INNOVATIVE REMEDIAL DESIGN OF BARREL ARCH DAM

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ABSTRACT

The California Department of Toxic Substances Control (DTSC) has addressed the stability of the 100 year-old, 45 foot-high Eastman Multiple Barrel Arch Dam. Argonaut Dam near Jackson, California, was built to contain tailing waste from nearby gold mining and processing. By the early 1920s, tailing had completely filled the dam's storage capacity with very soft tailings with high concentrations of arsenic, lead, and mercury. During DTSC environmental review of the now abandoned mining operation, the United States Army Corps of Engineers (USACE) performed a failure mode analysis of the dam. Although severely weathered, statically the dam met modern safety stability criteria. However, under seismic loading with the potentially liquefiable tailing loading the dam's arches, stability criteria were not met. A failure of the dam would release the contaminated tailings as a "mud" flow into a downstream urban area.

Consequently, DTSC and AECOM undertook a design to stabilize Argonaut Dam. The Argonaut dam remediation project objectives were to correct structural deficiencies identified by the USACE and address appropriate flood water management for the site. After comprehensive alternatives review, seismic structural deficiencies were addressed by an innovated combined cellular concrete and earthern embankment buttress on the downstream face of the dam. Stormwater deficiency was addressed by constructing a spillway system that has the capacity to handle the design 200-year return frequency storm event in Argonaut basin.

1. INTRODUCTION

The Argonaut dam is a Concrete Multi-Arch (CMA) dam, designed and constructed around 1916 by John S Eastwood for the purpose of storing mine tailings. Eastwood designed and constructed over ten similar CMA dams in California during this period. The CMA dam is located in Amador County, approximately one mile north of Jackson, CA, at the corner of the Argonaut Drive and Sutter Streets. The dam is about 420 to 450 feet long and 46 to 50 feet tall at its highest point. The dam consists of 13 contiguous arches and includes tie beams for bracing between buttress walls for six of the tallest arches.

Historical documents indicate that a spillway along the left abutment of the dam might have been a part of original construction. However, it has since been removed and demolished (cause unknown). It is understood that the CMA dam plinth was keyed into bedrock having a base thickness of 1.65 ft. In addition, the drawings indicate that "in order to get a firm foundation, the excavation was carried to an average depth of 6 ft. into this bedrock requiring the removal of 1167 cu. yd. of earth and rock."

The Argonaut dam vicinity map is shown in Figure 1. Plan, elevation and a site photo of the dam are provided in Figures 2, 3, and 4.

The CMA dam retains tailing waste from the historic gold mining/processing, to within 12 to 18 inches of the dam crest. The tailings contain toxic substances as noted above including arsenic and other heavy metals from gold ore proceedings.

Figure 1 : Site Vicinity

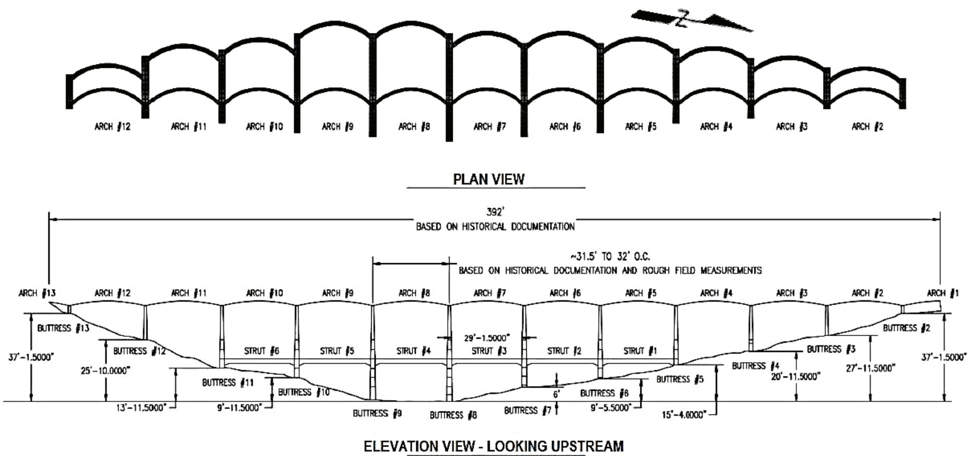

Figure 2 : Plan and Elevation View of Existing Dam

Figure 3 : Site Photo

Figure 4 : Site Photo (Pre Construction)

Following two inspections by the USACE and Environmental Protection Agency (EPA) in 2013 concerns of the dams overall stability were raised. Given the age of the dam (100 years), condition of the tie beams and relatively close proximity of the dam to downtown Jackson both a preliminary static and dynamic assessment were recommended.

The USACE performed geotechnical and structural evaluations of the Argonaut Mining tailing dams and found that the CMA dam to be structurally deficient (USACE, 2015). Based on this finding, the California Department of Toxic Substances Control (DTSC) in conjunction with AECOM has undertaken a stability and retrofit design project for the Argonaut CMA Dam.

With no longer any spillway structure, site stormwater management has been a problem as runoff from the site would pool behind the CMA dam and flow through a small hole which has been broken through in the concrete shell on the right side of the dam (date and responsible party unknown). In heavy runoff events flow would overtop the dam. In 2016, in order to re- route runoff around the dam away from the CMA structure, DTSC had a temporary berm installed behind the dam. The runoff is re-routed using pipe and temporary pump station.

The Argonaut CMA dam remediation project objectives are to correct the deficiencies identified by the USACE and address appropriate flood water management for the site.

2. SCOPE OF WORK

As part of addressing the stability and hydrologic deficiencies, AECOM performed two alternative studies, the first addressing stability deficiencies of the existing CMA dam and the second addressing the site floodwater management. Based on the analysis and review of alternatives including cost, dam safety, and constructability considerations, DTSC and AECOM selected an alternative for final design and construction. Information on the alternatives and the final design is provided in the subsequent sections.

2.1 Site Geotechnical Assessment

Initial geotechnical investigation of the tailings behind the CMA dam was conducted in 2014, under the direction of USACE. The tailings were classified as CL and were found to be of low shear strength, and liquefiable. Standard penetration blow counts were typically in the range of one to five blows per feet. The triaxial testing tailings indicate average effective friction angle of the tailings was 3 degrees with 20.6 kPa cohesion.

A second geotechnical investigation of the site was conducted by Weston in 2016, under the direction of USEPA. The objective of the geotechnical assessment was to obtain information on the geotechnical properties of soil overburden and rock at the downstream toe of the CMA dam. In addition, chemical analyses were conducted to determine whether arsenic, lead, and mercury in soils near the downstream of the dam pose a significant risk to human health or the environment.

A total of 10 borings were advanced at the downstream base of the dam. Six borings were installed within approximately 5 to 15 feet from the base of the CMA buttresses and four borings were installed between 40 and 135 feet downstream from the base of the dam. In general, hard rock was encountered at significantly shallower depths, with soil overburden measuring from 1 foot in thickness to 5 feet maximum depth.

Three test pits were excavated during the 2016 investigation at the base of the arches to expose the foundation of the concrete footings and examine the integrity of the rock underlying the foundation. Hard rock was encountered at 2 feet to 5.5 feet bgs. Two test pits, were located approximately 25 and 40 feet downstream of the dam. These test pits were excavated to a depth of 5 feet bgs.

In general, the thin overburden soils were classified as silt or clay with fine to medium grained sand. The bedrock was generally strong, although varying from intensely weathered to fresh, and very intensely fractured to slightly fractured, with a greater degree of fracturing and weathering near the surface. The beds were steeply dipping with two to three planes visible at the foundation of the dam buttresses. The uniaxial compressive strength of the bedrock varies between 11.8 ksf to 45.4 ksf.

Soil samples were collected from boring locations and test pit excavations at surface and subsurface depths. The following is a summary of the samples exceeding the Site Specific Screening Levels (SSSLs):

- Arsenic – 17 soil samples exceeded the SSSL of 100 milligrams per kilogram (mg/kg).
- Lead - Two soil samples exceeded the SSSL of 400 mg/kg.
- Mercury - Two soil samples exceeded the SSSL of 10 mg/kg.

In general, surface concentrations were higher than those detected in subsurface samples. The area around the CMA dam is fenced and therefore, unless soils are disturbed (e.g. during a retrofit of the dam), direct contact with human receptors is not a significant threat. However, surface contaminants may be able to migrate to groundwater or off-site to surface water via sediment transport mechanisms.

2.2 Stability Alternatives Evaluation

AECOM evaluated potential stability retrofit alternatives for the dam. The purpose of this evaluation was to develop and select a preferred alternative for design and construction. The following four alternatives have been considered, with input from USACE.

- Alternative 1: Reinforced Concrete Buttress wall with gravity arches

  Alternative 1 was developed by USACE and consists 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. The gravity arch would then tie into new buttress walls, which along with the gravity arch would be keyed into the rock foundation. Preliminary construction cost estimate for this alternative is $10.2M

- Alternative 2: Mass Concrete Gravity Dam

  Alternative 2 was developed by USACE and would completely fill in all of the arches with mass concrete and key into the rock foundation. Like the first option, the mass concrete approach would also use the existing arches as formwork. However, the mass concrete gravity dam would not need to be structurally tied into the existing dam structure. Preliminary construction cost estimate for this alternative is $11.8M

- Alternative 3: Roller Compacted Concrete Gravity Dam

  Alternative 3 is a roller compacted concrete gravity dam. This alternative is similar to Alternative 2, except that the gravity dam would be constructed using roller compacted concrete instead of mass concrete. Using roller compacted concrete would require a larger volume of concrete and dam footprint than the mass concrete gravity dam alternative. Preliminary construction cost estimate for this alternative is $9.4M

- Alternative 4: Earthen Embankment

  Alternative 4 consists of an earthen embankment buttress on the downstream face of the dam. The embankment would be constructed by excavating the overburden from the area downstream of the dam and placing a downstream earth fill buttress. The earth buttress would incorporate an inclined chimney and blanket drainage system to control seepage through the existing dam as well as underseepage. Seepage in the drainage layer would be directed to a collector drain and discharged downstream of the earthen embankment buttress. Preliminary construction cost estimate for this alternative is $4M

2.3 Recommended Remediation Alternative

Based on the analysis and review of the alternatives including cost, dam safety, and constructability considerations, DTSC and AECOM selected a modified Alternative 4 for final design and construction.

It became clear that the geometry of the existing concrete arches and buttress would cause significant practical earth embankment construction issues due to headroom restrictions. The overhanging downstream sloping concrete arches would impede proper placement and compaction of soil fill in between the existing buttress. The restricted headroom would limit the construction process to small equipment and potentially hand labor. Consequently the innovative option of using cementitious fill (light weight cellular concrete) for backfilling the buttress arch area that would not require compaction and could be pump in place was evaluated.

Based upon this evaluation, it was concluded that a cellular concrete fill in between the arches would meet both geometric constraints of construction as well as structural requirements for the remediation. Additional information on the cellular concrete fill is provided in the Dam Remediation Design section of this paper.

2.4 Recommended Composite Embankment Design

The composite embankment is composed of a cellular concrete fill in the area of the existing arches, a vertical drain/filter material, and a compacted engineered earth fill. The downstream face of the embankment has a slope of 2H:1V, and the upstream face will be near vertical and built directly against the vertical drain (see Figure 5).

Cellular concrete will be placed in 2 to 3 feet lifts to limit heat of hydration and cracking. Consequently the contractor will be required to coordinate placement of cellular concrete, vertical drain/filter, and earth embankment fills accordingly.

No onsite source of soil borrow was identified however multiple local potential sources were identified and samples taken. Testing results indicate borrow materials meeting the geotechnical criteria (see Table 1) can be obtained within a 15 mile radius of the site. Contractor also imported drain/filter materials from offsite sources. The compatibility between the drain/filter materials and the fill material was checked and approved by AECOM Engineer.

Table 1 : Material Properties for Geotechnical Design

Criteria	Vertical and Horizontal Drain/Filter	Soil Embankment	Cellular Concrete	Existing Mine Tailings	Rock Foundation
Plasticity Index	Non Plastic	Between 5 and 15	-N/A	N/A	N/A
Percent Fines (percent passing #200)	Less than 3%	Less than 25%	N/A-	N/A	N/A
Maximum Particle Size	3/8 inches	2.5 inches	N/A-	N/A	N/A
Density	120 pcf	120 pcf	50 pcf	70 pcf	130
Strength	Φ = 28 degrees, Cohesion = 0 psf-	Φ = 30 degrees, Cohesion = 50 psf	100 psi	Φ = 0 degrees	Φ = 35 degrees, Cohesion = 0 psf
Soil Resistivity Testing	Non Corrosive	Non Corrosive	N/A	N/A	N/A
Minimum Resistivity	>1000 ohm-cm	>1000 ohm-cm	N/A	N/A	N/A
Chloride	<500 ppm	<500 ppm	N/A	N/A	N/A
Sulfate	<2000 ppm	<2000 ppm	N/A	N/A	N/A
pH	5.5-10	5.5-10	N/A	N/A	N/A

2.5 Geotechnical Analyses

The slope stability of the proposed soil embankment was assessed using SLOPE/W software package by Geo-Slope International Ltd. A typical cross-section along the centerline of the soil embankment was analyzed using geotechnical parameters and material properties listed in Table 1. The results of the slope stability analysis is listed in Table 2. The results meets the factor of safety criteria of 1.5 for static, 1.3 for OBE, and 1.2 for MDE.

Factor of safety against slope failure were estimated under two different scenarios (designated as Case A and Case B) as follows.

- Case A: Static
- Case B: Psuedo-Static

Factors of safety for both circular and infinite slope stability failures were analyzed. For seismic loads, Peak Ground Acceleration (PGA) of 0.1175g with a maximum design earthquake return period of 949 years (MDE Condition) and PGA of 0.0685g with operational basis earthquake return period of 144 years (OBE condition) were utilized. These values are obtained from USACE study performed on site in January 2015 (USACE, 2015).

Table 2 : Slope Stability Results

Condition	Static	Pseudo-Static	
		OBE	MDE
Circular Failure	1.5	1.3	1.3
Infinite Slope Failure	1.5	1.3	1.2

Following the completion of structural and infinite slope stability analysis, a sliding analysis was undertaken for a condition where upstream tailings have liquefied and the effect of CMA dam structure was ignored. Under seismic loading, additional driving forces (active lateral earth pressures) will be imparted on the soil embankment from liquefied upstream tailings. The material properties used for sliding analysis are listed in Table 1 and the results of sliding analysis are summarized in Table 3.

Table 3 : Sliding Analysis Summary

Item	Value/Description
Analysis Type	Liquefied upstream tailings
Interface friction angle (soil embankment/underlying bedrock)	0.6
Unit weight of liquefied tailings	70 pcf
Factor of safety against sliding (Criteria 1.3)	Minimum Result - 3

2.6 Stormwater Management Alternatives Evaluation

The hydrologic conditions of the Argonaut Dam drainage area and downstream drainage areas were investigated to estimate the peak discharge for a 200-year event (0.5% probability of exceedance storm) with flow of 140 cfs. In addition, hydraulic analysis of the downstream stormwater system was performed to evaluate if 140 cfs flow can be conveyed easterly to Jackson Creek. The results of the hydraulic analysis of the existing downstream stormwater system show that the maximum peak flow that could be safely conveyed from the dam into the system was 48 cfs.

As part of the site stormwater management, several alternatives were evaluated consisting of spillway only, spillway with water detention behind the dam, improvements to existing infrastructure to accommodate the 140 cfs flow, and diversion of stormwater away from the site to Jackson Creek.

3. DAM REMEDIATION CONCEPT

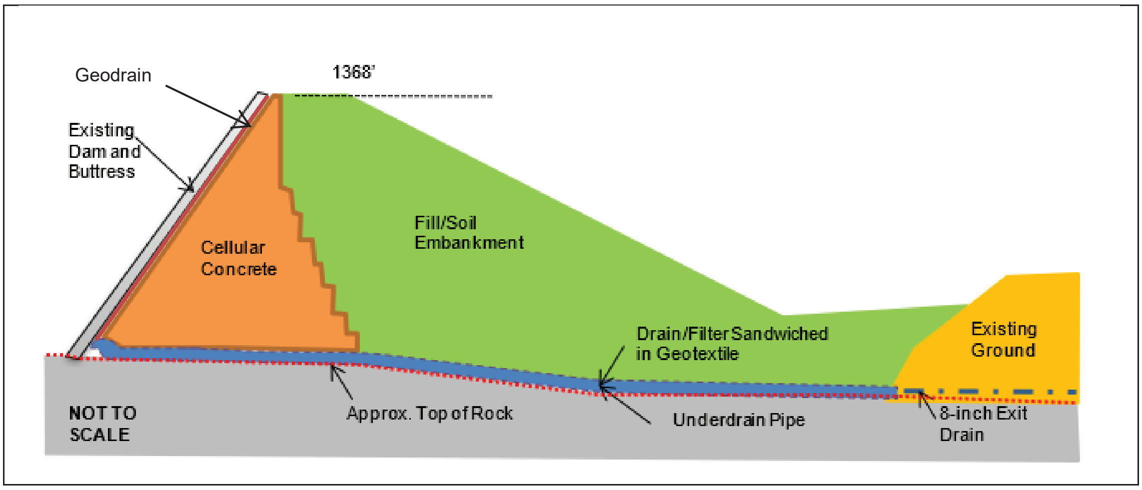

Figure 5 : Typical Cross-Section of Composite Embankment

The design composite downstream embankment buttress is shown in Figure 5. The area within the existing dam arches and buttress will be filled in two to three feet lifts of cellular concrete with a sand drain/filter separating it from the soil buttress (fill). The remaining fill can then be placed with conventional earth moving equipment. The cellular concrete, filter/drain and proposed fill would be staged in lifts by the contractor as the composite embankment is raised.

3.1 Spillway System Design

Due to the geometry and location constrains, the spillway system is designed to be a combination of spillway chute and a spillway pipe. This includes a graded entrance channel upstream of Argonaut Dam, intake and spillway chute, stand pipe, spillway pipe, and an outlet structure tied to the existing 36” culvert below Argonaut drive.

The graded entrance channel collects and conveys the stormwater to the intake and spillway chute. The spillway chute is connected to the standpipe, which will reduce the flow velocity. The stand pipe is connected to the spillway pipe, where the stormwater is conveyed to the outlet structure, which is further connected to the existing 36-inch RCP pipe. Overall, the conveyance feature transfers the runoff to the outlet structure, and the outlet structure provides energy dissipation to the entrance of the existing 36-inch pipe.

4. CONSTRUCTION

Construction period lasted between Spring 2018 and Fall 2018 and constructed of the following phases:

4.1 Phase 1 – Site Preparation

The site preparation consisted of establishing Best Management Practices for stormwater around the site, setting up the perimeter air monitoring equipment, and survey of the existing conditions. In addition, instrumentation consisting of seismographs and tilt meters were installed at strategic locations near the dam. The purpose of the instrumentation was to notify the Contractor in real time should the dam tilt, move, vibrate, or otherwise appear to be adversely impacted by site operations. The instrumentation was installed approximately a week prior to the beginning of work to establish the baseline conditions. Monitoring stopped after the construction was completed.

4.2 Phase 2 – Buttress soil and rock excavation

Excavations of soil and rock were performed for dam retrofit and stormwater system installation. Approximately 6,900 cubic yards of material was excavated. In the dam’s arch area in order to reduce the likelihood of dam failure, removal of the material from within the arches was performed in a sequenced manner so that no two adjacent arches were devoid of material at the same time. The excavated contaminated soils from below the dam were transported to the stockpile area northeast of dam. All equipment leaving both the site and stockpile area was decontaminated in accordance with the Construction Specifications.

4.3 Phase 3 – Embankment Construction

4.3.1 *Horizontal Drain/Filter and Underdrain Pipe Installation*

After excavation of the soils on the downstream side of the dam, the exposed weathered bedrock surface was visually inspected and approved by a licensed AECOM Geotechnical Engineer. Filter fabric was placed over the weathered bedrock surface and a minimum of 2 feet of 7/8-inch crushed drain rock material was placed and subsequently covered with filter fabric to constitute the horizontal drain/filter.

An 8-inch perforated HDPE underdrain pipe was set in the horizontal drain/filter within the low point of the valley created after soil and rock excavation to capture and convey water within the horizontal drain/filter to the existing drainage channel east or Argonaut Drive.

4.3.2 *Geodrain*

A vertical geodrain product lining (JDrain 400) was placed on the downstream face of the arches and embedded into the horizontal drain/filter. The geodrain is designed to capture and collect seepage coming through the concrete arches and convey it to the horizontal drain/filter layer. The seepage will then be conveyed by the drain/filter layer to the 8-inch underdrain pipe.

4.3.3 *Cellular Concrete and Embankment Fill Material*

Cellular concrete was pumped into the arch areas in maximum 3-foot lifts that were left to cure. Prior to pumping cellular concrete, 3-foot-high plywood forms were constructed in front of the arches to hold cellular concrete in place during curing. The first lift of cellular concrete was placed within 3 days of material removal from each arch.

Embankment fill material consisting of sandy gravel was imported from local quarry. The fill material was placed on top of filter fabric using a bulldozer and compacted with a sheep's foot roller in 12-inch lifts. The fill was compacted to minimum of 97 percent of standard Proctor maximum dry density. Nuclear density testing of the fill materials was performed at regular intervals to ensure compaction met the project requirements.

Agricultural fill was spread along the face of the embankment fill in a 12-inch thick lift. Hydroseed was sprayed on top of agricultural fill, followed by fertilizer.

4.4 Phase 4 – Stormwater Conveyance System

4.4.1 *Entrance Channel, Stormwater Drain Structure, and Drop Structure*

The entrance channel that conveys runoff to a concrete stormwater drain structure was graded parallel to the crest of the dam on the upstream side from south near Arch 9 to north near Arch 2. The entrance channel was graded to drain to the grated weir and intake.

Three layers of hydroseed and one spray of straw was spread on the entrance channel. TRM was then placed on the prepared entrance channel to prevent erosion. The TRM extended to anchor trenches outside the entrance channel and was additionally anchored to the prepared subgrade within the channel.

The intake includes a grated weir that is designed to trap and stop trash from entering the stormwater drain structure. The structural concrete work for the stormwater drain structure was undertaken once the prepared subgrade was approved by an AECOM on-site geotechnical engineer. Approximately 16-inches of soil were over-excavated at the drop structure to reach competent, weathered bedrock. Plain concrete backfill was placed to raise the excavated elevation to design subgrade elevation. The stormwater drain structure and the drop structure were connected using rebar that extends at least 30 inches into the drop structure and epoxied in place.

4.4.2 *42-Inch Ductile Iron Pipe and Manhole*

Stormwater will be conveyed from the drop structure to the outlet structure via a 42-inch diameter ductile iron pipe (DIP). Subgrade for the 42-inch DIP included excavating a trench through the embankment fill and placing bedding sand. The trench was then backfilled with compacted embankment fill.

A precast concrete manhole was installed to access the 42-inch DIP approximately mid-way between the drop structure and outlet structure

4.4.3 *Outlet Structure and 36-Inch HDPE Pipe*

The 42-inch DIP will convey runoff to the outlet structure, which connects to the existing 36-inch HDPE pipe running beneath Argonaut Drive. The outlet structure includes a stub out for a second 36-inch diameter pipe to allow for future construction of an additional outlet pipe under Argonaut Drive. On the far (east) side of Argonaut Drive, runoff discharges from the 36 inch HDPE pipe onto a rock outfall within the existing drainage channel.

Figure 6 : Site Photo (Post Construction)

5. CONCLUSION

A comparative evaluation of the four improvement alternatives has been performed, considering advantages, constraints, and conceptual construction costs for each alternative. The preferred alternative of earthen embankment with cellular concrete is a viable and cost effective solution compared to the structural repair alternatives for the CMA.

The proposed composite embankment bears directly on the underlying competent soil or bedrock. Due to contaminated soils, site preparation and excavation works were completed prior to the construction of the remedial works to allow the stabilization work to be completed with minimal to no environmental hazards.

ACKNOWLEDGMENTS

DTSC staff has provided managerial and technical guidance throughout the project. AECOM is responsible for all civil and hydrologic aspects of the project. Geotechnical investigation of the site was conducted by Weston Solutions (Weston) under the direction of US Environmental Protection Agency (USEPA). Topographic survey of the site was performed by KPFF Consultants (KPFF). Construction Contractor for the site was Engineering Remediation Resources Group, Inc. (ERRG).

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