ARGONAUT DAM STABILITY AND RETROFIT ALTERNATIVE INVESITGATION JACKSON, CA

PHASE I TECHNICAL REPORT

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EXECUTIVE SUMMARY

General Background

The purpose of this report is to investigate the stability of the Argonaut Dam according to current Engineering Manuals (EMs) and Regulations (ERs). The multiple arch dam was constructed around 1916 for the purpose of retaining mining tailings. The dam over the past century has been neglected in terms of inspections, maintenance, and repair. Concerns of the dam's structural integrity combined with its close proximity to buildings have resulted in a preliminary structural stability evaluation. The results of the stability evaluation showed that the dam failed to meet the minimum EM requirements for multiple load cases with the most severe being sliding shear through concrete. The dam should undergo a 35% repair / retrofit evaluation to avoid possible failure and loss of life downstream.

Structural:

Phase I Analysis Summary and Recommendations

The analysis of the tallest concrete arch section revealed failure in several stability checks. The most concerning of these failures was sliding shear through the concrete cross section. The analysis was performed using a finite element model (FEM), which evaluated the tributary arch section for four different load cases that involved both static and dynamic loadings. It is recommended that the dam undergo a 35% design evaluation, which will compare various repair methods that can be used to stabilize the dam against the four load cases evaluated within this report.

1. INTRODUCTION

1.1 BACKGROUND

The Argonaut dam is a multiple arch concrete dam constructed around 1916 for the purpose of storing mining tailings. The dam, from historic documents, was described as being 420-450ft long and 46-50ft tall at its highest point. The dam consisted of 13-14 arches, which ranged in thickness from 18” at the base to 12” at the top. The arches along with the 15 buttress walls were reinforced with a 1” or 1 1/8” 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’-1”. 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.

Following two inspections by the US Army Corps of Engineers (USACE) and Environmental Protection Agency (EPA) in 2013 concerns of the dams overall stability were raised. 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. The assessment of the dam, detailed herein, will be 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. The inspectors included Chris Abela, PE, and Ken Pattermann, GE, of USACE, and Dan Shane of the EPA.

The following paragraphs provide a brief timeline of the Argonaut Dam construction and inspection history:

1916 **-** During spring and summer the Argonaut Dam was constructed by John S. Eastwood for the purpose of storing the tailings from the new quartz mill, which was being erected. The total number of cubic yards of concrete in the structure was recorded to be 123 and the total over-all cost of the contract was $21,680. Construction of the structure began on the first of March and was completed in July.

The dam was noted to be 450ft in length, 50ft at its highest point, and consisted of 14 arches and 15 buttresses. It was designed on the assumption that the tailings from the quartz mill are the equivalent of a liquid having a unit weight of 75pcf.

1917 **-** Argonaut Mine No. 478 (OJ) was constructed.

1929 **–** An application for a prior dam construction was filed with the Division of Water Resources.

1930 **–** Inspection performed by S. A. Hart on 21 May. Concrete work seems to be of good quality. There has been some leakage through the dam face. For the most part it was minor and there are very few wet spots or seeps. There is some leakage beneath the toe, particularly at the center of the structure. The entrance to the spillway is badly choked with cat tails and willows, etc. The application states that a 16" pipe drains from the bottom of the dam. There is an old 16" steel pipe lying in the rubbish at the toe of the dam, but it does not appear to act as a drain. No evidence of a gate could be found. It has probably been filled and is no longer used.

1930 **–** Inspection of the dam performed by W.H. Homes, G.E. Goodall, and I. Nelidov on 5/25/30. A note was left in the record of inspection for structural action. (Unknown what structural action note was in reference to.)

1931 **–** Inspection of the dam was performed by G. McKinlay with a note specifying that the spillway was in satisfactory condition. A follow up inspection was performed by G. E. Goodall with a note saying debris conditions were in question and as a result samples were obtained. (It is unknown why the debris conditions were in question.)

1932 **–** Inspection of the dam was performed by S. A. Hart. Hart left several drawings, which recorded his observations of the arches and crack locations. In addition, Hart also provided estimates of stresses in the arch crown, and abutments.

1932 **–** A certificate of approval for the dam was issued. (Who and or What the certificate was issued for is unknown)

1933 **–** A Legislative Act amended the Division's responsibilities from dams impounding 10 acre-feet to those impounding 15 acre feet, which excluded Argonaut Dam.

1933 **–** The dam was inspected by G. F. Engle. Engle made several observations during the inspection, which included:

- Water flowing over one of the two spillway entrance channels at a depth of 3"
- Crown cracks in most of the arches on the downstream face about 1ft in length
- Several "spring line cracks" in arches #6-#12 were found and noted by the engineer to be new developments.
- The inspector postulated that "If these cracks did not exist at the time of former inspections (roughly a year ago) then something has happened to the structure since then. The recent earthquake disturbance centering in Nevada (approximately 2 months ago) may have influenced the present condition."

1937 **–** Argonaut Dam was inspected to confirm that there had been no changes to its condition.

1975 **–** Argonaut Dam was inspected to confirm that there had been no changes to its condition.

2010 **–** Department of Toxic Substances Control, which has been the regulatory agency for the environmental investigation and cleanup of the Argonaut Mine tailings, observed the current condition of the dam stating the following (Author: Tami Trearse Engineering Geologist):

"My observations of the current condition of the dam have caused me concern. The integrity of the dam was brought up during the Jackson City's site visit with their consultants and DTSC on the proposed Sutter Street Extension that would be located in close proximity to the dam. The apparent disrepair of the dam is of great concern to DTSC.

Assessment of the condition of the dam is not within DTSC's jurisdiction. I contacted the California Department of Water Resources (DWR) which oversees dams within the state. I spoke with Larry Ford at DWR about the Argonaut Dam. Mr. Ford explained that Dams that impound less than 15 acre feet of water are the responsibility of the County in which the dam is located. Amador County should have received a letter from DWR around 1933 when the dam left DWRs jurisdiction and became the responsibility of Amador County."

Pictures were attached to the report showing the condition of the dam during heavy rain events, in which water can be clearly seen spilling over the top of the dam. The picture was dated April 4, 2006.

The letter continues with the following statements:

"The dam is located in a steep ravine/creek bed which ends near Highway 49 and across from the commercial area of the city of Jackson. There is a potential for the dam to fail and tailings to move down gradient along the creek bed or Sutter Road to Highway 49."

"This letter is to inform both the land owner and the Amador County Public Works Department of the observed condition and possible consequences of a failure of the Argonaut Dam. It may be prudent for responsible entities to assess the need to take preventative measures associated with the condition of the dam."

2013 **–** On July 9th, an inspection of the dam was performed by USACE and the EPA. From the inspection the following recommendations were made by USACE (Chris Abela PE and Ken Pattermann GE):

- The dam should undergo a preliminary seismic evaluation in accordance with USACE standards.

- Vegetation downstream of the dam should be cleared and removed exposing the remaining condition of the arches, buttress braces, and buttress walls.

- A second site visit after the vegetation has been cleared should be performed by a structural engineer to investigate the condition of the remaining 10 arches that could not be previously inspected.

- If the seismic study is funded, several concrete core samples should be taken to determine the compressive strength of the existing concrete. Sampling of the concrete cores should be performed under the guidance of the appropriate ACI codes and ASTM standards.

***2013* –** On November 15th a second inspection of the dam was performed by USACE and the EPA. With the vegetation removed 13 arches were visible for inspection. Observations regarding the condition of the arches, buttress walls, and tie beams were documented in a memorandum. Based on the observations the following recommendations were provided:

- The dam should undergo a preliminary static stability and seismic evaluation in accordance with USACE standards, (ER 1110-2-1156, EM 1110-2-2100, & EM 1110-2-6053) to identify stability and seismic deficiencies.

- Upon receiving funding for the preliminary static stability and seismic study, several concrete core samples are recommended to be taken for determining the compressive strength and unit weight of the existing concrete. Sampling of the concrete cores should be performed under the guidance of the appropriate ACI codes and ASTM standards.

- Provide recommendations on the repair and future analysis of the dam in the event that stability and/or seismic deficiencies are found.

- Conduct Standard Penetration Test (SPT) drilling and geotechnical lab testing on foundation soils. (Pattermann and Abela, 2013)

- Conduct geotechnical seepage and stability analysis of the upper earth tailings dam. (Pattermann and Abela, 2013)

In regards to earthquakes and arch dams, according to the Federal Energy Regulatory Commission (FERC) of 600 dam incidents (including failures) only 2 have involved multiple arch dams. These two multiple arch dam failures included: Gleno Dam in Italy, which was completed in 1923 and failed only 30 days after filling, and Leguaseca Dam in Spain, which was completed in 1958 and failed in 1987 due to deterioration from aging and freeze thaw cycles.

According to (FERC, 1999) from a seismic perspective arch dams have an excellent record of performance with respect to earthquake motion. No failure has occurred in an arch dam as a result of an earthquake. However, it should be noted that very few maximum credible earthquakes (MCE) have occurred close enough to arch dams to truly test their performance and durability. In addition, (FERC 1997) also noted that buttresses, like those used in multiple arch dams, when unreinforced or unbraced, are susceptible to damage from lateral earthquake loading.

1.2 SCOPE

This report involves the investigation of the following structures:

1) (1) Tributary Arch Dam with Buttress Walls

The following scope of work identifies the tasks to complete:

Task 1. - Evaluate the static and dynamic stability of a tributary section of the dam and provide recommendations.

As the investigation of Argonaut Dam progresses the task list will increase to include other important failure modes such as localized failure in one of the arch sections, a cross canyon seismic loading condition, and or a flood inundation map that looks at the downstream consequences of a dam breach. However, the intent of the Phase I investigation presented here-in, is to evaluate the stability of the dam in the upstream downstream direction and determine if further action is warranted.

Figure 1 Site Location Map

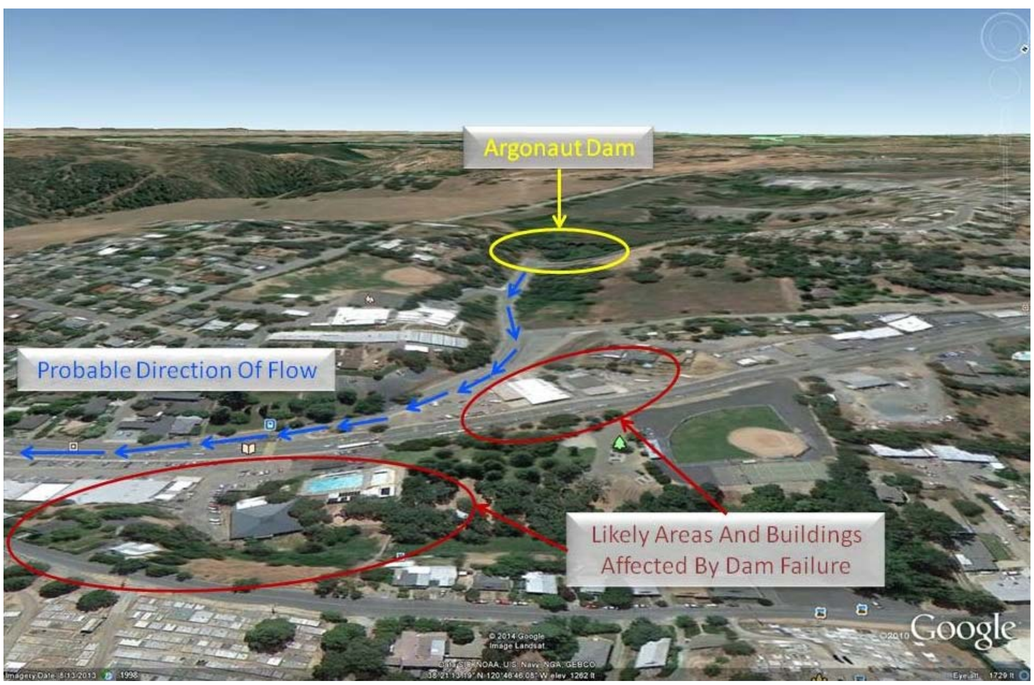

Figure 2 Site Elevation Map

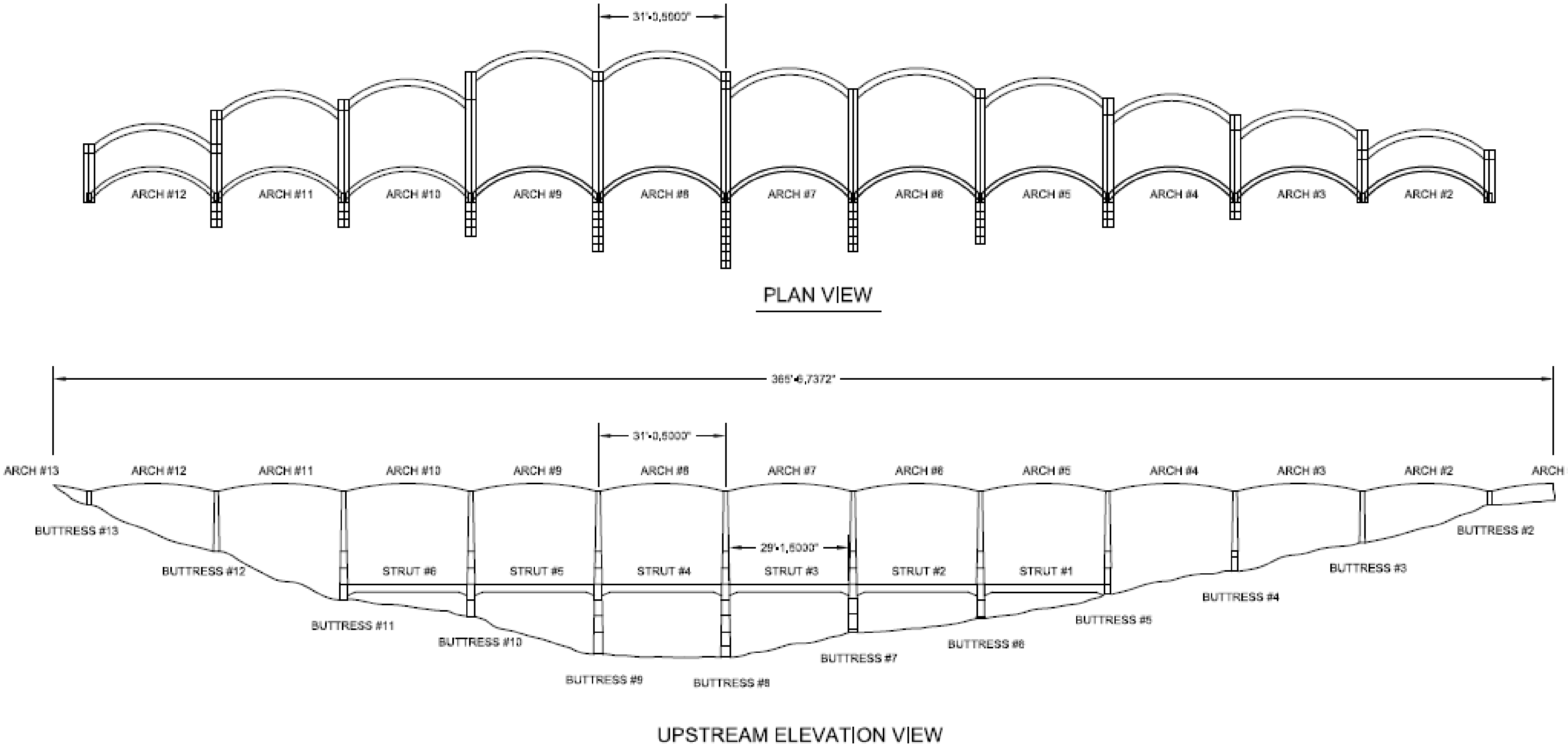

Figure 3. Plan and Elevation View of Dam

1.3 STABILITY ANALYSIS OF THE DAM

Chapter 7 of EM 1110-2-2100, which is titled, “Evaluating and Improving Stability of Existing Structures” along with ER 1100-2-1156 describe a procedure to be followed when evaluating the static and dynamic stability of a concrete structure. The procedure is broken down into Phases with Phase I (Preliminary Analysis and Evaluation) being the first step in analyzing the structures stability. Under this phase it is recommended that a 2D analysis be performed to determined stability adequacy of the dam and to determine if the analysis should progress to Phase II, which consists of a more comprehensive analysis.

As a result of the complex geometry, a 3D finite element analysis (FEA) model versus a 2D FEA model was used for the stability investigation. The model for this phase of the investigation was composed of only the tallest arch, Arch #8. The arch was analyzed for sliding, floatation, overturning, and bearing stability under both static and dynamic load cases related to usual, unusual, and extreme events. It is important to note that because USACE documents don’t specifically address the analysis of multiple arch dams, ER and EM documents were supplemented with Federal Energy and Regulatory Commission (FERC) documentation, which does provide guidance on the analysis of multiple arch dams. In addition, due to concerns that the failure of the dam has the potential to cause loss of life, the structure was assumed to be critical.

The following tables were taken from EM 1110-2-2100 to illustrate the stability guidance that was followed in Phase I of this investigation.

Table 1. Load Condition Probabilities

Load Condition Categories	**Annual Probability (p)**	**Return Period (tr)**
Usual	Greater than or equal to 0.10	Less than or equal to 10 years
Unusual	Less than 0.10 but greater than or equal to 0.0033	Greater than 10 years but less than or equal to 300 years
Extreme	Less than 0.0033	Greater than 300 years

- *Usual* - loads refer to loads and load conditions, which are related to the primary function of a structure and can be expected to occur frequently during the service life of the structure. A usual event is a common occurrence and the structure is expected to perform in the linearly elastic range.

- *Unusual* - loads refer to operating loads and load conditions that are of infrequent occurrence. Construction and maintenance loads, because risks can be controlled by specifying the sequence or duration of activities, and/or by monitoring performance, are also classified as unusual loads. Loads on temporary structures which are used to facilitate project construction, are also classified as unusual. For an unusual event some minor nonlinear behavior is acceptable, but any necessary repairs are expected to be minor.

- *Extreme*- loads refer to events, which are highly improbable and can be regarded as emergency conditions. Such events may be associated with major accidents involving impacts or explosions and natural disasters due to earthquakes or flooding which have a frequency of occurrence that greatly exceeds the economic service life of the structure. Extreme loads may also result from the combination of unusual loading events. The structure is expected to accommodate extreme loads without experiencing a catastrophic failure, although structural damage which partially impairs the operational functions are expected, and major rehabilitation or replacement of the structure might be necessary.

Table 2. Required Factors of Safety for Sliding - Critical Structures

Site Information Category	Usual	Unusual	Extreme
Well Defined	1.7	1.3	1.1
Ordinary	2.0	1.5*	1.1*
Limited**	-	-	-

*For preliminary seismic analysis without detailed site-specific ground motion, use FS=1.7 for unusual and FS=1.3 for extreme. See further explanation in section 3.11 b.
**Limited site information is not permitted for critical structures

Table 3. Required Factors of Safety for Flotation - All Structures

Site Information Category	Usual	Unusual	Extreme
All Categories	1.3	1.2	1.1

Table 4. Requirements for Location of the Resultant - All Structures

Site Information Category	Usual	Unusual	Extreme
All Categories	100% of Base in Compression	75% of Base in Compression	Resultant Within Base

<u>Allowable bearing capacity values</u> were based on a geotechnical investigation report, Pattermann (2014), which yielded a value of 45 ksf.

1.4 LOADS AND LOADING CONDITIONS

The selection of the unit weights and strength for the various materials for the project were based on historical documents of similar structures, inspection notes on the Argonaut Dam, a soil boring investigation by URS Corp., and recommendations by Ken Pattermann GE.

Dead Load:

- Unit weight of concrete = 144 pcf (Argonaut Dam inspection notes)
- Compressive strength of concrete = 2500 psi (Estimated compressive strength based on Webber Creek Dam)

Live Loads & Hydraulic Loads:

- Unit weight of water = 62.5 pcf
- Unit weight of moist backfill / mining tailings = 90 pcf (Argonaut Dam inspection notes and Recommendations from Ken Pattermann)
- Unit weight of saturated backfill soil = 115 pcf (Recommendation from Ken Pattermann based in part on URS geotechnical report)

From the URS geotechnical report the ground water elevation within the mining tailing was determined using the boring logs of SB-40 and SB-41, which were the closest to the dam. The boring logs estimated the ground water table to be ~5ft below the ground surface on the upstream side. During the two field inspections the ground surface was measured to be ~3ft below the top of the arches. Together, these values put the ground water table ~8ft below the top of the arches. Based on Pattermann (2014) a value for the drained peak friction angle of the soil, 3 deg, was used.

From the unit weights and historical observations, the usual and unusual static load cases defined by EM 1110-2-2100 were evaluated. The usual static load case was based on the existing conditions found during the URS soil exploration with the ground water ~8ft below the top of the arches. For the unusual load case, the water elevation behind the dam was assumed to be at the top of the arches. This assumption came as a direct result of a picture taken in 2006, which showed water spilling over the arches during a large rain event. It is important to note that due to the lack of information on the Argonaut Dam it was difficult to assume any value related to an extreme water event. Therefore, for Phase I of this investigation, only the usual and unusual loading conditions were considered. However, this assumption may be revisited during future Phases of the project.

Uplift pressure loads under the buttress walls and arch base were applied per the recommendations of the Federal Energy Regulatory Commission (FERC, 1997) chapter 10. This document provided two examples of uplift pressure distributions for buttress dams, one without a slab connecting buttress walls and one with a slab connecting buttress walls. In the case of the Argonaut dam the selected uplift pressure distribution was that of no slab connecting the two buttress walls. This resulted in a small trapezoidal pressure distribution under the arch base, which quickly transitions to a uniform uplift pressure distribution under the buttress walls. See figures below for illustration.

In regards to the seismic loads, the USGS website was used to determine both the operational basis earthquake (OBE) and maximum design earthquake (MDE) earthquake peak ground acceleration (PGA) values:

Project site location:
Latitude = 38.34889
Longitude = -120.774167

Seismic Loads:

- OBE with a return period of 144 Years = 0.0685g
- MDE with a return period of 949 years = 0.1175g

According to EM 1110-2-2100 an OBE earthquake event is considered an unusual event while the MDE load case is considered an extreme event.

Because the initial construction of the arch was to hold back mining tailings, the most appropriate loading conditions within EM 1110-2-2100 in regards to stability evaluation matched those of a retaining wall design, which has been provided in the table below.

Based on both the static and dynamic loads the following loading conditions were evaluated to determine the stability of the tallest arch:

Table 5. Description and Classification of Load Cases

Load Case	**Loading Description**	**Classification**
LC – 1	Normal Operating	U
LC – 2	Normal Operating + Short Duration Loads	UN
LC – 3	Normal Operating + OBE	UN
LC – 4	Normal Operating + MDE	E

(a) Loading Condition LC - 1 - Normal Operating.

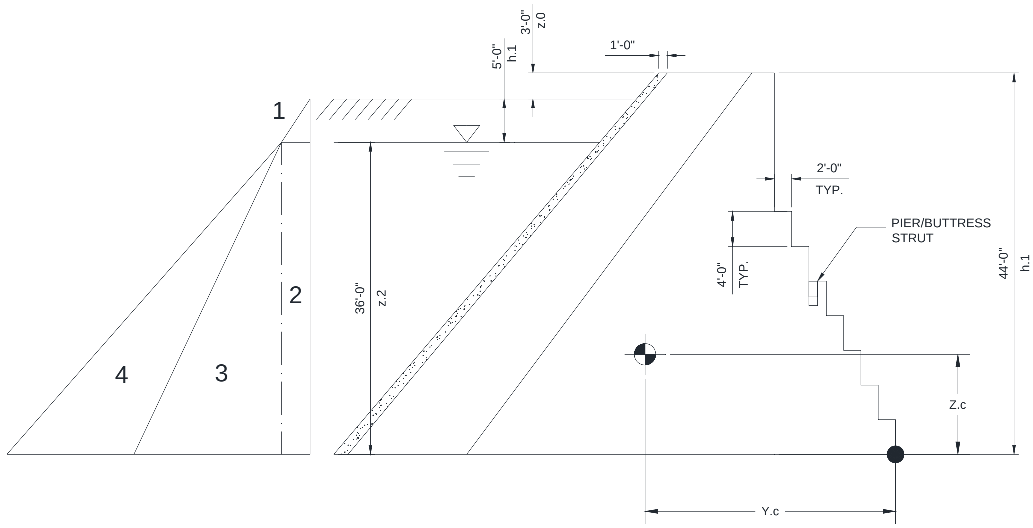

Figure 4. Free Body Diagram of Soil and Water Lateral Loads Acting on Arch for LC-1 (See Appendix for Additional Figures and Loads)

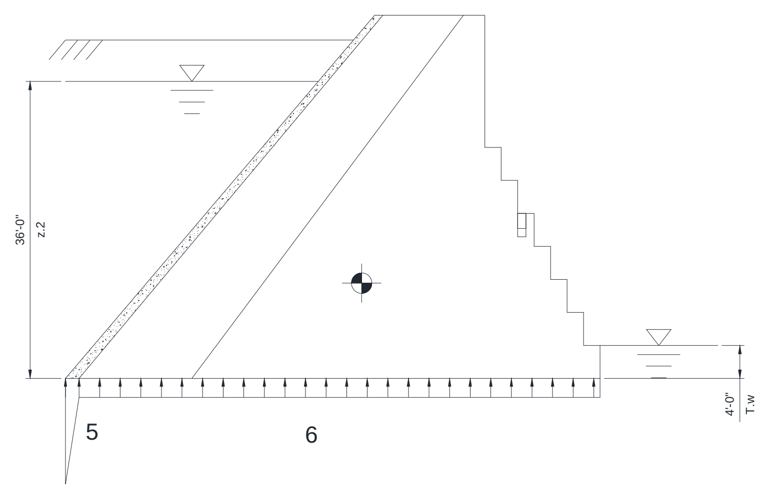

Figure 5. Uplift Pressure Diagram Under Arch for LC-1, LC-3, & LC-4

- Backfill is placed to the final elevation (the backfill is dry, moist, or partially saturated as the case may be).

- Surcharge loading, if present, is applied (stability should be checked with and without surcharge).

- Any existing lateral and uplift pressures due to water are applied.

- Construction loads, which are not considered short-duration loads.

(b) Loading Condition LC - 2 - Normal Operating + Short Duration Loads. This case is the same as LC-1 except the water table level in the backfill rises, for a short duration, or another type of loading of short duration is applied.

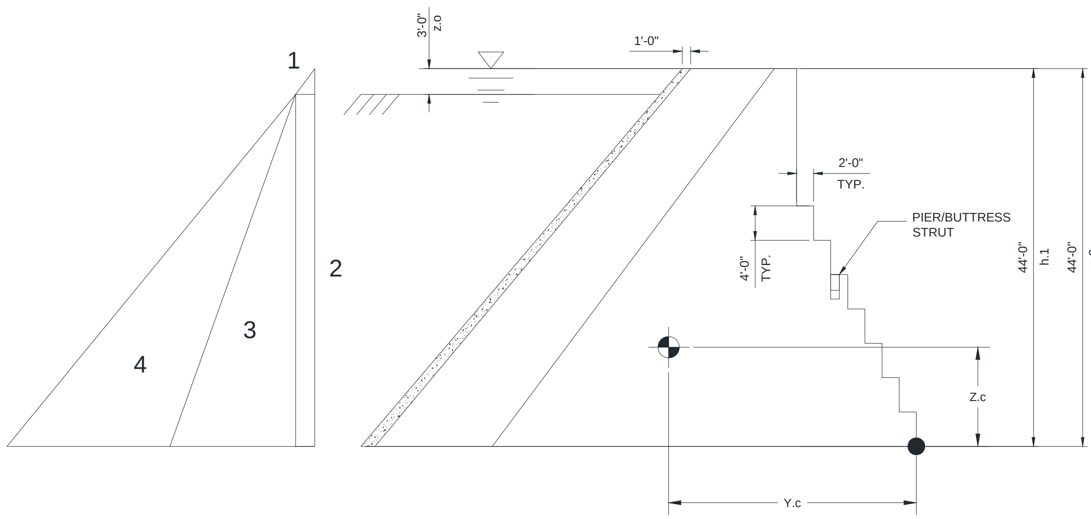

Figure 6. Free Body Diagram of Soil and Water Lateral Loads Acting on Arch for LC-2 (See Appendix for Additional Figures and Loads)

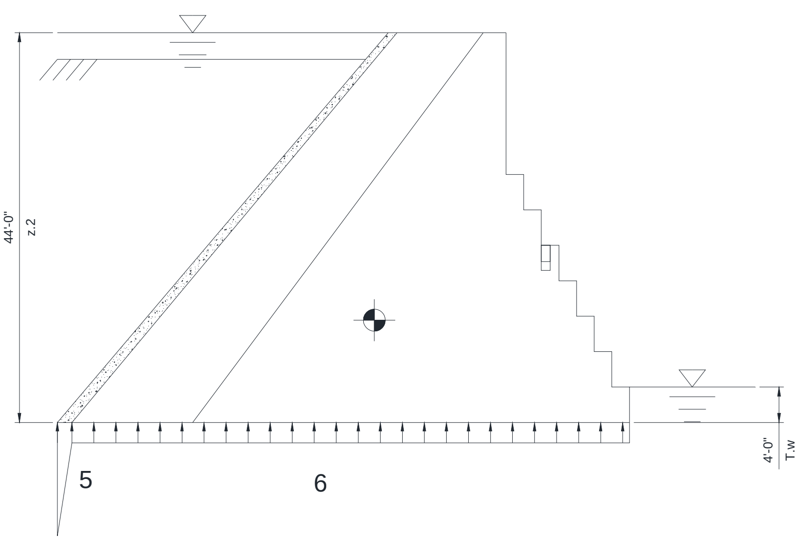

Figure 7. Uplift Pressure Diagram Under Arch LC-2

(c) Loading Condition LC - 3 - Normal Operating + OBE. This is the same as Case LC -1 except with the addition of OBE induced lateral and vertical loads. The uplift is the same as for Case LC -1 .

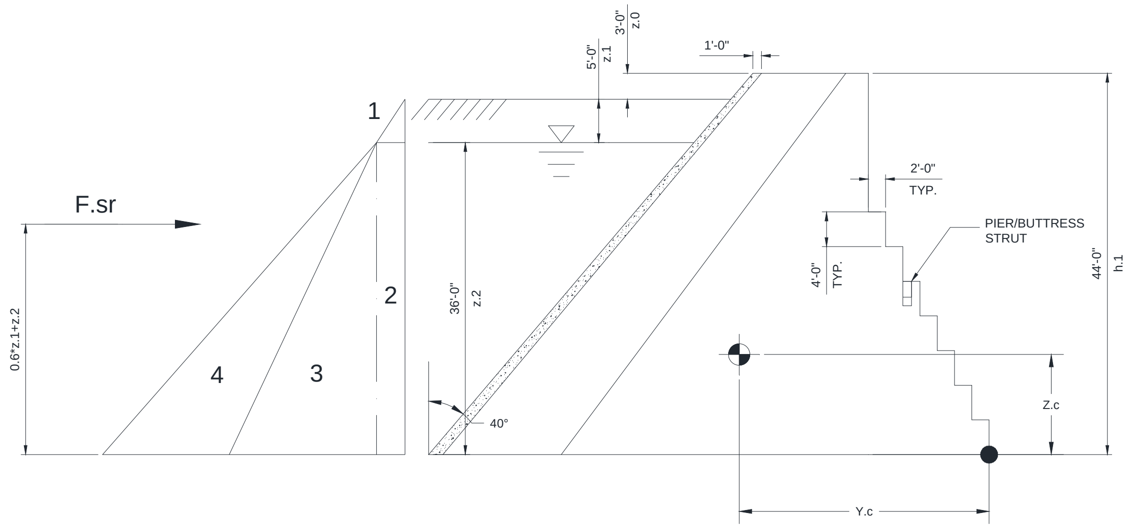

Figure 8. Free Body Diagram of Soil and Water Lateral Loads Acting on Arch for LC-3 (See Appendix for Additional Figures and Loads)

(d) Loading Condition LC - 4 - Normal Operating + MDE. This is the same as Case LC -1 except with the addition of MDE induced lateral and vertical loads. The uplift is the same as for Case LC -1 .

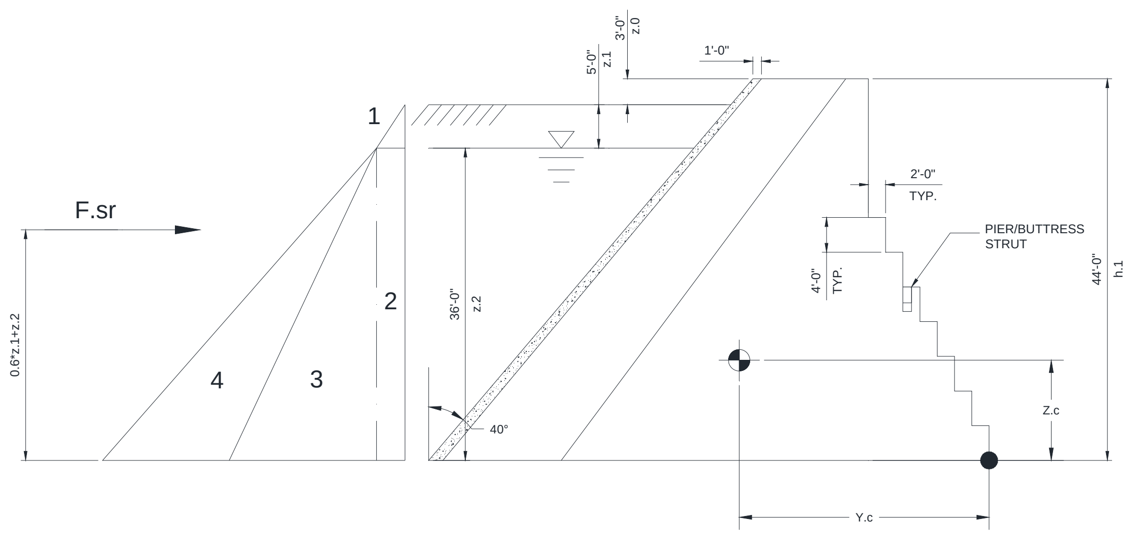

Figure 9. Free Body Diagram of Soil and Water Lateral Loads Acting on Arch for LC-4 (See Appendix for Additional Figures and Loads)

1.5 CONSTRUCTION OF THE FINITE ELEMENT ANALYSIS MODEL

To evaluate the stability of the dam, a finite element analysis (FEA) model of Arch #8 (the tallest arch) was modeled in SAP2000. Construction of the 3D model began by drawing 3D faces in AutoCAD that represented either the buttress walls or the arch. These 3D faces were then imported into SAP2000 and extruded into solids between sets of 3D faces. This method was determined to be the easiest way to create the model and maintain dimensional accuracy. However, in some cases, due to the complex geometry of both the arches and buttress walls, edge constraints were used in locations to insure a smooth stress distribution across solids whose edges did not align. It is important to note that the red springs within the model are a function of SAP2000 restraining an edge from rotating, which is common when edge constraints are assigned. The following excerpt was taken from the CSI Knowledge Base:

"Solid objects activate only three translational degrees of freedom (DOF) at each joint location. Edge-constraint implementation then activates the three rotational DOF. Since solid objects do not provide rotational stiffness, internal rotational springs are generated at affected joints to provide rotational stiffness such that local numerical instabilities do not occur"

An attempt was made to maintain an aspect ratio of 3:1 for all solids throughout the model. However, due to geometrical constraints this rule may have been exceeded in some locations. It is also important to note that the tie beams that spanned between the buttress walls were not included into the model due to their distressed state. The figure below illustrates the final model created.

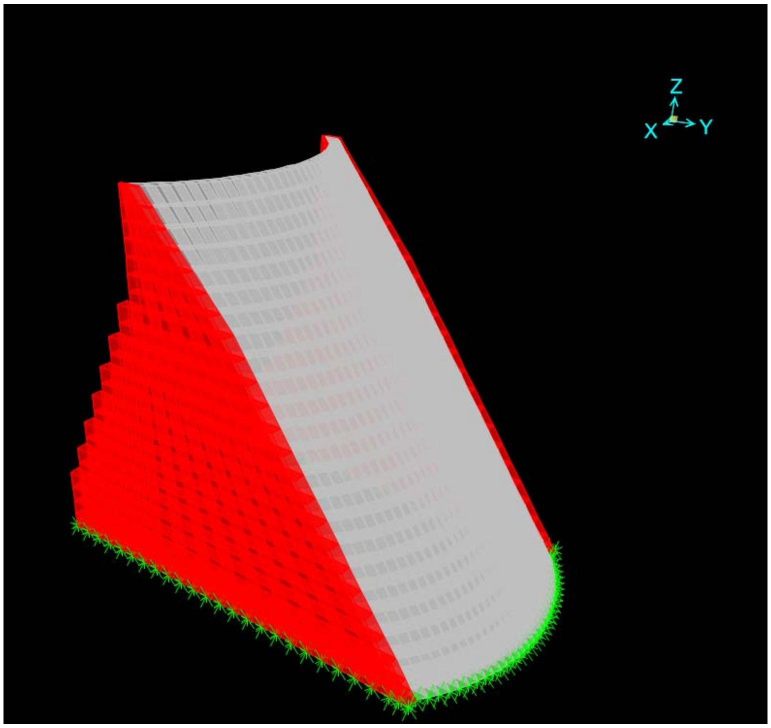

Figure 10. Finite Element Model of Arch #8

The entire base of the foundation was supported on gap links in the Global Z direction with an assumed stiffness of 1000 psi/in. Following a more detailed geotechnical investigation a new value of 1013 psi/in was determined for the modulus of subgrade reaction, see Pattermann (2014). Due to the similarity between the assumed and actual modulus of subgrade reaction values, the FEA models were not updated. Restraints were also applied in the Global X and Y direction to account for the partial embedment of the arch and buttress walls.

Loads were applied to the model using surface pressures based on joint patterns for the water loads and soil loads, see Appendix A for calculations. For the dynamic loading of the soil a joint load was applied at 0.6*height of the soil which was based on EM 1110-2-2100, EM 1110-2-2502, and the assumption that the soil was a non-yielding backfill due to the arch shape. For the inertial loading of the arch and buttress walls, a multiplier was applied into the SAP model as 2/3s*PGA for both the OBE or MDE earthquakes. For further information regarding this seismic coefficient method, please refer to EM 1110-2-2100 section 4.7b and EM 1110-2-6053 section 4.2a. It should be noted that because only the stability of a single arch and its corresponding buttress walls was checked, none of the loads within the model were factored. The figures below illustrate examples of the applied loads and the deflected shape of the model.

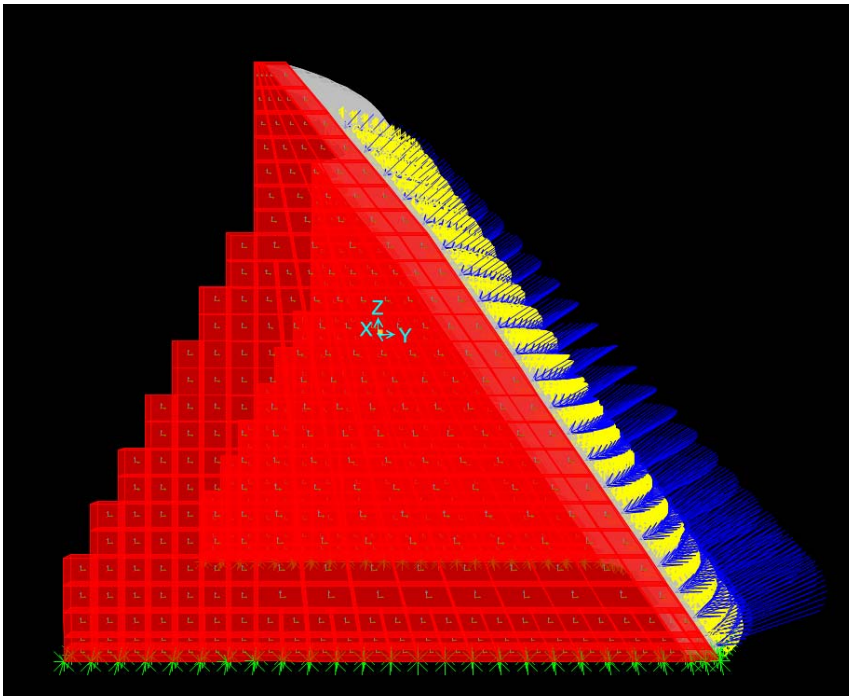

Figure 11. Applied Soil Pressure Loads to Arch

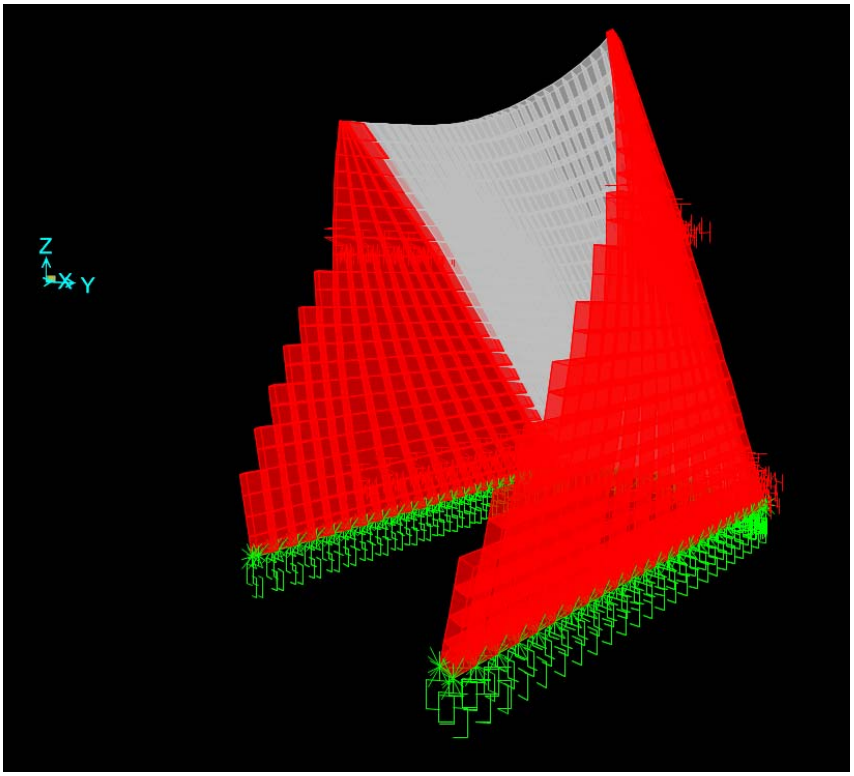

Figure 12. Deflection of Arch #8 Isometric View

1.6 STABILITY RESULTS

To evaluate sliding at the base, an internal friction angle (ϕ) of 35degrees was used along with a strength of cohesion (c) value of 10 psi per the recommendations of Ken Pattermann GE. Based on these values the following table provides the results for the sliding factor of safety for each load case based on Eq. 3-1 of EM 1102-2-2100:

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

Table 6. Stability Results for Sliding along Foundation

Load Case	Normal Force (N) (Kip)	Sliding Force (T) (Kip)	Factor of Safety	Required Factor of Safety	Result
LC-1	2302	2543	0.72	2.0	FAIL
LC-2	2690	3137	0.67	1.5	FAIL
LC-3	2319	2858	0.64	1.7*	FAIL
LC-4	2319	3067	0.60	1.3*	FAIL

*See Table 2 for selection of Factor of Safety value.

The normal force and sliding forces were determined from the global reactions within the SAP2000 model. The normal force was captured through the global Z direction, and the sliding force was captured through the global Y direction.

From the results, it is evident that the sliding stability factor of safety fell well short of what is required by the EMs for all load cases. In addition, comparisons between the FEA results and observed field conditions do not correlate well with one another. Based on the results, which show a factor of safety less than 1, evidence of sliding should be visible. Two possibilities are postulated to explain the discrepancy between the model and site conditions. First, the FEA model, which only looks at the tallest arch, may be too simplistic and consequently overly conservative. A more robust model with multiple arches would better capture the current field conditions. Second, it is unknown how deep the walls and arch extend below the ground surface. It is possible that the walls and arch are acting like a shear key preventing sliding from occurring. Nevertheless, given the amount that the safety factor is being exceeded, this failure mode is still of great interest / concern.

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 presents the range of safety values for the sliding shear through concrete failure mode:

Table 7. Internal Stability Results for Sliding Shear through Concrete

Load Case	Normal Force (Kip)	Sliding Force (Kip)	Factor of Safety for μ = 0.6	Factor of Safety for μ = 1.4	Required Factor of Safety (Based Table 6 Values)	Result
LC-1	2430	2348	0.58	1.35	2.0	FAIL – FAIL
LC-2	2753.16	3012.62	0.55	1.28	1.5	FAIL – FAIL
LC-3	2365.48	2745.44	0.52	1.21	1.5	FAIL - FAIL
LC-4	2365.48	2953.15	0.48	1.12	1.1	FAIL - OK

From the results it can be seen that only one of the selected friction coefficient values reached the minimum required factor of safety, see LC-4. Of concern, is the extremely low factor of safety values for the friction coefficient value of 0.6 (concrete cast against a hardened surface). Given the amount of efflorescence and algae along some of the cracked planes it is highly unlikely that a friction coefficient of 1.4, which correlates to monolithically placed concrete, is still currently in existence. The figures below illustrate the assumed shear plane failure location and the shear stress within the arch and buttress walls.

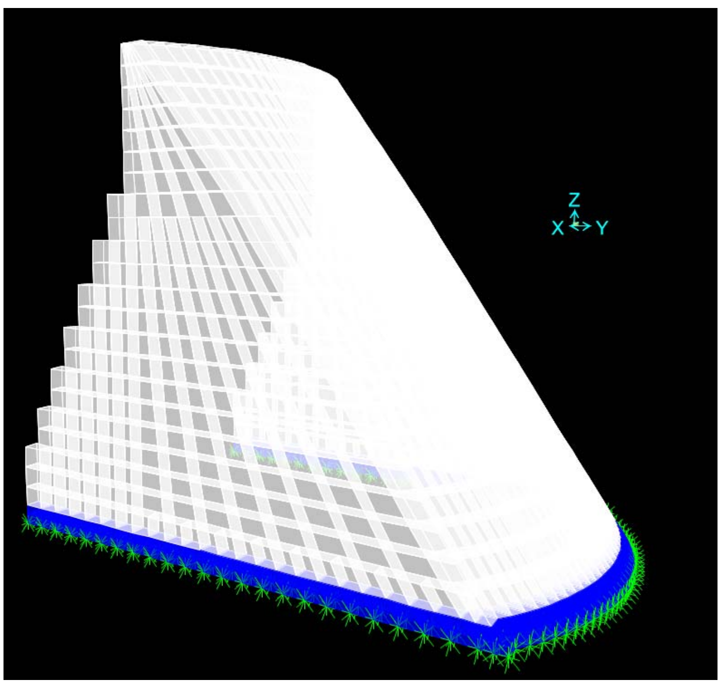

Figure 13. Assumed Shear Plane Failure Location Shown in Blue

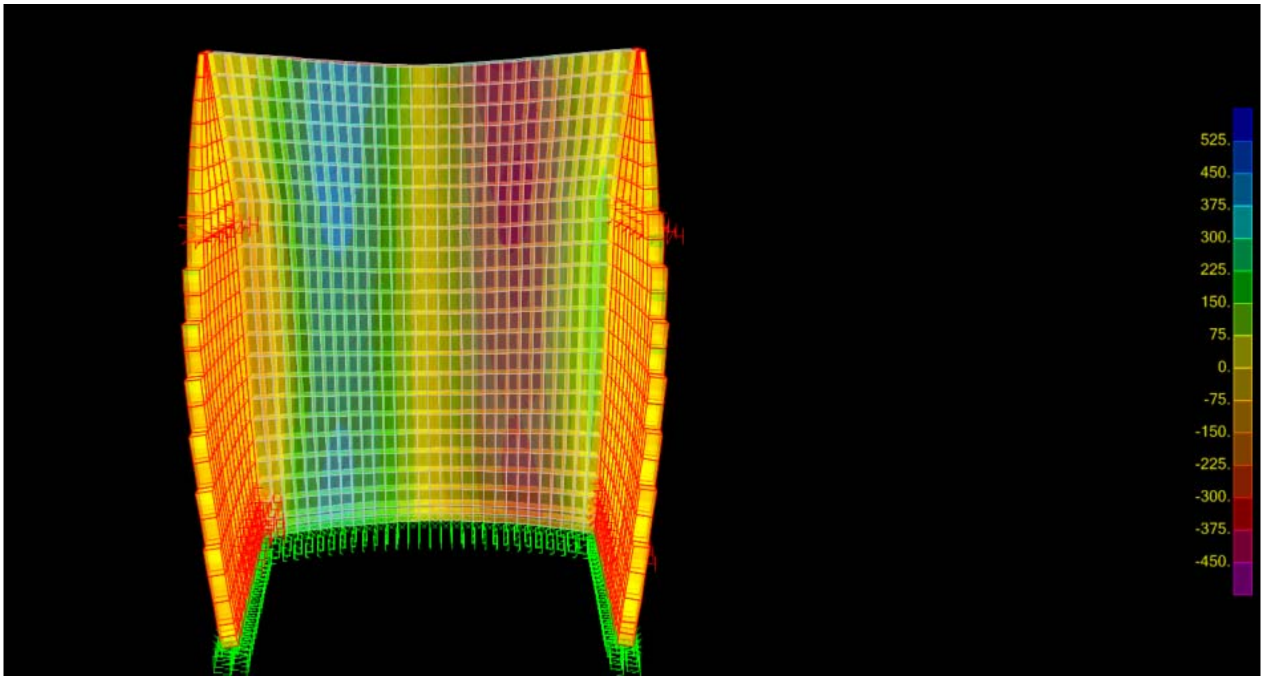

Figure 14. Shear Stress in Arch (psi)

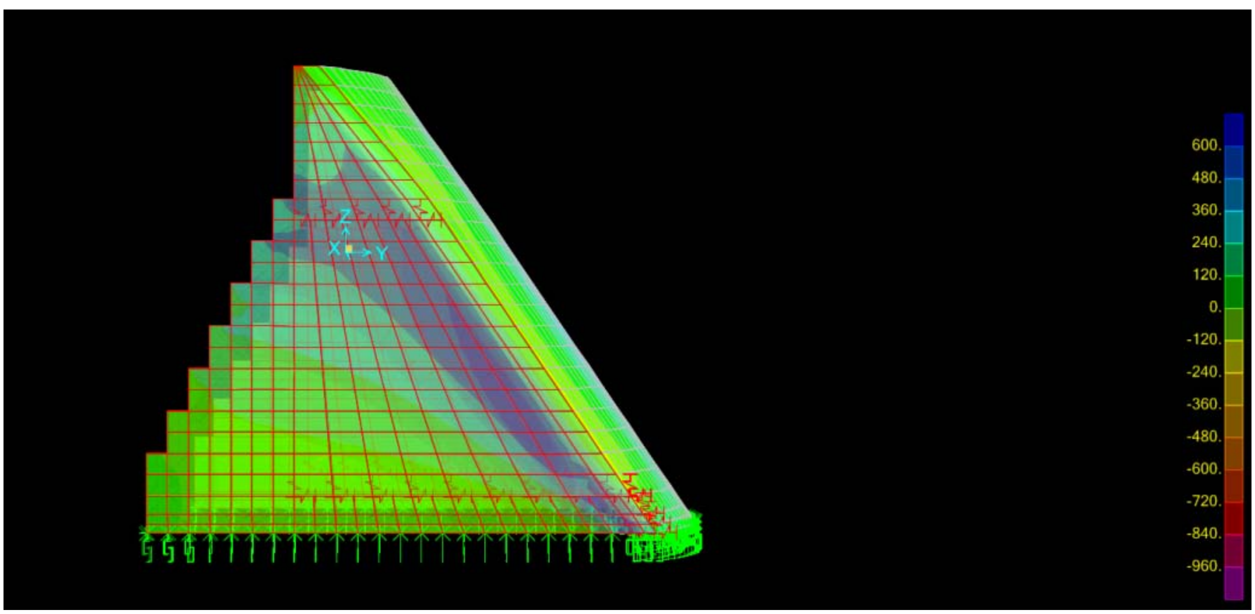

Figure 15. Shear Stress in the Buttress Wall (psi)

The factor of safety for floatation 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 8. Stability Results for Flotation

Load Case	Total Vertical Load W_S+W_C+S (Kip)	Uplift Force U (Kip)	Factor of Safety	Required Factor of Safety	Result
LC-1	2302	80.81	28.49	1.3	OK
LC-2	2690	92.38	29.12	1.2	OK
LC-3	2319	80.81	28.70	1.2	OK
LC-4	2319	80.81	28.70	1.1	OK

To determine the percent of the base in compression, the FEA model was 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 9. Stability Results for % Base in Compression (Arch & Buttress Walls)

Load Case	Total Number of Base Joints in Compression	Total Number of Base Joints	% Base In Compression	Required % Base In Compression	Result
LC-1	128	160	80.00%	100%	FAIL
LC-2	115	160	71.88%	75%	FAIL
LC-3	111	160	69.38%	75%	FAIL
LC-4	103	160	64.38%	> 0%	OK

Table 10. Stability Results for % Base in Compression (Buttress Walls Only)

Load Case	Total Number of Base Joints in Compression	Total Number of Base Joints	% Base In Compression	Required % Base In Compression	Result
LC-1	102	102	100%	100%	OK
LC-2	102	102	100%	75%	OK
LC-3	102	102	100%	75%	OK
LC-4	100	102	98.04%	> 0%	OK

The first table looks at the entire base of arch 8, which includes the two buttress walls and the arch itself. Under this analysis the arch fails to meet the EM's minimum requirement for base in compression for load cases 1-3. However, intuitively for this structure to fail the buttress walls must also lift up and possibly overturn. Therefore, a second analysis was performed, which looked at the percent base in compression of the buttress walls only. Under this approach the buttress walls were determined to be 100% in compression for nearly all load cases and were not in danger of destabilizing. Between the values of the two results it was determined that the structure met the minimum percent base in compression requirements of the EM and was stable for this load case.

To evaluate the bearing capacity of Arch #8, 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, solid element 912, which has an area of 2.54 ft^2 was determined to be the element in each load case that had the highest gap link reaction values surrounding it. The tabulated values for the max bearing pressure have been provided below.

Table 11. Stability Results for Bearing Capacity

Load Case	Area of Selected Solid Element (ft2)	Load Acting On Element (kip)	Max Bearing Pressure (ksf)	Allowable Bearing Pressure (ksf)	Result
LC-1	2.54	65.37	25.74	45.00	OK
LC-2	2.54	89.72	35.32	45.00	OK
LC-3	2.54	82.46	32.46	51.75	OK
LC-4	2.54	94.67	37.28	67.50	OK

From the results it can be seen that within each load case the allowable bearing capacity was not exceeded. It is important to note that EM 1110-2-2100 allows a 15% increase in bearing capacity for unusual load cases and a 50% increase in bearing capacity for extreme load cases.

1.7 DISCUSSION OF STABILITY RESULTS

From the results of the various stability checks, the most concerning failure mode was sliding shear through the concrete. During inspection of the dam, no visible modern (square or circular) reinforcing bars could be seen, even in the sections that were heavily damaged. In addition, the use of ASTM A15 and ASTM A16 steel was not used as reinforcement until 1911 and 1913 respectively, see CRSI 2001. Based on this information it was assumed that no vertical reinforcement existed in the dam. Unfortunately, due to the lack of vertical reinforcement, the visible signs of corrosion (circled in yellow in Figures below), and the visible horizontal cracked concrete planes (shown with arrows in Figures below), there is a high probability that this failure mode may occur. The concern of this failure mode was justified through the results, which showed failure of every load case despite the range of both high and low frictional coefficients values explored.

Concern for the other stability modes that failed also exists, however the results in some cases, such as sliding may improve following the construction of a larger FEA model. In regards to the percent base of compression, although it is not ideal for the arch to be lifting off its base, it was observed that the buttress walls in nearly all load cases were 100% in compression. If the buttress walls also showed signs of significant lift off, this would present a stability concern.

Figure 16. Corrosion of Cables within Arch #8 and Weakened Concrete Planes

Figure 17. Typical Observed Condition of Arches

1.8 PATH FORWARD

From the preliminary results of the Phase I investigation, it is recommended that a larger FEA model composed of a minimum of four of the tallest arches be built to reinvestigate stability values and explore repair / retrofit options to the 35% design stage. The repair / retrofit options will be evaluated from the standpoint of feasibility, constructability, and cost. Following the retrofit comparison, a recommendation on which repair option to pursue up to the 65% design submittal will be provided. It should be noted that seismic loadings, which will be evaluated further in the 35% design submittal, will follow EM 1110-2-2100 and EM 1110-2-6053 regarding progressive analysis methodology, which may increase the complexity of the analysis and FEA model. Also of importance is that the 35% design submittal will not investigate multiple extreme events happening simultaneously (i.e. MDE and Flood) as the chances if this occurring is extremely remote.

1.9 REPAIR / RETROFIT OPTIONS

Several options are available to repair / retrofit multiple arch dams. Historically, many multiple arch dams have been repaired due to seismic governing load cases; however these same methods can also be used to upgrade dams for static load cases. Below are examples of similar dams that were repaired using various methods:

- Bear Valley Dam (Multiple Arch): Upgrades included filling the bays between the arches with mass concrete. (Hansen & Nuss 2013)

- Littlerock Dam (Multiple Arch): Designed also by J.S. Eastwood, used roller compacted concrete (RCC) to infill between the arch buttress to form a gravity dam section. (Hansen & Nuss 2013)

- Weber Dam (Multiple Arch): RCC was placed adjacent to the largest buttress walls to increase lateral stiffness. (Hansen & Nuss 2013)

- Stony Gorge Dam (Ambursen Slab and Buttress Dam): Simply supported diaphragm walls were place between the buttresses to support the buttresses in the cross canyon direction.

- Big Tujung Dam (Thin Arch Dam): Used steel dowels grouted into the existing dam to tie to a concrete overlay, which thickened the arch.

The most applicable of these options to the Argonaut dam include mass concrete, roller compacted concrete, and a concrete overlay. The simply supported diaphragm walls would not help the Argonaut dam significantly to resist horizontal shear forces through the existing concrete. It is recommended that the 35% design submittal explore and compare these three repair options for use to resist the same four load cases investigated in this report.

1.10 REFERENCES

1. Concrete Reinforcing Steel Institute (CRSI), (2001) "Evaluation of Reinforcing Bars in Old Reinforced Concrete Structures". Engineering data report number 48.

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

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

4. ER 1110-2-1156, (2014). Engineering Regulation "*Engineering and Design -Safety of Dams - Policy and Procedures,* United States Army Corps of Engineers, Washington, DC.

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.

Appendix A