STABILIZING THE OLD COLUMBIA DAM: 
MAKING THE MOST OF LIMITED INFORMATION 
COLUMBIA, TENNESSEE

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2H. Grady Adkins, Jr., P.E.  
3James O. Clark

ABSTRACT

The Old Columbia Dam is a low hazard, 572-foot-long concrete gravity dam on the Duck River in Columbia, Tennessee, built during the 1930s and acquired by the City of Columbia in 1963. The chief purpose of the Dam is to provide a water supply to a raw water intake for the city of Columbia that is located directly upstream of the Dam. The run-of-river type dam is approximately 20 feet high and has been regularly overtopped by past flood events, including at least one 100 year flood and routinely by 20 year events. Previous studies commissioned by Columbia Power and Water Systems (CPWS) indicated that factors of safety for stability were less than the minimum required by standard engineering guidelines for dam safety.

CPWS contracted Paul C. Rizzo Associates, Inc. (RIZZO) to design a stabilization system for the Dam. This paper will provide a case study of the successful stabilization of the Dam, including the means and methods employed to supplement available information and reduce risk. During the design, RIZZO utilized all of the available data from detailed site surveys, field tests and inspections, and previous studies to accurately model the structure and rock foundation for use in the stability analyses and remedial design. The acquisition and use of the data was critical since as-built construction records and original design documents for the structure were unavailable. During the construction the contractor was required to work with the lake at a near full pool level to maintain the sole water supply for the city of Columbia. Due to the Dam’s susceptibility to overtopping, the contractor was required to be prepared for quick demobilization in response to changing river conditions. Only one emergency demobilization was required during the performance of the work. The project was completed in August 2009 with the installation of 22 permanent restressable anchors, including one instrumented anchor, installed on the crest of the Dam.

INTRODUCTION

The Old Columbia Dam generally consists of 20 to 24 foot-tall sections with ogee crests and 0.65H:1V downstream slopes, and it has been assumed that the dam foundation was not grouted or keyed into the underlying bedrock at the time of construction. The dam structure is characterized by three distinct sections: the Sluice Gates and Bridge Section, the Non-Overflow
Section, and the Overflow Section. The Sluice Gates and Bridge Section, closest to the right abutment, is 115.38 feet long and includes two stoplog sluice bays and a bridge walkway to the powerhouse, with the powerhouse located near the center of the structure. The stoplog sluice bays were intended to provide the capability to flush silt from the Dam and control pool levels, but are currently inoperable. As a result, considerable sediment and debris buildup has occurred on the upstream face of the Dam. The Non-Overflow Section of the spillway to the left of the powerhouse is 104.21 feet long. An island composed of sediment lies directly upstream of this Section. The Overflow Section of the spillway is 152.25 feet long and is adjacent to the left abutment. Figure 1 shows the Dam under normal headwater conditions.

Figure 1: Old Columbia Dam: Headwater at Normal Operating Level

Old Columbia Dam was acquired by the City of Columbia in 1963 from the Tennessee Valley Authority (TVA). Prior to this acquisition, the Dam and Hydroelectric Plant were run under TVA procedures and requirements. According to available records, FERC licenses have been issued to three entities since the City of Columbia obtained the Dam in 1963. Available records indicate the most recent operation of the Hydroelectric Plant was from January 19 through January 22, 2002. Power generation during this period was stopped when the powerhouse was damaged by flooding. Currently, the chief purpose of the Dam is to provide a water supply to a Columbia Power and Water Systems (CPWS) intake directly upstream. This required the reservoir levels to be within five feet of the established normal pool elevation. The normal pool elevation is established at near full pool level, approximately 1 to 2 feet from the crest of the
Dam; the Non-Overflow Section Dam crest is at elevation 558.7, and the river, under normal pool conditions, is at an approximate elevation 557.5 feet.

The Old Columbia Dam is classified as a low hazard potential dam by the FERC. A Part 12 D Safety Inspection Report for the Dam, although not required by FERC Regulations, was commissioned by CPWS and prepared in January 2005. This report contained the following findings related to the stability of the Dam:

- The Sluice Gates and Bridge Section of the spillway was found not to be stable; and
- Stability factors of safety for other sections of the Dam were less than the minimum required by FERC Engineering Guidelines. It was recommended that the entire Dam be stabilized by the use of post tensioned anchors.

RIZZO was contracted to design a stabilization system for the Dam. The Project Team responsible for the installation of the anchoring system included Brayman Construction Corporation (Brayman) and S&ME Incorporated (S&ME), the core drilling sub-contractor.

GEOLGY AND SITE CHARACTERISTICS

The Old Columbia Dam is constructed on the Duck River in Maury County, Tennessee, on the east side of the city of Columbia. In this area, the Duck River lies within the Outer Nashville Basin Physiographic Section of the Interior Low Plateaus Physiographic Region. The Nashville Basin Physiographic Section lies within the Nashville Dome, the dominant geologic structure of the region. The Nashville Dome is a structural uplift in the continental crust that has experienced initial uplift in Late Ordovician to Silurian time (Bradley and Leach, 2003) followed by tectonic uplift in the Pennsylvanian Period related to the Appalachian Orogen, and Mesozoic and Cenozoic uplift due to erosion and subsequent isostatic uplift sequences (Frazier and Schwimmer, 1987).

The Duck River Channel at the site location is incised into the Ordovician Carters Limestone of the Stones River Group. The Old Columbia Dam is founded on the Carters Limestone which is a thinly to moderately bedded, slightly cherty, fine grained limestone. The primary fracture set in the dam foundation rock mass of the Carters Limestone is sub-horizontal fractures along bedding planes. These fractures are noted to be closed to slightly open, with slightly weathered to fresh fracture faces. Regional fracture sets associated with the uplift of the Nashville Dome (Brahana and Bradley, 1986) and noted in RIZZO’s field reconnaissance include subvertical parallel sets and are significant in karst feature development. These subvertical sets were not significant within the dam foundation.

The elevations above the river channel are comprised of the Ordovician Bigby Cannon Limestone and Hermitage Formation of the Nashville Group. The Bigby Cannon Limestone is primarily a calcarenite while the Hermitage Formation is an argillaceous to sandy, thinly bedded limestone with shale zones. The Duck River channel exhibits relatively steep banks, often with rock outcrops of the Carters Limestone as part of the river banks.
The site lies within a seismically stable region of central Tennessee, east of the New Madrid Seismic Zone and west of the East Tennessee Seismic Zone, with peak acceleration with 2 percent probability of exceedence within 50 years of 0.14g (USGS, 2008).

On the abutments of the Dam, vegetation consists of trees and shrubs. There were no wetlands in the construction area. The Duck River downstream of the Dam is approximately 400 feet wide and varies in depth from 1 to 4 feet at normal flow conditions. The alluvial substrate consists of silt, sand, and gravel. Upstream of the Dam, there is an island made of sediment carried by the river and deposited behind the Dam.

DESIGN

STABILITY ANALYSES

Stability analyses were performed using the computer program known as Computer Analysis of concrete gravity DAMs (CADAM) for all three sections of the Dam that previously were determined to be of concern: the Sluice Gates and Bridge Section on the right abutment, the Non-Overflow Spillway Section in the center of the structure, and the Overflow Spillway Section on the left abutment. Construction records were unavailable, and section geometry including dimensions, elevations, foundations, water levels upstream and sediment levels, slopes, and curves were taken from the previous Part 12 D safety inspection report commissioned by CPWS in 2005 (R.W. Beck, January 2005). Figure 2 illustrates a typical Dam cross section. For stability analyses, the critical load case was taken as the “usual” load case; at flood conditions, tailwater elevations rapidly rise and serve to significantly raise safety factors. Uplift pressure was assumed to be linear between headwater and tailwater, with no reduction due to drains (no drainage in existing structure). Concrete compressive strength was assumed to equal 2,500 psi (360 ksf); the compressive strength of the concrete was chosen to be a conservative value based on the results of a limited field testing program performed with a Schmidt hammer. No cohesion and a peak friction angle of 55 degrees were assumed between the concrete and bedrock at the base. A residual friction angle of 37 degrees was used to represent post peak stress conditions. Minimum factors of safety and analysis methodology were adopted from the Federal Energy Regulatory Commission (FERC) Document “Engineering Guidelines for the Evaluation of Hydropower Projects.” Table 1 shows the stability and stress criteria used in the analyses. The stability analyses showed that for un-anchored conditions of the Dam, resulting factors of safety were lower than the acceptable factors of safety required by FERC guidelines. Based on the results of the stability analyses, a series of post tensioned anchors was designed to bring the residual sliding factor of safety of each of the dam sections to the minimum of 1.2. Table 2 shows the results from the stability analyses of un-anchored and anchored conditions.

<table>
<thead>
<tr>
<th>TABLE 1: OLD COLUMBIA DAM STABILITY CRITERIA</th>
</tr>
</thead>
<tbody>
<tr>
<td>LOAD CONDITION</td>
</tr>
<tr>
<td>----------------</td>
</tr>
<tr>
<td>USUAL</td>
</tr>
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</table>

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TABLE 2: STABILITY ANALYSES RESULTS

<table>
<thead>
<tr>
<th>SECTION</th>
<th>UN-ANCHORED CONDITIONS</th>
<th>ANCHORED CONDITIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>PEAK STRENGTH SLIDING</td>
<td>RESIDUAL STRENGTH SLIDING</td>
</tr>
<tr>
<td>SLUICE GATES AND BRIDGE</td>
<td>0.9</td>
<td>0.5</td>
</tr>
<tr>
<td>NON-OVERFLOW SPILLWAY</td>
<td>0.7</td>
<td>0.4</td>
</tr>
<tr>
<td>OVERFLOW SPILLWAY</td>
<td>1.2</td>
<td>0.6</td>
</tr>
</tbody>
</table>

Figure 2: Typical Dam Section

ANCHOR DESIGN

The anchor design was performed using the same parameters as the initial stability analysis for each dam section and in accordance with the guidelines established in the fourth edition of the
“Recommendations for Prestressed Rock and Soil Anchors” (2004) produced by the Post Tensioning Institute (PTI). An incremental analysis of anchor loads was made to determine the magnitude of the vertical load applied 1.5 feet from the upstream face of the structure needed to achieve the desired sliding FS and to shift the resultant location to the middle third of the sections analyzed. Based on the results of the stability analyses for anchored conditions, minimum required stabilizing loads for each dam section were used to compute anchor spacing, dimensions of the anchor hole diameter, required bond length, and free length. For the purposes of anchor bond length design, the allowable bond strength of the existing limestone bedrock at the site was considered to be 50 psi (0.050 ksi); this value corresponds to the ultimate bond stress of soft limestone in Table 6.1 in the PTI manual. The ultimate bond stress value used is assumed to be conservative based on available data. Table 3 provides the design summary of the anchoring system. A rock mass failure analysis was performed after the minimum anchor spacing and depth were determined. This computation was completed by using Naval Facilities Engineering Command (NAVFAC 7.3, Figure 7, pg 7.3 – 99) guidelines for shallow anchors; an allowable pullout value based on ultimate rock shear strength of 900 psf, with a cone of 60 degrees, and ignoring the weight of the rock in cone. The resulting value was greater than that required in the design. As noted in Table 3, anchors share equal design parameters with the exception of the spacing between anchors in each dam section. This design was finalized after several conferences with the contractor were completed and discussion of available drill tooling and corrugated sheath diameter were incorporated for more efficient constructability procedures. Figure 2 illustrates a typical Dam section with anchor.

### Table 3: Summary of the Anchoring System

<table>
<thead>
<tr>
<th>Section</th>
<th>Vertical Stabilizing Load (KLF)</th>
<th>Design Anchor Load (Kips)</th>
<th>Number of Strands per Anchor</th>
<th>Bond Length (FT)</th>
<th>Free Length (FT)</th>
<th>Diameter of the Drill Hole (IN)</th>
<th>Design Anchor Spacing (FT)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sluice Gates and Bridge Section</td>
<td>11.6</td>
<td>240</td>
<td>7</td>
<td>30</td>
<td>30</td>
<td>6</td>
<td>Center of Each Bay (Critical Spacing 20.6 feet)</td>
</tr>
<tr>
<td>Non-Overflow Spillway Section</td>
<td>16.0</td>
<td>240</td>
<td>7</td>
<td>30</td>
<td>30</td>
<td>6</td>
<td>15.0</td>
</tr>
<tr>
<td>Overflow Spillway Section</td>
<td>9.6</td>
<td>240</td>
<td>7</td>
<td>30</td>
<td>30</td>
<td>6</td>
<td>25.0</td>
</tr>
</tbody>
</table>

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CORROSION PROTECTION

The anchors installed at Old Columbia Dam required Class 1 corrosion protection for permanent anchors, as recommended in PTI guidelines. The elements of a Class 1 corrosion protection system for anchors in the drill hole after a passing water pressure test are as follows: full sheathing of the free length of individual anchor strands with a grease filled polypropylene sheath; full sheathing of the anchor length with a corrugated polyethylene sheath; embedment of the strands and sheath in a suitable Portland cement grout; and a properly controlled and protected anchor head assembly.

The anchor head corrosion protection consists of a trumpet pipe that provides an overlap with the corrugated HDPE sheath, a steel cap filled with corrosion inhibiting grease bolted to the anchor head and painted with anticorrosive mastic, and a final encapsulation with low strength grout. The low strength, non-shrink grout was required to aid removal of the grout in case restressing of the anchors is required in the future. Final cover of the completed anchor recess was provided by a high strength grout. Figure 3 illustrates the details of a typical final anchor head assembly.

![Anchor Head Detail with Recess Dimensions](image)

**Figure 3: Anchor Head Detail with Recess Dimensions**

INSTALLATION

EXPLORATION

Prior to full mobilization to the site by the contractor, a detailed site survey was completed to confirm dam dimensions and to establish control points on site. Upon review of the survey data,
one anchor on the Non-Overflow Section of the spillway was eliminated from the design. This was not surprising since construction details and detailed site surveys were not available during the design phase. Due to a limited construction budget and access issues resulting from steep rock outcrops as part of the river banks, only one exploratory core boring was completed. This was performed to provide information about the condition of the Dam and the underlying limestone at the location of the anchor located closest to the powerhouse on the Sluice Gates and Bridge Section of the Dam. The boring was drilled using HQ size core tooling to a terminal depth of 82.40 feet below the top of the Dam. A total of 22.45 feet of core was recovered from the dam section, with 59.95 feet of rock cored below the dam foundation contact, which was approximately 15 feet past the five foot overdrill of the planned anchoring depth. In general, the concrete was found to be in good condition with no sand pockets or large voids encountered. The foundation rock was generally considered to be composed of moderately hard to soft limestone, which confirmed assumptions used in the preliminary stability analyses.

CONSTRUCTION LIMITATIONS

Constructability issues included space constraints and environmental concerns. The Duck River is protected along much of its length as a scenic river. The immediate vicinity of the Dam does not have this classification; however, a habitat for an endangered species of mussel exists both up and downstream of the structure. Environmental controls regarding water quality were strict during construction, particularly in regards to drill cuttings and grout placement; the contractor installed containment bays on the crest of the Dam to comply with this requirement. The water intake upstream of the Dam required the contractor to work with the Dam at normal operating levels, established at a near full pool level. The contractor had approximately 1 foot of working freeboard on the Dam, which required platform construction and the use of a barge for safe installation procedures. The continuous flow of water on the Overflow Section impeded platform construction on the crest of the Dam, and the contractor installed temporary plywood flashboards on the specified anchor locations to divert water flow and used a platform constructed on the front of the barge unit to drill, insert, and test anchors. Unable to control the water level and with the susceptibility to overtopping of the Dam, the contractor was also required to work with a quick demobilization readiness during the entire construction phase. Precautions taken during work included: daily weather monitoring, relocating the barge unit to a safe location at the end of every shift, relocating the equipment to high ground if a flood was suspected, removing expensive installation equipment from platforms at the end of every shift, and completing construction of anchors in strategic groups to ensure that exposed strands and anchor heads were not left vulnerable to damage in case overtopping of the Dam occurred. For example, the contractor worked on one Section of the Dam until anchors were inserted, before proceeding to the next section Dam Section.

SLUICE GATES AND BRIDGE SECTION

The contractor initiated anchor installation on the Sluice Gates and Bridge Section of the Dam. Installation on this dam section was performed from the bridge of the Dam. The bridge was used as a platform for drilling, inserting, and testing of the anchors; the contractor cored through the bridge to make the proposed locations of the anchors on the Dam accessible to the driller. During the work period for the Sluice Gates and Bridge Section, a high headwater event stopped
construction for approximately 2 weeks. The contractor was installing sediment containment bays when a warning call by CPWS informed of possible overtopping of the Dam. The contractor immediately stopped production work and moved the equipment to higher ground. The contractor returned to the site when the pool elevation returned to normal levels. There was no damage caused by this flood event and construction continued as scheduled when the staff returned. Figure 4 shows the Dam during the flood event.

![Figure 4: The Old Columbia Dam during the Flood Event](image)

**NON-OVERFLOW SECTION**

On the Non-Overflow Section of the spillway, the anchors were installed by using a barge to construct platforms on the crest of the Dam. The installation included an instrumented anchor. The instrument seemed to be working properly at first, but placement of the low strength and high strength non-shrink grout in the recess apparently caused the instrument to fail. As a result, a replacement instrumented anchor was installed on the Sluice Gates and Bridge Section of the Dam; the contractor installed an additional anchor with a Load Cell next to and at a distance of 6 feet from the center of the anchor located on the first bay of the Sluice Gates and Bridge Section of the Dam. On the Non-Overflow Section of the spillway, at the left abutment, the contractor also used platforms to allow for the installation. This section originally called for a total of three anchors; however, an anchor was eliminated due to the relatively short section of the Dam at that location. To confirm the depth of the dam section, a pilot hole was drilled at the planned location of the eliminated anchor, and the concrete rock interface was confirmed by camera. The hole was then backfilled with the anchor grout mix and capped with high-strength non-shrink grout. Figure 5 shows the platform construction on the Non-Overflow Section next
to the powerhouse, and Figure 6 shows the Non-Overflow Section on the left abutment after anchor installation was completed.

**OVERFLOW SECTION**

On the Overflow Section of the spillway, platforms were constructed on the front of the barge and drilling was performed from those platforms. The continuous flow of water over this section had, to this point, concealed a small sluice gate until the time of construction. When the flow of water over this section was reduced, this sluice gate, not identified on any records, was revealed. The contractor moved the barges directly upstream of the Dam to reduce the flow over the crest and exposed this section of the Dam at the specified anchor location. RIZZO analyzed the location of the closest anchor with respect to the location of the gate and concluded that a 3.3 foot distance was adequate cover for the anchor to be installed in its original proposed location. During the installation of another anchor in the Overflow Section of the spillway, a construction joint required the respective anchor to be moved from its original proposed location. The anchor was moved 2 feet west along the centerline of the crest to avoid the construction joint. As a result, the remaining anchors were shifted 2 feet west to maintain minimum spacing between anchors. Figures 7 and 8 show the previously unidentified gate in the Overflow Section of the spillway. Figure 9 shows the plywood flashboards on the Overflow Spillway Section, as well as the platforms used on the Non-Overflow Section.
ANCHOR INSTALLATION PROCEDURE

The 22 anchors installed in the Old Columbia Dam were installed according to the following general procedure: preparation of anchor location; pilot hole and recess drilling; anchor hole
drilling; alignment verification and water testing; consolidation grouting (as necessary); anchor installation; load testing; and recess backfilling.

Prior to the drilling of the pilot hole and anchor recess, collection bays for cuttings and other materials were constructed by building plywood platforms supported by jacks with anchored tarp walls. **Figures 10 and 11** show the collection bays built by the contractor. The recess construction was first initiated by drilling the pilot hole. Completion of the Recess allowed for a 1-inch non-shrink grout pad to be placed and the trumpet pipe with the sub-bearing plate installation to be completed. The trumpet pipe length was designed to overlap the corrugated HDPE sheathing by at least 12 inches to ensure a corrosion protection transition between the corrugated HDPE sheath and trumpet pipe. Following the installation of the trumpet pipe and sub-bearing plate, the drilling of the anchor hole was initiated. The free length was completed 10 feet past the concrete-rock interface of the Dam. The anchor’s bond length was completed 40 feet past the concrete-rock interface or 30 feet past the free length. A downhole camera was used to confirm concrete-rock interface in select locations. A 5 foot overdrill depth was also required to provide enough space for debris to settle if cleaning did not remove it entirely. A DK527 Davey percussion hammer drill with a button bit operated with compressed air was used to drill the hole.

![Figure 10: Cutting Containment Bays: Bridge Section](image1)

![Figure 11: Cutting Containment Bay: Inside](image2)

The alignment of the hole was verified by the use of a single shot micro-mechanical borehole surveying instrument known as a Tropari. This survey instrument is operated by a timing device, which measures borehole direction from the earth’s magnetic field. The survey instrument was used at 3 depths in each hole: the first survey was taken at the mid-point of the dam, the second survey was taken at the concrete rock interface elevation, and the final survey was taken at the bottom of the hole. The Project specifications required the anchor hole be drilled to an angle tolerance of +/- 1 degree. Every hole drilled by the contractor was within this tolerance.

A passing water pressure test was required for insertion of the anchor. The Project specifications followed the criterion recommended by the PTI Manual. If the leakage from the hole over a
10 minute period exceeded 2.75 gallons of water, the hole had to be consolidation grouted, redrilled and retested. The water pressure test was conducted at 5 psi in excess of the hydrostatic head. In the case of consolidation grouting, the contractor monitored the flow rate and volume during grouting by the use of a magnetic flow meter. The anchor hole was grouted by tremie and usually a 12 hour window elapsed before redrilling of the anchor hole was initiated. This was done to ensure that the grout did not gain a higher strength than the surrounding rock. According to the water pressure tests results and documented grout takes, the Sluice Gates and Bridge Section of the Dam proved to need the most subsurface treatment in regards to consolidation grouting.

The anchor manufacturer, Williams Form Engineering Corporation (WILLIAMS), assembled the tendons at the factory. The steel strands were inserted into the corrugated HDPE sheath and were held together by the end cap of the anchor. The anchor tendons were inspected for visible damage when delivered to the job site and immediately prior to installation. The addition of spacers and an outside tremie pipe by the contractor on the site completed the anchor assembly for installation. The spacers were placed at a 12 inch distance from each end of the corrugated HDPE sheathing, and then at a 10 foot spacing. Each anchor had a total of 7- PVC spacers. **Figure 12** shows the polypropylene sheath of each anchor strand inside the corrugated HDPE sheath and **Figure 13** shows an anchor being inserted.

The anchor tendon was inserted into the hole by the use of a crane; the crane picked up the anchor and several laborers guided the anchor down into the hole. Once the anchor was inserted into the hole, it was grouted by the tremie method in stages to ensure that the corrugated sheath did not collapse during the grouting process. The anchor assembly was inserted with two tremie pipes, one on the inside of the corrugated HDPE sheathing and one on the outside of the corrugated HDPE sheathing. Grout flow rates and volumes were monitored by the use of a magnetic flow meter to ensure that the flow rate did not exceed 9 gallons per minute. Grout cylinder samples were obtained and cured for confirmation of unconfined compression tests for each anchor.
LOAD TESTING

The testing of an anchor was performed to ensure the tendon had enough tensile capacity to provide the required stabilizing load, and the lock-off of an anchor was performed to transfer the specified stabilizing load to the structure. Two types of tests were performed during the Dam’s anchoring system installation: performance test and proof test. Five anchors were performance tested and 17 anchors were proof tested.

After the unconfined compression test on the anchor grout cylinders confirmed the grout had gained the necessary strength, initiation of the load testing began. PTI Manual guidelines were incorporated in the specifications for the anchor load testing procedures. During the load testing of an anchor, an Alignment Load, a Test Load, and a Lock-Off Load of 10 percent, 133 percent, and 110 percent of the Design Load were applied respectively. An anchor was required to pass a creep test, total elastic movement criteria, and a lift-off test to be approved by the Engineer on site. The creep test required an anchor to be held at a test load of 133 percent of the Design Load and monitored for the first 10 minutes; if an anchor elongated more than 0.04 inches in the first 10 minutes, then the anchor was required to pass a 60 minute test, which required the anchor to creep less than 0.08 inches. The total elastic movement criteria range was calculated using PTI guidelines, and at installation the upper and lower elongation limits were also utilized to analyze performance of an anchor. After the anchor was locked-off, an initial lift-off test was performed on the anchor to verify that the specified load was transferred to the structure; the anchor was required to stay within 5 percent of the anchor lock-off load value. A four day lift off test was performed on one anchor per dam section; a total of four four-day lift off tests were performed, and one 17-day lift off test was performed on one anchor, which was performed in the Non-Overflow Section at the left abutment. The extended lift off tests indicated the anchors had stabilized, and no creeping of the anchor was occurring at the locations tested. Every anchor installed passed the 10 minute creep test, the total elastic movement criteria, and the initial and extended lift-off tests. Figure 14 illustrates a sample of the graphical analyses of the test data.
MONITORING SYSTEM

Two instrumented anchors were installed during the anchoring program. The first load cell was installed on the Non-Overflow Section of the spillway. This load cell functioned correctly until the anchor’s recess was backfilled with low strength non-shrink grout. Once the load cell was determined inoperable, the contractor proposed to install an additional instrumented anchor on the bridge section of the Dam, under the guidance of a representative from the instrument manufacturer. The additional anchor was installed on the Sluice Gates and Bridge Section of the Dam with a second load cell and was the final anchor installed for the Project. The load cell has shown a downward trend since the beginning of the monitoring period. Based on a review of the available data, including drilling records and extended lift-off testing that does not show consequential creep of any of the anchors, a monitoring program has been continued with readings taken one day a month over the course of the year. When a suitable data set has been
collected showing the trend over seasonal changes, the data will be evaluated and the need for any additional measures determined.

**SUMMARY OF GROUT MIXES & TESTING**

Professional Service Industries, Inc. (PSI) performed quality control testing of the grout cubes and cylinders sampled at the time of placement. Five different grout mixes were used for construction of the anchors: high strength non-shrink grout was used to construct the one inch grout pad under the sub-bearing plate, to cap the low strength non-shrink grout used to backfill recesses, and to patch the boreholes through the bridge of the Dam at the right abutment; low strength non-shrink grout was used to backfill the recess around the anchor head; trumpet pipe grout was used to grout in the trumpet pipe at the anchor head, which allowed for later drilling and cuttings control, and provided permanent corrosion protection at the anchor head; consolidation grout was used to grout anchor holes that failed water pressure tests; and anchor grout was used to install the anchor tendons and corrugated sheathing.

**CONCLUSION**

A total of 22 anchors, including one instrumented anchor, were installed using state of the practice means and methods to stabilize the Old Columbia Dam. During the construction of the anchoring system, unexpected conditions were noted and caused the original design to change. The detailed survey confirmed dam dimensions and required an anchor to be eliminated due to distances measured; the construction joints noted near anchor locations required anchors locations to be moved; and a relatively short Non Overflow Section of the Dam at the Left abutment also required an anchor to be eliminated. The unidentified gate on the Overflow Section of the spillway section of the dam did not require a change in planned locations of the nearby anchors, but provides an insight into the potential problems associated with poor construction documents. The testing performance on the installed anchors and the double corrosion protection system employed indicate that the required stabilizing loads have been applied to the Old Columbia Dam to meet the minimum factor of safety criteria for each dam section.

**REFERENCES**


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