US Army Corps
of Engineers

Sacramento District

ARGONAUT MINE DAM - PHASE II TECHNICAL REPORT STABILITY AND RETROFIT ALTERNATIVE INVESTIGATION JACKSON, CA

EPA / USACE Superfund Program
Argonaut Mine Dam Inspection and Site Support
Amador County, California
Work Authorization Assignment No.04
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EXECUTIVE SUMMARY

The Argonaut dam is a multiple arch concrete dam constructed around 1916 for the purpose of storing mining tailings. The federal Environmental Protection Agency (EPA) contacted the U.S. Army Corps of Engineers (USACE) in May 2013 regarding concerns for the stability of the dam, and provided funding for USACE site inspections, initial engineering analyses, and development of Phase I and Phase II reports. Following two inspections by USACE and EPA in 2013, concerns of the dams overall stability were raised. The Corps Sacramento District then began a structural stability analysis of the existing dam in accordance with USACE standards ER 1110-2-1156 Safety of Dams Policies and Procedures, and EM 1110-2-2100 Stability Analysis of Concrete Structures. The structural stability analysis of the existing dam, as shown in the Phase I Technical Report in January 2015, indicated the dam does not meet USACE stability criteria.

This Phase II report was developed to investigate feasible construction options (and their estimated costs) to structurally stabilize the dam according to the USACE standards and criteria referenced above. USACE only considered structural stabilization of the existing dam, except for seismic loading. A seismic hazard analysis (deterministic and probabilistic) is recommended for the next phase of design and analysis.

Per direction from the EPA, impacts to other site factors such as upstream drainage, hazardous waste, environmental considerations, and cultural resources were not analyzed by USACE. The results of two options investigated are summarized in the table below:

Option	Estimated Construction Cost
1. Buttress & Gravity Arch	$11,220,321
2. Mass Concrete	$12,776,152

Excludes State and EPA costs for administration/management

Details of the two options, including construction risk summaries for each, are included in Section 3 Construction Options Analysis.

1. INTRODUCTION

1.1. BACKGROUND

The Argonaut dam is a multiple arch concrete dam constructed around 1916 for the purpose of storing mining tailings. An aerial photo plan view of the Argonaut Mine Dam and adjacent City of Jackson is shown in Figure 1. Figure 2 is a plan and elevation representation of the multiple arch / buttress concrete structure. The dam, from historic documents, was described as being 420 to 450 feet long and 46 to 50 feet tall at its highest point. The dam consisted of 13-14 arches, which ranged in thickness from 18 inches at the base to 12 inches at the top. The arches along with the 15 buttress walls were reinforced with a 1 inch or 1 1/8 inch diameter steel wire rope, which daylights out the back of the walls. In addition, tie beams have been constructed between 6 of the arches to brace the tallest of the buttress walls. The spacing between the arches is approximately 30 feet – 1 inch. The foundation of the dam may be composed of a weathered clayey siltstone of low plasticity [according to borings (SB-40 & SB-41) conducted just upstream of the dam conducted by URS in 2008]. However, the exact weathering condition of the siltstone is unknown. There is also a letter from 1931 (by C. Marliave) that states the "dam rests upon a hard dense diabasic rock of good quality." But no data was provided as to how this was determined.

EPA contacted USACE in May 2013 to discuss their concerns regarding stability of the existing dam. Following two inspections by USACE and the EPA in 2013 concerns of the dams overall stability were raised. The inspectors included Chris Abela, PE, and Ken Pattermann, GE, of USACE, and Dan Shane of the EPA. Given the age of the dam, condition of the tie beams and relative close proximity of the dam to downtown Jackson both a preliminary static and dynamic assessment were recommended. An assessment of the dam was performed in accordance with USACE standards ER 1110-2-1156 Safety of Dams Policies and Procedures, and EM 1110-2-2100 Stability Analysis of Concrete Structures, which resulted in the Phase I report. A third cursory inspection was conducted with USACE and the EPA on October 7, 2015. The inspection was followed by a meeting with USACE and the EPA regarding the completed Phase I report, and topics/strategy for the upcoming Phase II report.

A brief timeline of the Argonaut Dam construction and inspection history is included in the Phase I Report.

1.2. USACE SCOPE of SERVICES

Per agreements in place to date, the USACE scope of services for this project involves the following:

I. Structural stability analysis of the exposed section of the existing dam in accordance with USACE standards ER 1110-2-1156 Safety of Dams Policies and Procedures, and EM 1110-2-2100 Stability Analysis of Concrete Structures. The existing dam is defined as all exposed and intact arches and buttress walls (damaged or buried arches are not included). This analysis was delivered in the Phase I report.

II. Develop a Phase II report that provides two alternatives that could be constructed to structurally stabilize the dam according to USACE engineering essetstandards. This includes the supporting structural analysis and design efforts to develop the options to a reconnaissance level design, and estimated construction costs for each for comparison purposes. Concept-level construction risks for each option will also be developed.

III. USACE has utilized all existing information available to date to perform for the analyses in Phases I and II. Information not available to date, but that will be required (whether provided by USACE, EPA, or another party) for the development of construction plans and specifications, includes the following:

   a. LiDAR and/or field topographic survey to include existing physical features, property lines, utilities, trees/vegetation, and topographic data to the accuracy required to generate two foot contours on site plan drawings.

   b. Subsurface geotechnical data in sufficient detail to be used to develop a geotechnical engineering report, which is required for the analysis of the existing foundations and the design of the new structure foundations. Very little is known about the existing foundations, which is critically required for any subsequent stabilization measures. This may include soil and rock borings, trenching, laboratory analyses, and ground penetrating radar surveys.

   c. Environmental Site Assessment(s) to characterize any potential hazardous waste sites that may be encountered by a construction contractor.

   d. Historic engineering drawings and other information from John S. Eastwood currently archived at the UC Riverside library.

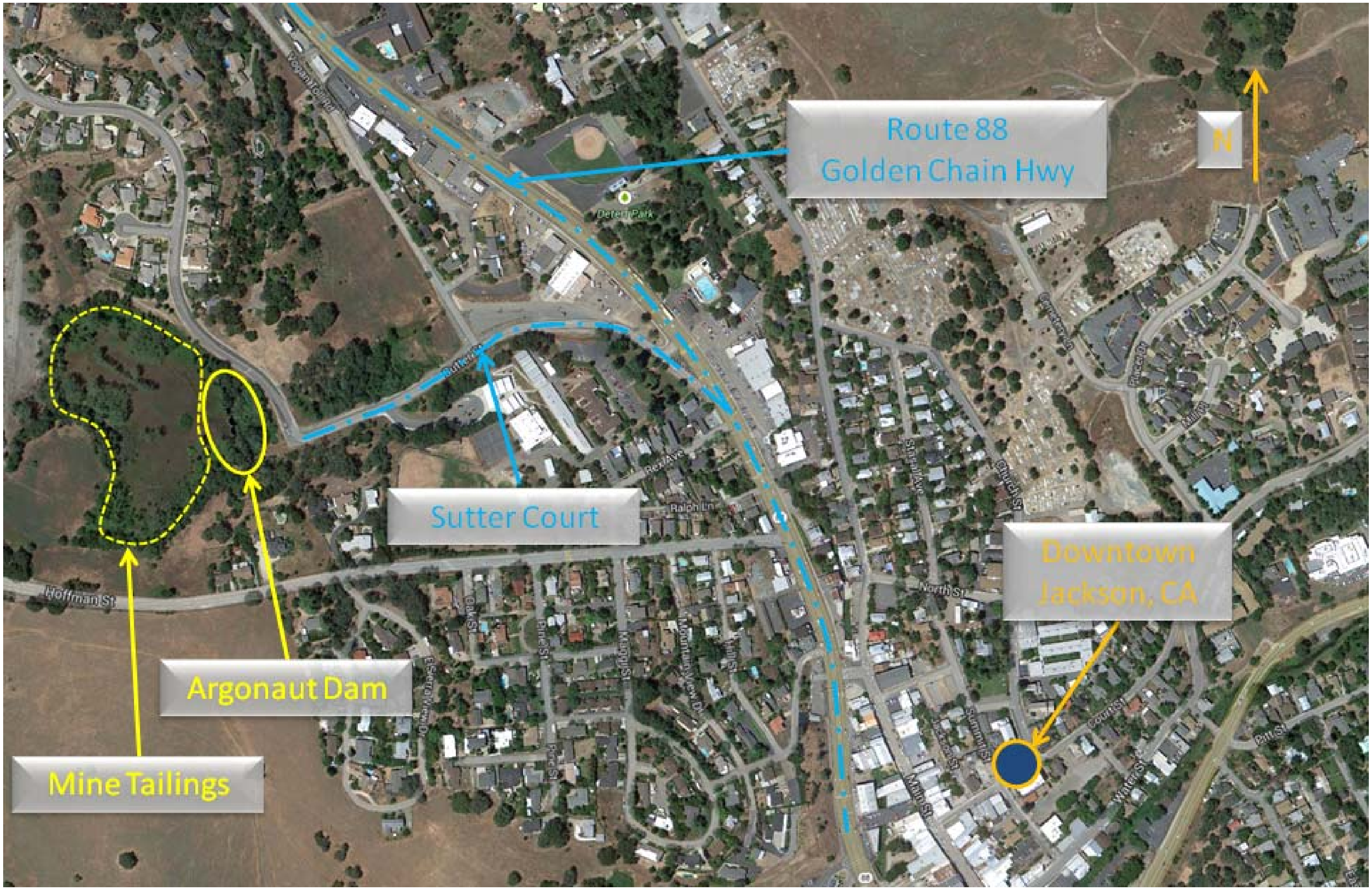

Figure 1 Site Location Map

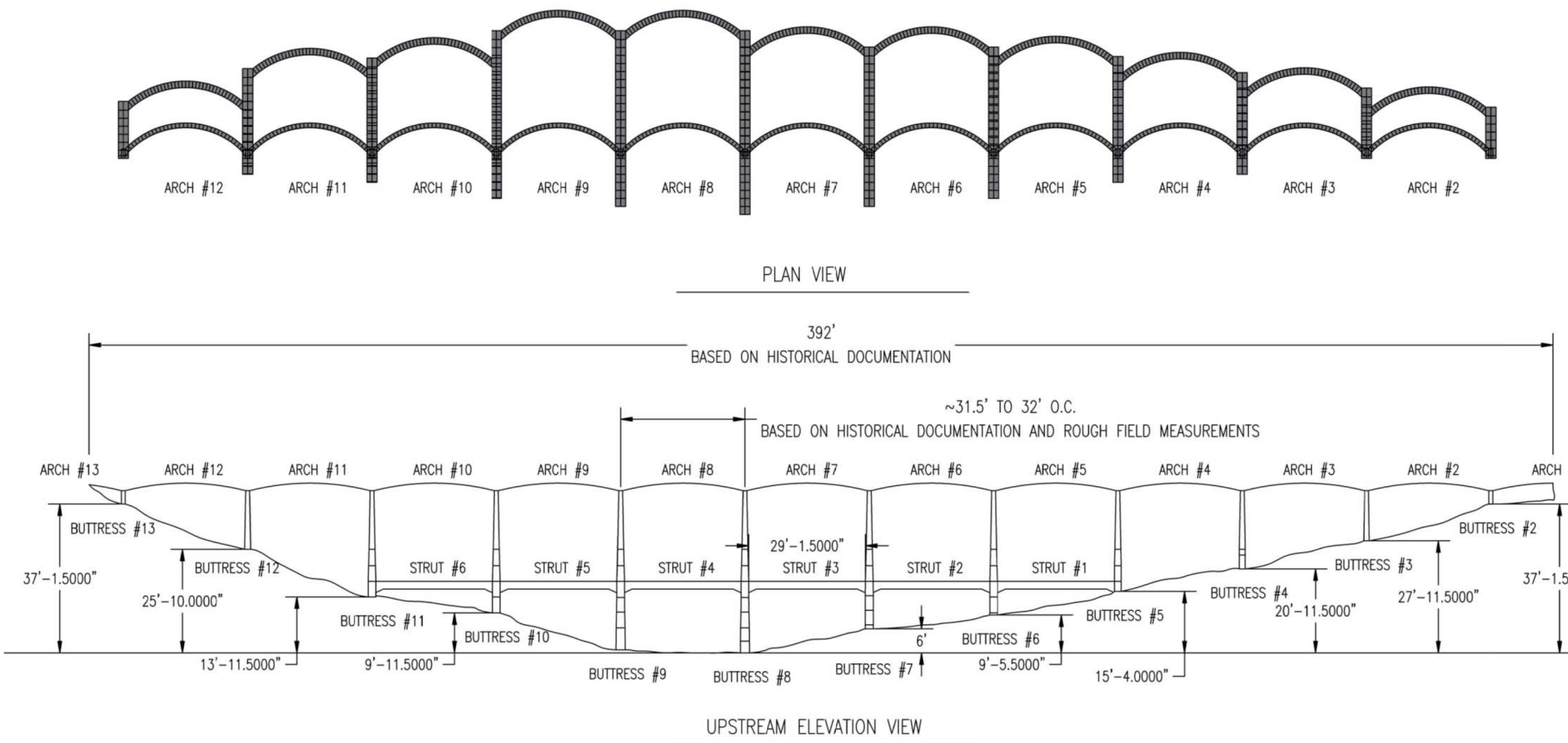

Figure 2. Plan and Elevation View of Dam

2. STABILITY ANALYSIS OF THE EXISTING DAM AND RETROFIT OPTIONS

2.1. EXISTING CONDITION –Multiple Arch Dam Overview

From multiple site visits and various historical documents, a rough approximation of the entire existing dam was modeled to determine the stability of the dam under existing conditions and during a heavy rainfall event, which has been reported to happen periodically. The dam has 11 remaining arches that hold back tailings with each arch supported with a pair of buttress walls. Several assumptions had to be made with regards to the foundation under the dam, especially if the dam was keyed into the rock and whether or not the key was in good structural condition. Because the dam was unreinforced with the exception of hoist cables, the dam was assumed to not have a good functional key. In addition, given the poor condition of the tie beams (see figures below), which were bracing some of the buttress walls, the beams were assumed to not have any remaining structural capacity and were left out of the analysis. A rendering illustrating the existing dam has also been provided below. Further details and discussion on the existing condition of the dams has been provided in the Phase I report.

Figure 3 Existing Condition of Tie Beams (Example 1)

Figure 4 Existing Condition of Tie Beams (Exampple 2)

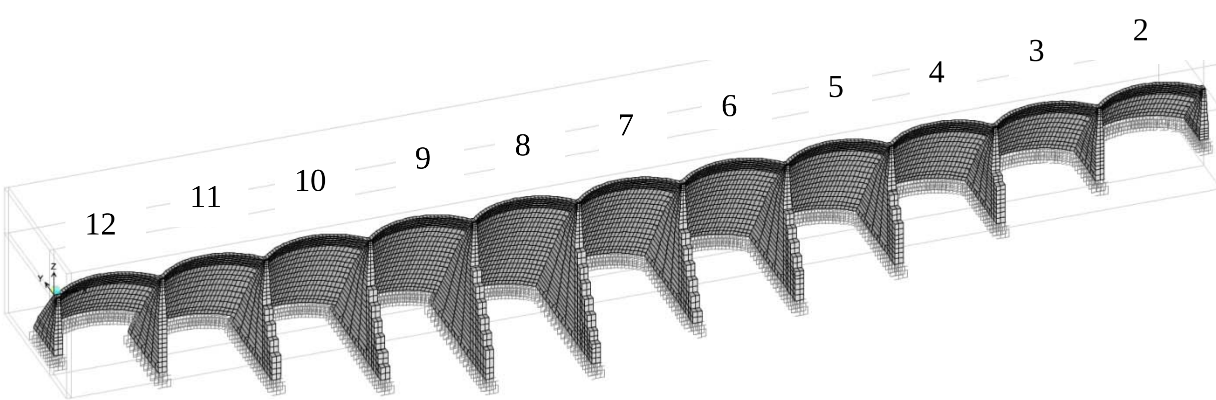

Figure 5 Existing Condition Dam FEA Model

2.2. OPTION 1 –Reinforced Concrete Buttress Walls with Gravity Arches Overview

The first retrofit option investigated is composed of a mass gravity arch with new thickened buttress walls. This option would use the existing dam as formwork and construct a new vertical mass gravity arch on the downstream side. A vertical arch versus a sloped barreled arch (like the existing dam) was selected to help simplify the construction of the dam. The gravity arch would then tie into the new buttress walls, which along with the gravity arch would be keyed into the rock foundation. For the shorter arches located at the sides of the dam, it was elected to completely fill in the arches with mass concrete, versus using a vertical arch approach. A rendering of Option 1 has been provided in the figures below.

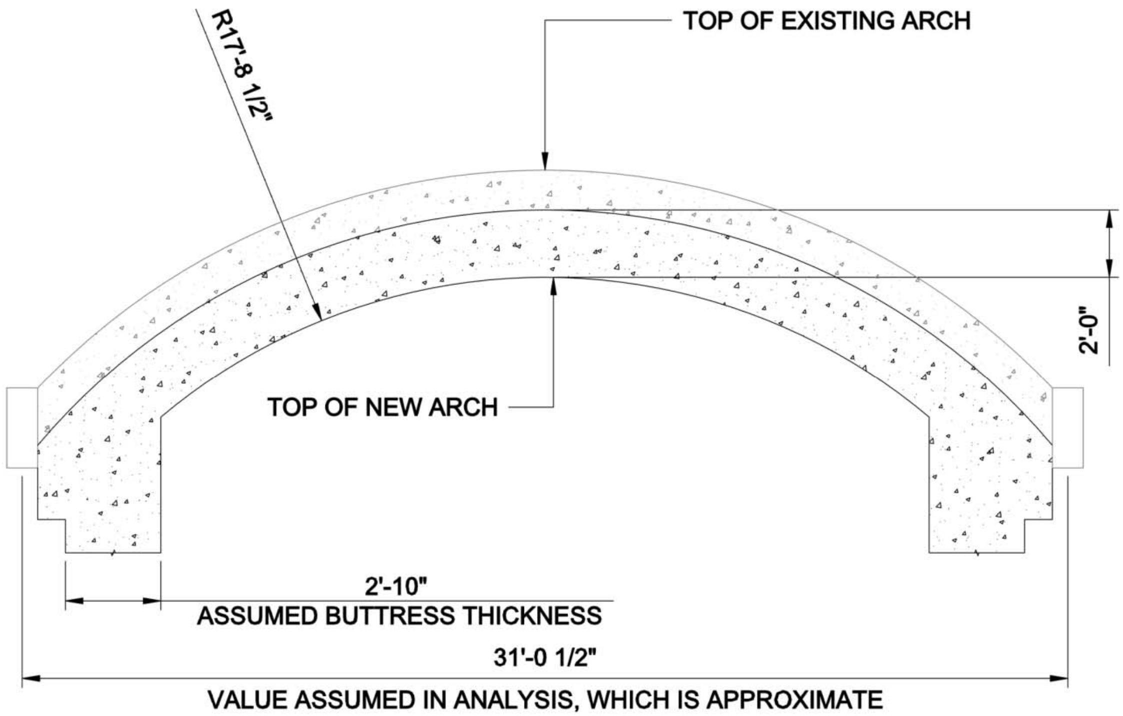

Figure 6 Assumed Dimensions At top of Arches (thickness at base of arches varies)

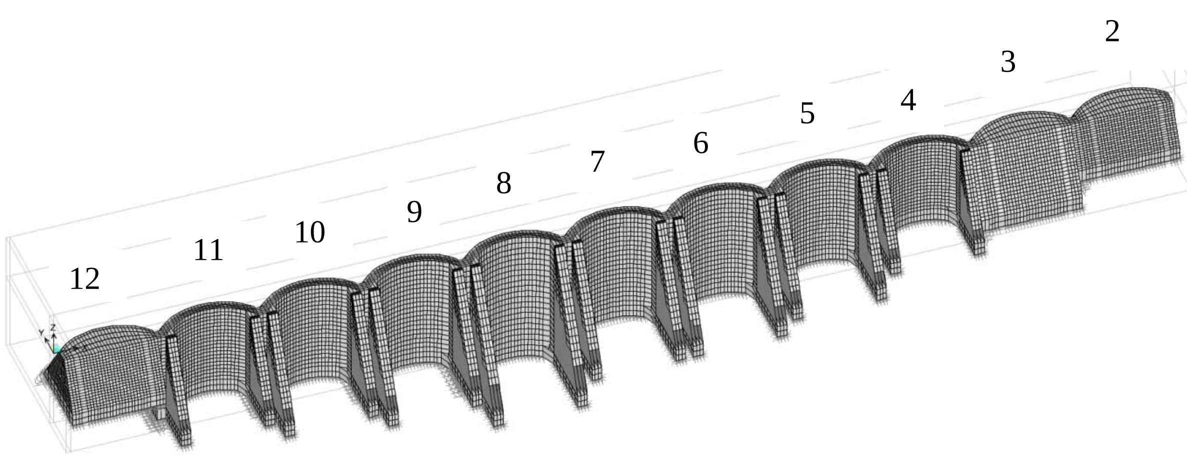

Figure 7 Upstream Isometric View of Option 1 (Looking down onto the dam)

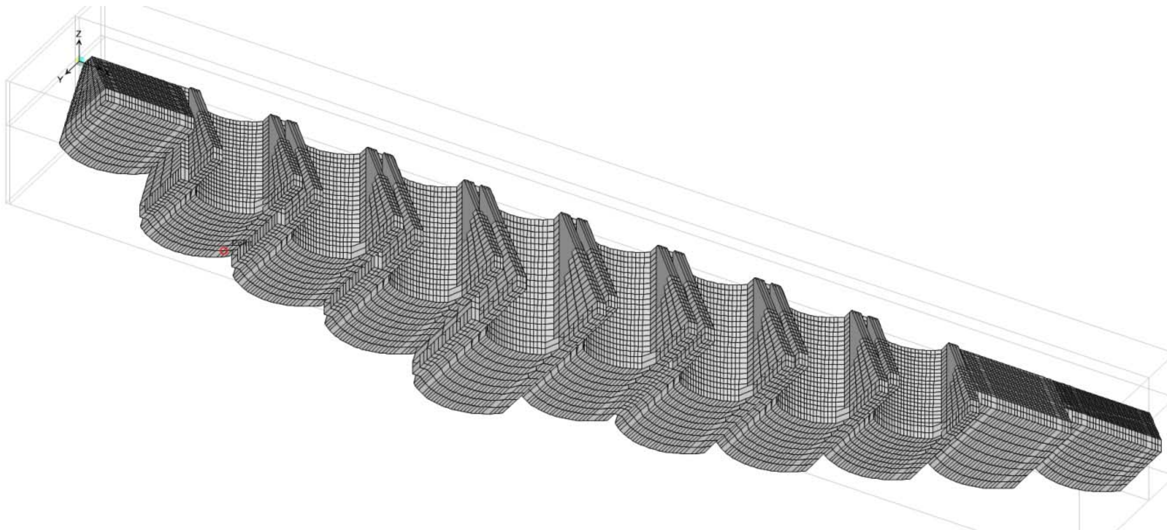

Figure 8 Upstream Isometric View of Option 1 (Looking up underneath the dam)

2.3. OPTION 2 – Mass Concrete Gravity Dam Overview

The second retrofit option investigated would completely fill in all of the arches with mass concrete and key into the rock foundation. A mass concrete solution from literature searches has been a common approach in retrofitting multiple arch dams. Like the first option, the mass concrete approach would also use the existing arches as formwork. The downside of this approach is the sheer quantity of concrete needed to fill in all of the arches, which would most likely lead to higher costs. However, mass gravity dams are common place and historically have performed very well. The figure below illustrates the mass concrete gravity dam solution.

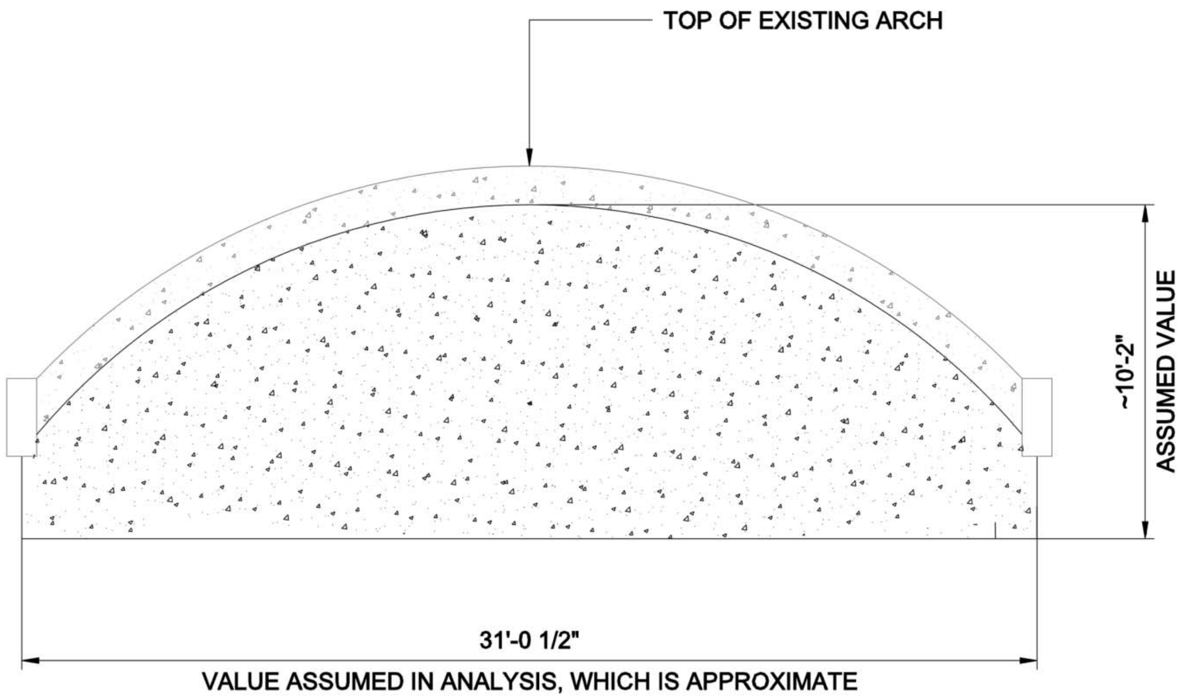

Figure 9 Assumed Dimensions At top of Mass Concrete (thickness at base varies)

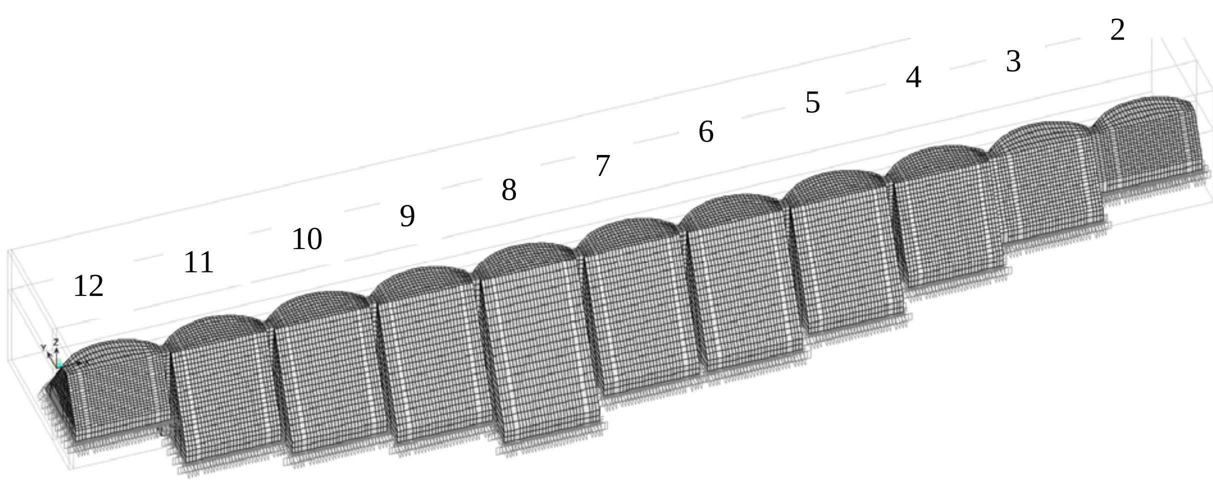

Figure 10 Upstream Isometric View of Option 2 (Looking down onto the dam)

2.4. Construction of Finite Element Models

All finite element models were first drawn as 3D shell elements in AutoCAD and then imported and extruded into solid elements in SAP2000. In some cases the solid elements, which were auto meshed in SAP2000, did not align properly, however, to prevent the misaligned solids from acting independent of one another all solids were assigned edge constraints. The imported shell elements were separated into groups that included new arches, new buttress walls, new foundation, new mass concrete, and others. To apply loads to the models, Option 1 and Option 2 used dummy solid elements that represented the existing dam to apply loads to in regards to soil and water pressures. The dummy elements were constructed such that they would have the same stiffness properties of concrete, but their unit weight was reduced to prevent concrete quantity estimates from being too conservative (150 lb/ft^3 reduced to 1 lb/ft^3). The Existing Condition model did not have dummy elements and the loads were applied directly to the concrete arches and the upstream front face of the buttress walls. The foundation support system under the new arches and new buttress wall was given vertical stiffness properties that matched the rock deformation modulus provided in the Phase I technical report (~1000 lb/in^2/in). The stiffness properties were assigned to area springs, which were assigned to the foundations of Option 1 and Option 2 models. The area spring element was defined as a compression only element that would act on the solid face 4 in the inward direction. The As-Is model used gap links to represent the foundation, which had to be individually drawn. This method was used for the Existing Condition model because certain solid elements on the foundation could not be conveniently rotated such that the same solid face would compose the entire foundation. Therefore, an average solid element area was used to calculate the gap link stiffness. To stabilize the model, the foundation joints of the new arches and new buttress wall was restrained in the upstream downstream and cross canyon directions. Because compression only area springs were used for the foundation, all models were run under a nonlinear load case. For the foundation support of the dummy elements linear springs were used with a stiffness value of only 0.1 kip /in. This small stiffness value was inputted to avoid the linear springs from attracting too much loads away from the area springs, but at the same time provide enough support to the dummy elements.

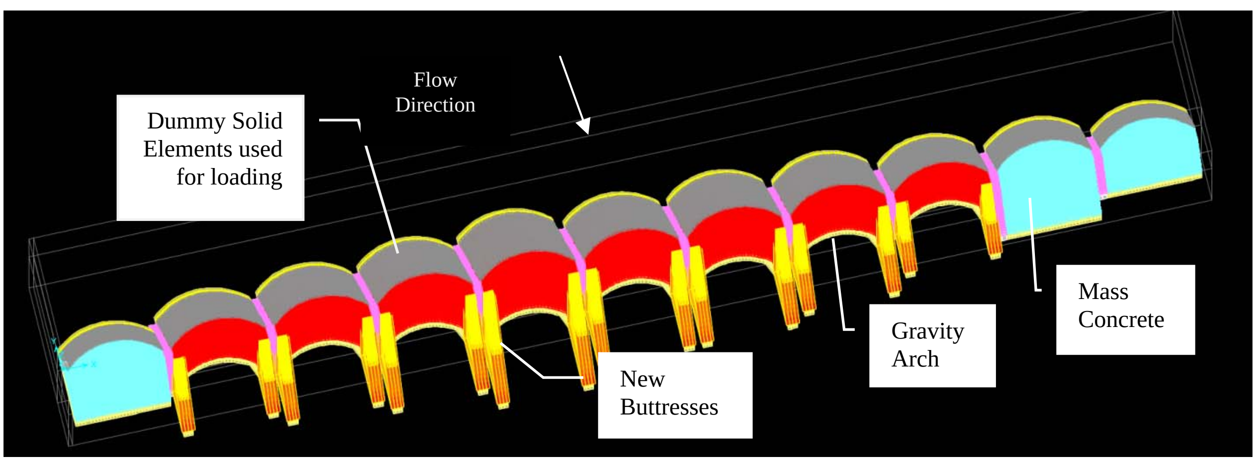

Figure 11 Solid Model Labels

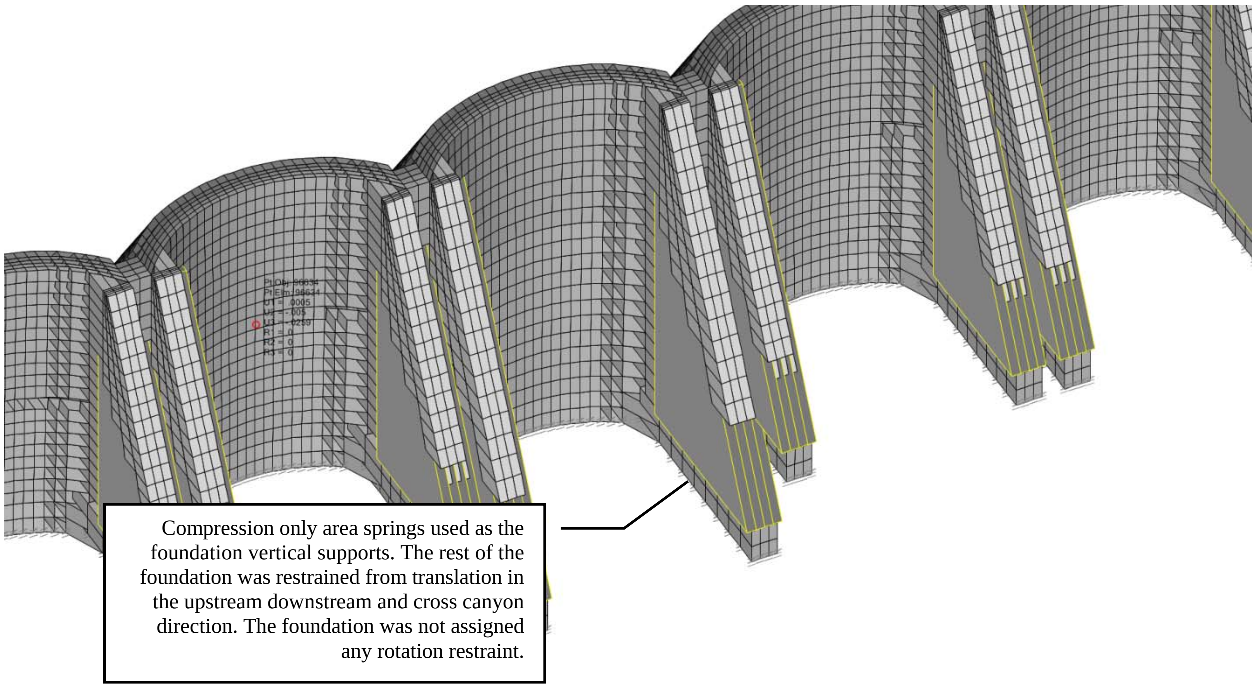

Figure 12 Solid Model Foundation Supports

2.5. Load Cases and Applied Loads

The load cases examined consisted of Load Case 1 and Load Case 2 with the assumptions described below:

I. Load Case 1 (Usual) – Existing conditions of the dam
   - Soil 3ft below top of dam for all arches
   - Ground water 5 ft below top of soil
   - Soil assumed to be at rest

II. Load Case 2 (Unusual) – Historic condition photographed of the dam during a heavy rain event.
   - Soil 3ft below top of dam for all arches
   - Water elevation assumed to be acting 3ft above the existing arches or 6ft above top of soil
   - Soil assumed to be at rest and fully saturated.

The seismic load cases were not evaluated for this draft of the report. However, it is fully intended to evaluate these load cases in either future drafts of the Phase II report or in the DDR if the project heads to construction. The seismic load cases are as follows:

III. Load Case 3 (Unusual) – Seismic OBE
   - Soil 3ft below top of dam for all arches
   - Ground water 5 ft below top of soil

- Soil assumed to be at rest

IV. Load Case 4 (Extreme) – Seismic MDE
   - Soil 3ft below top of dam for all arches
   - Ground water 5 ft below top of soil
   - Soil assumed to be at rest

From the Phase I analysis, the seismicity of the area was found to be low and the load case of most concern was determined to be Load Case 2, which is associated with a heavy rainfall event. Although, the seismic load cases are also important, to help accommodate the aggressive schedule the load cases were prioritized and it was believed that given the relatively low seismic acceleration values in the area the overall design approach of either option would not substantially change.

The soil and water loads applied to the model were based off the Geotechnical report performed as part of the Phase I analysis. For further information regarding the soil property values, the reader is directed to Appendix A, which provides the various load inputs for the SAP models.

All loads within the model were applied as surface pressures typically combined with a joint pattern. A free body diagram of load case 1 and some of its assumptions has been provided below. The surface pressure loads for **Load Case 1** included:

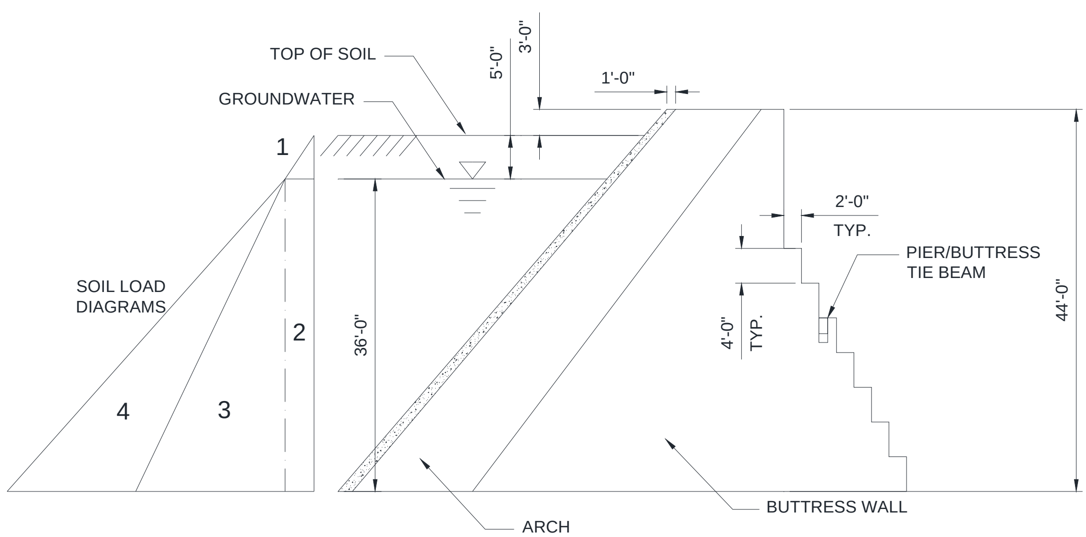

Figure 13 Free Body Diagram Illustrating Load Case 1 (Uplift Pressure Not Shown)

a) Moist Soil Lateral Earth Pressure:
= **85.29 pcf** (applied with a joint pattern)

b) Vertical Pressure of Moist Soil Acting on Saturated Soil:
= **426.45 psf** (applied as a uniform constant surface pressure)

c) Saturated Soil Lateral Earth Pressure:
= **49.75 pcf** (applied with a joint pattern)

d) Water Lateral Pressure:
= **62.5 pcf** (applied with a joint pattern)

e) Water Uplift Pressure:
= **62.5 pcf** (applied with a joint pattern associated with the line of creep method, see Appendix A for line of creep calculations.)

All loads within the model were applied as surface pressures typically combined with a joint pattern. A free body diagram of load case 1 and some of its assumptions has been provided below. The surface pressure loads for **Load Case 2** included:

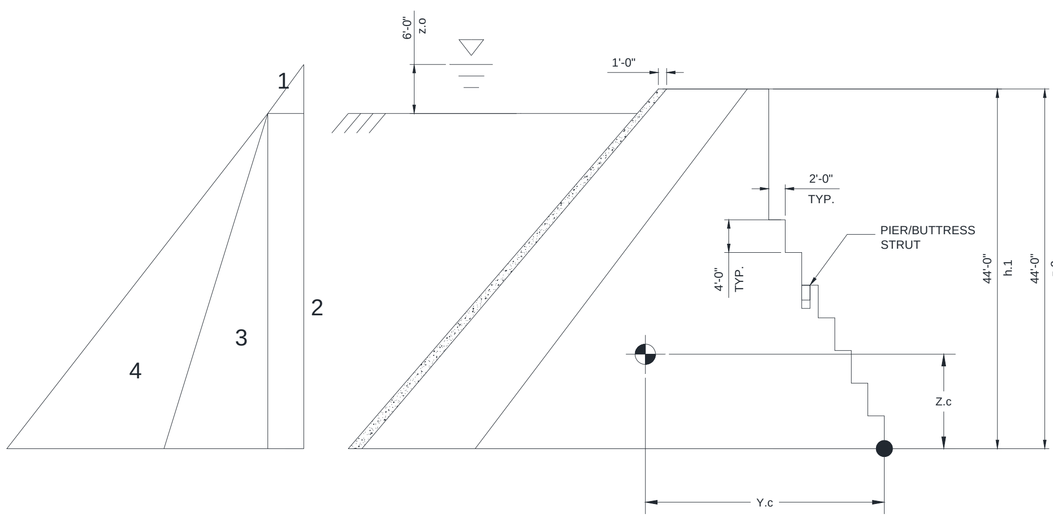

Figure 14 Free Body Diagram Illustrating Load Case 2 (Uplift Pressure Not Shown)

f) Vertical Pressure of Water acting on Saturated Soil:
= **375 psf** (applied as a uniform constant surface pressure)

g) Saturated Soil Lateral Earth Pressure:
= **49.75 pcf** (applied with a joint pattern)

h) Water Lateral Pressure:
= **62.5 pcf** (applied with a joint pattern)

i) Water Uplift Pressure:
= **62.5 pcf** (applied with a joint pattern associated with the line of creep method, see Appendix A for line of creep calculations.)

The figures below illustrate examples of the saturated soil lateral pressure load and uplift pressure load applied to arches respectively in SAP2000:

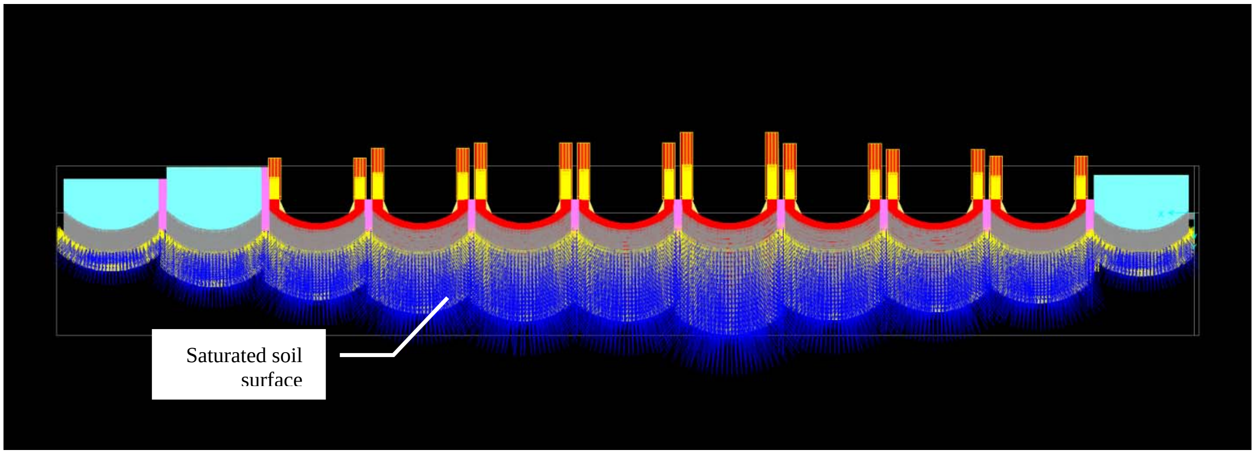

Figure 15 Saturated Soil Surface Pressure Acting on the Dam

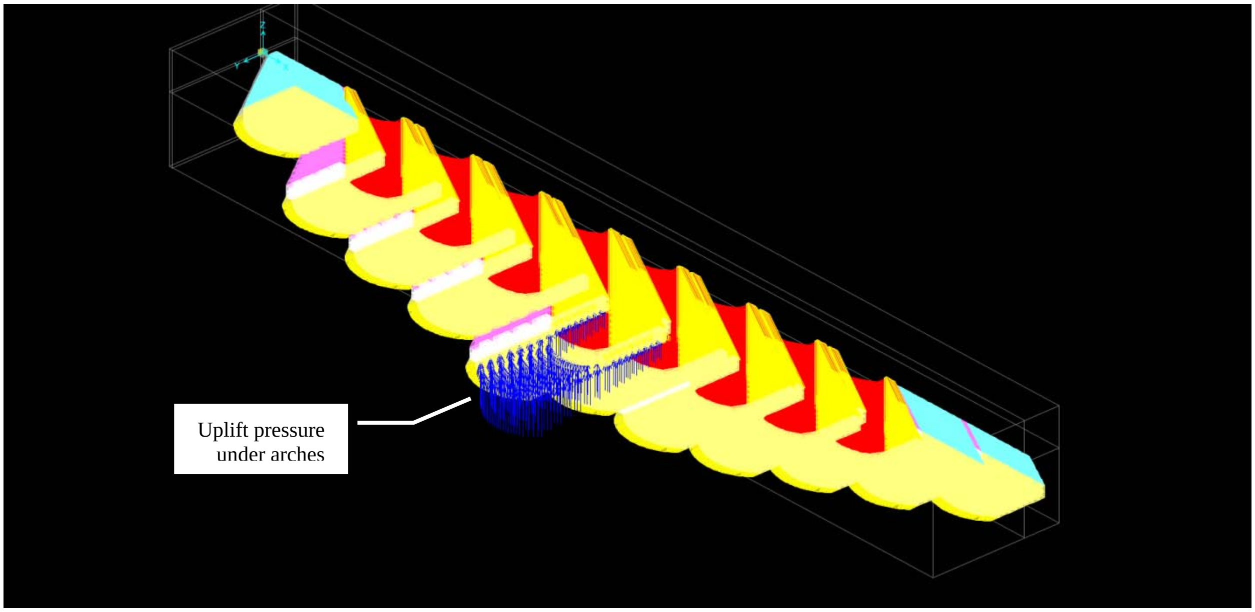

Figure 16 Example of the Applied Uplift Pressure underneath Every Arch

2.6. Stability Results and Assumptions

Early model runs revealed that in order for the foundation to meet USACE stability criteria detailed in EM 1110-2-2100 the foundation would need to be keyed into the existing rock. The final results were run under the assumption that the foundation would be keyed into the surrounding rock up to 4 ft deep, but only contribute a small percent to the overall sliding stability calculation. In this case, only 15% of the rocks allowable bearing capacity was used to achieve acceptable stability values for all of the arches. The dimensions of the key and the bearing capacity of the rock are expected to change following additional site reconnaissance and soil lab results. To extract reactions / demands from the various models, sections cuts were taken along the foundations of each arch, which provided the forces in all 3 directions.

The stability values reviewed and discussed in the Phase I report were reevaluated in the Phase II report for the As-Is condition, Option 1 Retrofit Alternative, and Option 2 Retrofit Alternative. The following are the various stability failure modes reviewed for Load Case 1 and 2:

I. Sliding at the Foundation
II. Sliding along Assumed Weakened Concrete Failure Plane (Existing Condition)
III. Flotation
IV. Overturning / Percent Base in Compression
V. Bearing Capacity

Please see Appendix A for MathCAD hand calculations

To evaluate **<u>sliding at the base</u>**, an internal friction angle (ϕ) of 35 degrees was used along with strength of cohesion (c) value of 10 psi per geotechnical recommendations. Based on these values the following table provides the results for the sliding factor of safety for each load case and arch based on guidance from on Eq. 3-1 of EM 1102-2-2100:

$$FS = \frac{N*\tan\phi + c*L+Key}{T}$$

Table 1 Factor of Safety for Sliding Load Case 1

AS-IS LC1				Option 1 LC1				Option 2 LC1			
#	FS Sliding	Required	PASS/ FAIL	#	FS Sliding	Required	PASS/ FAIL	#	FS Sliding	Required	PASS/ FAIL
Arch 2	2.05	2.00	PASS	Arch 2	11.61	2.00	PASS	Arch 2	9.67	2.00	PASS
Arch 3	2.10	2.00	PASS	Arch 3	8.80	2.00	PASS	Arch 3	6.98	2.00	PASS
Arch 4	2.52	2.00	PASS	Arch 4	5.46	2.00	PASS	Arch 4	6.57	2.00	PASS
Arch 5	3.49	2.00	PASS	Arch 5	6.87	2.00	PASS	Arch 5	5.42	2.00	PASS
Arch 6	6.92	2.00	PASS	Arch 6	12.79	2.00	PASS	Arch 6	5.16	2.00	PASS
Arch 7	1.66	2.00	FAIL	Arch 7	2.91	2.00	PASS	Arch 7	4.18	2.00	PASS
Arch 8	3.35	2.00	PASS	Arch 8	5.37	2.00	PASS	Arch 8	4.70	2.00	PASS
Arch 9	1.30	2.00	FAIL	Arch 9	2.53	2.00	PASS	Arch 9	5.79	2.00	PASS
Arch 10	3.22	2.00	PASS	Arch 10	4.21	2.00	PASS	Arch 10	3.40	2.00	PASS
Arch 11	1.29	2.00	FAIL	Arch 11	2.11	2.00	PASS	Arch 11	4.16	2.00	PASS
Arch 12	1.79	2.00	FAIL	Arch 12	3.95	2.00	PASS	Arch 12	2.70	2.00	PASS

Table 2 Factor of Safety for Sliding Load Case 2

AS-IS LC2				Option 1 LC2				Option 2 LC2			
#	FS Sliding	Required	PASS/ FAIL	#	FS Sliding	Required	PASS/ FAIL	#	FS Sliding	Required	PASS/ FAIL
Arch 2	0.82	1.50	**FAIL**	Arch 2	7.94	1.50	**PASS**	Arch 2	3.07	1.50	**PASS**
Arch 3	0.71	1.50	**FAIL**	Arch 3	4.74	1.50	**PASS**	Arch 3	2.54	1.50	**PASS**
Arch 4	1.22	1.50	**FAIL**	Arch 4	2.91	1.50	**PASS**	Arch 4	3.75	1.50	**PASS**
Arch 5	0.74	1.50	**FAIL**	Arch 5	3.16	1.50	**PASS**	Arch 5	3.39	1.50	**PASS**
Arch 6	0.98	1.50	**FAIL**	Arch 6	3.62	1.50	**PASS**	Arch 6	3.57	1.50	**PASS**
Arch 7	0.63	1.50	**FAIL**	Arch 7	2.47	1.50	**PASS**	Arch 7	2.75	1.50	**PASS**
Arch 8	0.98	1.50	**FAIL**	Arch 8	3.09	1.50	**PASS**	Arch 8	3.69	1.50	**PASS**
Arch 9	0.96	1.50	**FAIL**	Arch 9	2.56	1.50	**PASS**	Arch 9	2.55	1.50	**PASS**
Arch 10	0.81	1.50	**FAIL**	Arch 10	3.37	1.50	**PASS**	Arch 10	3.19	1.50	**PASS**
Arch 11	0.77	1.50	**FAIL**	Arch 11	2.88	1.50	**PASS**	Arch 11	2.55	1.50	**PASS**
Arch 12	0.70	1.50	**FAIL**	Arch 12	3.68	1.50	**PASS**	Arch 12	2.18	1.50	**PASS**

The factor of safety for **<u>flotation</u>** was also evaluated and the results have been tabulated below. By inspection it can be observed that flotation of the tributary arch is not a concern and safety factor values far exceed the required minimum values.

$$FS = \frac{W_S + W_C + S}{U}$$

Table 3 Factor of Safety for Flotation Load Case 1

AS-IS LC1				Option 1 LC1				Option 2 LC1			
#	FS Flotation	Required	PASS / FAIL	#	FS Flotation	Required	PASS / FAIL	#	FS Flotation	Required	PASS / FAIL
Arch 2	3.32	1.3	**PASS**	Arch 2	3.85	1.3	**PASS**	Arch 2	3.63	1.3	**PASS**
Arch 3	5.49	1.3	**PASS**	Arch 3	4.00	1.3	**PASS**	Arch 3	3.86	1.3	**PASS**
Arch 4	9.85	1.3	**PASS**	Arch 4	3.82	1.3	**PASS**	Arch 4	4.19	1.3	**PASS**
Arch 5	16.86	1.3	**PASS**	Arch 5	4.40	1.3	**PASS**	Arch 5	4.31	1.3	**PASS**
Arch 6	15.88	1.3	**PASS**	Arch 6	4.45	1.3	**PASS**	Arch 6	4.34	1.3	**PASS**
Arch 7	19.86	1.3	**PASS**	Arch 7	3.99	1.3	**PASS**	Arch 7	4.44	1.3	**PASS**
Arch 8	19.90	1.3	**PASS**	Arch 8	3.88	1.3	**PASS**	Arch 8	4.32	1.3	**PASS**
Arch 9	18.68	1.3	**PASS**	Arch 9	4.59	1.3	**PASS**	Arch 9	4.43	1.3	**PASS**
Arch 10	24.71	1.3	**PASS**	Arch 10	4.37	1.3	**PASS**	Arch 10	4.31	1.3	**PASS**
Arch 11	14.39	1.3	**PASS**	Arch 11	3.80	1.3	**PASS**	Arch 11	4.12	1.3	**PASS**
Arch 12	11.00	1.3	**PASS**	Arch 12	4.09	1.3	**PASS**	Arch 12	3.96	1.3	**PASS**

Table 4 Factor of Safety for Flotation Load Case 2

AS-IS LC2			
#	FS Flotation	Required	PASS / FAIL
Arch 2	6.01	1.2	PASS
Arch 3	9.33	1.2	PASS
Arch 4	14.12	1.2	PASS
Arch 5	20.89	1.2	PASS
Arch 6	19.06	1.2	PASS
Arch 7	23.27	1.2	PASS
Arch 8	23.08	1.2	PASS
Arch 9	22.11	1.2	PASS
Arch 10	14.66	1.2	PASS
Arch 11	14.61	1.2	PASS
Arch 12	5.36	1.2	PASS

Option 1 LC2			
#	FS Flotation	Required	PASS / FAIL
Arch 2	3.70	1.2	PASS
Arch 3	3.47	1.2	PASS
Arch 4	3.74	1.2	PASS
Arch 5	4.72	1.2	PASS
Arch 6	5.01	1.2	PASS
Arch 7	4.31	1.2	PASS
Arch 8	4.54	1.2	PASS
Arch 9	6.08	1.2	PASS
Arch 10	5.37	1.2	PASS
Arch 11	4.89	1.2	PASS
Arch 12	4.74	1.2	PASS

Option 2 LC2			
#	FS Flotation	Required	PASS / FAIL
Arch 2	3.39	1.2	PASS
Arch 3	3.81	1.2	PASS
Arch 4	4.22	1.2	PASS
Arch 5	4.31	1.2	PASS
Arch 6	4.37	1.2	PASS
Arch 7	4.48	1.2	PASS
Arch 8	4.38	1.2	PASS
Arch 9	4.47	1.2	PASS
Arch 10	4.35	1.2	PASS
Arch 11	4.09	1.2	PASS
Arch 12	3.87	1.2	PASS

To determine the **percent of the base in compression / overturning**, the FEA models were used in which the total numbers of joints in compression were compared to the total number of joints within the base. This procedure although relatively simplistic, does provide a reasonable estimate on the percent of the base in compression. The stability results for this failure mode are provided below:

Table 5 Factor of Safety for Percent Base In Compression / Overturning Load Case 1

AS-IS LC1				Option 1 LC1				Option 2 LC1			
#	FS Overturning	Required	PASS / FAIL	#	FS Overturning	Required	PASS / FAIL	#	FS Overturning	Required	PASS / FAIL
Arch 2	51%	100%	**FAIL**	Arch 2	100%	100%	**PASS**	Arch 2	100%	100%	**PASS**
Arch 3	71%	100%	**FAIL**	Arch 3	100%	100%	**PASS**	Arch 3	100%	100%	**PASS**
Arch 4	79%	100%	**FAIL**	Arch 4	100%	100%	**PASS**	Arch 4	100%	100%	**PASS**
Arch 5	83%	100%	**FAIL**	Arch 5	100%	100%	**PASS**	Arch 5	100%	100%	**PASS**
Arch 6	92%	100%	**FAIL**	Arch 6	100%	100%	**PASS**	Arch 6	100%	100%	**PASS**
Arch 7	66%	100%	**FAIL**	Arch 7	100%	100%	**PASS**	Arch 7	100%	100%	**PASS**
Arch 8	95%	100%	**FAIL**	Arch 8	100%	100%	**PASS**	Arch 8	100%	100%	**PASS**
Arch 9	87%	100%	**FAIL**	Arch 9	100%	100%	**PASS**	Arch 9	100%	100%	**PASS**
Arch 10	84%	100%	**FAIL**	Arch 10	100%	100%	**PASS**	Arch 10	100%	100%	**PASS**
Arch 11	75%	100%	**FAIL**	Arch 11	100%	100%	**PASS**	Arch 11	100%	100%	**PASS**
Arch 12	56%	100%	**FAIL**	Arch 12	100%	100%	**PASS**	Arch 12	100%	100%	**PASS**

Table 6 Factor of Safety for Percent Base In Compression / Overturning Load Case 2

AS-IS LC2			
#	FS Overturning	Required	PASS / FAIL
Arch 2	51%	75%	FAIL
Arch 3	71%	75%	FAIL
Arch 4	79%	75%	PASS
Arch 5	83%	75%	PASS
Arch 6	92%	75%	PASS
Arch 7	66%	75%	FAIL
Arch 8	95%	75%	PASS
Arch 9	87%	75%	PASS
Arch 10	84%	75%	PASS
Arch 11	75%	75%	PASS
Arch 12	56%	75%	FAIL

Option 1 LC2			
#	FS Overturning	Required	PASS / FAIL
Arch 2	100%	75%	PASS
Arch 3	100%	75%	PASS
Arch 4	100%	75%	PASS
Arch 5	95%	75%	PASS
Arch 6	94%	75%	PASS
Arch 7	100%	75%	PASS
Arch 8	100%	75%	PASS
Arch 9	94%	75%	PASS
Arch 10	100%	75%	PASS
Arch 11	100%	75%	PASS
Arch 12	100%	75%	PASS

Option 2 LC2			
#	FS Overturning	Required	PASS / FAIL
Arch 2	100%	75%	PASS
Arch 3	100%	75%	PASS
Arch 4	100%	75%	PASS
Arch 5	100%	75%	PASS
Arch 6	100%	75%	PASS
Arch 7	100%	75%	PASS
Arch 8	100%	75%	PASS
Arch 9	100%	75%	PASS
Arch 10	100%	75%	PASS
Arch 11	100%	75%	PASS
Arch 12	100%	75%	PASS

To evaluate the **bearing capacity** an allowable bearing capacity of 45 ksf was used based on the geotechnical investigation report by Pattermann (2014). To determine q.max or the maximum bearing pressure, the max joint reaction was selected and then divided by its tributary area. The tabulated values for the max bearing pressure have been provided below.

Table 7 Factor of Safety for Bearing Capacity Load Case 1

LC1			
#	Bearing Demand (ksf)	Allowable Bearing (ksf)	PASS / FAIL
AS - IS	10.46	45.00	**PASS**
OPTION 1	6.81	45.00	**PASS**
OPTION2	9.30	45.00	**PASS**

Table 8 Factor of Safety for Bearing Capacity Load Case 2

LC2			
#	Bearing Demand (ksf)	Allowable Bearing (ksf)	PASS / FAIL
AS - IS	15.00	45.00	**PASS**
OPTION 1	8.80	45.00	**PASS**
OPTION2	5.46	45.00	**PASS**

A second sliding shear stability failure mode was also checked, which dealt with **sliding shear through concrete** versus sliding shear at the base of the dam. This stability check was added due to observations made in 2013, in which it was observed that the embedded cable had corroded and possibly created a weakened plane in the concrete. Because the friction coefficient of the concrete / weakened plane is unknown, a range of values were checked using ACI 318-11. For simplicity the required factor of safety for sliding shear at the base was also used to compare values for sliding shear through concrete. The table below illustrates a range of safety factors for each existing arch using different friction values for the concrete.

Table 9 Factor of Safety for Sliding Through Concrete Plane Load Case 1

AS-IS LC1 (ACI Friction = 0.6)			
#	FS Sliding	Required	PASS / FAIL
Arch 2	1.09	2.00	FAIL
Arch 3	1.37	2.00	FAIL
Arch 4	1.76	2.00	FAIL
Arch 5	2.56	2.00	PASS
Arch 6	5.14	2.00	PASS
Arch 7	1.22	2.00	FAIL
Arch 8	2.51	2.00	PASS
Arch 9	0.94	2.00	FAIL
Arch 10	2.48	2.00	PASS
Arch 11	0.95	2.00	FAIL
Arch 12	1.33	2.00	FAIL

AS-IS LC1 (ACI Friction = 1.4)			
#	FS Sliding	Required	PASS / FAIL
Arch 2	2.54	2.00	PASS
Arch 3	3.19	2.00	PASS
Arch 4	4.11	2.00	PASS
Arch 5	5.97	2.00	PASS
Arch 6	12.00	2.00	PASS
Arch 7	2.84	2.00	PASS
Arch 8	5.85	2.00	PASS
Arch 9	2.19	2.00	PASS
Arch 10	5.80	2.00	PASS
Arch 11	2.21	2.00	PASS
Arch 12	3.09	2.00	PASS

Table 10 Factor of Safety for Sliding Through Concrete Plane Load Case 2

AS-IS LC2 (ACI Friction = 0.6)			
#	FS Sliding	Required	PASS / FAIL
Arch 2	0.53	1.50	**FAIL**
Arch 3	0.51	1.50	**FAIL**
Arch 4	0.91	1.50	**FAIL**
Arch 5	0.57	1.50	**FAIL**
Arch 6	0.76	1.50	**FAIL**
Arch 7	0.48	1.50	**FAIL**
Arch 8	0.76	1.50	**FAIL**
Arch 9	0.73	1.50	**FAIL**
Arch 10	0.60	1.50	**FAIL**
Arch 11	0.56	1.50	**FAIL**
Arch 12	0.46	1.50	**FAIL**

AS-IS LC2 (ACI Friction = 1.4)			
#	FS Sliding	Required	PASS / FAIL
Arch 2	1.23	1.50	**FAIL**
Arch 3	1.19	1.50	**FAIL**
Arch 4	2.13	1.50	**PASS**
Arch 5	1.33	1.50	**FAIL**
Arch 6	1.77	1.50	**PASS**
Arch 7	1.13	1.50	**FAIL**
Arch 8	1.76	1.50	**PASS**
Arch 9	1.69	1.50	**PASS**
Arch 10	1.40	1.50	**FAIL**
Arch 11	1.31	1.50	**FAIL**
Arch 12	1.08	1.50	**FAIL**

2.7. Discussion of Stability Results

From the results, the following conclusions were made regarding the **Existing Dam**:

1. The dam under (sliding stability) current EM guidance could be classified as being potentially unstable under the load cases examined.

2. The flotation stability results were found to be acceptable and complied with current EM guidance.

3. Although the percent base in compression stability results showed widespread failure, these results were understood as the dam not meeting current guidance, but not that the dam was necessarily unstable. This is illustrated by the fact that the stability results improved between Load Case 1 and Load Case 2 in which the percent base in compression requirement was lowered due to the loading event being classified as Unusual.

4. The allowable bearing capacity check was determined to be acceptable and well under the limiting value.

5. Without reinforcement a weakened concrete plane (stability for sliding through a concrete plane) due to a poorly prepared lift lines could lead to dam instability.

From the results the following observations were made regarding retrofit **Options 1 & 2**:

1. Sliding stability for both options was found to be the governing stability failure mode that required the use of a foundation key to achieve acceptable stability results in accordance with current EM guidelines.

2. The flotation stability results were found to be acceptable and easily complied with current EM guidance.

3. Overturning or the percent base in compression stability failure mode was not found to be difficult to achieve for either retrofit option.

4. The allowable bearing capacity check was determined to be acceptable and well under the limiting value.

5. A stability check related to a weakened concrete plane was not evaluated given the early design phase of the retrofit options. However, it is envisioned that any potential weakened concrete plane will be designed to resist all forces across the plane with the use of reinforcement and or dead weight of the structure to achieve acceptable stability results.

In the case of all models, the sliding stability of the foundation was determined to be the governing stability failure mode and the failure mode of most concern. For the Existing Dam, given the age of the structure, failure through a weakened plane is also of concern. All of the remaining failure modes were not found to be as critical as the sliding failure mode and appeared to be able to achieve or nearly achieve acceptable stability results in accordance with EM guidance. It is important to note that the stability results have been reported based on each arch being analyzed individually. This approach is somewhat conservative as the entire dam system (existing or retrofit) will act integrally, which may increase stability values.

2.8. Path Forward and Recommendations

Based on both the Phase I and Phase II modeling results the Argonaut Dam should be retrofitted using either Option 1 or Option 2 presented here-in. The selected option should progress to a 35% - 50% design level complete with plans, specifications, and a design draft report. At this phase, it is recommended that the selected option design undergo some form of independent technical review before progressing to the 95% design level to help negate any critical flaws being overlooked. Furthermore, at the 35% - 50% design phase all load cases should be evaluated including seismic. Based on this analysis either option could be designed (and potentially optimized) to achieve acceptable stability results.

2.9. References

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2. Federal Energy Regulatory Commission, (1997) "Chapter 10 Other Dams" Engineering Guidelines for the Evaluation of Hydropower Projects.

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5. EM 1110-2-2100, (2005). Engineering Manual *Stability Analysis of Concrete Structures*, United States Army Corps of Engineers, Washington, DC.

6. EM 1110-1-1905, (1992). Engineering Manual *Bearing Capacity of Soils*, United States Army Corps of Engineers, Washington, DC.

7. EM 1110-2-6053, (2007). Engineering Manual Earthquake Design and Evaluation of Concrete Hydraulic Structures, United States Army Corps of Engineers, Washington, DC.

8. Federal Energy Regulatory Commission, (1997) "Chapter 10 Other Dams" Engineering Guidelines for the Evaluation of Hydropower Projects.

9. Federal Energy Regulatory Commission, (1999) "Chapter 11 Arch Dams" Engineering Guidelines for the Evaluation of Hydropower Projects.

10. Pattermann, K. R., Abela, C. M. (2013) "Argonaut Tailings Storage Dam – Initial Inspection", United States Army Corps of Engineers Memorandum, July 30, 2013.

11. Pattermann, K.R., (2014) “Argonaut Dam and Tailings Storage Site”, Geotechnical report December 2014.

12. Abela, C. M. (2014) “Argonaut Tailings Storage Dam – Initial Inspection”, United States Army Corps of Engineers Memorandum, July 30, 2013.

13. Abela, C. M., Pattermann, K. R.,. (2013) “ARGONAUT DAM STABILITY AND RETROFIT ALTERNATIVE INVESITGATION” United States Army Corps of Engineers Phase I Technical Report June 2015.

2.10. Acronyms

- OBE = Operational Basis Earthquake
- MDE = Maximum Design Earthquake
- DDR = Design Draft Report
- SAP = Structural Analysis Program
- US = Upstream
- DS = Downstream
- FS = Factor of Safety
- AS-IS = Existing Condition of the Dam
- LC = Load Case

3. COST ANALYSIS FOR RETROFIT OPTIONS 1 AND 2

3.1 OPTION 1 –Reinforced Concrete Buttress Walls with Gravity Arches Cost Overview

The combination mass and reinforced concrete option (shown schematically in Figure 4 & 5 and figure below) consists of the following main features:

I. Site preparation work downstream of the existing dam as shown in Figure 13 (drawing C-100), to include clearing of trees and vegetation, construction of a gravel site access road with rock base entrances, and drainage structures (pipes, catch basins, etc.) as required around the new concrete foundations.

II. For three outermost arch vaults, mass (lightly reinforced) concrete will be placed downstream of the existing dam versus vertical arches as shown below.

    a. For the interior eight arch vaults, the existing buttresses will become encapsulated with new concrete to help form a new ~8 foot thick buttress wall, which will help support the new mass gravity arches. The new mass gravity arches will be placed against the existing concrete using it as formwork. The top of the concrete will be at the same elevation as the top of the existing dam. The new mass gravity arches will extend vertically and not follow the existing sloped barrel arch design. The existing tie beams currently bracing the existing buttress walls will be demolished and not replaced.

III. Both the mass concrete and reinforced concrete overlay sections will be entirely founded on competent rock and keyed into the foundation, which will be cleaned and prepped for the placement of concrete. For the purposes of this study, it is assumed that competent rock is ~eight feet below the existing visible buttress foundations. The existing arch and buttress foundations will remain in place. The upcoming geotechnical explorations and subsequent report will determine the elevation of competent rock as design progresses, and quantities/costs will be adjusted accordingly.

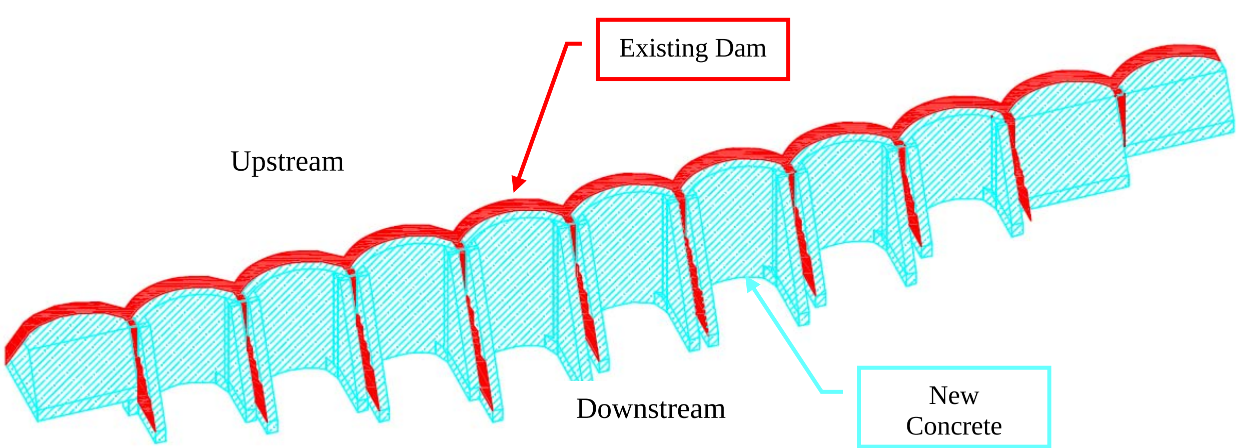

Figure 17 Option 1 Layout

3.2 OPTION 2 – Mass Concrete Gravity Dam Cost Overview

The mass concrete option (shown schematically in the figure below) consists of the following main features:

I. Site preparation work downstream of the existing dam as shown in Figure 13 (drawing C-100), to include clearing of trees and vegetation, construction of a gravel site access road with rock base entrances, and drainage structures (pipes, catch basins, etc.) as required around the new concrete foundations.

II. For every arch vault, mass (lightly reinforced) concrete will be placed downstream of the existing dam. The concrete will be placed against the existing dam to completely fill the space under and downstream of the arches, and the concrete buttresses and struts will be completely encapsulated. The top of the mass concrete will be at the same elevation as the top of the existing dam, and will be placed at a 2:1 slope downstream to the foundation. The sloped mass concrete will not be formed to match the arch shape.

III. The mass concrete will be entirely founded on and keyed into competent rock, which will be cleaned and prepped for the placement of concrete. For the purposes of this study, it is assumed that competent rock is eight feet below the visible existing buttress foundations. The existing arch and buttress foundations will remain in place. The upcoming geotechnical explorations and subsequent report will determine the elevation of competent rock as design progresses, and quantities/costs will be adjusted accordingly.

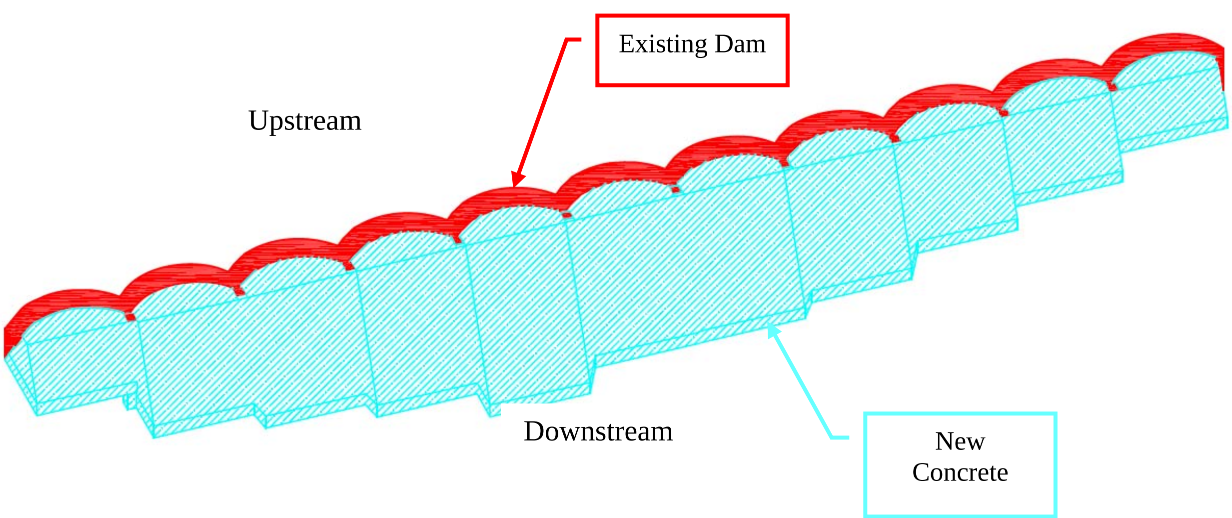

Figure 18 Option 2 Layout

3.3. Options Cost Assumption Comparison and Risk Summary

3.3.1. Project Description: These are preliminary and show two options, A) mass concrete placement behind existing dam with a 2:1 backside slope from the crest, and B) reinforcement of the existing buttress with a 2:1 slope downstream and then concrete infill on the backside of the dam from the crest down. The preliminary CAD drawings show a 4 foot foundation excavation depth however, the PDT opted to use 8 foot excavation depth. A site drawing was provided showing an approximate 400 foot road constructed across the downstream face of the dam and a small staging area on the south end of the site. It is assumed that the road and various site improvements will be constructed prior to commencement of work and left in place after completion of the project with no salvage value. Site-work assumptions are assumed the same for both options.

3.3.2. Site-work Assumptions: Clear and grub areas, remove vegetation, then placement of filter fabric and gravel over all areas affected. Necessary commercially sourced fill is brought in to slope the road at 1:10 It is assumed that a small settling pond will be constructed within the downstream area and a pump and necessary hose to pump any collected water upstream on the site above the dam. Approximately 400 feet of chain-link fence will be installed along the roadway with 2 vehicle gates, and temporary fence placed around the construction limits on the dam. A small graveled area is also prepared for the concrete delivery/pumping for the dam reinforcement on the north end of the dam. A temporary transformer and site power is installed to provide site power for the construction activates and to power the pump for the settling pond. All items are assumed purchased as they will be constructed and left for the construction contractor to utilize. At the end of the project the improvements are assumed left in place.

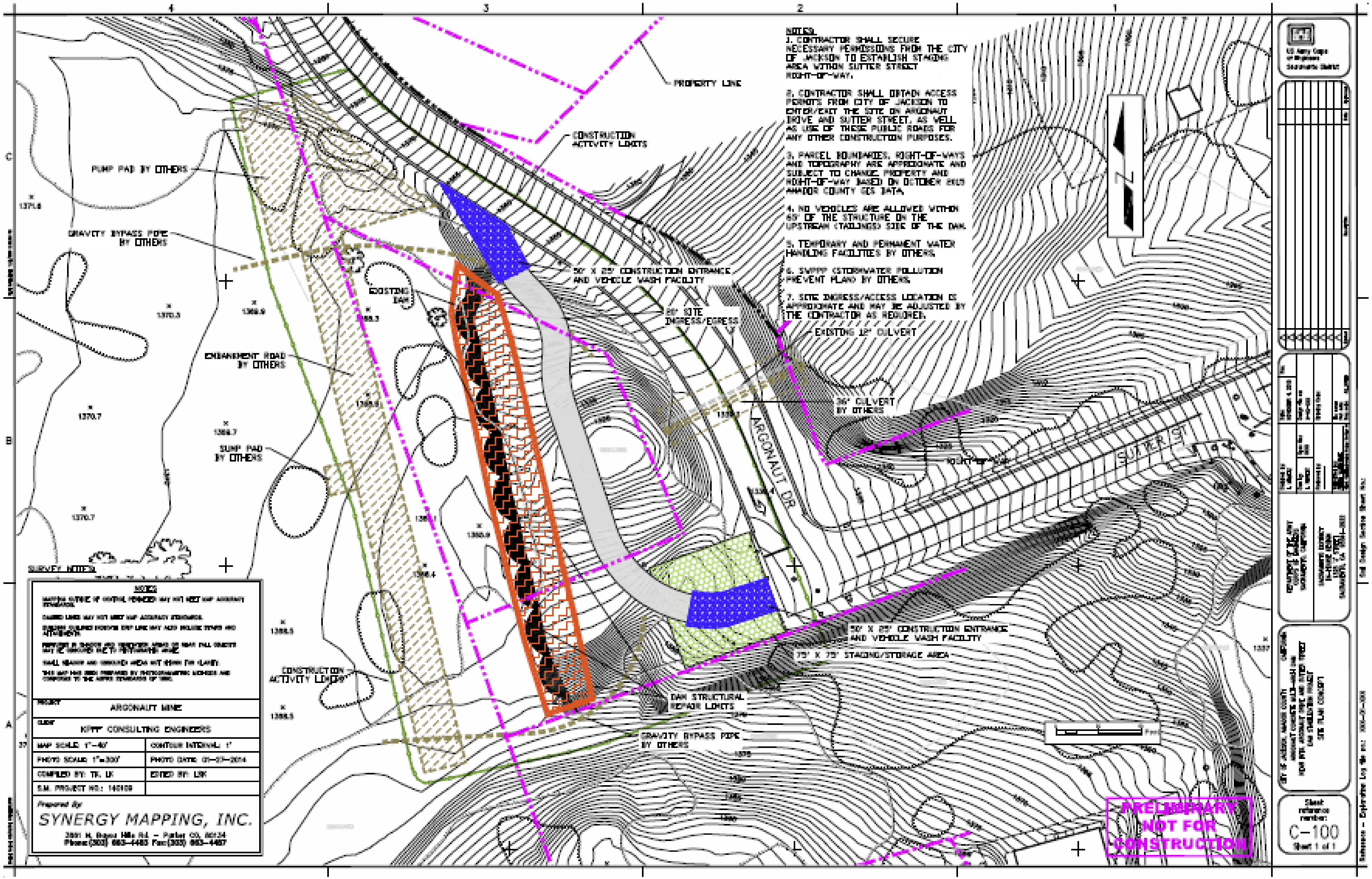

Figure 19 Site Work Layout

3.4. Option 1: Reinforced Buttresses Walls with Mass Gravity Arches

3.4.1. Overall Approach: This approach widens and extends the buttress downstream and then places mass concrete with a vertical face in each bay from approximately the crest down. Bays 2, 3, and 12 are reinforced with mass concrete as in Option 2.

3.4.2. Foundation Excavation: The foundation area is assumed to be excavated with an excavator mounted hammer, loaded and hauled to an onsite disposal area. Some of this material may be contaminated with mining byproducts typical of the site. There is a small allowance for removal of some contaminated materials in the dam preparation estimate. A small network of trenches is excavated out for a pair of HDPE foundation drains per bay. It is assumed for each bay that the deepest buttress will drive the overall depth. (This will slightly overstate the volumes and results in the overall height for the three deepest bays from crest to bottom of excavation to approach 50 feet in height. Since the majority of this excavation will be closer to the existing dam concrete via the mass option, slower productivity was utilized for the excavation. No extraordinary measures were assumed for the foundation excavation and preparation and based on the findings of the geotechnical investigations extraordinary measures such as grouting, micro piles, or drilled shafts could be required to stabilize the foundations.

3.4.3. Existing Dam Preparation: It is assumed that all exposed areas on the downstream area of the dam will be sandblasted and the debris vacuumed and hauled offsite to disposal. The existing struts between bays will be removed and hauled offsite for disposal and the final foundation excavation is washed prior to concrete placement. (Same approach as Option 2)

3.4.4. Formwork and Placement: There are three types of placements involved in this approach- the buttress reinforcement (s similar to a column), the sloped mass concrete AKA Option 2 on the ends, and a DS vertical face mass placement in the remaining bays. Quotes for rental formwork were obtained and utilized in the estimate for the formwork pricing.

3.4.5. Buttress: Each Bays Buttress is widened 3 feet and extended downstream at a 2:1 slope from the crest. For the cost estimate it is assumed that the deepest bay would drive the overall buttress length- this results in a slightly larger volume of concrete than measured in the CAD file and also would simplify construction as well as make the design slightly more conservative as the shorter bays would have slightly larger buttress lengths. The overall buttress when reinforced will be approximately 8 feet wide at the base and extend downstream ½ of their height. The tallest buttress is approximately 48 feet high and placements are assumed to be done in 10 foot lifts. Five placement days are assumed for all of the buttress lifts combined as the quantities are low overall (600-700cy). A light finish as well as curing compound is applied to the buttress work.

3.4.6. Buttress Reinforcement: it is assumed that there would be 12”x12” grid of #10 reinforcement ran in each buttress with 6-12” spacing between each grid. This would extend forward of the upstream buttress formwork so that it can be tied to the mass reinforcement along the sides and downstream faces.

3.4.7. Mass Placements: For this approach the quantity of concrete is much smaller than in option 2 overall however the same 5' lift restriction was assumed. This results in 84 placement areas, however, in this case the small overall quantities result in about 33 placement days assuming approximately 220cy per day. A three foot levelling lift then five foot lifts are assumed in each bay. The following placement diagram was developed from the top of dam down showing the volumes in each area (buttress excluded).

Table 11 Option 1 Concrete Yard Volume Layout

2	**3**	4	5	6	7	8	9	10	11	12
43 cy	***59 cy***	21 cy	21 cy	21 cy	21 cy	21 cy	21 cy	21 cy	21 cy	***43 cy***
73 cy	***100 cy***	31 cy	31 cy	31 cy	31 cy	31 cy	31 cy	31 cy	31 cy	***73 cy***
102 cy	***141 cy***	52 cy	52 cy	52 cy	52 cy	52 cy	52 cy	52 cy	52 cy	***110 cy***
116 cy	***161 cy***	73 cy	73 cy	73 cy	73 cy	73 cy	73 cy	73 cy	73 cy	***132 cy***
69 cy	***96 cy***	93 cy	93 cy	93 cy	93 cy	93 cy	93 cy	93 cy	93 cy	***79 cy***
		104 cy	114 cy	114 cy	114 cy	114 cy	114 cy	114 cy	104 cy	
		62 cy	124 cy	135 cy	135 cy	135 cy	135 cy	124 cy	62 cy	
			75 cy	145 cy	156 cy	156 cy	156 cy	75 cy		
				87 cy	166 cy	166 cy	166 cy			
					100 cy	100 cy	100 cy			

Once the buttress placements are complete and cured they provide an excellent surface to brace the formwork on and this could provide an opportunity to reduce formwork costs with a step form technique. Concrete is assumed all pumped and a delivered price of $130/cy to the site is assumed. . The three end bays that are mass concrete with the DS face sloped are small enough that they do not significantly affect the placement approach. These will require some small amount of additional side formwork as they are brought up in conjunction with the remaining bays. All formwork and reinforcement crews have a crane and a manlift, and all placement crews have a manlift assigned to them.

3.4.8. Mass Reinforcement: It is assumed a 12"x12" layer of #10 reinforcement would be applied on the front, back, and sides of each bay. A 3x3 pattern of anchors was utilized to secure the reinforcement to the existing dam where applicable. The backs and side reinforcement under the dam crest will tie to the buttress reinforcement as applicable.

Table 12 Cost Estimate for Option 1

Option 1- Buttress Reinforcement with Gravity Arches							
04	**DAMS**						
	Mob/Demob construction Contractor	1	EA	$210,529	$63,159	$273,688
	Sitework Preparation	1	EA	$393,316	$117,995	$511,311
	Foundation Preparation and Excavation	2536	CY	$154,388	$46,316	$200,704
	Prepare Existing Dam	1	EA	$333,224	$99,967	$433,191
	Concrete Reinforcement of Dam	7700	CY	$6,055,620	$1,816,686	$7,872,306
30	**Design (USACE estimate)**	1	EA	$1,000,000		$1,000,000
31	**Construction Management (10% of construction)**	1	EA	$714,708	$214,412	$929,120
TOTAL				**$8,861,785**	**$2,358,536**	**$11,220,321**

3.5. Option 2: Mass Concrete

3.5.1. Overall Approach: Stabilize the dam with mass concrete placement behind existing dam with a 2:1 backside slope from the crest downstream.

3.5.2. Foundation Excavation: The foundation area is assumed to be excavated with an excavator mounted hammer, loaded and hauled to an onsite disposal area. A small network of trenches is excavated out for a pair of HDPE foundation drains per bay. It is assumed for each bay that the deepest buttress will drive the overall depth. (This will slightly overstate the volumes and results in the overall height for the three deepest bays to approach 50 feet in depth.) No extraordinary measures were assumed for the foundation preparation pending the results of the site investigations. Actual conditions encountered could require grouting, micro piles or drilled shafts to bring the foundation up to a suitable standard to build on.

3.5.3. Existing Dam Preparation: It is assumed that all exposed areas on the downstream area of the dam will be sandblasted and the debris vacuumed and hauled offsite to disposal. The existing struts between bays will be removed and hauled offsite for disposal and the final foundation excavation is washed prior to concrete placement.

3.5.4. Formwork and Placement: For this option it is assumed that the bay between each buttress will be in filled with mass concrete with 5 foot lifts. Due to the varying height of each bay the downstream depth varies for each bay. It is assumed that there will be a 3’ levelling lift, then each lift beyond that will be placed in 5’ heights. A placement diagram was developed showing the CY in each placement by bay across the dam. Due to the 8 foot foundation assumption, the overall height in the deepest arch is approximately 48feet, requiring 10 lifts.

Table 13 Option 1 Concrete Yard Volume Layout

2	3	4	5	6	7	8	9	10	11	12
43 cy	39 cy	98 cy	98 cy	66 cy	52 cy	56 cy	52 cy	98 cy	98 cy	43 cy
73 cy	59 cy	118 cy	118 cy	95 cy	66 cy	73 cy	66 cy	118 cy	118 cy	73 cy
102 cy	100 cy	137 cy	137 cy	122 cy	95 cy	106 cy	95 cy	137 cy	137 cy	110 cy
116 cy	141 cy	158 cy	158 cy	150 cy	122 cy	138 cy	122 cy	158 cy	158 cy	132 cy
69 cy	161 cy	182 cy	182 cy	177 cy	150 cy	172 cy	150 cy	182 cy	187 cy	79 cy
	96 cy	194 cy	213 cy	205 cy	177 cy	204 cy	177 cy	213 cy	205 cy	
		117 cy	231 cy	234 cy	205 cy	239 cy	205 cy	232 cy	123 cy	
			138 cy	246 cy	238 cy	278 cy	236 cy	139 cy		
				148 cy	255 cy	296 cy	249 cy			
					153 cy	178 cy	149 cy			

The formwork will be cantilever braced from the placement below it. Side forms will be required on every other bay as it is assumed that a staggered approach will be utilized to bring up the concrete. A ROM daily placement of about 220 cy per day was assumed. This would require 61 placement days based on volume, however, there are also 84 placement areas and due to the large volume of concrete in many of them an average of 73 days was assumed for costing the placement. A quote for formwork was obtained and utilized to cost the formwork. It was assumed that adequate formwork to work 8 bays nearly simultaneously would be rented. Several local vendors were contacted and the price of concrete was verified. Due to the typical USACE Specifications for concrete, additives, and testing 130/cy was used in the estimate for delivered concrete. All concrete is assumed pumped to placement and allowances were added for this loss. A light sandblast is assumed between each placement to aid in adhesion and a light rub finish applied as well as curing compound applied. All formwork and reinforcement crews have a crane and a manlift, and all placement crews have a manlift assigned to them.

3.5.5. Reinforcement: it was assumed a 12”x12” layer of #10 Reinforcement would be applied on the front, back, and sides of each bay. A 3x3 pattern of anchors was utilized to secure the Reinforcement to the existing dam where applicable.

Table 14 Cost Estimate for Option 2

Option 2-Mass Concrete		Quantity	U/M	Estimate	Contingency	Total
04	**Dams**					
	Mob/Demob Construction Contractor	1	EA	$210,470	$63,141	$273,611
	Sitework Preparation	1	EA	$393,316	$117,995	$511,311
	Foundation Preparation and Excavation	3700	CY	$206,889	$62,067	$268,956
	Prepare Existing Dam	1	EA	$342,409	$102,723	$445,132
	Mass Concrete Reinforcement of Dam	12305	CY	$7,081,987	$2,124,596	$9,206,583
30	**Design Cost(USACE Estimate)**	1	EA	$1,000,000		$1,000,000
31	**Construction Management (10% of construction)**	1	EA	$823,507	$247,052	$1,070,559
TOTAL				**$10,058,578**	**$2,717,573**	**$12,776,152**

3.6. Other Estimate Assumptions:

3.6.1. Labor: Adequate labor is available in the area to perform the work. Davis Bacon rates for Amador County from 11/6/2015 are utilized.

3.6.2. Equipment: M2 Equipment library 2014 Region 7 is utilized.

3.6.3. Materials: All materials are commercially sourced. There are several batch plants and rock quarries within 30 miles capable of meeting the demand for the project.

3.6.4. Cost Book: M2 English 2012b is utilized.

3.6.5. Unusual Conditions: The project site is privately owned but the dam repair is under control of the EPA and State of California. It is assumed that access to the site will be arranged by one of these two agencies and that the owner will not intervene to stop the project.

The upstream areas are contaminated with mine byproducts and no work or equipment will be allowed upstream of the dam. It is assumed that a small area will be developed from the north end of the dam for concrete delivery and pumping.

3.6.6. Schedule: The project is slated to advertise so that construction can begin in the summer of 2016.

3.6.7. Overtime: The project assumes 5-10 overtime would be utilized to for construction

3.7. Project Construction Risks:

3.7.1 Project Risks: There are several areas that have the potential to cause a large cost variance on the project. Each alternative is subject to them somewhat to varying degrees. The following risks are noted for consideration as the project progresses:

3.7.1.1 Unknown Foundation Conditions. The existing site conditions of rock for the foundation are currently unknown. The results of the investigation are critical as to which alternative may pose the least risk and consideration of the potential variances in the site conditions and the cost and schedule implications of each design should be considered. For example: the mass alternative may be more tolerant of poor foundation conditions but would still require a reasonable bearing strength to key in to resist sliding forces. Depending on the results of the geotechnical investigations and required foundation repairs, the buttress alternative with a smaller area of more robust repair may be a less costly option. The cost and schedule sensitivity of each design should be determined based on the range of foundation conditions that could be experienced across the alignment and the design developed accordingly. Based on the findings a combination of the two alternatives may be an option where bays you have mixed repairs based on each arches differing geotechnical conditions.

3.7.1.2 Interim Construction Stability. The existing dam could be effected by construction activities and consideration to the proximity of excavation and reasonable restrictions need to be developed for both excavation and concrete placement activities. Working from the ends towards the middle may provide additional stability to the structure during construction while working alternating bays simultaneously may provide cost and schedule savings.

3.7.1.3 Project Delivery Schedule. The project has an aggressive delivery schedule in order to advertise and be constructed in the FY 16 construction season. This poses several risks in itself to the project.

3.7.1.3.1 Design Assumptions. The design may not have time to be optimized and may result in a higher than assumed cost due to conservative assumptions that will need to be made to make progress to meet the schedule.

3.7.1.3.2 Bidding Climate. With the aggressive schedule many contractors may be booked for FY 16 and would take on this project only at an attractive price or may rely more heavily on subcontractors due to limited crew availability.

3.7.1.4 Site Contamination. The alternative estimates assumed that a significant cost would expended to clean and cap the downstream area to provide an essentially clean work area for the repair contractor. This could make the project more attractive to bidders. If this is not completed prior to the construction contractor coming to the site or is combined into a single contract, this could affect the type of contractor bidding the project as well as the overall project costs. The alternatives estimates assumed a conservative subcontracting approach assuming that this area may not be done in time or that construction contractors may be leery to bid as prime contractor.

3. 8 Project Risk and Cost Perspectives:

A memorandum was developed by Dan Hertel as part of his Agency Technical Review of this report, and is both excerpted below and included in full in Appendix B. The purpose of Mr. Hertel's memo is to assist decision makers involved in design selection that, from a risk and cost perspective, these alternatives receive adequate comparison. Mr. Hertel's directly quoted observations below are made only in the overall interest of the project and to facilitate decision makers in the selection of an alternative for final design.

- The current cost estimate for the Mass Concrete Buttress (Option 2) includes 171 tons of reinforcing steel at a direct cost of $553,000. With markups and contingencies, this translates to over $ 1 million in cost. I believe that the final design for this alternative could include significantly less reinforcing steel, perhaps only on the downstream face, if any. This consideration has the potential to lower the cost of the Mass Concrete Buttress option by approximately $500,000 to $1,000,000. This should be explored further.
- The current geometry of the Mass Concrete Buttress (Option 2) calls for an approximately 1:1 downstream slope of the buttress. It seems possible to me that this geometry could be refined slightly to reduce the volume of mass concrete. If the downstream slope of the buttress could be steepened to a 0.8:1 slope as with many gravity dams, or if each bay could utilize a "U" or "V" notch in the downstream face, a reduction of as much as 1200 CY of mass concrete might be realized. With markups and contingencies, this reduction in concrete volume could translate to a savings of about $600,000. This should be explored further.
- Neither alternative is very complicated, but the Mass Concrete alternative is definitely more straightforward, which has several advantages. The Arched Buttress alternative (Option 2) has the added element of forming and placing the high buttress walls. From a scheduling and access perspective, this added element may complicate the sequence of work. Buttress walls would need to be placed prior to forming and placing mass concrete. If some buttress wall work is planned concurrent to some mass concrete work, then there would likely be a conflict of access, since most access will be from the downstream access road. There is the potential for added complication if the contractor is limited in the number or sequence of foundation excavation for dam stability during construction. This would make the Arched Buttress alternative (Option 2) even more risky, less desirable, and add duration to the construction schedule.
- I would expect the Mass Concrete (Option 2) to receive a few more bids. The project does not require any real concrete expertise. Mass concrete is fairly straight forward. The buttress option requires a little more expertise in forming the buttresses and the arched (radius) walls.
- Option 2, Mass Concrete, is a simpler, more straightforward project from a construction point of view. I would expect a faster construction schedule on this option, with fewer construction and schedule risks.