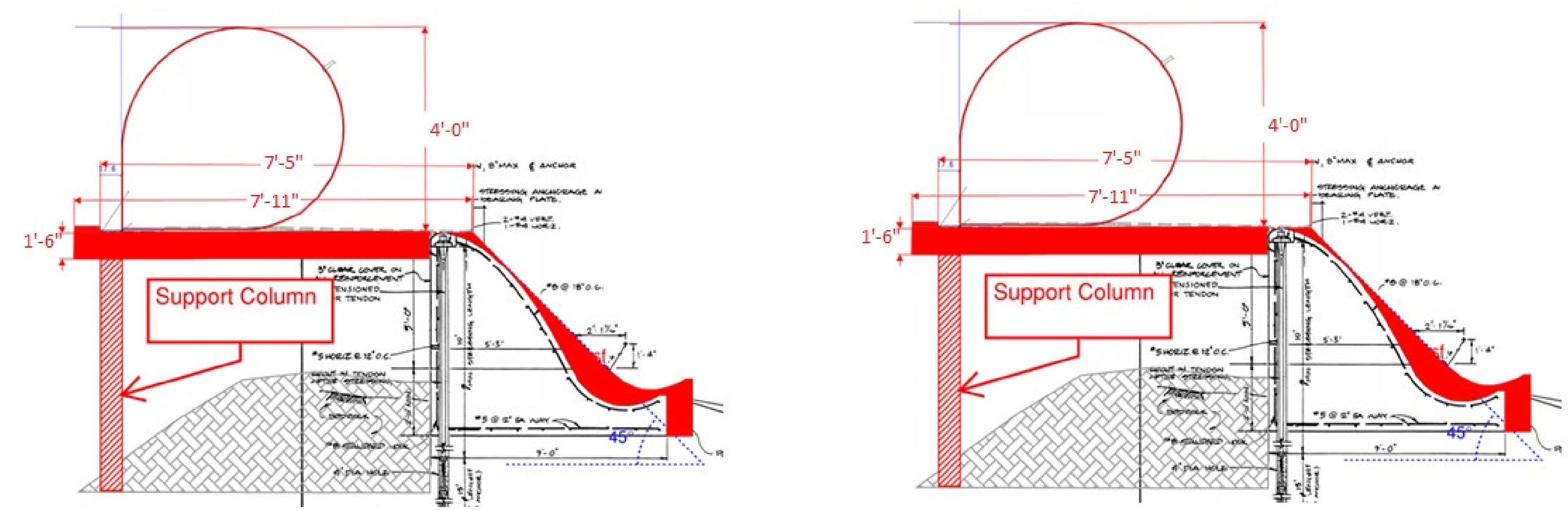


Plan View of Existing Spillway

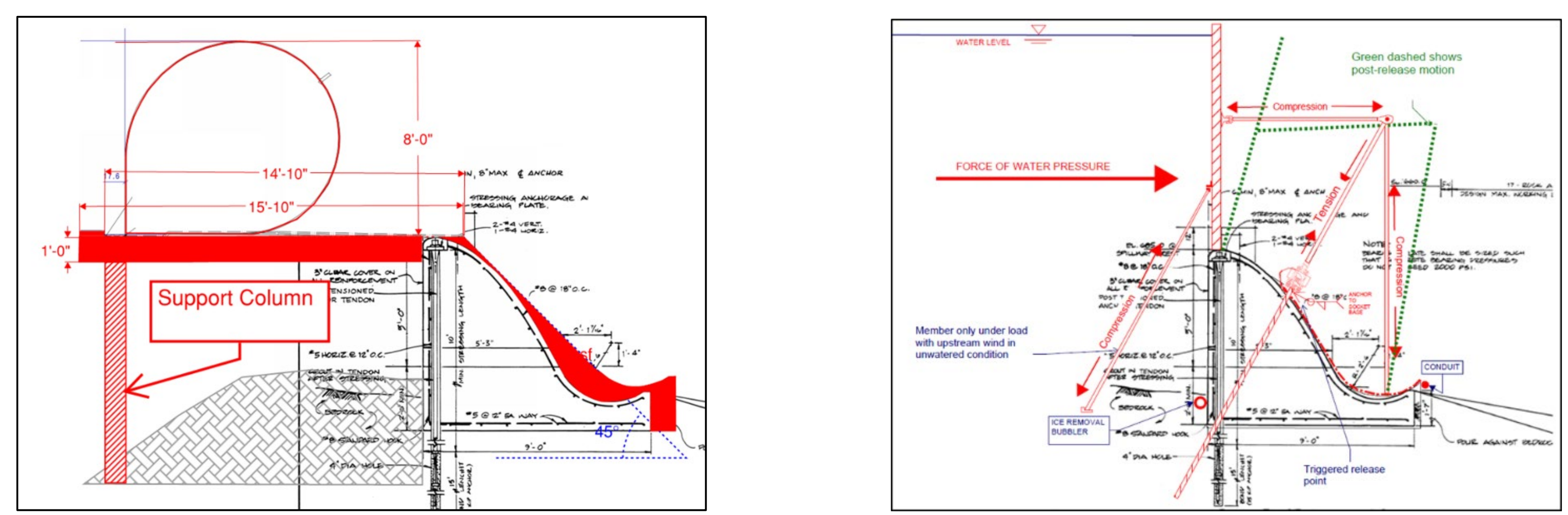
Section A **2 ft Alternative** **Section B**

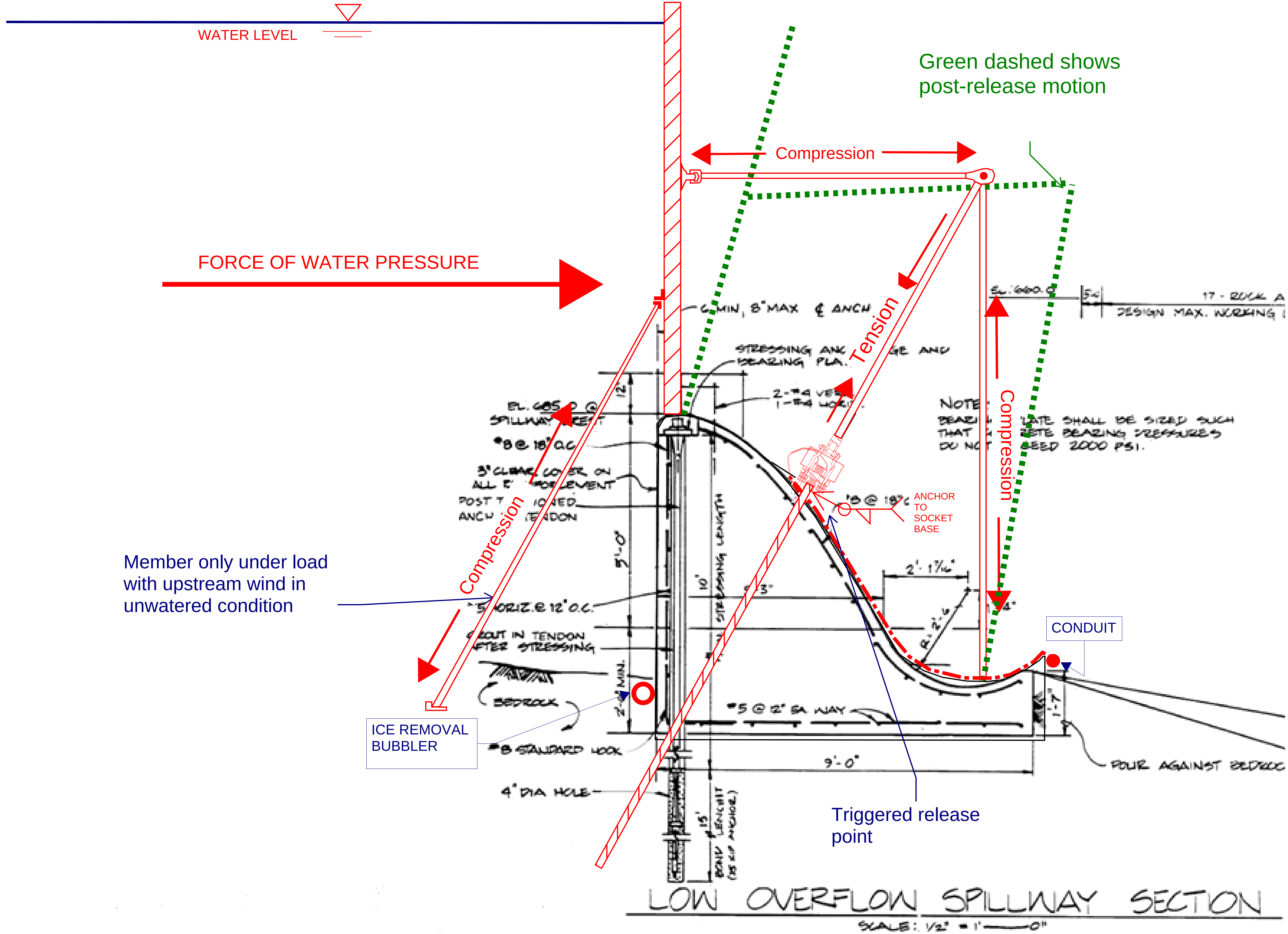


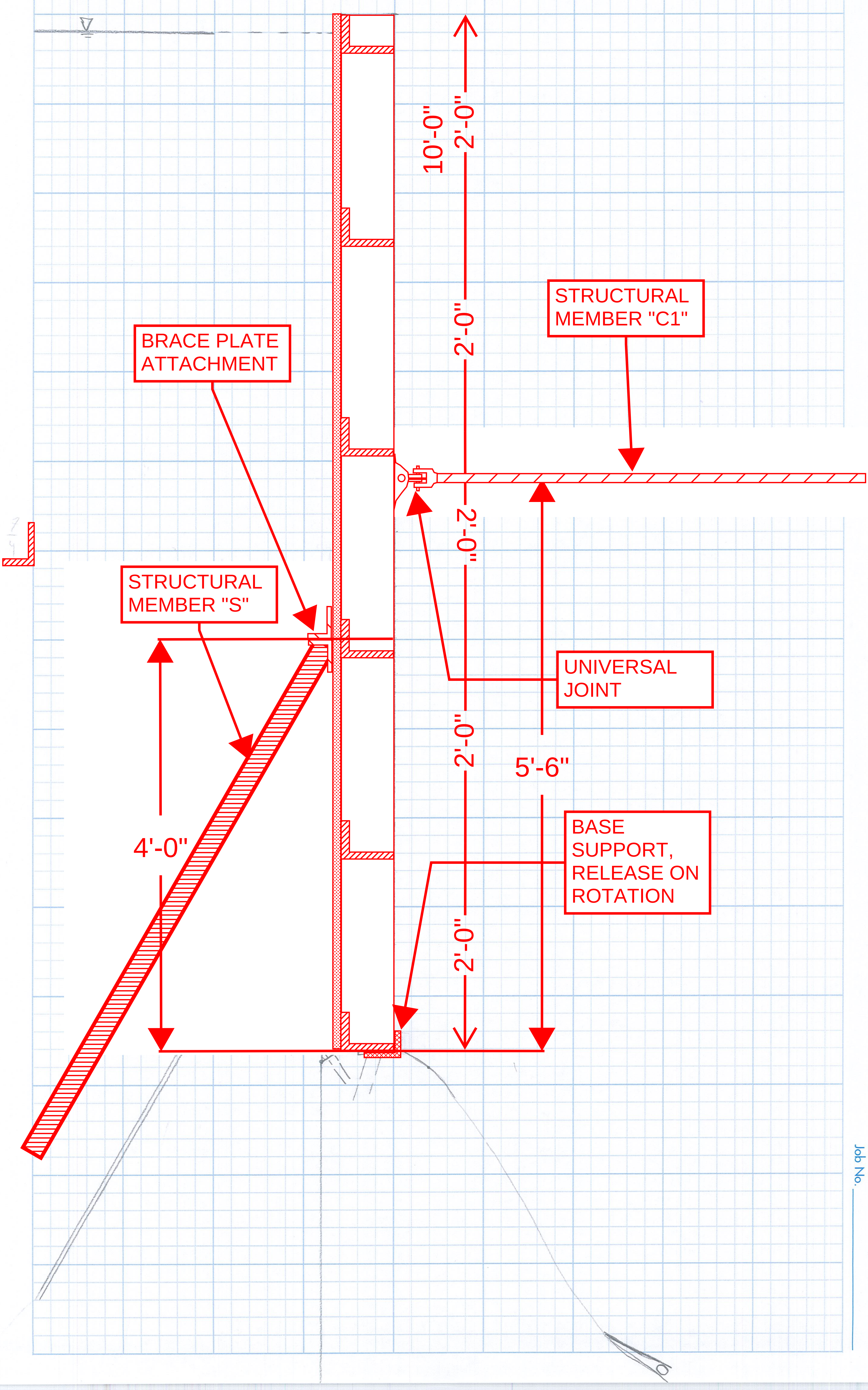
Section A **4 ft Alternative** **Section B**



Section A **8 ft Compound Alternative** **Section B**



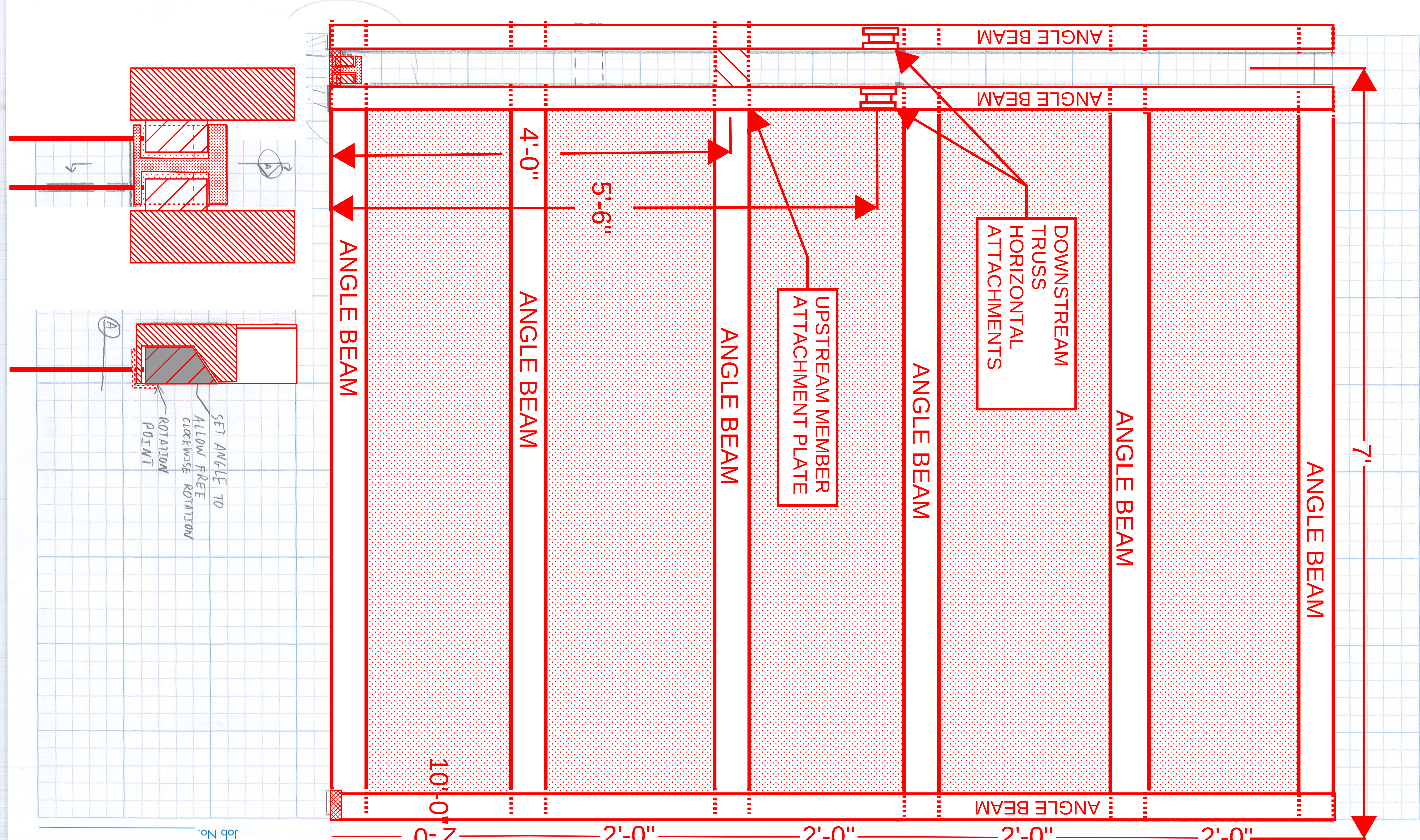

WATER LEVEL
Green dashed shows post-release motion
Compression
FORCE OF WATER PRESSURE
Tension
Compression
Compression
ANCHOR TO SOCKET BASE
Member only under load with upstream wind in unwatered condition
CONDUIT
ICE REMOVAL BUBBLER
Triggered release point
NOTE: BEARING PLATE SHALL BE SIZED SUCH THAT CONCRETE BEARING PRESSURES DO NOT EXCEED 2000 PSI.
EL. 685.0 @ SPILLWAY CREST
EL. 660.0
#8 @ 18" O.C.
3" CLEAR COVER ON ALL REINFORCEMENT
POST TENSIONED ANCHOR TENDON
6" MIN, 8" MAX ℄ ANCH
STRESSING ANCHORAGE AND BEARING PLA.
2-#4 VERT, 1-#4 HORIZ.
STRESSING LENGTH
GROUT IN TENDON AFTER STRESSING
BEDROCK
#8 STANDARD HOOK
#5 @ 12" EA WAY
4" DIA HOLE
9'-0"
5'-0"
2'-1 7/16"
R. 2'-6
POUR AGAINST BEDROCK
LOW OVERFLOW SPILLWAY SECTION
SCALE: 1/2" = 1'-0"



MCMILLEN
DESIGN with Vision. BUILD with Integrity

Project
Description
Prepared By

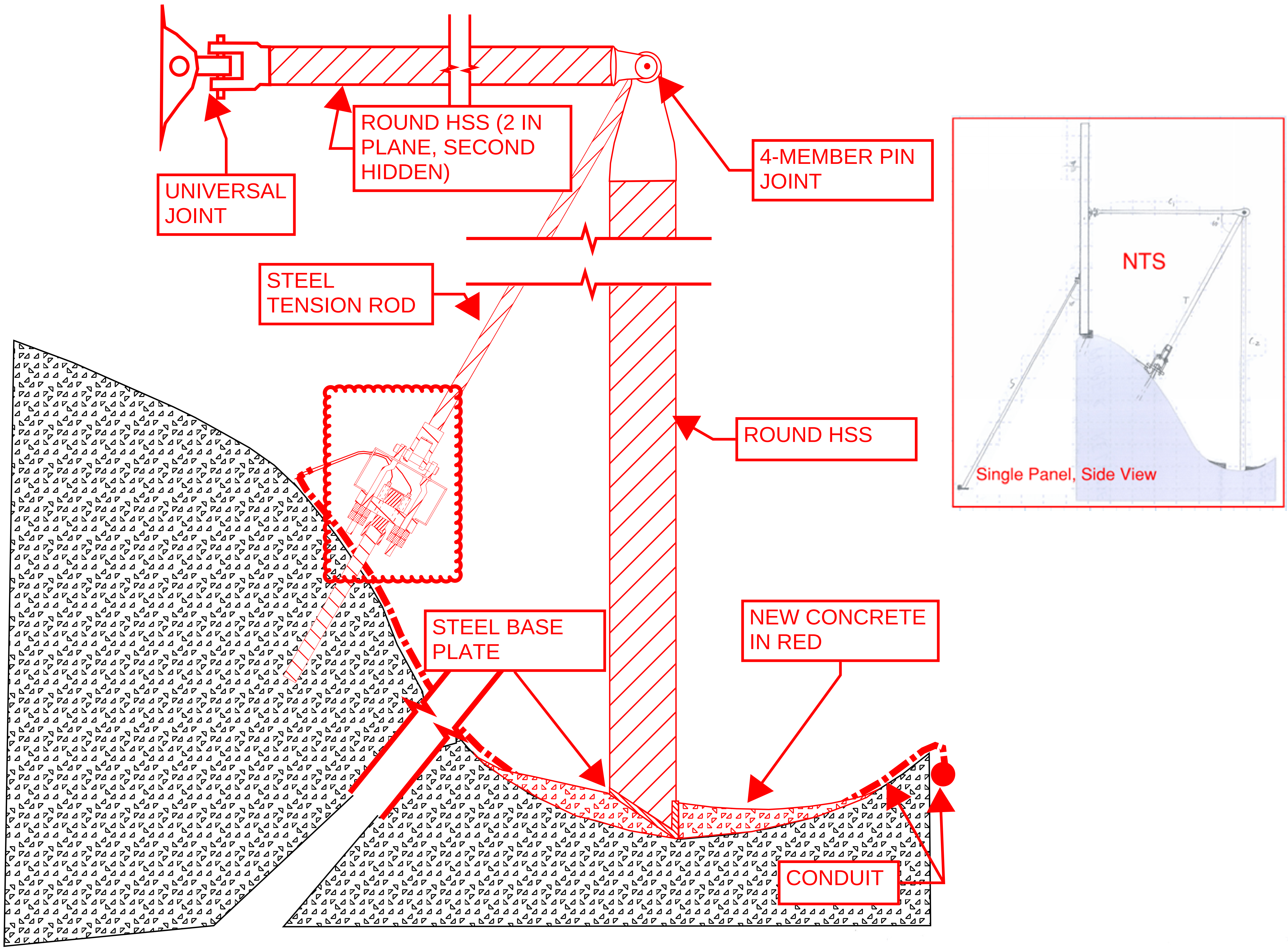
Page of
Date 10/31
Checked
Job No.



MCMILLEN
DESIGN with Vision. BUILD with Integrity.

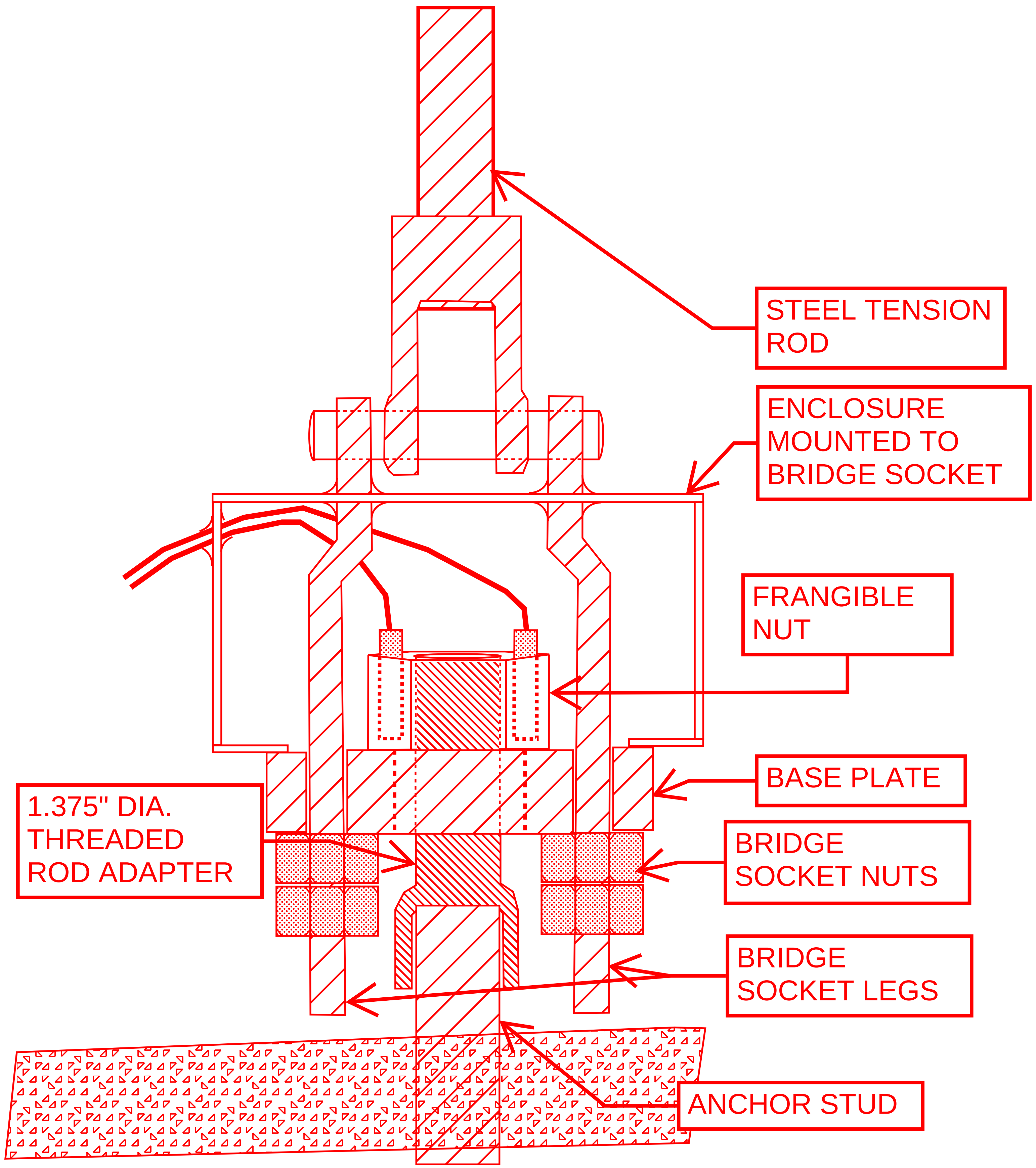
Project
Description
Prepared By

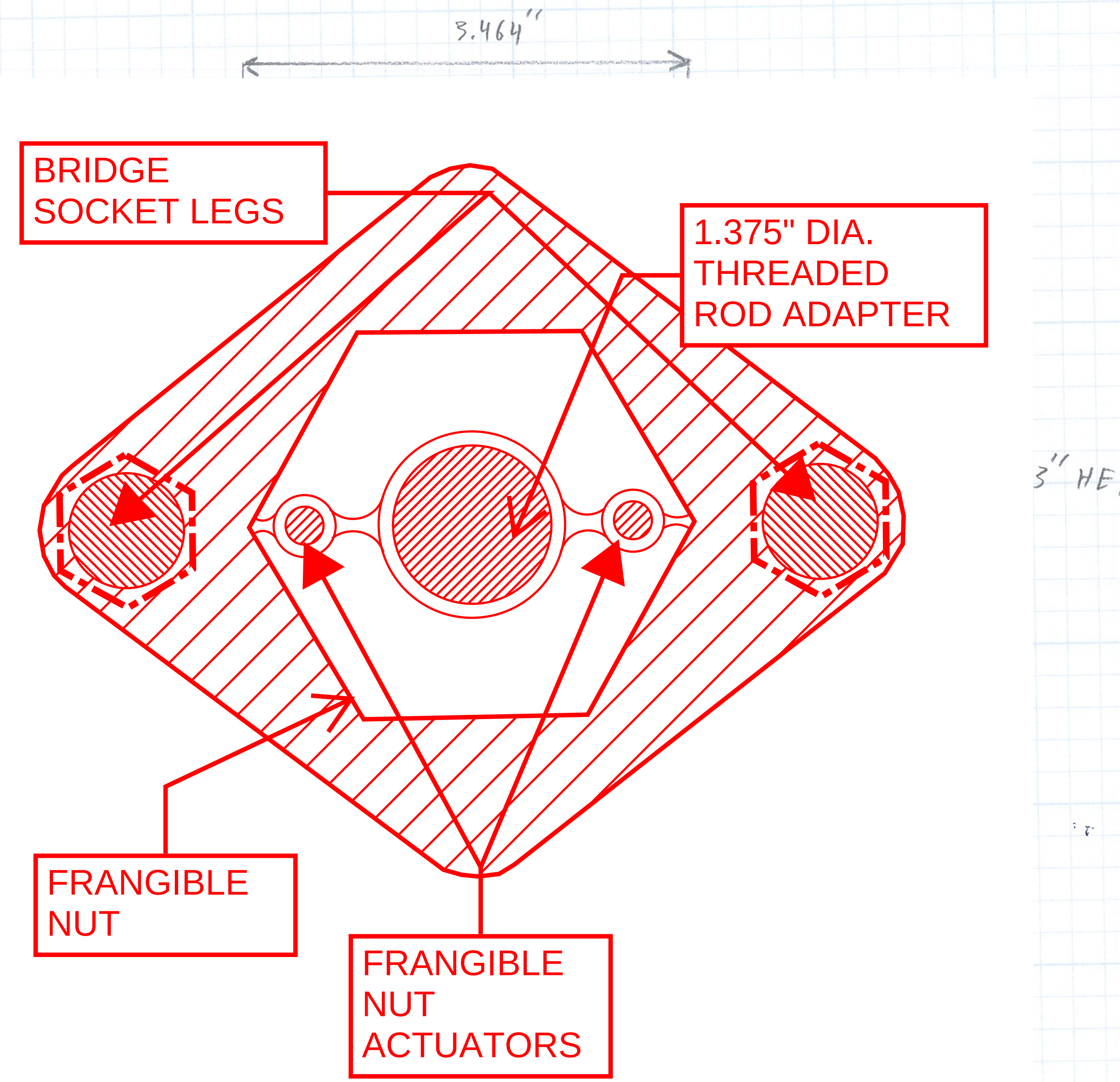
Job No.
Checked
Date 10/31
Page of


UNIVERSAL JOINT
ROUND HSS (2 IN PLANE, SECOND HIDDEN)
4-MEMBER PIN JOINT
STEEL TENSION ROD
ROUND HSS
STEEL BASE PLATE
NEW CONCRETE IN RED
CONDUIT
NTS
Single Panel, Side View

McMILLEN JACOBS ASSOCIATES

PROJECT		SHEET
SUBJECT		DATE
BY	CHECKED	PROJECT NO.





* May be able to reduce some of these inspection steps.

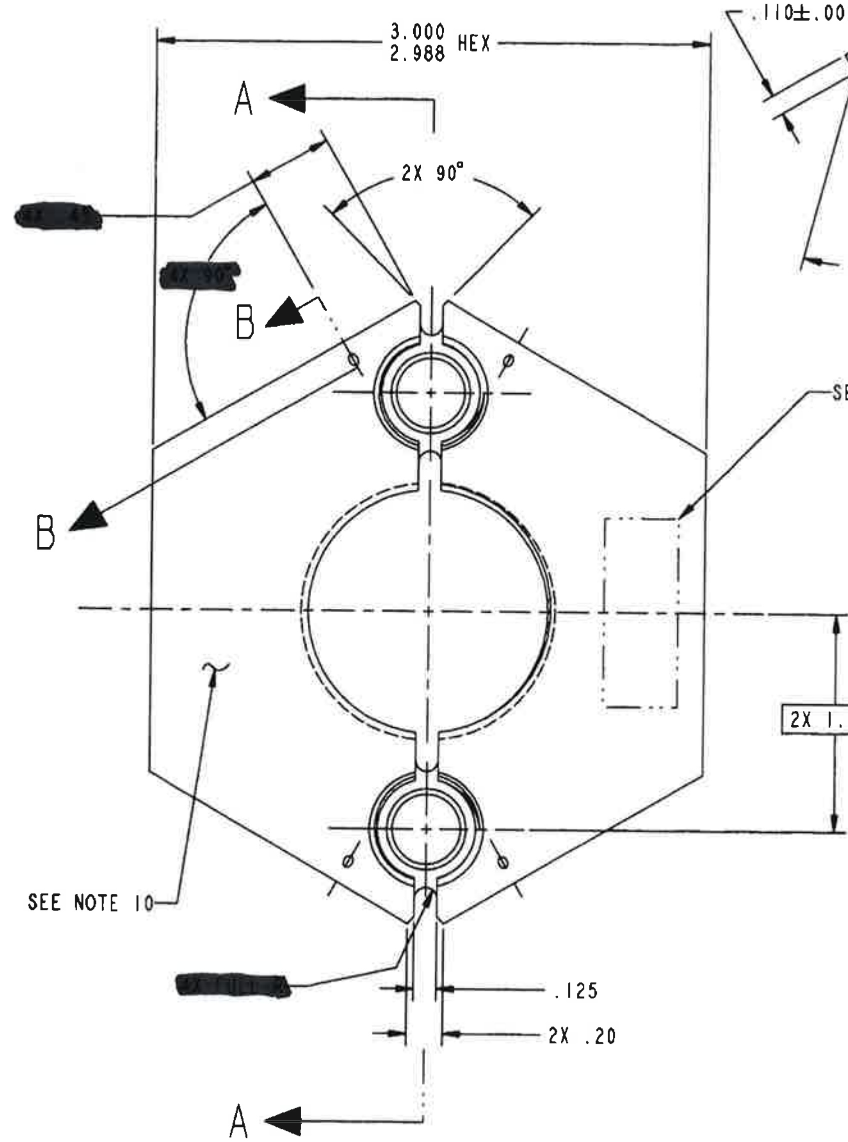
NOTES: UNLESS OTHERWISE SPECIFIED.

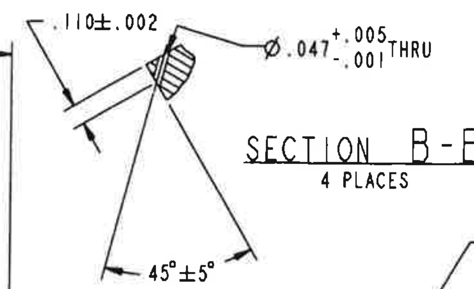
1. INTERPRET DIMENSIONING AND TOLERANCING IN ACCORDANCE WITH ANSI Y14.5M - 1982.
2. MATERIAL: INCONEL 718, HEAT TREATED, IN ACCCORDANCE WITH AMS 5662 TO ACHIEVE AN ULTIMATE TENSILE STRENGTH OF 190 TO 210 KSI. THE HRC SHALL BE 41 TO 44.
3. HEAT TREATMENT, IF CONDUCTED PRIOR TO MACHINING, SHALL BE IN ACCORDANCE WITH MIL-H-6875 IN A SINGLE LOT. ACTUAL OVEN TEMPERATURE PLOTS SHALL BE PROVIDED WITH CERTIFICATIONS.
4. BAR STOCK SHALL BE ULTRASONICALLY INSPECTED IN ACCORDANCE WITH MIL-STD-2154, CLASS AA, PRIOR TO FABRICATION.
5. THREE (3) STANDARD TENSILE TEST COUPONS FROM THE SAME LOT OF RAW MATERIAL SHALL BE MACHINED FROM THE HEAT TREATED BAR STOCK AND TESTED PER ASTM E8. THE ACTUAL MECHANICAL PROPERTIES SHALL BE RECORDED AND SUBMITTED WITH THE CERTIFICATIONS. THE FRACTURED TEST COUPONS SHALL BE DELIVERED WITH THE DATA.
6. ALL THREADS SHALL BE INSPECTED AFTER MACHINING PRIOR TO PROOF TEST, AFTER PROOF LOAD TEST, AND AFTER DRY FILM LUBRICATION. ALL UNIFIED INCH THREADS SHALL BE INSPECTED 100% IN ACCORDANCE WITH FED-STD-H28/20. SYSTEM 22. BUTTRESS THREAD FORMS SHALL BE INSPECTED PER RECOMMENDED GAUGING PRACTICES DEFINED IN ANSI B1.9.
7. ALL SURFACE FINISHES SHALL BE RMS 63 OR BETTER, IN ACCORDANCE WITH ANSI B46.1.
8. MACHINED NUTS SHALL BE DYE PENETRANT INSPECTED IN ACCORDANCE WITH ASTM E1417 FOLLOWING THE PROOF LOAD OF NOTE 12.
9. PASSIVATE IN ACCORDANCE WITH ASTM A967.
10. EACH NUT SHALL HAVE THE ROCKWELL OR BRINELL HARDNESS MEASURED IN THE AREA INDICATED IN ACCORDANCE WITH ASTM E18. THE HARDNESS SHALL BE RECORDED BY SERIAL NUMBER.
11. LASER MARK OR ELECTROCHEMICAL ETCH THE FOLLOWING INFORMATION IN THE AREA INDICATED. CHARACTERS SHALL BE .06 MINIMUM HIGH.

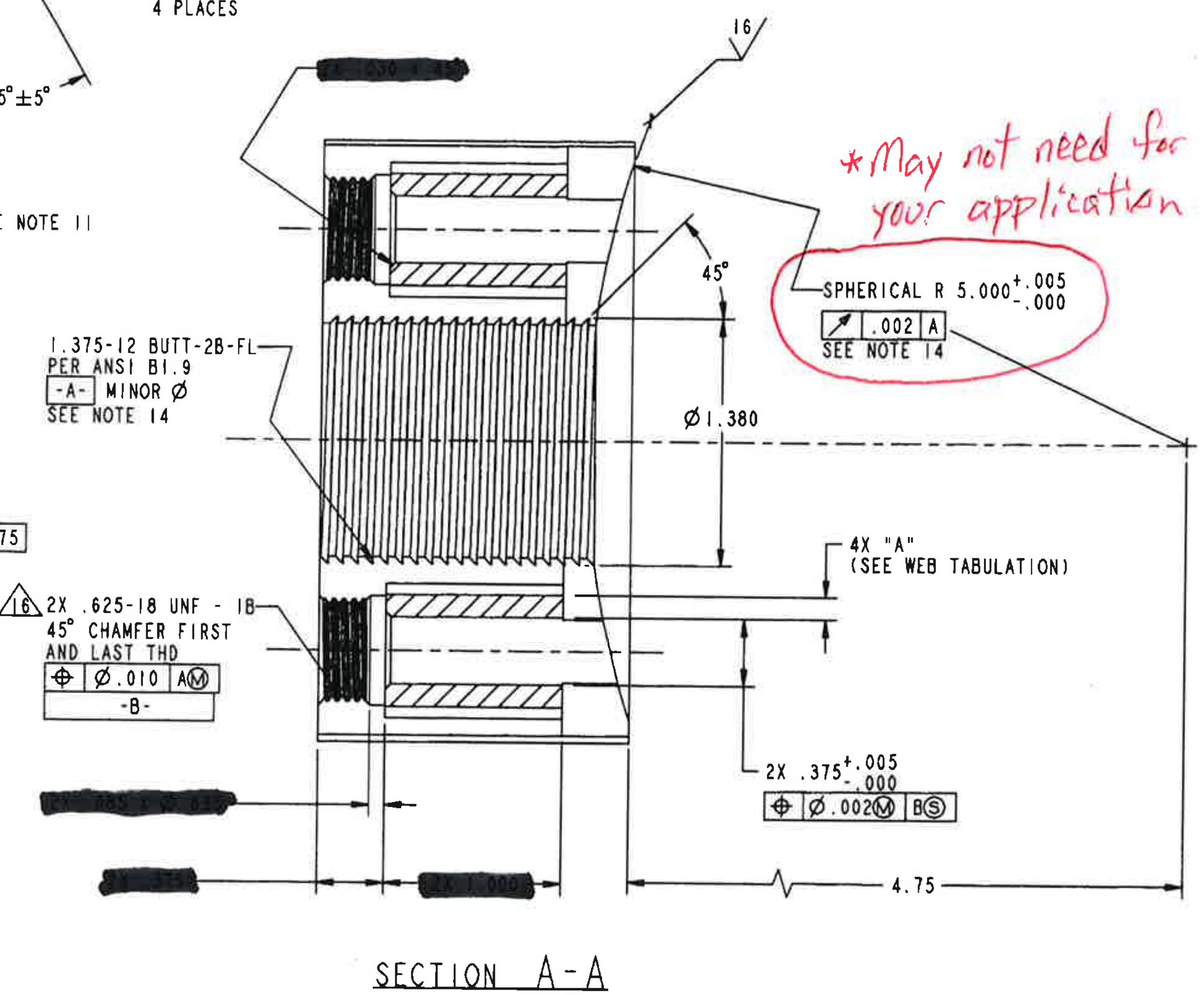
    P/N: 108270-X
    LOT NO: SEB-XXXXXX-XX
    SERIAL NO: XXXX

12. EACH NUT SHALL BE TENSIONED TO A PROOF LOAD OF 197,000LB. THE PROOF LOAD SHALL BE PERFORMED AND EVALUATED AS SPECIFIED IN SAE J995 OR ASTM F606.
13. EACH NUT SHALL BE PACKAGED INDIVIDUALLY AFTER MACHINING FOR PROCESSING AND TESTING TO ENSURE PROTECTION OF THE SPHERICAL SURFACE AND THREADS.
14. DRY FILM LUBRICATE NOTED THREADS AND SPHERICAL SURFACE IN ACCORDANCE WITH MIL-L-004610, TYPE I AFTER DYE PENETRANT INSPECTION PER NOTE 8.
15. MARK A RED BAND AS INDICATED, 360° AROUND BASE OF HEX (DASH NUMBERS -2 AND -3 ONLY), SEE WEB TABULATION.

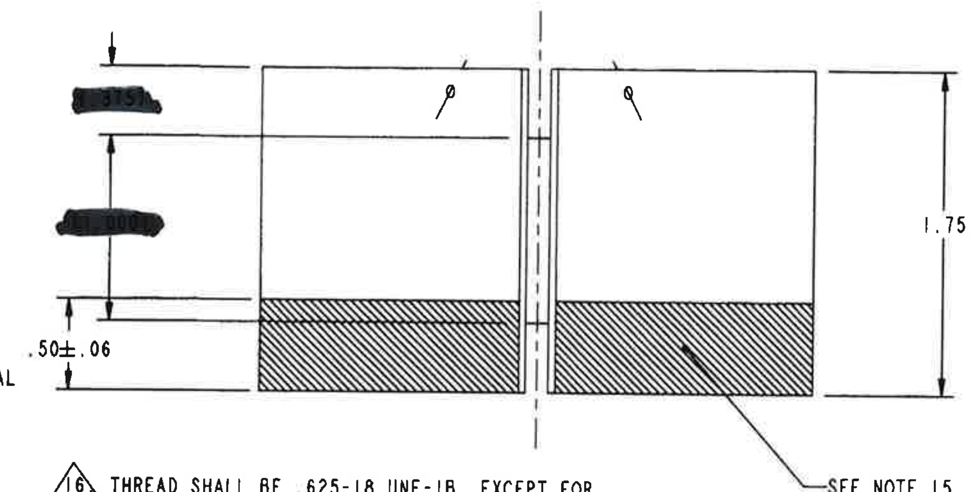
REVISIONS				
ZONE	LTR	DESCRIPTION	DATE	APPROVED
	A	SEE E.O. 021220	98/6/18	RW
	B	SEE E.O. 022174	98/10/13	RW
	C	SEE E.O. 022240	99/1/18	RW







WEB TABULATION			
DASH NO.	"A" DIMENSION	REMARKS	BAND
-1	[redacted]	NOMINAL WEB	NONE
-2	[redacted]	115% NOMINAL WEB	RED
-3	[redacted]	120% NOMINAL WEB	RED



16 THREAD SHALL BE .625-18 UNF-1B, EXCEPT FOR THE FOLLOWING:

PITCH DIAMETER
.5980
.5930

ITEM NO.	QTY REQD	CAGE CODE	PART OR IDENTIFYING NO.	NOMENCLATURE OR DESCRIPTION

LIST OF MATERIALS OR PARTS LIST

UNLESS OTHERWISE SPECIFIED
DIMENSIONS ARE IN INCHES
MACHINE MADE FINISHES TO BE 125
INTERPRET DRAWING IN ACCORDANCE WITH STANDARDS PRESCRIBED BY DOD-D-1000
TOLERANCES ON
.XX ±.01 .XXX ±.005 ANGLES ±1°
MATERIAL:
SEE NOTE 2

CONTRACT NUMBER

		DATE
DRAWN	DGM/EJK	98-5-21
CHECKED	DGM	98-6-12
APPROVED	ROB WEINHEIMER	98-6-12

PACIFIC SCIENTIFIC
ENERGETIC MATERIALS CO. CHANDLER ARIZONA

FRANGIBLE NUT BODY
(1 3/8")

SIZE D | CAGE CODE 54181 | 108270

SCALE: 2/1 | SHEET 1 OF 1

NEXT ASSY | USED ON
APPLICATION

108270

Appendix B

B1-2 ft concrete Raise concrete volume and over-turning Estimate

B2- Rubber Dam Calculations

B3-Compound Spillway Calculations

B4-Flashboard Panel & Truss calculations

B1- Concrete Raise Calculations

Concrete Volume:

As shown in Figure 3-1, the upstream to downstream width of the existing spillway is 9 feet. This means that matching the ogee curve along that width would require 9 cubic feet of concrete per every foot raised along 1 foot of spillway span. A Solidworks model of the existing spillway was used to confirm that the volume for a 1-foot raise over a 1-foot span section is 9 cubic feet (see Figure A-1).

The first 100 feet of existing spillway is sloped, meaning the spillway is only raised 1 foot at the Dike end and is raised a full 2 feet at the inside end of the sloped section. The average spillway raise in this section is 1.5 feet. The total volume of concrete needed is found as follows:

$$9\frac{feet^3\ of\ concrete}{(feet\ of\ raise)*(feet\ of\ span)}*100 feet\ span*1.5\ feet\ average\ raise$$
$$+\ 9\frac{feet^3\ of\ concrete}{(feet\ of\ raise)*(feet\ of\ span)}*350\ feet\ span*2\ feet\ raise$$
$$=\mathbf{7,650\ feet^3}$$

The arc length of the ogee shape is approximately 12.17 feet. Because the shape is consistent across the whole spillway including the sloped section, the total upper surface area of the existing spillway is:

$$12.17\ feet\ arc\ length*450\ feet\ span=\mathbf{5,475\ feet^2}$$

Figure A-1. Properties of new concrete for a 1-foot raise over 1 foot of span

Overturning

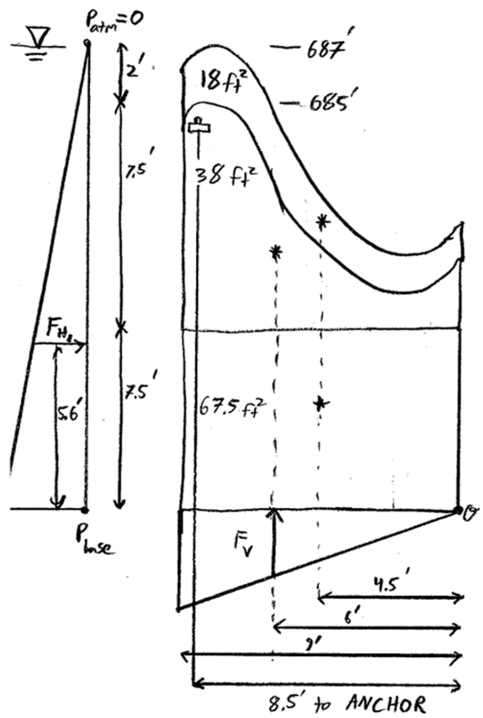
The raise in the pool elevation increases the hydrostatic forces acting on the spillway at full pool and at the PMF. Overturning analysis must be done to confirm that the new design will not cause the factor of safety for overturning to drop below 1.0 in either scenario. The assumptions in the previous overturning analysis in the STI (2018), along with new design assumptions, are listed in Table A-1. These were used for the new construction overturning analysis here. The post-tensioned anchors imbedded in the existing crest have a force contribution resisting overturning that can be counted towards the PMF condition but cannot be used for full pool overturning.

Table A-1. Concrete raise components

STI (2018) Assumptions	
Density of Concrete	150 lb./ft^3
Density of Water	62.4 lb/ft^3
Existing Post-tensioned Anchor Force Per Span	23,000 lb/ft
Monolith Height (Vertical distance from crest to upstream reservoir bed)	15 ft
Design Assumptions	
Cross-Sectional Area of Main Ogee Section (upper 7.5-foot section of monolith height)	38 ft^2
Cross-Sectional Area of Rectangular Spillway Base Section (lower 7.5-foot section of monolith height)	67.5 ft^2
Cross-Sectional Area of 2-Foot Concrete Section Placed on Existing Weir for Concrete Raise Option	18 ft^2
Cross-Sectional Area of New Downstream Ramp Concrete for Rubber Dam Option	7 ft^2
Cross-Sectional Area of Upstream Platform for Rubber Dam Option	14 ft^2
Rubber Dam Weight per Square Foot of Material	3.95 lb/ft^2

Weight of 7-foot Picket Panel	1,395 lb

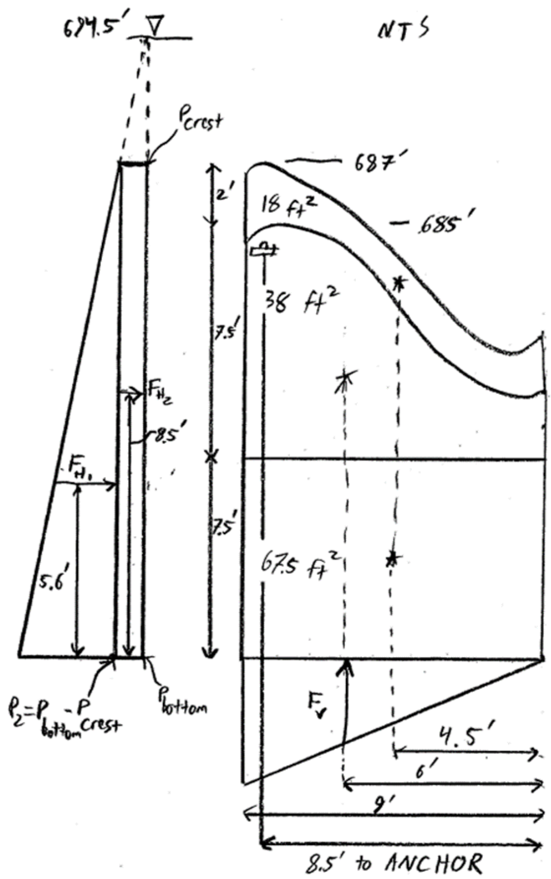
Figure A-2 represents the case for a 2-foot concrete raise at the new full pool elevation of 687'. The forces contributing to overturning are the horizontal hydrostatic force acting on the upstream side of the crest, and the vertical hydrostatic forces from the water assumed to be creating an uplift pressure on the bottom of the spillway. The restoring forces are the weight of the three defined concrete sections, the main ogee, its rectangular base, and the new concrete placed on top. The net moment is negative, which is defined as the counterclockwise, restoring direction. This means the weight of the crest alone is enough to prevent overturning at full pool, with a calculated safety factor of 1.15.



2' Raise	FULL POOL		Water Level	687				
			Crest Level	687				
			Base Level	670				
			Head Base	17				
			Head Crest	0				
Overturn (+1)/ Correcting (-1)	Overturn/Correcting	Average Pressure (psf)	Active area ft2	Active Volume ft3	Force lb	Moment Arm ft	Net Moment (ft-lb)	
1	Horizontal Force FH1 (triangular dist.)	530.4	17		9016.8	5.6667	51,095	
1	Vertical Force FV (triangular distribution)	530.4	9		4773.6	6	28,642	
-1	Weight of Main Ogee			38	5700	6	-34,200	
-1	Weight of Ogee rectangular base			67.5	10125	4.5	-45,563	
-1	Weight of 2-foot raise			18	2700	4.5	-12,150	
No Seal						Total	-12,176	
						FoS	1.15	

Figure A-2. Overturning forces for a 2-foot raise at full pool

Figure A-3 shows the case of a 2-foot concrete raise at PMF conditions, water level 694.5'. The hydrostatic forces will be higher than full pool, but in this case the post-tensioned anchors can be included to the restoring forces in the full pool case. The result is still a negative net moment, and a factor of safety of 1.80 against overturning. Because neither the full pool nor the PMF cause overturning, no additional stabilization is needed for the 2-foot raise.



2' Raise	PMF		Water Level	694.5				
			Crest Level	687				
			Base Level	670				
			Head Base	24.5				
			Head Crest	7.5				
Overturn (+1)/ Correcting (-1)	Overturn/Correcting	Average Pressure (psf)	Active area ft2	Active Volume ft3	Force lb	Moment Arm ft	Net Moment (ft-lb)	
1	Horizontal Force FH1 (triangular dist.)	530.4	17		9016.8	5.666667	51,095	
1	Horizontal Force FH2 (rectangular dist.)	468	17		7956	8.5	67,626	
1	Vertical Force FV (triangular distribution)	764.4	9		6879.6	6	41,278	
-1	Weight of Main Ogee			38	5700	6	-34,200	
-1	Weight of Ogee rectangular base			67.5	10125	4.5	-45,563	
-1	Post-tensioned anchor				23000	8.5	-195,500	
-1	Weight of 2-foot raise			18	2700	4.5	-12,150	
No Seal							-127,414	
							1.80	

Figure A-3. Overturning forces for a 2-foot raise at PMF

Appendix B2, Rubber Dam Calculations

Discharge Characteristics

A rubber dam uses level sensors to maintain the desired pool level by deflating or inflating automatically. Without releasing all the additional storage, the rubber dam can pass flows up to the capacity of the spillway with the dam completely deflated. To evaluate rubber dam viability, the spillway was analyzed assuming a maximum water level of 694.5 feet, which maintains 0.5 feet of freeboard relative to the dam, and a rubber dam thickness of 2 inches.

Following guidelines from a rubber dam manufacturer, the design includes a 15.83-foot wide sill as the base of the 8-foot rubber dam (or a 7.92-foot wide sill for a 4-foot rubber dam), three independent rubber dam sections, and a 1.5H:1V concrete slope on either side of each dam section. The rubber dam over the 1% sloped section of existing spillway follows this slope, meaning the top of the rubber dam next to the dike would be 1 foot above the full pool level.

The capacity of the lowered rubber dam spillway was found by breaking the full span into sections (see Figure B-1). The capacities for some of these sections, like the sloped concrete abutments or the existing sloped spillway crest, were calculated as if they had uniform head based on the average crest elevation. For sections with a broad-crested sill and a downstream slope, the USGS curve was used to find the discharge coefficient. This depends on head and sill width. The abutments and piers would be broad-crested weirs without a downstream slope, so they would have a discharge coefficient of 2.64 independent of head. Discharge coefficient is used in the following equation to determine flow capacity:

$$Q = C * b * H^{3/2}$$

Q – Water flow in cfs passed by the spillway.

C – Discharge coefficient resulting from the shape of the spillway.

b – Effective length of the spillway section in feet.

H – Total head feet upstream of the spillway relative to the spillway crest.

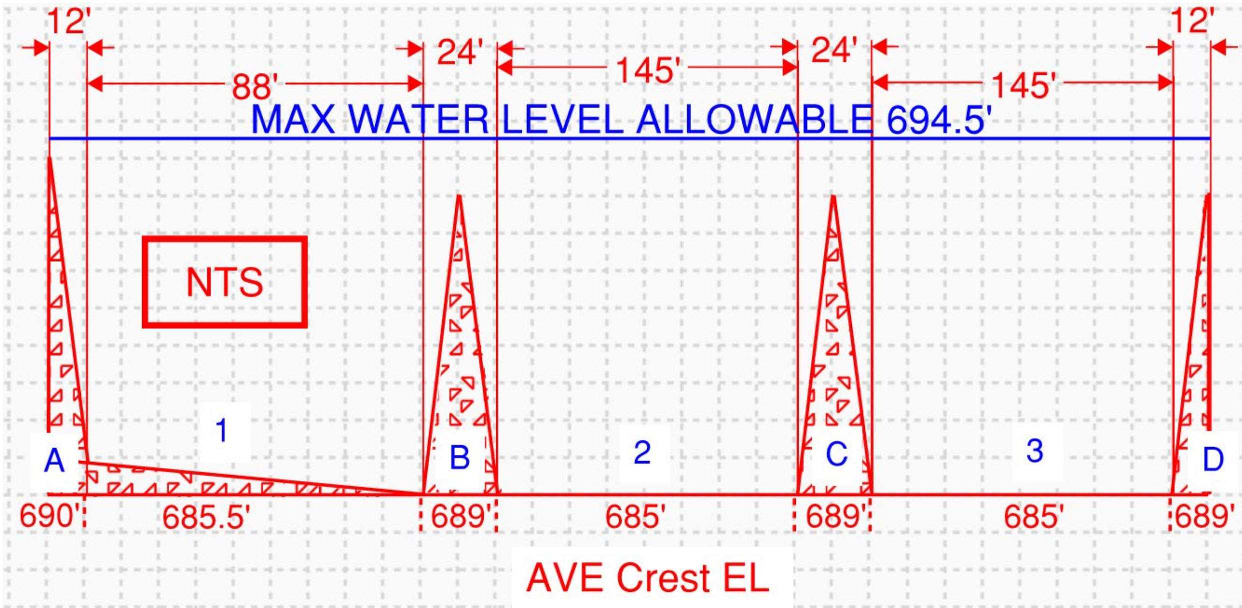


Figure B-1. Section view of deflated rubber dam at maximum allowable water level

Table B-1 shows the values needed to determine the 8-foot rubber dam discharge coefficients for the sections labeled in Figure B-1. The ratio of head to broad-crested weir width (h/L) uses the 15.83-foot recommended sill width under the rubber dam and assumes there are negligible head losses due to approach velocity. Using the h/L ratio for the sloped rubber dam section and the flat rubber dam sections, the USGS chart gives discharge coefficients of 2.71 and 2.725 respectively (see Figure B-2).

Table B-1. Full-span 8-foot rubber dam discharge coefficients

Water Level	Section	Average Crest Elevation (ft)	Rubber Dam Thickness (ft)	Head (ft)	h/L	C
PMF: 694.5’	Sloped Rubber Dam Sill (1)	685.5	0.16	8.84	0.558	2.71
PMF: 694.5’	Flat Rubber Dam Sill (2 and 3)	685	0.16	9.34	0.590	2.725
PMF: 694.5’	Dike-adjacent Abutment (A)	690	0.16	4.34	N/A	2.64
PMF: 694.5’	Hillside Abutment (D)	689	0.16	5.34	N/A	2.64
PMF: 694.5’	Center Piers (B and C)	689	0.16	5.34	N/A	2.64

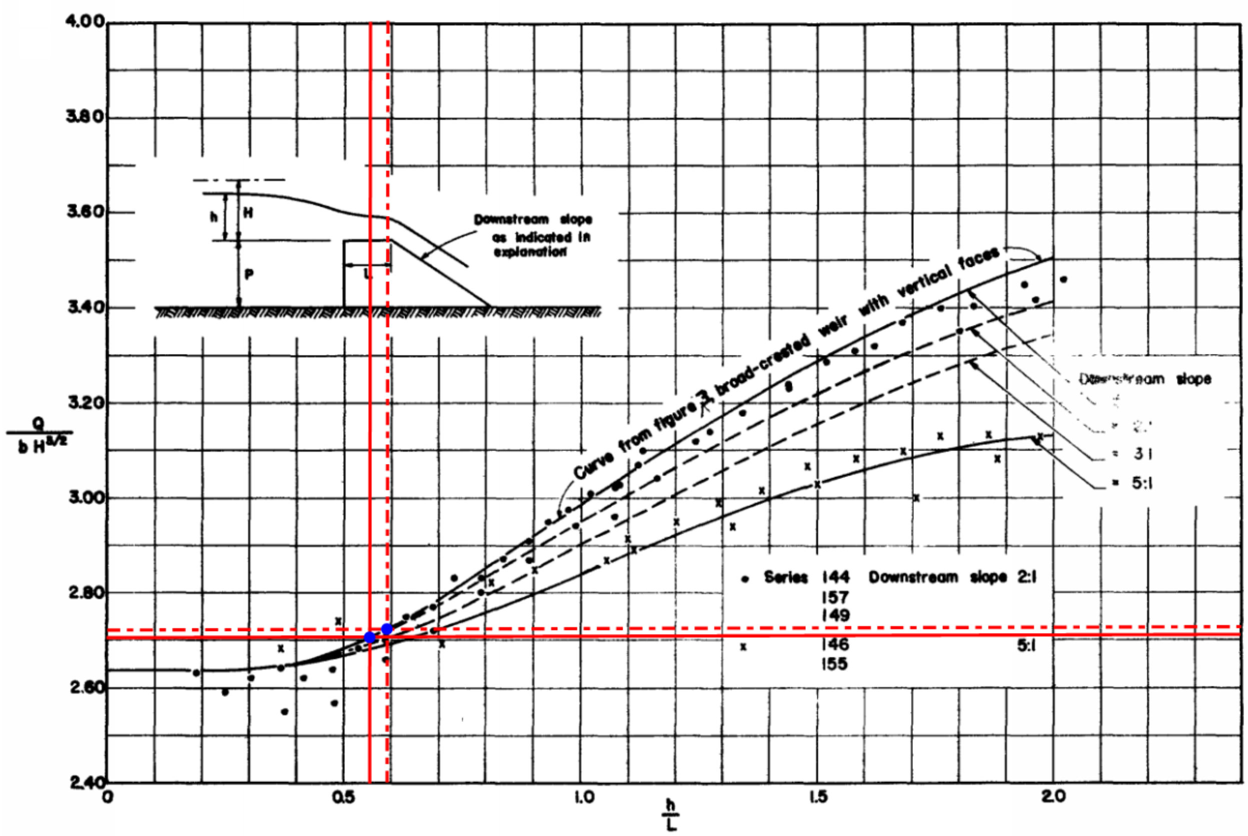
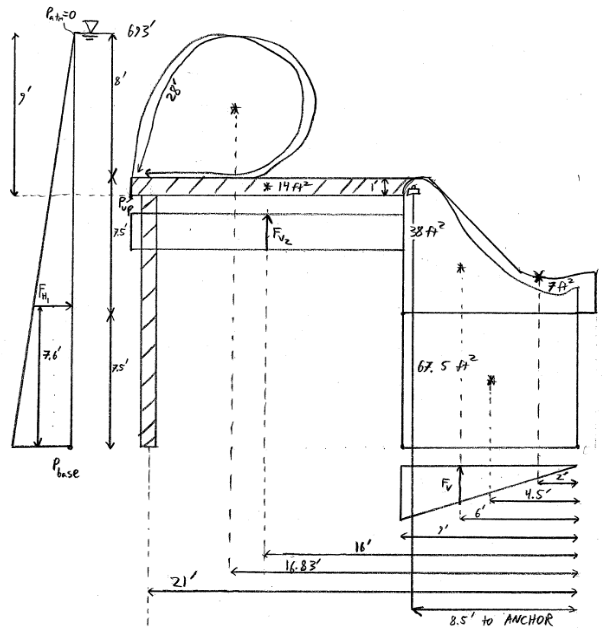


Figure B-2. Determination of discharge coefficients for rubber dam sections during PMF

Figure 4-4 showed the summation of the flow capacities of all spillway sections, using the equation $Q = C * b * H^{3/2}$ and using the coefficients determined from Figure B-2. This gives a total spillway capacity of 31,066 cfs, which is less than the 37,135 cfs required to pass the PMF.

Overturning

In addition to the horizontal hydrostatic force acting on the rubber dam and concrete base, there would be an overturning force from hydrostatic pressure acting upward on the bottom of the sill. The full pool water level is much higher with the rubber dam, so the hydrostatic forces would be higher than with the concrete raise option. There would be more weight from the platform, the concrete placed on the downstream face of the crest, and from the rubber dam material. The concrete placed on the spillway face would also help slightly by adding to the weight.



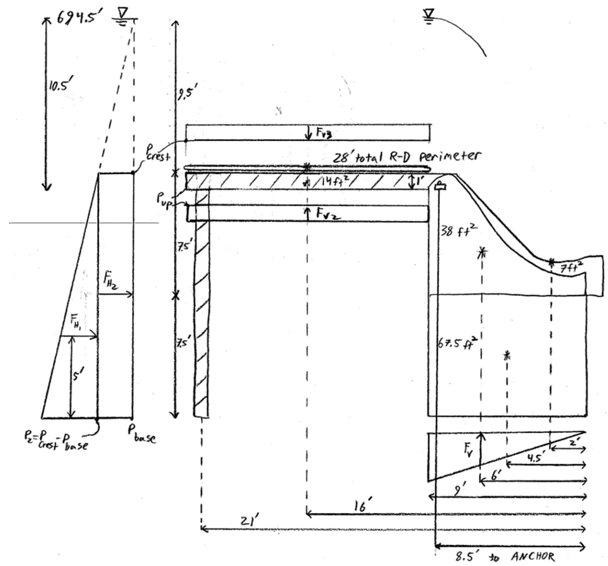
Rubber Dam	FULL POOL		Water Level	693			
			Crest Level	685			
			Base Level	670			
			Head Base	23			
			Head Crest	8			
			Level under platform	684			
			Head under platform	9			
Overturn (+1)/ Correcting (-1)	Overturn/Correcting	Average Pressure (psf)	Active area ft2	Active Volume ft3	Force lb	Momen t Arm ft	Net Moment (ft-lb)
1	Horizontal Force FH1 (triangular dist.)	717.6	23		16504.8	7.66667	126,537
1	Vertical Force FV (triangular distribution)	717.6	9		6458.4	6	38,750
1	Vertical Force FV2 (horizontal dist. Up on	561.6	14		7862.4	16	125,798
-1	Weight of Main Ogee			38	5700	6	-34,200
-1	Weight of Ogee rectangular base			67.5	10125	4.5	-45,563
-1	Weight of downstream ramp addition			7	1050	2	-2,100
-1	Weight of platform			14	2100	16	-33,600
-1	Weight of rubber dam		28		110.6	16.83	-1,861
No Seal							173,762
							0.40
Moment arm new anchors	9	Necessary force per foot:	19,307	lb/ft			
Seal							135,011
							0.46
Moment arm new anchors	9	Necessary force per foot:	15,001	lb/ft			

Figure B-3. Overturning forces for a rubber dam at full pool

Figure B-3 represents the case of a rubber dam section with the new full pool of 693'. The assumptions for the calculations can be seen in Table A-1 in Appendix A. In this case, the high overturning forces introduced from the increased head and new uplift area is much more significant than the increased weight from the new materials. Because of this the net moment is positive, indicating that the crest would overturn at full pool.

One way to help counteract the overturning moment is to seal the upstream bottom edge of the crest to remove the uplift force; however, this only reduces the overturning moment, it does not eliminate it. To counteract the overturning moment, grouted rock anchors will be attached to the upstream face of the crest. Assuming this creates a 9-foot moment arm, the required force would be 19,307 lb per foot of span without sealing the upstream face, and 15,001 lb per foot of span if the upstream face were sealed.

The preliminary design for this attachment includes 1-3/4" threaded rebar anchors grouted into the bedrock at 6-foot or 5-foot spacing, depending on if the upstream face were sealed or left unsealed respectively. This satisfies the force distribution required.



Rubber Dam	PMF		Water Level	694.5			
			Crest Level	685			
			Base Level	670			
			Head Base	24.5			
			Head Crest	9.5			
			Level under platform	684			
			Head under platform	10.5			
Overturn (+1)/ Correcting (-1)	Overturn/Correcting	Average Pressure (psf)	Active area ft2	Active Volume ft3	Force lb	Moment Arm ft	Net Moment (ft-lb)
1	Horizontal Force FH1 (triangular dist.)	468	15		7020	5	35,100
1	Horizontal Force FH2 (rectangular dist.)	592.8	15		8892	7.5	66,690
1	Vertical Force FV (triangular distribution)	764.4	9		6879.6	6	41,278
1	Vertical Force FV2 (horizontal dist. Up	655.2	14		9172.8	16	146,765
-1	Weight of Main Ogee			38	5700	6	-34,200
-1	Weight of Ogee rectangular base			67.5	10125	4.5	-45,563
-1	Weight of downstream ramp addition			7	1050	2	-2,100
-1	Weight of platform			14	2100	16	-33,600
-1	Downward pressure on top of platform	592.8	14		8299.2	16	-132,787
-1	Post-tensioned				23000	8.5	-195,500
-1	dam		28		110.6	16	-1,770
No Seal							-155,687
							1.54
Moment arm new anchors	9	Necessary force per foot:	-17,299	lb/ft			
Seal							-196,965
							1.79
new anchors	9	foot:	-21,885	lb/ft			

Figure B-4. Overturning forces for a rubber dam at PMF

Figure B-4 shows the case for the rubber dam at PMF conditions, with a water level of 694.5'. In this case, the rubber dam would be deflated, so there would be a downward pressure force on the platform as well as the uplift pressure. Additionally, the existing post-tensioned anchors would could towards preventing overturning too. Though the hydrostatic pressures also increase slightly, the result is a negative moment and a safety factor of 1.54 without an upstream seal, and 1.79 with an upstream seal. Measures to prevent overturning would still need to be taken for the full pool condition, but this shows that full pool, not PMF, is the limiting case.

Table B-2. Full-span rubber dam components

Component	Description	Material	Size	Weight
Rubber Dam Control System	Full control system, level sensors, and air compressor responsible for automatically managing the height of the rubber dam.	Various	N/A	N/A
Section 1 Rubber Dam	Section of rubber dam over the sloped 100-foot section of the existing spillway.	Rubber	8 feet high inflated; span of 112 feet including conical end sections	N/A
Section 2 Rubber Dam	One of two sections of rubber dam laid on the 350-foot flat section of the existing spillway. This section is the one closer to the Dike.	Rubber	8 feet high inflated; span of 169 feet including conical end sections	N/A
Section 3 Rubber Dam	One of two sections of rubber dam laid on the 350-foot flat section of the existing spillway. This section is the one farther form the Dike.	Rubber	8 feet high inflated; span of 169 feet including conical end sections	N/A
Dike Abutment	Sloped 1.5H:1V concrete abutment supporting the end of rubber dam Section 1 adjacent to the Dike.	Concrete	760 cubic feet (28.14 cubic yards)	114,000 lb
First Pier	Concrete center pier, sloped 1.5H:1V on both sides, which sits in between rubber dam Sections 1 and 2.	Concrete	1,520 cubic feet (56.28 cubic yards)	228,000 lb
Second Pier	Concrete center pier, sloped 1.5H:1V on both sides, which sits in between rubber dam Sections 2 and 3.	Concrete	1,520 cubic feet (56.28 cubic yards)	228,000 lb
Far Abutment	Sloped 1.5H:1V concrete abutment supporting the end of rubber dam Section 3 adjacent to the far hillside.	Concrete	760 cubic feet (28.14 cubic yards)	114,000 lb
Concrete Sill	Concrete slab extending back from the existing spillway crest to provide a base for the rubber dam.	Concrete	6,300 cubic feet (233.33 cubic yards). [i.e. 14 cubic feet per foot span]	945,000 lb

New Spillway Face Concrete	Concrete placed over existing spillway to turn the ogee shape to a broad-crested weir with 1:1 downstream slope.	Concrete	3,150 cubic feet (116.67 cubic yards). [i.e. 7 cubic feet per foot span]	472,500 lb
Support Rods	Vertical members supporting the upstream end of the new concrete sill.	Epoxy-coated Steel	To be sized with further design details for the platform sill	N/A

4-Foot Raise

The design for a 4-foot rubber dam would be achieved in the same way as an 8-foot raise, scaling down the dimensions of the platform sill and abutments needed to accommodate the smaller rubber dam. Table B-3 summarizes the properties for each rubber dam or support section relevant to the hydraulics at PMF conditions. The results of the hydraulic calculations are shown in Figure 4-5. For cost estimates, the volume of concrete used for the platform sill was halved, as were the procurement and installation costs for the rubber dam. Applying the same overturning procedure as detailed above for an 8-foot rubber dam, the dam would still overturn in the full pool condition. The amount of force needed to resist overturning is less significant, reflected in a lower cost for upstream anchors.

Table B-3. Full-span 4-foot rubber dam discharge coefficients

Water Level	Section	Average Crest Elevation (ft)	Rubber Dam Thickness (ft)	Head (ft)	h/L	C
PMF: 694.5’	Sloped Rubber Dam Sill (1)	685.5	0.16	8.84	1.117	3.050
PMF: 694.5’	Flat Rubber Dam Sill (2 and 3)	685	0.16	9.34	1.180	3.100
PMF: 694.5’	Dike-adjacent Abutment (A)	688	0.16	6.34	N/A	2.64
PMF: 694.5’	Hillside Abutment (D)	687	0.16	7.34	N/A	2.64
PMF: 694.5’	Center Piers (B and C)	687	0.16	7.34	N/A	2.64

Appendix B3- Compound Spillway Calculations

Discharge Characteristics

The compound spillway is broken into an operational spillway and an emergency spillway. The operational spillway would be a rubber dam, extending from the dike and covering a portion of the spillway span. The emergency spillway would be permanent unless an extreme flood exceeded the operational capacity. The emergency structure would be triggered in this event to release additional flow up to at least the PMF, ideally leaving behind only the original spillway crest with minimal modification.

As with the full rubber dam concept, the capacity of the rubber dam in the compound spillway is found by considering individual sections (see Figure C-1). Sections with a sloped crest are also determined the same way, assuming a constant head across the section span based on the average crest elevation. Section 1 of the rubber dam over the sloped spillway crest along with abutment A and pier B would be the same as with the full rubber dam design. There would be an additional rubber dam section, Section 2, the width of which is shown here as 37 feet for a baseline. The far side of the second rubber dam section would have a sloped abutment adjacent to a 2-foot pier. This pier separates the operational spillway from the emergency spillway, which would take up the remaining span of the spillway to the far hillside.

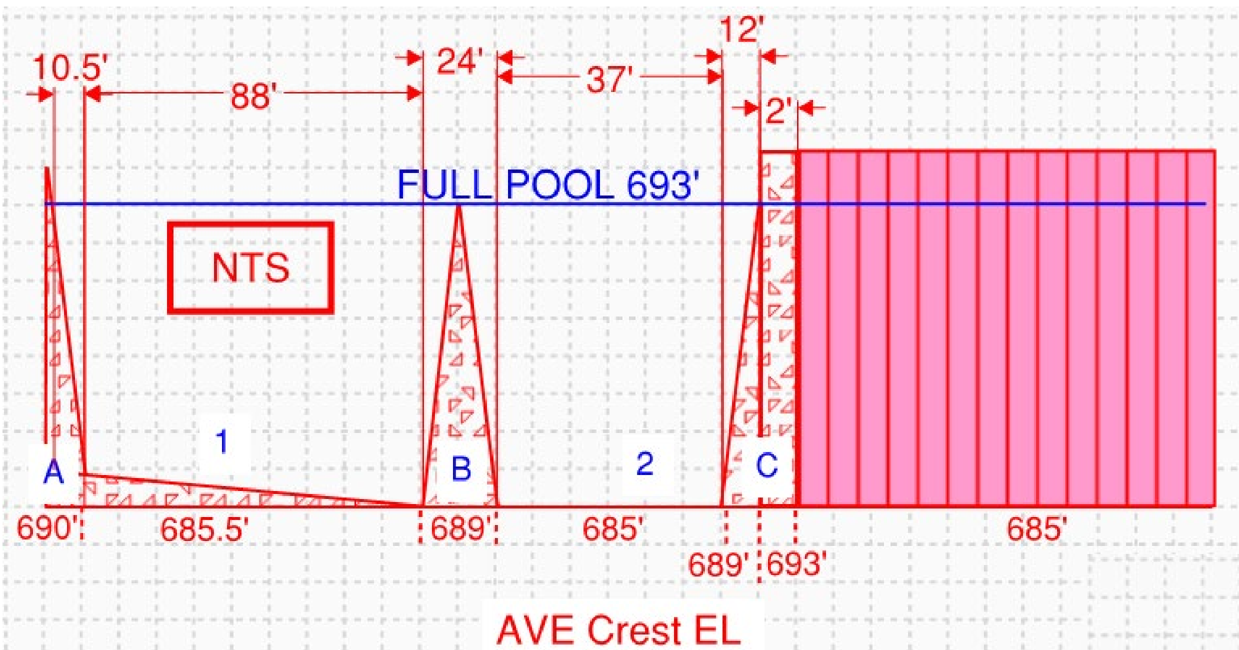


Figure C-1. Section view of compound spillway

As with the full rubber dam, the deflated thickness is assumed to be 2 inches and the horizontal width of the sill is set to 15.83 feet per the manufacturer recommendations. The rubber dam would need to pass operational flows without exceeding the full pool water level. At the PMF, the water level would be higher than the full pool. Discharge characteristics for both conditions are given in Table C-1. The PMF discharge matches Table B-1. The h/L ratios for full pool discharge give a discharge coefficient of 2.66 for the sloped rubber dam sill section and 2.68 for the flat rubber dam sill using the USGS chart in Figure

C-2. The abutments and piers are broad-crested weirs with a discharge coefficient of 2.64 at both water levels.

Table C-1. Compound Spillway discharge coefficients

Water Level	Section	Average Crest Elevation (ft)	Rubber Dam Thickness (ft)	Head (ft)	h/L	C
Full Pool: 693.0’	Sloped Rubber Dam Sill (1)	685.5	0.16	7.34	0.464	2.66
Full Pool: 693.0’	Flat Rubber Dam Sill (2 and 3)	685	0.16	7.84	0.495	2.68
Full Pool: 693.0’	Dike-adjacent Abutment (A)	689.5	0.16	3.34	N/A	2.64
Full Pool: 693.0’	Center Piers (B and C)	689	0.16	3.84	N/A	2.64
PMF: 694.5’	Sloped Rubber Dam Sill (1)	685.5	0.16	8.84	0.558	2.71
PMF: 694.5’	Flat Rubber Dam Sill (2 and 3)	685	0.16	9.34	0.590	2.725
PMF: 694.5’	Dike-adjacent Abutment (A)	690	0.16	4.34	N/A	2.64
PMF: 694.5’	Center Piers (B and C)	689	0.16	5.34	N/A	2.64

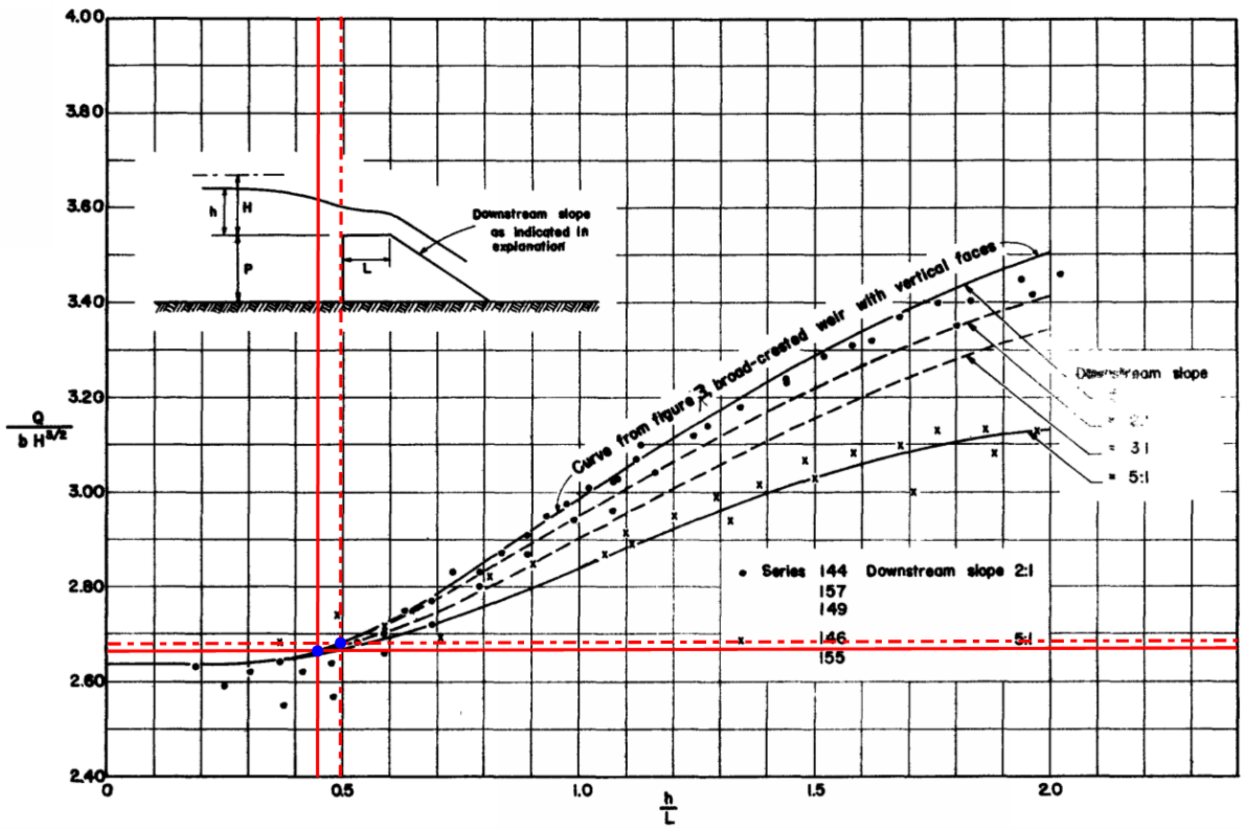


Figure C-2. Determination of discharge coefficients for rubber dam sections at full pool water level

Full Pool Section	Crest Elevation ft	Deflated Thickness ft	Water Level ft	Head ft	b ft	L ft	h/L	C	Q cfs
A	689.5	0.16	693.0	3.34	10.5	15.83	0.211	2.64	169
1	685.5	0.16	693.0	7.34	88	15.83	0.464	2.660	4655
B	689	0.16	693.0	3.84	24	15.83	0.243	2.640	477
2	685	0.16	693.0	7.84	37	15.83	0.495	2.680	2177
C	689	0.16	693.0	3.84	12	15.83	0.243	2.64	238
									7716

Figure C-3. Flow calculations for operational spillway at full pool discharge

Figure C-3 shows a spreadsheet calculation for the maximum operational flow in the baseline case with a total operational spillway length of 175 feet. This uses the same equation as with the full rubber dam calculations, $Q = C * b * H^{3/2}$. The water level being at full pool instead of PMF does limit the submerged span of the Dike-adjacent abutment by 1.5 feet, which is why Section A in Figure C-3 is only 10.5 feet.

L Add	L Operational RD	L Abutments	L Secondary RD	Head	C	Flow add abut	Add Flow	Tot Flow	% Record	RD PMF	EM PMF	EM length	EM Head	EM min C
14	114	14	0	0	0	408	0	5062	93%	6,945	30,190	336	9.5	3.07
50	150	38	12	7.84	2.680	884	706	6,245	114%	8,661	28,474	300	9.5	3.24
75	175	38	37	7.84	2.680	884	2,177	7,716	141%	10,605	26,530	275	9.5	3.29
100	200	38	62	7.84	2.680	884	3,648	9,187	168%	12,550	24,585	250	9.5	3.36
125	225	38	87	7.84	2.680	884	5,118	10,658	195%	14,494	22,641	225	9.5	3.44
150	250	38	112	7.84	2.680	884	6,589	12,128	222%	16,439	20,696	200	9.5	3.53
175	275	38	137	7.84	2.680	884	8,060	13,599	249%	18,384	18,751	175	9.5	3.66
200	300	38	162	7.84	2.680	884	9,531	15,070	276%	20,328	16,807	150	9.5	3.83
225	325	38	187	7.84	2.680	884	11,001	16,541	303%	22,273	14,862	125	9.5	4.06
250	350	62	188	7.84	2.680	1361	11,060	17,076	313%	23,132	14,003	100	9.5	4.78
275	375	62	213	7.84	2.680	1361	12,531	18,547	340%	25,077	12,058	75	9.5	5.49
300	400	62	238	7.84	2.680	1361	14,002	20,018	367%	27,022	10,113	50	9.5	6.91
325	425	62	263	7.84	2.680	1361	15,473	21,489	394%	28,966	8,169	25	9.5	11.16
350	450	60	290	7.84	2.680	1361	17,061	23,077	423%	31,066	6,069	0	9.5	#DIV/0!

Figure C-4. Calculated operational and PMF discharge characteristics for compound spillways

Figure C-4 compares the operational and PMF discharge characteristics for different combinations of operational and emergency spillways. The first row of the spreadsheet represents the case of only using the sloped section of rubber dam; all additional rows represent adding secondary rubber dam sections to the operational spillway.

L Add – This is the total span in feet of spillway needed for the operational spillway in addition to the sloped 100-foot section. The first row shows 14 feet for using only the sloped section of rubber dam because the abutment and pier at the end of the rubber dam would extend 14 feet onto the flat span of the existing spillway.

L Operational RD – This is the total span in feet that would function as the operational spillway.

L Abutments – This is the total width in feet of abutments and piers for supporting the rubber dam sections. The increase from 14 feet to 38 feet in the spreadsheet comes from the assumption that at that point the additional operational spillway on the flat spillway section would be split among two rubber dams. Similarly, the total abutments and piers span would increase to 62 feet if the operational spillway were split into three segments. See Figure C-1 for pier and abutment assumptions.

L Secondary RD – This is the total effective span in feet for the secondary rubber dam section(s).

Head – This is the head in feet available for any additional rubber dam sections on the flat crest of the spillway.

C – This is the discharge coefficient for any additional rubber dam sections on the flat crest of the spillway.

Flow add Abut – This is the flow in cubic feet per second added from all abutments and piers in the operational spillway during discharge at the full pool water level. In Figure C-1 this includes Sections A, B, and C. The flow calculations for abutments and piers use the values in Table C-1 for full pool and the equation $Q = C * b * H^{3/2}$.

Add Flow – This is the flow in cubic feet per second added by any additional rubber dam sections on the flat crest of the spillway. It is calculated using the head (Head) and discharge coefficient (C) for any additional rubber dam sections on the flat crest of the spillway, the length of secondary rubber dam sections (L Secondary RD), and the equation $Q = C * b * H^{3/2}$.

Tot Flow – This is the total flow capacity in cubic feet per second of the operational rubber dam without exceeding the full pool water level. It is found by adding the full pool capacity of the sloped rubber dam section 4,655 cfs (see Figure C-3), the flow over all abutments and piers (Flow add Abut), and the additional flow from any additional rubber dam sections of the flat crest of the spillway (Add Flow).

% Record – This is the factor of the highest estimated flood on record, 5,457 cfs, that the operational spillway can pass without exceeding the full pool water level. In the baseline case shown in Figure C-3 and highlighted in Figure C-4, the total operational spillway span is 175 feet which corresponds to an operational capacity of 7,716 cfs or 141% of the record flow.

RD PMF – This is the flow in cubic feet per second that the total operational spillway (The sloped rubber dam section, any flat rubber dam sections, and all piers and abutments) could pass during a PMF event. The PMF discharge characteristic values in Table C-1 are used with the equation $Q = C * b * H^{3/2}$ for each case. The final row is the full rubber dam case, which is why the RD PMF value of 31,066 cfs matches the maximum capacity calculated for the full rubber dam option in Appendix B.

EM PMF – This is the flow in cubic feet per second that the emergency spillway must be able to pass during a PMF event. It is the PMF less the capacity of the operational spillway during the PMF.

EM Length – This is the length in feet of the emergency spillway. It is found by subtracting the total "L Operational RD" from the total length of the existing spillway, 450 feet.

EM Head – This is the head in feet assumed to be available to pass the PMF over the emergency spillway. It is found by assuming a maximum water level of 694.5 feet and assuming that the crest of the spillway after the triggered release will be at the current crest elevation of 685 feet.

EM Min C – This is the discharge coefficient required of the emergency spillway to pass the required flow during the PMF event. The discharge coefficient at 9.5 feet of head for the existing spillway is 4.23, so any combinations requiring a higher discharge coefficient from the emergency section would not pass the PMF. These are highlighted in red in Figure C-4. It is found by solving the below equation for C.

$$Q = C * b * H^{3/2}$$

Q – Water flow in cubic feet per second passed by the spillway.

C – Discharge coefficient resulting from the shape of the spillway.

b – Effective length of the spillway section in feet.

H – Total head feet upstream of the spillway relative to the spillway crest.

Summary results of compound spillway calculated discharges. The operational spillway will pass approximately 7700 cfs, 40% over the maximum observed spill flow of 5457 cfs with a full pool level of 693 feet. The combined operational and emergency spillway will pass the PMF with a SG reservoir water level at approximately 694.5 feet if the emergency spillway has a coefficient of discharge of 3.29 or greater. These values are shown in the yellow highlighted row of Figure C-4.

Appendix B4 Panel & Truss Design

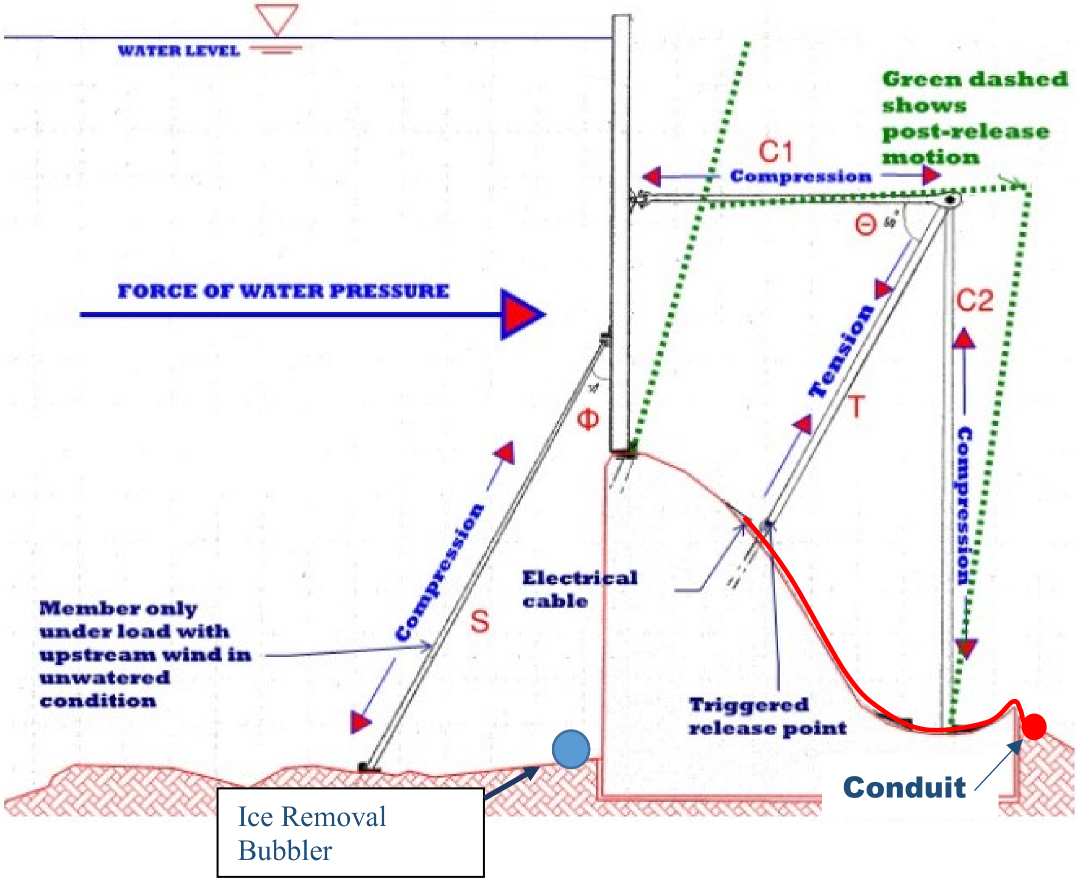


Figure D-1. Overview of components

Truss Design

The truss is defined as the members labeled C1, C2, and T in Figure D-1. One truss will be used at each meeting point of two panels. This means there will be truss structures spaced every 7 feet across the span of the emergency spillway. Each truss supports one-half the weight of each of the two panels that meet at its location; in total, each truss must be able to support loads equivalent to those acting on one full panel.

It is important to note that the C1 member of the truss is comprised of two individual beams. Figure D-3 shows this in a plan view of the truss. For the general force calculations, C1 represents these two individual members combined to provide a total horizontal force on the panel joint.

The worst (highest load) case for the elements of the truss comes from holding back the panel with a full 10 feet of head. In theory the head behind the emergency spillway should ever exceed the 8 feet of additional capacity from the new full pool, but in case of delay between a flood exceeding the operational spillway capacity and the release of the emergency spillway, 10 feet of head will be considered.

The relevant forces to the truss system are the hydrostatic pressure acting on the panel and the force of wind blowing from upstream to downstream. The force from downstream wind would be supported by the upstream compression member “S”, so it does not load the truss system. The Separation Device Design calculation sheets detail how the forces required of the truss elements were found. The results are summarized in Table D-1.

Table D-1. Truss component maximum force

Truss Component	Maximum Force (- is Compression, + is Tension)
C1 (Total force on both members)	-13.24 kip
C2	-22.93 kip
T	26.48 kip

The selection of structural members for the truss based on these forces is documented in the Structural Member Sizing calculation sheets. The two horizontal and one vertical compression members were chosen to be round HSS weathering steel beams. The tension member was chosen to be a weathering steel rod. These components are included in Table D-2.

The truss also includes the pin joint for the four structural members; the base plate and new concrete used at the bottom of the C2 beam; the 2-axis universal joints connecting the two C1 members to the panel; and the frangible nut attachment assembly which includes an anchor stud from the existing concrete, a threaded adapter from the stud for the nut to attach to, a base plate against which the nut is tightened, two bridge socket steel legs holding the base plate to a joint with the tension rod, a weatherproof enclosure around the nut built up from the base plate, and the frangible nut itself.

Frangible Nut Attachment

Figure 6-3 shows the detail for the frangible nut. It would tighten an anchor stud to the bridge socket base plate. The part of the anchor stud that threads into the nut would be a small adapter from the main anchor. The frangible nut can damage male threading when it triggers, so this adapter would be a sacrificial piece that could be replaced without needing to replace or rethread the main anchor stud.

At detonation the nut would split along the built-in fracture line. Both boosters fire when an electrical signal is sent to the nut, but if only one fires this is sufficient to break the nut. No longer fastened, the anchor stud adapter would slide out the through-hole in the base plate. The weatherproof enclosure, nut fragments, bridge socket, and tension rod would all be completely detached from the spillway.

Figure D-2 shows the frangible nut attachment and enclosure in context of the entire truss. Detonation wires to the nut would run through weatherproof gaskets in the enclosure to a conduit installed on the spillway face. This conduit would run the length of the emergency spillway. It would run spanwise below the flip bucket of the existing spillway to avoid being crosswise in the flow. Every 7 feet at each truss a conduit line would branch from the main span and profile the ogee face to provide a connection to the frangible nut.

Table D-2. Truss and frangible nut connection component list

Component	Description	Material	Size	Weight
Horizontal C1 Beam #1	Horizontal compression member connected to one panel's vertical support at the point where two panels meet.	Weathering Steel	Round HSS 2.375"x0.125"; 3.01 lb/ft; 7 ft	21 lb
Horizontal C1 Beam #2	Horizontal compression member connected to the other panel's vertical support at the point where two panels meet.	Weathering Steel	Round HSS 2.375"x0.125"; 3.01 lb/ft; 7 ft	21 lb
Vertical C2 Beam	Vertical compression member in the truss running from the pin joint to a newly shaped concrete wedge on the spillway face.	Weathering Steel	Round HSS 4.000"x0.188"; 7.66 lb/ft; 10.5 ft	80 lb
Angled Tension Rod "T"	Tension member holding the truss and panel in place. Runs from the pin joint to the frangible nut assembly.	Weathering Steel	1.25" Diameter Rod; 4.18 lb/ft; 9 ft	38 lb
Pin Joint	4-member pin connection for 3 round HSS and 1 steel rod, including end connection adapters for members.	Steel	4-member joint with central pin	N/A
C2 Base Plate	Base plate of vertical C2 member for contact with spillway face. Welded to C2, contact with spillway face maintained by compression only.	Steel	Sized to vertical compression member	5 lb
C2 Base Concrete	Concrete placed over ogee shape as base to vertical truss member. Shaped to prevent horizontal movement of C2 but allow rotation. Profile to direct water over gap left by C2 washing away.	Concrete	½ ft^3 per truss, 1 foot max of effective span	75 lb
C1 Beam #1 Universal Joint	2-axis joint welded to the vertical support beam of the panel and fitted with an adapter to the C1 horizontal member. 1 of 2 per panel.	Steel	Sized to handle individual C1 member compressive loads	N/A
C1 Beam #2 Universal Joint	2-axis joint welded to the vertical support beam of the panel and fitted with an adapter to the C1 horizontal member. 2 of 2 per panel.	Steel	Sized to handle individual C1 member compressive loads	N/A
Anchor Stud	Anchor drilled into existing spillway. Supports full tensile load of T member.	Steel	Sized for tension load	N/A

Threaded Adapter	Female threaded on one end to fit onto anchor stud allowing for a replacement if the original is damaged, and male threaded on the other end to fit into frangible nut.	Steel	Female end sized to anchor stud. Male end at 1.375" diameter.	N/A
Nut Base Plate	Steel plate against which the frangible nut tightens. Transfers the tension load from the frangible nut and threaded adapter to the bridge socket legs.	Steel	Sized for bending moment of transferring tension and to provide necessary clearances.	5 lb
Steel Bridge Socket Legs	Two rods linking tension rod to nut base plate. Fastened to base plate with nuts which are tightened to pre-load the truss at installation.	Steel	Sized for tensile load and shear.	N/A
Pin Joint to Tension Rod	Pinned connection between bridge socket legs and an adapter from the tension rod.	Steel	Sized for legs and 1.25" Diameter rod.	N/A
Weatherproof Enclosure	Box built up from the base plate to house the frangible nut.	TBD	Sized based on manufacturer clearance recommendations.	N/A
Frangible Nut	Nut designed to take the full tensile load of the truss and act as the planned failure point of the truss. Fractures on a pre-determined line with electrically actuated explosive boosters.	Various	Off-the-shelf 1.375" nut.	N/A
Conduit	Conduit running from the emergency release operation point to each of the frangible nuts. This includes conduit to the spillway, along the full 450-foot spillway located just below the flip bucket footer and branching at each frangible nut location to run along the face of the spillway up to the nut enclosure.	Conduit	Sized for length of spillway and chosen release operation point.	N/A

Upstream Components

As with all options requiring a semi-permanent barrier, the truss-supported panels would require a bubbler system running on the upstream face of the spillway to prevent ice loading on the panel face. This would consist of air tubing with holes to release bubbles which create an upward flow of relatively warmer water from below the ice. This would prevent ice from forming against the panels. The bubbler would be run with an approximately 7.5 horsepower (hp) air compressor and run the length of the emergency spillway.

Spaced every 7 feet along the full width of the emergency spillway there would be support members bracing the panels against potential downstream-to-upstream winds. This is the member labeled “S” in Figure D-1. The member would not be permanently attached to either the panel or the upstream bedrock so it would be able to wash away in an emergency release. It would be held in place by being wedged in compression between the bedrock base and the panel. The truss tension member would be pre-loaded to ensure that the horizontal C1 truss member is always in compression so the S member would never be required to provide tension support. It would either be unloaded resting against the panel or providing compression. Guides above and beside the attachment point would keep the S member from falling free when it would be unloaded.

In Figure D-4 the attachment point for the S member would be two angle beam sections. The upper would be welded to the panel, and the lower would be welded to the S member. They would be positioned so the S member could brace the panel without sliding up the face. On the bedrock, the S member would be similarly braced so it would be supported by the ground without sliding upstream. The maximum force would occur in a dewatered condition with 50 pounds per square foot wind loading from the upstream direction, which would produce **10.12 kips** of compressive force.

Panel Design

The picket panels making up the emergency spillway would be 10 feet total height and 7 feet wide measured from the centerlines of the gap between panels. See Figures D-4 and D-5 for typical section views of a panel.

The upstream face would be a steel skin plate. Backing this plate would be horizontal cross beams designed to take the full load of the water. All of the cross beam forces would be countered by vertical beams on either end of the panel. There would be a gap between adjacent vertical beams. This gap is included in the 7-foot per panel effective width. Reaction forces from the panel base, upstream support, and downstream truss support would all act on the vertical beams.

There would be 6 horizontal cross beams spaced every 2 feet up the full 10-foot height. Because the hydrostatic forces are much higher for the lower beams, two different beam sizes were used. The bottom 4 beams are a common angle beam size, and the top two beams are a different angle beam size. The beams were chosen to keep the same depth dimension for all 6 beams.

The skin plate also has different loading depending on depth and was sized in sections. There are 5 2-foot spans of skin plate between the cross beams. The bottom two form the thickest skin plate section. The next two spans up form a middle thickness skin plate section. The top span is its own section of the thinnest skin plate.

The vertical beams take the full load acting on the panel acting as transmitted from the cross beams. The reaction forces from the base connection to the spillway crest, the horizontal truss support, and the upstream compression member all act directly on the vertical panel beams. The beams were sized for the maximum moment at the fully watered, most extreme case. They would be angle beams, chosen to have the same depth dimension as the cross beams.

The basic requirements of the base connection of panel to spillway crest are to prevent horizontal movement either upstream or downstream, to prevent the vertical upward movement, and to allow the panel to rotate clockwise and disconnect from its base when the release is triggered.

A concept for preventing horizontal movement towards downstream can be seen in Figure D-4. The downstream side would be a vertical tab preventing horizontal movement. Figure D-6 shows the connection pieces designed to prevent vertical motion and horizontal motion in the upstream direction. Wedge pieces would be welded to each of the two vertical members on each panel. In the gap between panels, there would also be a custom footer anchored into the spillway crest. The tabs on the panels would be overhung by an angled top section of the custom piece. This would prevent vertical motion without the base of the panel also moving downstream, which is prevented by the vertical tabs anchored into the spillway. Upstream motion would be prevented by the overhang as well, in combination with a vertical face upstream of the tabs. The angle would be set to allow the panel to rotate around the downstream tab when the support from the truss goes away. The panel would then be free to wash away.

The remaining component is the method for spanning the gap between panels. The design calculations account for water behind a 7-foot span for each panel, but in Figure D-5 the physical design shows that the outer edge of the vertical support beams does not span the full 7 feet to allow room for the base connection. Because the structural members already account for the full weight, for now it is assumed that sections of steel plate will span the gap, attached strongly enough to resist water or wind pressure, but lightly enough that the panels would easily separate when the emergency spillway would be triggered. This detail will be finalized with the details of the base connection to account for the final width of the gap needed.

Table D-3. Panel and upstream component list

Component	Description	Material	Size	Weight
Upstream Compression S Beam	Angled beam bracing panel against upstream bedrock to provide support against wind loadings downstream to upstream.	Weathering Steel	Round HSS 2.875”x0.250”; 7.02 lb/ft; approx. 11 ft	77.22 lb
Bubbler System	Air tubing system with 7.5 hp air compressor. Active bubbler length running the span of the emergency spillway, total tubing running the full 450-foot spillway span.	Flexible piping and air compressor	400 feet bubble tubing; 370 feet self-sink tubing; 7.5 hp air compressor	N/A
Bedrock Bracing for S Member	10 kip compression bracing at the point where the angled compression member hits the bedrock. Includes some end cap for S member.	Concrete/ Steel	TBD	TBD

Angle Beam for S Member Panel Brace	Angle beam section welded to the panel-adjacent end of the S member.	Steel	Approx. 1 ft	5-10 lb
Angle Brace on Panel for S Member	Angle beam section welded to the panel where the S member makes contact.	Steel	Approx. 1 ft	5-10 lb
Bottom 4 Cross Beams	Lower 4 cross beams sized to take the larger loads.	Weathering Steel	5”x3”x7/16” Angle Beam; 4x7 ft; 11.3 lb/ft	316 lb
Top 2 Cross Beams	Upper 2 cross beams sized for lower loads.	Weathering Steel	5”x3”x1/4” Angle Beam; 2x7 ft; 6.6 lb/ft	92 lb
Bottom 2 Skin Plate Spans	Skin plate over bottom 4 feet of panel sized for larger loads.	Steel	3/8” Plate; 28 sq. ft; 15.32 psf	429 lb
Middle 2 Skin Plate Spans	Skin plate over 4 feet section spanning 4 feet from the base to 8 feet from the base. Sized for medium loads.	Steel	1/4" Plate; 28 sq. ft; 10.21 psf	286 lb
Top Skin Plate Span	Skin plate over the top 2 feet of the panel.	Steel	3/16” Plate; 14 sq. ft; 7.650 psf	107 lb
Vertical Panel Beams	Two beams, one on either side of the panel supporting the cross beams and transferring the panel forces to the support base and structures.	Weathering Steel	5”x3”x5/16” Angle Beam; 2x10 ft; 8.2 lb/ft	164 lb
Base Support	Custom metal base mounting piece and two metal tabs welded to the panel edges.	Steel	1 assembly per panel including the piece in the gap and the two total studs welded to each panel.	Approx. 50 lb

Table D-4. Partial-span rubber dam components

Component	Description	Material	Size	Weight
Rubber Dam Control System	Full control system, level sensors, and air compressor responsible for automatically managing the height of the rubber dam.	Various	N/A	N/A
Section 1 Rubber Dam	Section of rubber dam over the sloped 100-foot section of the existing spillway.	Rubber	8 feet high inflated; span of 112 feet including	N/A

			conical end sections	
Section 2 Rubber Dam	Section of rubber dam laid on part of the 350-foot flat section of the existing spillway.	Rubber	8 feet high inflated; span is TBD, but base case is 61-foot span including conical end sections	N/A
Dike Abutment	Sloped 1.5H:1V concrete abutment supporting the end of rubber dam Section 1 adjacent to the Dike.	Concrete	760 cubic feet (28.14 cubic yards)	114,000 lb
Center Pier	Concrete center pier, sloped 1.5H:1V on both sides, which sits in between rubber dam Sections 1 and 2.	Concrete	1,520 cubic feet (56.28 cubic yards)	228,000 lb
Far Abutment	Sloped 1.5H:1V concrete abutment supporting the end of rubber dam Section 2 adjacent to the emergency spillway pier.	Concrete	760 cubic feet (28.14 cubic yards)	114,000 lb
Emergency Spillway Pier	Rectangular 2-foot span pier separating the operational rubber dam spillway from the emergency flashboard spillway.	Concrete	317 cubic feet (11.73 cubic yards)	47,490 lb
Concrete Sill	Concrete slab extending back from the existing spillway crest to provide a base for the rubber dam.	Concrete	14 cubic feet per foot span. [175-foot base case gives 2,450 cubic feet]	945,000 lb
New Spillway Face Concrete	Concrete placed over existing spillway to turn the ogee shape to a broad-crested weir with 1:1 downstream slope.	Concrete	7 cubic feet per foot span [175-foot base case gives 1,225 cubic feet]	472,500 lb
Support Rods	Vertical members supporting the upstream end of the new concrete sill.	Epoxy-coated Steel	To be sized with further design details for the platform sill	N/A

Appendix C

C1-Rubber Dam application in very cold climate

C2- Bubbler System from Northern Alberta

C3-Frangible Nut Manufacturer’s Technical Data

Great Falls Sluice Way, Bathurst, New Brunswick

The original stoplog bays were replaced by a concrete ogee spillway, topped with a five-metre high rubber dam. A submerged gate allows operators to draw the reservoir below the ogee crest and a control system allows deflation in response to rising water levels. In winter and early spring rapid ice jam break-ups have historically caused flooding problems at the site.

Consulting firms RSW of Montreal and ADI of Fredericton were commissioned to finalize the rehabilitation concept, carry out detailed design engineering and supervise construction. The retrofit included objectives to:

achieve 50 years of service life without major repairs

conform to current criteria for stability and spillway capacity increase hydroelectric generation, if possible

minimize ice problems minimize environmental impact during construction be cost effective in construction and operation maintain access to the south abutment over the sluiceway, and prevent powerhouse flooding.

The alternative of installing a new rubber dam and submerged gate on the sluiceway proved to be the best way to meet these objectives. The rubber dam has the ability to be deflated, even in the event of power failures. This would allow the dam to pass ice, especially during floods caused by ice breakups. The cost of the rubber dam (10 to 15 per cent less than the next best alternative) was also an important factor.

Most rubber dam operators accept the risk of losing reservoir storage due to acts of vandalism. However, the risk of vandalism is not as high as it might seem, as the dam material is highly resilient and can be quickly repaired without special tools or training. Furthermore, the Great Falls complex is a run-of-river facility and the rubber dam retains only a small storage volume. The water volume could be replaced relatively quickly if the dam were to deflate.

Quote presented to:

MCMILLEN JACOBS ASSOCIATES

By:

Kenneth Rourke

On : 2020-01-28

570 Knowlton Rd., Lac-Brome, Quebec J0E 1V0
450 243-0976 - info@canadianpond.ca

Canadianpond.ca

Canadian Pond.ca Products Ltd.
570 Knowlton Rd.
Lac-Brome, Quebec J0E 1V0
450 243-0976
www.canadianpond.ca

Quote	1965
Date	28-Jan-2020
Salesperson	Kenneth Rourke
Terms	Payable in advance
Shipped	BEST WAY
Reference	1965
Expiration	27-Feb-2020

Client:

,

Contact : Leif Fredericks

Shipped to: 2873184
McMillen Jacobs Associates
1471 Shorline Drvie, Suite 100
Boise, ID, ID83702
(208) 287-3184
Contact :

Products and services offered by Canadianpond.ca Products Ltd

Project description

PRELIMINARY BUDGET ESTIMATE 27,900.00

BASED ON THE FOLLOWING SCOPE OF SUPPLY
Total of 400' of 3/4" of Bubble Tubing®
Total of 370' of 3/4" Torpedo self-sinking Tubing
6-line manifold
Harware including valves and accessories
7.5HP (5.5kW) Busch Compressor, 220V, 1ph, 57CFM, 31 psig
Textured Painted Cabinet for Rotary Claw Comp.
Heater, inlet filter, oil

EXCLUSIONS
Installation
Shipping
On-site Supervision
Insolation of exposed lines

Please note that our complete legal name for your Purchase Order is Canadian Pond.ca Products Ltd.

Sub-total :	$27,900.00
GST - HST: 89011 6908 RT 0001	$0.00
QST: 1203144331 TQ 0001	$0.00
TOTAL :	$27,900.00

Canadianpond.ca Product Ltd General Sales Terms & Conditions

Agreement
These terms and conditions constitute the entire agreement and shall supersede and replace all prior oral and written agreements and can be modified or cancelled only by written agreement signed by the Buyer and Canadianpond.ca Products Ltd (CPP). The Buyer expressly waives all terms and provisions in any Buyer's correspondence, purchase order or other forms which negate, limit, extend or otherwise conflict with this agreement. The Buyer's acceptance must be based solely upon the provision of this agreement.

Disclaimer
Canadianpond.ca Products Ltd, its directors and employees disclaim all responsibility and offer no warranty, expressed or implied, as to the accuracy or reliability of the information contained in this proposal. This proposal is made to the best of their knowledge, according to the deadlines and according to the information provided by the client and its stakeholders. In no event shall Canadianpond.ca Products Ltd., its directors or its employees be liable for any damages, losses, claims or obligations of any kind, whether direct or indirect, arising out of or relating to the use of the information presented in this proposal. CPP, its directors and its employees cannot be held responsible for circumstances beyond their control or for any changes made by the client and its stakeholders or related to environmental conditions.

Canadianpond Products Ltd. is not responsible for the lack of performance and issues related to the wrong use of the product or its components by the purchaser, its agents or its user.

Design disclaimer
The plans and drawings included in CPP's offers are preliminary, based on the information provided by the client and should not be used for the construction or installation of the system. All information contained in these documents is property of CPP. Duplication or transmittal of these documents is strictly prohibited without the prior written consent of CPP. The results and performances proposed in these documents are theoretical and may be influenced by many factors unknown during the design phase. Systems are designed based on the information provided by the client. Changes or errors in the information provided originally could result in a lack of performance of the system and might need to be revised to address the issue. This can create fees and delays that are out of CPP's control. CPP is not responsible if the system is not installed and used as per our recommendation.

Electrical System Components
Installation and electrical connection are not included and must be performed in accordance with existing standards in the area where the system is installed, by a certified electrician. Some assemblies, including cabinet and compressors, are not certified, unless specified. Specific certification may be required and is not included in our service unless requested by the client.

Components and Assembly of Pressurized Parts
Standard 316 stainless steel Class 150 components are used, unless specified. Any pre-assembly of pressurized parts made at CPP's workshop are not certified. Depending on the province and the country, some certifications may have to be executed according to the client's requirements. This certification is the responsibility of the customer and is not included in our service.

Installation and product certificate of authorization
Some of our products installation may require a certificate of authorization from the governing authorities. It is the customer's responsibility to check with the authorities.

Installation service
Installation service can be provided when required. The offer will depend on the scope of work and requirements of the service required. Additional fees may apply for equipment installations in the presence of ice. Consult CPP for more details.

Payment terms
For all sales performed within Canada, all first orders are payable in advance. A credit application can then be opened for future purchases, conditional to approval, and for a minimum yearly purchase of 5000$. For all orders over 25 000$, even with approved terms, a deposit of 30% minimum will be requested, payable in advance.

For all sales outside Canada, orders are always payable in advance. Purchases of 5 000$ and more may be insured by EDC (Export Development Canada), pending approval. If approved, a 2% administration fee will be applied to the total of the invoice. Please note that the first order remains payable in advance.

As soon as the complete payment of the order is issued by the client, the products are the proprietary of the client.

Payment method

- In Canada, our preferred method of payment is EFT-Electronic Fund Transfers, but we also accept Certified cheques, Interac e-transfers and Credit Cards (VISA, MasterCard and American Express - max of 5000$).
- USA and International clients can pay by Wire Transfers or by Credit Card (VISA and MasterCard only - max of 5000$).

Taxes and Duties

- For sales in Canada, taxes related to the province of destination are included in our quotes.
- For sales in the United States, custom and duty fees are normally included and specified in our quotes. State taxes are payable at delivery, if applicable.
- For international sales, taxes and import duties are not included in our quotation and are payable to customs at delivery.

Delivery terms

Unless specified and for all destinations:

- Shipments in Canada considers FCA - 570 Knowlton Rd, Lac-Brome (Free Carrier) INCOTERM® 2010 for shipments with included shipping fees or "EXW - 570 Knowlton Rd, Lac-Brome" (Ex Works) INCOTERM® 2010 for orders picked up or not shipped by Canadianpond.ca Product Ltd (Shipped on client's transporter account).
- United States shipments normally consider DDP - Agreed Destination (Delivered, Duty Paid) INCOTERM® 2010.
- International shipments normally consider CPT - Agreed Port or Airport Terminal (Carriage Paid To) INCOTERM® 2010. Option on DAP- Agreed destination (Delivered At Place) INCOTERM® 2010 also possible when requested.

In all cases, at the exception of the deliveries in the United States with DDP INCOTERMS®, the responsibility for the delivery to the destination in good condition rests with the carrier. CPP will assist the buyer insofar as is reasonable in securing satisfactory adjustment of claims against the carrier, however, all claims for loss or damage must be made by the buyer against the carrier. CPP shall not be responsible for such loss or damage. CPP shall not be liable to the buyer or deemed to be in breach of the agreement by reason of any delay in performing or any failure to perform any of CPP's obligation in relation to the goods or any related services if the delay or failure is due to any causes beyond CPP's reasonable control.

Delivery dates

Every order will be made to meet the dates quoted however, any dates quoted for delivery of the goods are approximate only and CPP shall not be liable for any delay in delivery of the goods, however caused.

Shipping fees

CPP ships orders with the most efficient and cost-effective transporter, able to provide the best service based on the destination and size of the shipment. Shipping fees are normally considering a single and complete shipment. If an order must be split in two shipments, the shipping fees of both shipments will be charged. Orders shipped under a client's account will be charged a 15$ administration fee.

Warranties

Canadianpond.ca Products Ltd. warrants that all the goods sold are of merchantable quality and free of manufacturing defects at the time of delivery. The Bubble Tubing® and Torpedo™ tubing have a 1-year manufacturing warranty. For other products, the warranty provided by the manufacturer of the product applies. To claim a warranty, please contact CPP.

Limitation of liability

Under any circumstances should Canadianpond.ca Products Ltd. be liable for any prospective profit of for any special, indirect, consequential, punitive or exemplary damages such as but not limited to injuries to persons, damage or loss of other property or equipment, loss of profit or revenue, cost of capital, cost of purchased or replacement of goods or claims for service interruption. Should CPP, at its own discretion, determine that a product is defective, then CPP may replace or repair any defective goods.

Return of goods

No product returns are accepted prior to a written authorization from CPP. The returned of goods must be done in its original packaging and clearly show a Return Authorization Number provided by CPP to be accepted. Goods that have been in the water are not returnable nor credited. Restocking fees might apply for returned goods based on the nature of the product and the condition the item is found upon its return.

Storage

Storage fees can be invoiced if one or many items are on hold for an answer from the client or to be picked up by the client for a period above 30 days.

Title, Risk of loss and security interest

Risk of damage to or loss of the goods shall pass to the Buyer upon due tender of goods for delivery at the agreed point. Notwithstanding delivery and the passing of the risk in the goods, the property in the goods shall not pass to the Buyer until CPP has received payment in full. Until such time as the property in the goods passes to the Buyer, CPP shall be entitled at any time to require the Buyer to deliver the goods to CPP or to enter the premises of the Buyer or its agents where the goods are stored and to repossess the goods.

Force Majeure

CPP shall not be liable for any delay in the delivery of orders, due in whole or in part, directly or indirectly, to fire, acts of god, strike, shortage of raw materials, supplies or components, retooling, upgrading of technology, delays of carrier, embargo, government order or directive or any other circumstance beyond the control of Canadianpnod.ca Products Ltd.

Miscellaneous

The headings to each section are inserted for convenience of reference only and do not form part of this agreement. The parties hereto agree that this document be written in the English language.

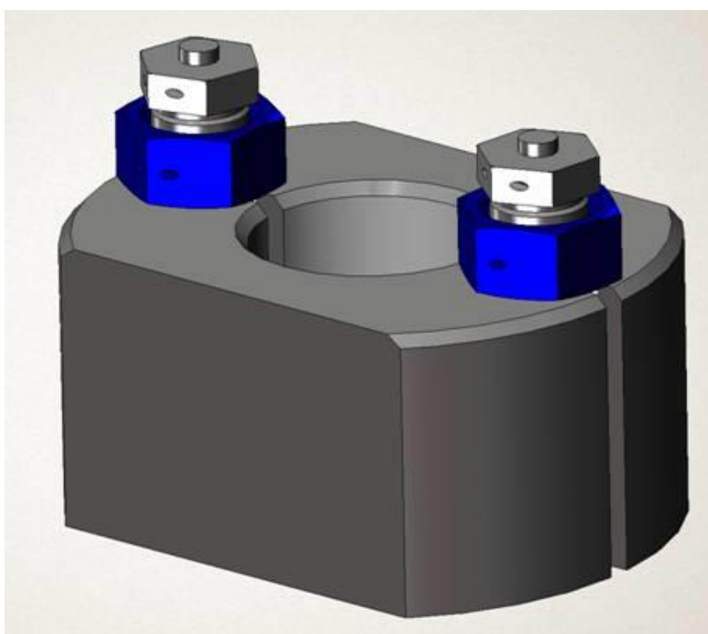
TECHNICAL PROPOSAL

948CA1302520-901 FRANGIBLE NUT (LIVE)
948CA1302520-601 FRANGIBLE NUT (INERT)

Submitted to:

By:
Pacific Scientific Energetic Materials Co.
7073 West Willis Road
Chandler, Arizona 85226

MARCH 23, 2011



1.0 PSEMC EXPERIENCE SUMMARY

1.1 PSEMC PN 108157 FRANGIBLE NUT ASSEMBLY (X-33)

The Frangible Nut design presented is a direct derivative of the NASA X-33 Frangible Nut (Figure 1) used in the launch vehicle hold down system prior to launch. The primary design attributes carried forward are summarized:

- Materials including heat treat conditions/mechanical properties
- Separation plane design
- Separate Booster with type and quantity of explosive materials

1.2 PSEMC FRANGIBLE NUT FAMILY

Pacific Scientific Energetic Materials Co. (PSEMC) has developed and qualified several variants of the Frangible Nut design including the NASA Space Shuttle Hold Down System, the Atlas V Hold Down System, and the Delta IV Hold Down System.

FAMILY OF FRANGIBLE NUTS

NASA Shuttle Hold Down Frangible Nut
Thread: 3.50-8 BUTT-2B
P/N 10306-004
Proof Load = 1,144,000 lbs

Atlas V Hold Down Frangible Nut
Thread: 2.00-8 BUTT-2B
P/N 108727
Proof Load = 302,000 lbs

Delta IV Hold Down Frangible Nut
Thread: 2.00-8 BUTT-2B
P/N 108415-1
Proof Load = 302,000 lbs

NASA X-33 Hold Down Frangible Nut
Thread: 1.375-12 BUTT-2B
P/N 108270
Proof Load = 197,000 lbs

PSEMC provides a complete kit including the frangible nut, stud, washers, booster assemblies, supernut, upper and lower containers as well as the associated hardware.

2.2 Nut Body Features

The body design will incorporate all critical dimensional attributes of the heritage NASA X-33 nut, most importantly those features at the separation plane (web). Material of construction is heat treated to an ultimate tensile strength of 190 to 210 KSI. This material selection is important as the body material, hardness, and tensile strength properties are key attributes of separation performance. This is also the material of choice for the entire family of frangible nuts. Proper material properties also assure the structural integrity. Final frangible nut load requirements will need to be finalized prior to qualification. The materials and geometry used in the proposed design have been proven by successful tests and launches.

2.3 Booster Assembly

The Booster Assembly houses a pressed RDX explosive. It interfaces to an ETL End-tip and the Nut Body. The output of the End Tip is a flyer plate impact to the bulkhead creating a shock wave into the RDX separation charge. The End Tip input end is sealed from environmental conditions by an integral bulkhead designed to withstand environmental conditions by providing a maximum $1x10^{-6}$ cc/sec/He leak rate at a one (1) atmosphere pressure differential. The output end is sealed using a steel closure TIG welded to the body to withstand the same environmental conditions.

2.3.1 Closure Disk

The Closure Disk is welded to the booster housing in order to withstand environmental conditions by providing a maximum $1x10^{-6}$ cc/sec/He leak rate at a one (1) atmosphere pressure differential.

2.3.2 Separation Charge

The Separation charge is RDX explosive which is a high velocity/high energy secondary explosive material. This material has a long history of usage in many explosive applications due to relative safe handling, stability and long service life, just to name a few of its positive attributes.

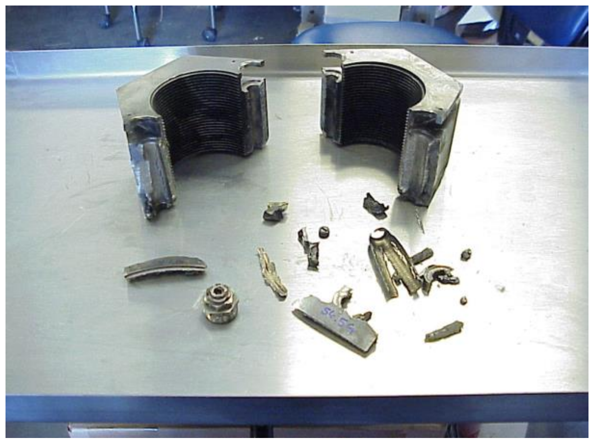
2.0 948CA1302520-901 FUNCTION

3.1 <u>948CA1302520-901 Frangible Nut Function</u>

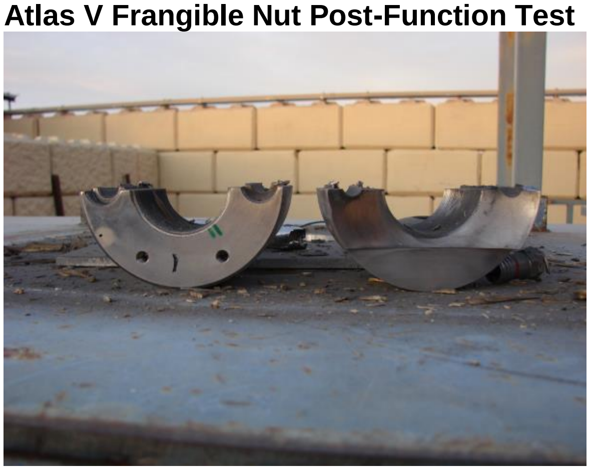
The PSEMC 948CA1302520-501 Frangible Nut is operated by an Explosive Transfer Line (ETL) End-tip threaded into the Booster. The flyer plate (bottom of end-tip) output from the ETL End-tip impacts the integral bulkhead in the booster housing sending a shock wave igniting the RDX output charge powder column. When the RDX booster charge detonates, it transfers shock waves through a web feature machined into the nut geometry and, along with extremely high internal pressures, producing a dependable clean fracture of the nut body at the separation plane. Two examples are shown below.

Frangible Nut Post-Function Tests

Atlas V Frangible Nut Post-Function Test



Shuttle Frangible Nut Post-Function Test



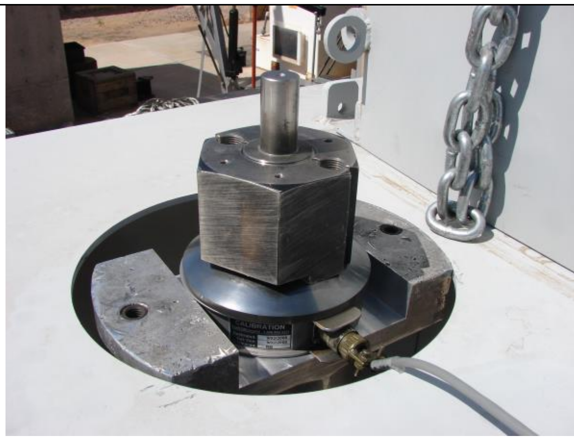
4.0 Capabilities and Related Experience

Based on the programs with the Shuttle, Atlas, Delta, and X-33 Frangible Nuts, PSEMC has the capability, equipment, and experience to develop the Orion Frangible Nut. The description and photos below show the test equipment used to apply the loads and function testing.

Pretensioner used for Pre-Load, Axial Load, and Proof Load

Test Setup to Validate Moment Load (e.g. Delta IV)

Functional Test Setup



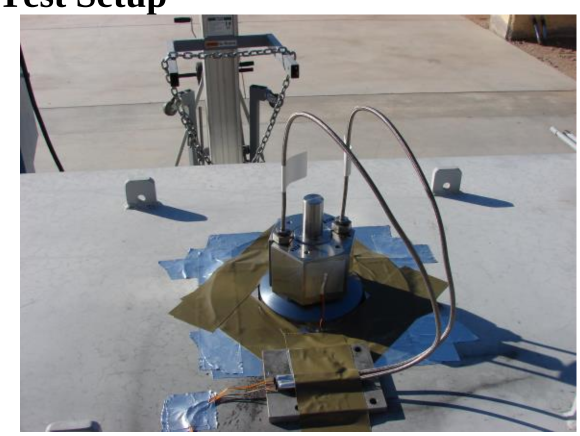
Measure Pre-Load Prior to Function

Test Setup – Temperature Conditioning

Pretensioner Used For Function Testing

Function Test Setup

Single Booster Firing



Dual Booster Firing

Econ Analysis Runs- Results vs. Bond Rate at $3/gal

Fuel Inflation Rate	1.00%		O&M Inflation %		3.00%			
Loan Rate	3.00%		Diesel Heat Rate (kW/gal)		14.8			
Bond Term (yrs)	30		Diesel Bulk Rate $/gal		3			
Construction Inflation %	4.50%							
Alternative	2020 Constr. Cost	2028 Constr. Cost	Annual Cost of Construction	Annual Benefit MWh	Benefit Diesel (gal)	Annual Maint. $	Fuel Price for B/cost =1	Present Value 30 yr Analysis
2 ft Concrete Raise	($913,500)	($1,299,000)	($66,274)	$691	$46,689	$0	$1.47	$3,006,531
4 ft rubber Dam	($5,898,600)	($8,388,400)	($427,970)	$1,382	$93,378	($5,000)	$4.53	($3,495,037)
8 ft Compound Spillway	($5,429,400)	($7,721,100)	($393,925)	$2,759	$186,419	($10,000)	$2.16	$7,295,773

Fuel Inflation Rate	1.00%		O&M Inflation %		3.00%			
Loan Rate	4.00%		Diesel Heat Rate (kW/gal)		14.8			
Bond Term (yrs)	30		Diesel Bulk Rate $/gal		3			
Construction Inflation %	4.50%							
Alternative	2020 Constr. Cost	2028 Constr. Cost	Annual Cost of Construction	Annual Benefit MWh	Benefit Diesel (gal)	Annual Maint. $	Fuel Price for B/cost =1	Present Value 30 yr Analysis
2 ft Concrete Raise	($913,500)	($1,299,000)	($75,121)	$691	$46,689	$0	$1.47	$2,732,265
4 ft rubber Dam	($5,898,600)	($8,388,400)	($485,102)	$1,382	$93,378	($5,000)	$4.53	($5,266,131)
8 ft Compound Spillway	($5,429,400)	($7,721,100)	($446,512)	$2,759	$186,419	($10,000)	$2.16	$5,665,570

Fuel Inflation Rate	1.00%		O&M Inflation %		3.00%			
Loan Rate	5.00%		Diesel Heat Rate (kW/gal)		14.8			
Bond Term (yrs)	30		Diesel Bulk Rate $/gal		3			
Construction Inflation %	4.50%							
Alternative	2020 Constr. Cost	2028 Constr. Cost	Annual Cost of Construction	Annual Benefit MWh	Benefit Diesel (gal)	Annual Maint. $	Fuel Price for B/cost =1	Present Value 30 yr Analysis
2 ft Concrete Raise	($913,500)	($1,299,000)	($84,502)	$691	$46,689	$0	$1.47	$2,441,469
4 ft rubber Dam	($5,898,600)	($8,388,400)	($545,677)	$1,382	$93,378	($5,000)	$4.53	($7,143,970)
8 ft Compound Spillway	($5,429,400)	($7,721,100)	($502,269)	$2,759	$186,419	($10,000)	$2.16	$3,937,114

2 Ft	8ft	8ft	2 ft concrete and Vertical Gates in Control Section	Quantity	Unit	Unit Price	Option 1 Price	Altern1 Total	Vert Gate Price	Vert Gate Total	Vert Gate Price	Vert Gate Total
			General Requirements/Mobilization/Demobilization					**$ 152,230.56**		**$ 1,071,864.43**		**$ 1,081,112.14**
x	x	x	General Requirements	15%	%		$ 91,338.33		$ 643,118.66		$ 648,667.28	
x	x	x	Mobilization / Demobilization	10%	%		$ 60,892.22		$ 428,745.77		$ 432,444.86	
			Site Prep and Access Roads					**$ 38,200.00**		**$ 38,200.00**		**$ 38,200.00**
x	x	x	Clear and Grub Laydown Area	5500.00	SY	$ 1.50	$ 8,250.00		$ 8,250.00		$ 8,250.00	
x	x	x	Import Pit Run Fill for Access Road	350.00	TN	$ 50.00	$ 17,500.00		$ 17,500.00		$ 17,500.00	
x	x	x	Import CSBC for Laydown Area and Access Road	50.00	TN	$ 75.00	$ 3,750.00		$ 3,750.00		$ 3,750.00	
x	x	x	Grading Access on Upstream Side of Dam	1000.00	SY	$ 1.50	$ 1,500.00		$ 1,500.00		$ 1,500.00	
x	x	x	Place Imported Fill for Access Rd (over Dam)	360.00	CY	$ 20.00	$ 7,200.00		$ 7,200.00		$ 7,200.00	
			Uplift Prevention Measures (Cutoff Wall/Rock Anchors)					**$ -**		**$ 172,527.51**		**$ 229,518.34**
		x	Rock Excavation	66.67	CY	$ 100.00					$ 6,666.67	
		x	Drill and Dowel Rebar	450.00	EA	$ 10.00					$ 4,500.00	
		x	Form/Rebar/Pour Concrete Seal Strip	66.67	CY	$ 1,250.00					$ 83,333.33	
			Mobilize Drill Rig (Drains)	1.00	LS	$ 15,000.00						
			Drill Drains Through Dam	10000.00	LF	$ 15.00						
	x		Upstream Side - Drill Rock Anchors (No Concrete Seal)	612.00	LF	$ 183.87			$ 112,527.51			
		x	Upstream Side - Drill Rock Anchors (Use Concrete Seal)	408.00	LF	$ 183.87					$ 75,018.34	
	x		Procure & Install Rock Anchor Attachment Plates (No Concrete Seal)	60.00	EA	$ 1,000.00			$ 60,000.00			
		x	Procure & Install Rock Anchor Attachment Plates (USE Concrete Seal)	40.00	EA	$ 1,000.00					$ 40,000.00	
			Dam Raise Concrete (2' Raise)					**$ 570,722.22**		**$ -**		**$ -**
x			Drill and Dowel Rebar	6000.00	EA	$ 15.00	$ 90,000.00					
x			Form/Rebar/Pour Concrete	281.48	CY	$ 1,500.00	$ 422,222.22					
x			Finish Concrete Slope	5850.00	SF	$ 10.00	$ 58,500.00					
			7 vertical gates, 15'-4" (125' Length includes piers)					**$ -**		**$ 2,930,354.41**		**$ 2,930,354.41**
	x	x	Rock Excavation for Fdn.	23.70	CY	$60.00			$ 1,422.22		$ 1,422.22	
	x	x	Procure and Install Structural Steel	20.00	TN	$10,000.00			$ 199,990.00		$ 199,990.00	
	x	x	Form/Rebar/Pour Concrete Piers	59.50	CY	$1,500.00			$ 89,250.00		$ 89,250.00	
	x	x	Form/Rebar/Pour Concrete Dam Slope, DS	86.53	CY	$1,500.00			$ 129,791.67		$ 129,791.67	
	x	x	Finish Concrete Sill	1300.00	SF	$5.00			$ 6,500.00		$ 6,500.00	
	x	x	Procure & Install Vertical Gate Mech & Electrical	7.00	EA	$312,550.00			$ 2,187,850.00		$ 2,187,850.00	
	x	x	Procure Electrical Equipment	1.00	EA	$2,150.52			$ 2,150.52		$ 2,150.52	
	x	x	Mechanical Building	300.00	SF	$200.00			$ 60,000.00		$ 60,000.00	
	x	x	Site Electrical	1.00	Ea	$119,000.00			$ 119,000.00		$ 119,000.00	
	x	x	Stabilization Anchors into rock below piers	112.00	LF	$1,200.00			$ 134,400.00		$ 134,400.00	
			Panel System (48 Panels) (325 ft)					**$ -**		**$ 1,165,154.55**		**$ 1,165,154.55**
	x	x	Fabricate Picket System	72.00	TN	$ 7,000.00			$ 504,000.00		$ 504,000.00	
	x	x	Procure Picket Attachment System (Frangible nuts and Anchors)	48	EA	$ 5,000.00			$ 240,000.00		$ 240,000.00	
	x	x	Rock Excavation	14.01	CY	$ 100.00			$ 1,400.67		$ 1,400.67	
	x	x	Form/Rebar/Pour Foundations	14.01	CY	$ 1,250.00			$ 17,508.42		$ 17,508.42	
	x	x	Drill and Install Anchoring System	48.00	EA	$ 1,500.00			$ 72,000.00		$ 72,000.00	
	x	x	Install Picket System	48.00	EA	$ 2,500.00			$ 120,000.00		$ 120,000.00	
	x	x	Install Electrical/Control Wiring	1.18	LS	$ 50,000.00			$ 59,090.91		$ 59,090.91	
	x	x	Grout Base of Dam	70.91	CY	$ 1,000.00			$ 70,909.09		$ 70,909.09	
	x	x	Procure Bubbler System (Quote + Freight Allowance $5000)	1.18	LS	$ 32,900.00			$ 38,881.82		$ 38,881.82	
	x	x	Install Bubbler System (Onsite crews +mfr supervision)	1.18	LS	$ 35,000.00			$ 41,363.64		$ 41,363.64	
								$ -		**$ -**		**$ -**

			Project Subtotal (Direct Costs Only) =					$ 608,922.22		$ 4,287,457.72		$ 4,324,448.55
			Project Subtotal (Direct and Indirect) =					$ 761,152.78		$ 5,359,322.16		$ 5,405,560.69
			Contingency (20%)					$ 152,230.56		$ 1,071,864.43		$ 1,081,112.14
			Grand Total =					913,400		$ 6,431,186.59		$ 6,486,672.83
			Accuracy Range									
			+50%					$ 1,370,100.00		$ 9,646,779.88		$ 9,730,009.24
			-30%					$ 639,380.00		$ 4,501,830.61		$ 4,540,670.98

2 ft	4 ft	4 ft	2 ft Concrete and 4 ft Rubber Dam	Quantity	Unit	Unit Price	Option 1 Price	Option 1 Total	4'-Rubber Dam Price	4'-Rubber Dam Total	4'-Rubber Dam Price	4'-Rubber Dam Total
			General Requirements/Mobilization/Demobilization					**$ 152,230.56**		**$ 973,772.11**		**$ 983,097.66**
x	x	x	General Requirements	15%	%		$ 91,338.33		$ 584,263.27		$ 589,858.59	
x	x	x	Mobilization / Demobilization	10%	%		$ 60,892.22		$ 389,508.84		$ 393,239.06	
			Site Prep and Access Roads					**$ 38,200.00**		**$ 38,200.00**		**$ 38,200.00**
x	x	x	Clear and Grub Laydown Area	5500.00	SY	$ 1.50	$ 8,250.00		$ 8,250.00		$ 8,250.00	
x	x	x	Import Pit Run Fill for Access Road	350.00	TN	$ 50.00	$ 17,500.00		$ 17,500.00		$ 17,500.00	
x	x	x	Import CSBC for Laydown Area and Access Road	50.00	TN	$ 75.00	$ 3,750.00		$ 3,750.00		$ 3,750.00	
x	x	x	Grading Access on Upstream Side of Dam	1000.00	SY	$ 1.50	$ 1,500.00		$ 1,500.00		$ 1,500.00	
x	x	x	Place Imported Fill for Access Rd (over Dam)	360.00	CY	$ 20.00	$ 7,200.00		$ 7,200.00		$ 7,200.00	
			Uplift Prevention Measures (Cutoff Wall/Rock Anchors)					**$ -**		**$ 174,395.63**		**$ 211,697.82**
		x	Rock Excavation	66.67	CY	$ 100.00					$ 6,666.67	
		x	Drill and Dowel Rebar	450.00	EA	$ 10.00					$ 4,500.00	
		x	Form/Rebar/Pour Concrete Seal Strip	66.67	CY	$ 1,250.00					$ 83,333.33	
	x		Upstream Side - Drill Rock Anchors (No Concrete Seal)	459.00	LF	$ 183.87			$ 84,395.63			
		x	Upstream Side - Drill Rock Anchors (Use Concrete Seal)	229.50	LF	$ 183.87					$ 42,197.82	
	x		Procure & Install Rock Anchor Attachment Plates (No Concrete Seal)	90.00	EA	$ 1,000.00			$ 90,000.00			
		x	Procure & Install Rock Anchor Attachment Plates (USE Concrete Seal)	75.00	EA	$ 1,000.00					$ 75,000.00	
			Dam Raise Concrete (2' Raise)					**$ 570,722.22**		**$ -**		**$ -**
x			Drill and Dowel Rebar	6000.00	EA	$ 15.00	$ 90,000.00					
x			Form/Rebar/Pour Concrete	281.48	CY	$ 1,500.00	$ 422,222.22					
x			Finish Concrete Slope	5850.00	SF	$ 10.00	$ 58,500.00					
			4-Foot Rubber Bladder Dam (450' Length)					**$ -**		**$ 3,682,492.81**		**$ 3,682,492.81**
	x	x	Rock Excavation for Fdn.	60.95	CY	$60.00			$ 3,657.14		$ 3,657.14	
	x	x	Procure and Install Support Columns	9.90	TN	$10,000.00			$ 99,000.00		$ 99,000.00	
	x	x	Form/Rebar/Pour Concrete Elevated Slab & Fdn.	248.45	CY	$1,500.00			$ 372,678.57		$ 372,678.57	
	x	x	Form/Rebar/Pour Concrete Dam Slope, DS	222.50	CY	$1,500.00			$ 333,750.00		$ 333,750.00	
	x	x	Form/Rebar/Pour Concrete Center Piers	52.00	CY	$1,500.00			$ 78,000.00		$ 78,000.00	
	x	x	Finish Concrete Slope	2925.00	SF	$5.00			$ 14,625.00		$ 14,625.00	
	x	x	Procure Rubber Bladder Dam	450.00	LF	$1,949.88			$ 877,443.90		$ 877,443.90	
	x	x	Procure Mechanical and Electrical Equipment	450.00	LF	$2,150.52			$ 967,734.74		$ 967,734.74	
	x	x	Mechanical Building	200.00	SF	$200.00			$ 40,000.00		$ 40,000.00	
	x	x	Install Rubber Bladder Dam	450.00	LF	$1,070.69			$ 481,810.34		$ 481,810.34	
	x	x	Install Mechanical and Electrical for Rubber Dam	450.00	LF	$919.54			$ 413,793.10		$ 413,793.10	
			Picket System (0 Picket Panels)					**$ -**		**$ -**		**$ -**
								$ -		**$ -**		**$ -**
			Project Subtotal (Direct Costs Only) =					**$ 608,922.22**		**$ 3,895,088.44**		**$ 3,932,390.62**
			Project Subtotal (Direct and Indirect) =					**$ 761,152.78**		**$ 4,868,860.55**		**$ 4,915,488.28**
			Contingency (20%)					**$ 152,230.56**		**$ 973,772.11**		**$ 983,097.66**
			Grand Total =					**$ 913,383.33**		**$ 5,842,632.66**		**$ 5,898,585.94**
			Accuracy Range									
			+100%					**$ 1,826,766.67**		**$ 11,685,265.33**		**$ 11,797,171.87**
			-50%					**$ 456,691.67**		**$ 2,921,316.33**		**$ 2,949,292.97**

Solomon Gulch Dam Raise
Engineer's Estimate

2 ft 1	8ft 2	8ft 3	2 ft Concrete and 8 ft rubber dam in Control Section	Quantity	Unit	Unit Price	Option 1 Price	Option 1 Total	Option 2 Price	Option 2 Total	Option 3 Price	Option 3 Total
			General Requirements/Mobilization/Demobilization					**$ 152,230.56**		**$ 890,640.97**		**$ 904,888.67**
x	x	x	General Requirements	15%	%		$ 91,338.33		$ 534,384.58		$ 542,933.20	
x	x	x	Mobilization / Demobilization	10%	%		$ 60,892.22		$ 356,256.39		$ 361,955.47	
			Site Prep and Access Roads					**$ 38,200.00**		**$ 38,200.00**		**$ 38,200.00**
x	x	x	Clear and Grub Laydown Area	5500.00	SY	$ 1.50	$ 8,250.00		$ 8,250.00		$ 8,250.00	
x	x	x	Import Pit Run Fill for Access Road	350.00	TN	$ 50.00	$ 17,500.00		$ 17,500.00		$ 17,500.00	
x	x	x	Import CSBC for Laydown Area and Access Road	50.00	TN	$ 75.00	$ 3,750.00		$ 3,750.00		$ 3,750.00	
x	x	x	Grading Access on Upstream Side of Dam	1000.00	SY	$ 1.50	$ 1,500.00		$ 1,500.00		$ 1,500.00	
x	x	x	Place Imported Fill for Access Rd (over Dam)	360.00	CY	$ 20.00	$ 7,200.00		$ 7,200.00		$ 7,200.00	
			Uplift Prevention Measures (Cutoff Wall/Rock Anchors)					**$ -**		**$ 172,527.51**		**$ 229,518.34**
		x	Rock Excavation	66.67	CY	$ 100.00					$ 6,666.67	
		x	Drill and Dowel Rebar	450.00	EA	$ 10.00					$ 4,500.00	
		x	Form/Rebar/Pour Concrete Seal Strip	66.67	CY	$ 1,250.00					$ 83,333.33	
			Mobilize Drill Rig (Drains)	1.00	LS	$ 15,000.00						
			Drill Drains Through Dam	10000.00	LF	$ 15.00						
	x		Upstream Side - Drill Rock Anchors (No Concrete Seal)	612.00	LF	$ 183.87			$ 112,527.51			
		x	Upstream Side - Drill Rock Anchors (Use Concrete Seal)	408.00	LF	$ 183.87					$ 75,018.34	
	x		Procure & Install Rock Anchor Attachment Plates (No Concrete Seal)	60.00	EA	$ 1,000.00			$ 60,000.00			
		x	Procure & Install Rock Anchor Attachment Plates (USE Concrete Seal)	40.00	EA	$ 1,500.00					$ 60,000.00	
			Dam Raise Concrete (2' Raise)					**$ 570,722.22**		**$ -**		**$ -**
x			Drill and Dowel Rebar	6000.00	EA	$ 15.00	$ 90,000.00					
x			Form/Rebar/Pour Concrete	281.48	CY	$ 1,500.00	$ 422,222.22					
x			Finish Concrete Slope	5850.00	SF	$ 10.00	$ 58,500.00					
			Rubber Bladder Dam 8' Diam. (175' Length)					**$ -**		**$ 2,297,936.35**		**$ 2,297,936.35**
	x	x	Rock Excavation for Fdn.	23.70	CY	$60.00			$ 1,422.22		$ 1,422.22	
	x	x	Procure and Install Support Columns	3.85	TN	$10,000.00			$ 38,500.00		$ 38,500.00	
	x	x	Form/Rebar/Pour Concrete Elevated Slab & Fdn.	193.84	CY	$1,500.00			$ 290,763.89		$ 290,763.89	
	x	x	Form/Rebar/Pour Concrete Dam Slope, DS	86.53	CY	$1,500.00			$ 129,791.67		$ 129,791.67	
	x	x	Finish Concrete Slope	1300.00	SF	$5.00			$ 6,500.00		$ 6,500.00	
	x	x	Procure Rubber Bladder Dam	175.00	LF	$3,899.75			$ 682,456.37		$ 682,456.37	
	x	x	Procure Mechanical and Electrical Equipment	175.00	LF	$2,150.52			$ 376,341.29		$ 376,341.29	
	x	x	Mechanical Building	200.00	SF	$200.00			$ 40,000.00		$ 40,000.00	
	x	x	Install Rubber Bladder Dam	175.00	LF	$2,141.38			$ 374,741.38		$ 374,741.38	
	x	x	Install Mechanical and Electrical for Rubber Dam	175.00	LF	$919.54			$ 160,919.54		$ 160,919.54	
	x	x	end pier concrete correction (4-24-20)	131.00	CY	$1,500.00			$ 196,500.00		$ 196,500.00	
			Panel System (40 Panels) 275 ft					**$ -**		**$ 1,053,900.00**		**$ 1,053,900.00**
	x	x	Fabricate Picket System	60.00	TN	$ 7,000.00			$ 420,000.00		$ 420,000.00	
	x	x	Procure Picket Attachment System (Frangible nuts and Anchors)	40.00	EA	$ 5,000.00			$ 200,000.00		$ 200,000.00	
	x	x	Rock Excavation	11.85	CY	$ 100.00			$ 1,185.19		$ 1,185.19	
	x	x	Form/Rebar/Pour Foundations	11.85	CY	$ 1,250.00			$ 14,814.81		$ 14,814.81	
	x	x	Drill and Install Anchoring System	40.00	EA	$ 3,500.00			$ 140,000.00		$ 140,000.00	
	x	x	Install Picket System	40.00	EA	$ 2,500.00			$ 100,000.00		$ 100,000.00	
	x	x	Install Electrical/Control Wiring	1.00	LS	$ 50,000.00			$ 50,000.00		$ 50,000.00	
	x	x	Grout Base of Dam	60.00	CY	$ 1,000.00			$ 60,000.00		$ 60,000.00	
	x	x	Procure Bubbler System (Quote + Freight Allowance $5000)	1.00	LS	$ 32,900.00			$ 32,900.00		$ 32,900.00	

2 ft 1	8ft 2	8ft 3	2 ft Concrete and 8 ft rubber dam in Control Section	Quantity	Unit	Unit Price	Option 1 Price	Option 1 Total	Option 2 Price	Option 2 Total	Option 3 Price	Option 3 Total
	x	x	Install Bubbler System (Onsite crews +mfr supervision)	1.00	LS	$ 35,000.00			$ 35,000.00		$ 35,000.00	
								$ -		$ -		$ -
			Project Subtotal (Direct Costs Only) =					$ 608,922.22		$ 3,562,563.87		$ 3,619,554.70
			Project Subtotal (Direct and Indirect) =					$ 761,152.78		$ 4,453,204.83		$ 4,524,443.37
			Contingency (20%)					$ 152,230.56		$ 890,640.97		$ 904,888.67
			Grand Total =					$ 913,383.33		$ 5,343,845.80		$ 5,429,332.04
			Accuracy Range									
			+50%					$ 1,370,075.00		$ 8,015,768.70		$ 8,143,998.06
			-30%					$ 639,368.33		$ 3,740,692.06		$ 3,800,532.43