

The Stabilization of Gilboa Dam, New York, Using High Capacity Rock Anchors: Addressing Service Performance Issues

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Abstract

Prior to the proposed reconstruction of Gilboa Dam in upstate New York, 79 very high capacity rock anchors were installed from the dam crest and downstream face to improve the interim stability of the dam. Given concerns over potentially compromising corrosion protection at the head, and other logistical reasons, it was elected not to install load cells in the permanent anchors. In order to satisfy potential concerns regarding the long-term performance of the anchors, many of which were installed in argillaceous rocks of variable properties, several “defenses” were put in place. These included a conservative design process; preproduction pull-out tests; the concept of off-site “sentinel” anchors (with load cells); stringent installation and testing procedures; and Performance Testing (i.e., progressive cyclic) on every anchor (not just on a limited number).

The paper describes each step in the assessment, design, construction and testing/evaluation process and, thereby, provides a comprehensive case history of a contemporary large dam stabilization using high capacity rock anchors. Units are provided in the actual, Imperial style used in the project. A conversion table is provided at the end of the paper.

Project Overview

Gilboa Dam is a major component of the New York City water supply system and is located in the Catskill Mountains approximately 120 miles north of New York City. Completed in 1927, the 180-foot-high dam consists of a 700-foot-long earth embankment and a 1,324-foot-long cyclopean concrete spillway. The Schoharie Reservoir, which is impounded by Gilboa Dam, can store up to 17.6 billion gallons and provides the City with a large percentage of its drinking water. The spillway was built on nearly horizontally interbedded layers of sandstone, mudstone, siltstone, and shale, with shale being the pre-dominantly weak rock unit. The spillway is composed of cyclopean concrete and has a stepped downstream face (Figure 1). A cutoff wall of varying depth was constructed near the upstream face.

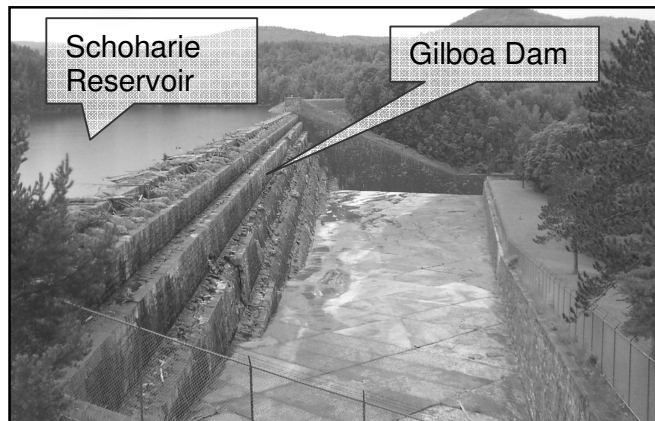


Figure 1. Gilboa Dam in summer 2003.

In early fall 2005, during the preliminary design phase for dam reconstruction, preliminary analyses showed that sliding stability of the spillway structure did not meet current New York State dam safety criteria and was marginal for the 1996 record flood (JV 19Dec 2005). Given the critical nature of the reservoir both in terms of public safety to over 8,000 residents living downstream and dependability for New York City's water supply, an interim stability improvement project was implemented for completion before the end of 2006, years prior to the major reconstruction. The design and construction phases of this interim project were completed in an unprecedented time frame of twelve months.

To help ensure successful completion of the job given the tight time frame, bid packages for rock anchor installation were distributed to three specialty contractors judged to have the appropriate experience in high capacity rock anchors for dams. In addition to conservative design assumptions, several construction measures were implemented to ensure the long-term performance of the anchors including pre-production anchors, "sentinel" anchors, corrosion protection of tendons, water-tightness testing of the anchor hole and sheathing, performance testing of all production anchors, extended lift-off tests of select anchors, and anchor head encapsulation.

Design Approach

The rock anchor system was designed in general accordance with criteria and guidance provided in the Post-Tensioning Institute Recommendations for Prestressed Rock and Soil Anchors (PTI 2004) for Class I encapsulation.

Geotechnical Investigation

The schedule for interim improvements did not permit the completion of a full-scale geotechnical field and laboratory investigation prior to selection of the foundation shear strength parameters for use in the rock anchor design. Investigations by others (Chas T. Main 1977 and GZA 2003) showed that the site is underlain by relatively flat lying sedimentary rocks primarily consisting of sandstone, siltstone, shale, and mudstone. Only four unconfined compressive strength tests had been conducted on rock specimens; two on sandstone samples, one on a shale sample, and one on an interbedded shale and sandstone sample (GZA 2003). Results indicated that the shale and sandstone have unconfined compressive strengths (UCS) ranging from approximately 15,000 to 25,000 psi.

To supplement and confirm the existing data, a subsurface and laboratory investigation program was performed concurrently with anchor design (JV 16May 2006). This investigation consisted of five borings located immediately downstream of the spillway structure at locations that were accessible without regard to, or hindrance of, active flow within the spillway. Figure 2 shows the location of one of these new downstream borings relative to the spillway location at one cross-section. Because the site stratigraphy is relatively flat, samples from these borings were considered to be indicative of the foundation rock conditions beneath the adjacent spillway structure.

UCS testing was performed on rock specimens obtained from the five borings. Representative specimens were selected from the sandstone, siltstone and shale rock units. To supplement the 4 UCS tests by GZA and to verify the ultimate strength used in the bond zone design, 15 UCS tests were conducted on the rock samples. The results of these UCS tests indicated compressive strengths ranging from approximately 6,000 to 13,000 psi for shale/siltstone samples and from 11,000 to 20,000 psi for sandstone samples.

Construction Considerations

Since anchor installation was conducted along the crest and downstream face of the spillway, measures were required to minimize the potential for lost construction time due

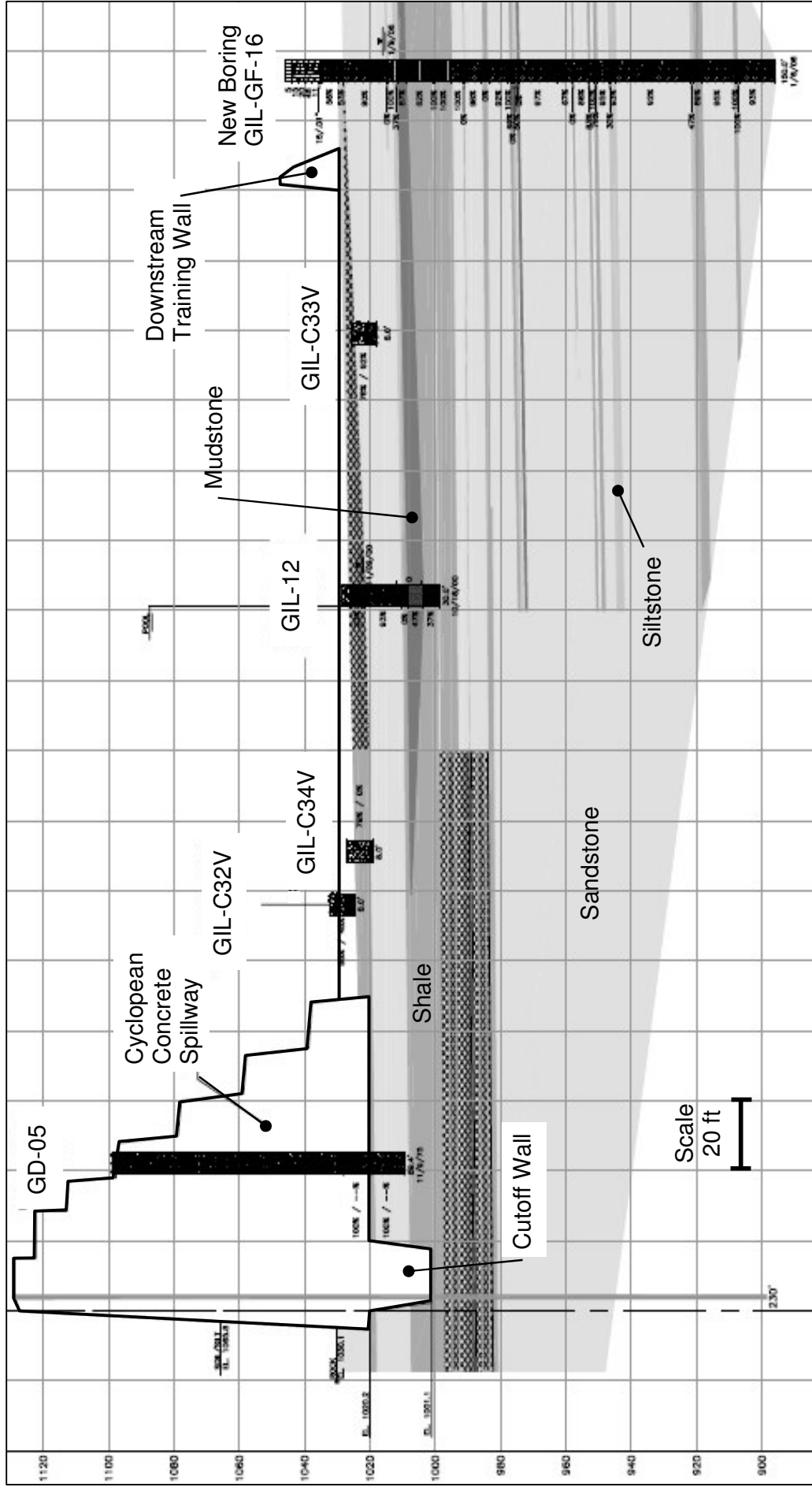


Figure 2. Preliminary site stratigraphy at Gilboa Dam. Gray shading indicates the interbedded sedimentary rock types (after JV 16May 2006).

to active flows from the Schoharie Reservoir. As provided in Kline and Cordell (2007), the following three measures directly impacted the anchor work:

- 1) Installation of a log boom across the reservoir to prevent debris from encroaching on the spillway crest.
- 2) Construction of a 5.5 feet deep by 220 feet long notch in Monoliths M15, M16, and M17 in order to isolate discharges, to the degree practical, from the majority of the spillway and side channel.
- 3) Installation of four siphons over the spillway structure to regulate the reservoir level during construction of the crest notch, and to slow rising reservoir levels prior to flood events that resulted in overtopping of the entire spillway and ultimately emergency demobilization.

The temporary notch and siphon system were considered in the anchor layout and design and also impacted construction efforts (see Figure 3).

Anchor Layout

Spillway stability analyses were conducted at seven cross-sections across the site to identify the required post-tensioned anchor loads (JV 19Dec 2005). Based on these results, vertical anchors were required along the entire length of the spillway crest, from Monolith M1 to Monolith M17. Furthermore, inclined anchors were needed along the downstream face in the central portion of the spillway. These central monoliths are taller than the eastern monoliths, but they have shallower cutoff walls resulting in shorter failure surfaces compared to the higher western monoliths with deeper cutoff walls. For Monoliths M6 through M11, inclined anchors were angled 48° from horizontal and were located at the corner of step numbers 3-4; for Monoliths M12 through M14, the anchors were inclined 45° from horizontal and were located at the corner of step numbers 4-5. A plan view of the anchor layout is shown in Figure 3, and a schematic of the general anchor configuration is depicted in Figure 4. A total of 79 anchors were installed including 47 vertical and 32 inclined anchors.

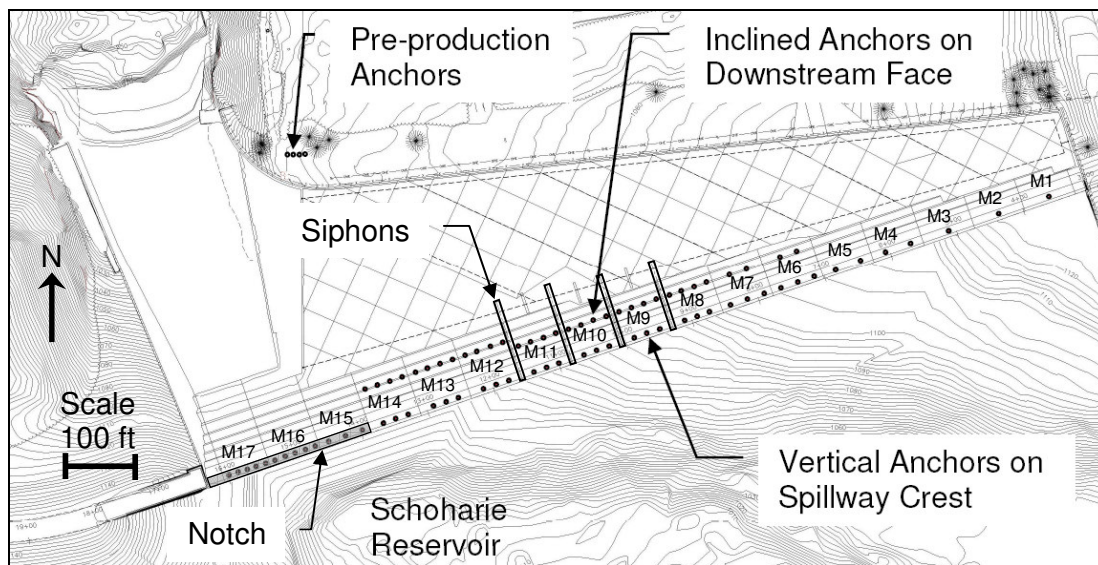


Figure 3. Plan view of Gilboa Dam showing the anchor layout.

The anchor demands were grouped into four ranges of anchor size, Groups A through D, based on their capacities and engineering judgment (Table 1). By grouping the anchors based on maximum required capacities and locking-off all the anchors to the same Design Load (DL) equal to 60% Guaranteed Ultimate Tensile Strength of the steel tendons (GUTS), some additional load was provided greater than that required to meet the minimum stability requirements.

Anchor Design

Computations were performed to determine the minimum required corrugated sheathing diameter, hole diameter, and bond length (JV 30May 2006). Based on the computed minimum diameters, commercially available products were selected to utilize readily available products minimizing lead times for material deliveries. Table 2 provides the design parameters as presented in the contract documents and also shows

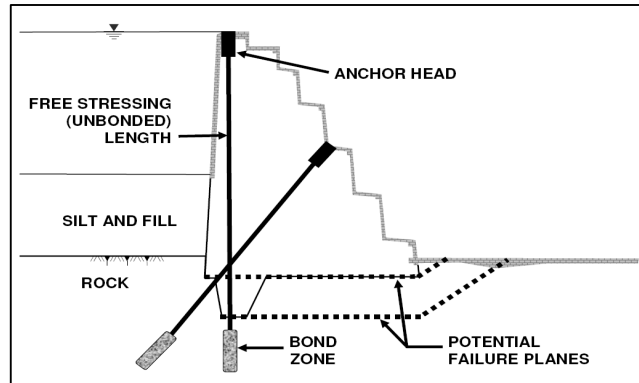


Figure 4. Typical section of the spillway with rock anchors.

changes (in bold) implemented by the contractor. These changes were reportedly made to minimize tool sizes and to ensure satisfactory performance during stressing.

Table 1. Anchor Groups

Group ID	Range of Number of Strands	Design Capacity Range (kips)	Selected Number of Strands	Individual Anchor Design Capacity (kips)
A	33-39	1160-1371	39	1371
B	40-45	1406-1582	45	1582
C	46-52	1617-1828	52	1828
D	53-58	1863-2039	58	2039

Table 2. Summary of Minimum Design Parameters

Number of Strands	Design Load (kips)	Drill Hole Diameter (inch)		Sheathing Diameter (inch)		Bond Length (ft)	
39	1371	12	15	8	10	31	41
45	1582	12	15	8	10	35	45
52	1828	14	15	8	10	35	45
58	2039	14	15	10	10	39	49

The bond length was calculated based on the selected drill hole diameter and the ultimate bond zone strength. To establish the ultimate bond stress, the interbedded site strata were presumed to be governed by sandstone and shale, which were the predominant strata in the bond zone. Typical ultimate bond strength values for preliminary design published in PTI (Table 6.1, 2004) for shale range from 30 to 200 psi and for sandstone range from 100 to 250 psi. These values were significantly less than the ultimate bond stresses of 1,500 to 2,500 psi based on UCS testing by GZA, which exceed the typical allowable bond stress in grout of 600 psi. Therefore, an ultimate bond stress of 200 psi was selected for design of the rock anchors, which is the upper bound for shale given in PTI. A working bond stress of 100 psi was selected providing a factor of safety of 2.0. These bond stresses were confirmed by the previously mentioned UCS testing on rock samples downstream of the spillway, by conducting a site specific pre-production test program early in construction, and by testing each installed anchor to a 33% overload to verify its load carrying capacity.

Free-Stressing Length

The free stressing length was selected to locate the top of the bond zone at a depth at least 10 feet below the base of the existing cutoff wall. The distance of 10 feet was intended to account for uncertainty associated with the location of the actual concrete/rock interface. For design purposes, the location of this interface was based upon the original construction

as-built (record) drawings (NYCDEP 1927). The concrete/rock interface was encountered within 5 feet of its predicted location in preliminary borings drilled through the crest, thus documenting with reasonable accuracy the as-built drawings. PTI (2004) recommends that the free stressing length of anchors should extend a minimum distance of 5 feet beyond potential failure planes (see Figure 4). Therefore, 10 feet was selected to provide additional assurance that the bond zone was at least 5 feet beyond the bottom of the cutoff wall. This additional anchor footage also allowed the contractor to order anchor materials prior to confirming the concrete/rock interface location during drilling.

Anchor Group Effects

Anchor group effects were evaluated to ensure that the interaction between anchors would not decrease the overall capacity of the anchored system. The pull-out resistance of an anchor was equated to the weight of an inverted cone of rock as presented in Littlejohn & Bruce (1977). This method assumes that a vertical plane develops where adjacent cones overlap and decreases the cone volume accordingly, and it also ignores the rock shear strength along the edges of the cone. Although vertical and nearly vertical fractures were encountered in the borings drilled concurrently with the anchor design, these fractures were typically discontinuous and had surface roughness, or undulations, such that shear failure of the rock would be required to form a continuous vertical failure surface. Furthermore, grouting was completed in the dam foundation during the original construction (NYCDEP 1927), which would have further “locked” the rock together. Therefore, despite the presence of vertical fractures, the inverted cone method was considered appropriate since the rock shear strength was ignored.

Corrosion Protection of Tendons

Corrosion protection of the anchor tendons was addressed by specifying permanent PTI Class I encapsulation. This protection included a grout filled sheathing extending the full length of the strand, the trumpet welded to the bearing plate, and an overlap of the trumpet by the sheathing. The anchor holes and sheathing were subjected to extensive testing to ensure water tightness during the successive phases of anchor construction.

After drilling, each anchor hole was required to pass a water test that limited water loss to 5.5 gallons per 10 minutes under a pressure head of 10 psi. If required due to failure of the water test, pregrouting and redrilling were performed until the anchor hole was sufficiently watertight. Furthermore, each anchor hole was videotaped to “see” the sidewalls of the anchor hole. Consequently, the infiltration of water through rock joints into the anchor hole could be observed, whether as a trickle or a jet. Jets of water were more concerning since they would more likely wash anchor grout from the hole reducing the degree of corrosion protection and the area of grout to rock contact within the bond zone.

Water testing was required for all sheathing, which consisted of full length corrugated sheathing for vertical anchors and of smooth sheathing in the unbonded zone and corrugated sheathing in the bond zone for inclined anchors. Regardless of the sheathing configuration, the criterion for water testing was water loss less than 2.75 gallons per 10 minutes under a pressure head of 5 psi. For the vertical anchors, the corrugated sheathing was tested both prior to and after installation into the anchor hole. For the inclined anchors, the smooth and corrugated sheathing were heat welded and then water tested prior to installation into the anchor hole. The sheathing was further tested after its placement into the hole both prior to and after tendon installation. This third additional water test was required because of the increased potential for damaging the sheathing during tendon installation on an angle since the annulus between the sheathing and wall of the anchor hole had not been grouted.

Anchor Heads

Future dam reconstruction at Gilboa Dam will include removal of the deteriorated concrete and overly stone masonry façade of the spillway up to a depth of 6.5 feet. To minimize anchor constructability problems caused by setting the top of the encased anchor at this depth, a steel reinforced concrete column was constructed prior to installation of the anchors. The column was built by removing the existing concrete to a depth of 10 feet and then refilling with steel reinforced, high strength concrete to the proposed bearing plate elevation approximately 2.5 feet below the existing face. This column is intended to protect the anchor during future construction and will transmit the anchor load to the structure below the limit of scaling. To facilitate removal of the deteriorated materials around the columns during reconstruction, a bond breaker was applied between existing and new concretes.

To provide a high level of corrosion protection of the anchor heads, the bearing plate, wedge plate, and tendon tails were coated with bitumastic material and then directly encased in concrete at all locations, except in the notch. Restressable anchors were considered in the design of the anchorage but were not utilized due to the increased potential for corrosion of restressable anchor heads as compared to traditional anchor heads. This is an especially important consideration at Gilboa Dam where the anchor heads are within the active spillway structure.

Although originally designed to be directly encased in concrete, questions regarding the final dam reconstruction instigated revision of the anchor heads in the notch during construction. The possibility of decommissioning anchors was realized, so access to the wedge plate was required in the 13 vertical anchors in the notch. The anchor heads in the 5.5 feet deep notch were, therefore, modified to include a steel cap in-filled with corrosion inhibiting grease immediately around the wedge plate. This steel cap was encased in a larger steel cap bolted to the bearing plate that was filled with expandable closed-cell foam. Finally, the entire anchor head was encased in concrete. This configuration was used for the anchor heads to provide access, if necessary, without damaging the anchors during reconstruction. Lastly, due to construction restraints and to ensure adequate concrete cover, the anchor tails were cut short thereby not allowing for future restressing.

Long-Term Anchor Performance

Although load cells are frequently incorporated into anchor heads to provide long-term monitoring of anchor loads, this was not feasible at Gilboa Dam due to the location of the anchors (i.e. in an active spillway) and future reconstruction activities (i.e. scaling of deteriorated concrete). In addition to the previously discussed design conservatism and anchor installation testing, extensive test methods were put in place to ensure satisfactory anchor performance including pre-production anchor tests, installation of “sentinel” anchors, Performance Testing all production anchors, and extended lift-off testing.

Pre-Production Anchors (“Sentinel” Anchors)

At the beginning of the anchor contract, an extensive pre-production anchor test program was performed to verify the bond stress and factor of safety used in design or establish the actual bond stress of the site strata, evaluate creep susceptibility of the site strata, and provide instrumented “sentinel” anchors at the site. Four pre-productions anchors were installed downstream of the spillway (see Figure 3) utilizing the same construction techniques and materials used in the production anchors but were smaller in size. Table 3 summarizes the test conditions, including anchor sizes and rock types, and the test results. The pre-production anchors were incrementally loaded and unloaded until an ultimate bond stress of 200 psi was exceeded. As shown, the required 200 psi bond stress was achieved in each anchor, so the factors of safety were greater than the value of 2.0 used in design. It is important to note that no anchor was taken to failure.

Table 3. Pre-Production Anchor Test Conditions and Results

Test Anchor Number	Number of Strands	Test Load (kips)	Bond Length (ft)	Bond Zone Lithology	Tested Average Bond Stress (psi)	Tested Factor of Safety
PP1	10	468	4.9	Shale	282	2.8
PP2	15	701	9.9	Shale	209	2.0
PP3	10	468	5.0	Sandstone	276	2.7
PP4	15	701	9.8	Sandstone	211	2.1

To evaluate creep susceptibility of the site strata, a constant load equivalent to 80% GUTS was applied to each pre-production anchor. The anchors were monitored for up to 75 hours (4500 minutes), which is significantly greater than typical creep tests. Elongation of the tendons indicated creep less than 0.04 inches per log cycle of time, which is acceptable.

Two pre-production anchors were equipped with permanent load cells after completion of the performance and creep testing and were locked-off at 70% GUTS. Load cell readings for these “sentinel” anchors were recorded intermittently for one year, or approximately 6 months beyond completion of the anchor contract. The results are shown in Figure 5, a semi-log plot of the anchor loads over time. Assuming a 100-year service life for the dam, it is apparent that 91% to 92% of the anchor lock-off load will be available, which corresponds closely to the design loads of the anchors within the spillway.

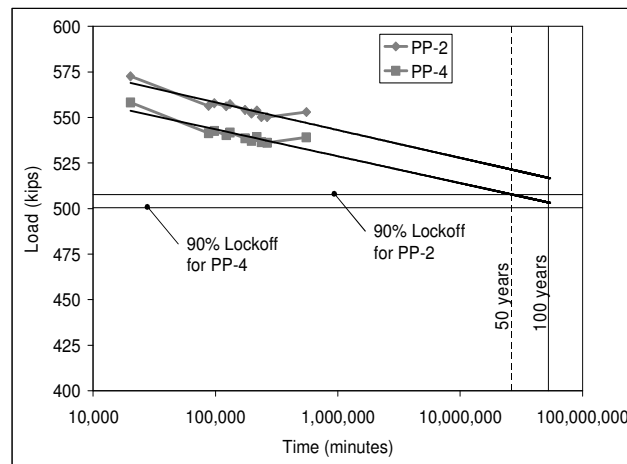


Figure 5. Results of load-cell monitoring of “sentinel” anchors.

Performance Testing

All 79 production anchors, vertical and inclined, installed in the spillway were subjected to Performance Testing. This exceeds standard practice of Proof Testing the majority of the anchors and Performance Testing only the first 2 or 3 anchors and 2% thereafter. Due to the limited amount of subsurface information available during design, it was deemed prudent to apply this additional level of testing to ensure satisfactory anchor performance. The Performance Testing was in accordance with PTI (2004), which included application of an Alignment Load to each strand (10% Design Load per strand), cyclic loading and unloading to a maximum load of 133% Design Load, and then creep testing at the maximum load. During stressing, both dial gauges and load cells were utilized to measure the applied load and tendon elongation.

With the exception of one anchor, which is discussed below, all of the rock anchors were successfully stressed per the contract documents. Of these 78 anchors, 73 anchors performed adequately during Performance Testing and passed the 10 minute creep test, which required the strand elongation not to exceed 0.04 inches. Due to excessive elongation during the 10 minute creep test, the 5 remaining anchors were subjected to the 60 minute creep test, which they subsequently passed with strand elongation not exceeding 0.08 inches. Typical Performance Test results are shown in Figure 6 in terms of total, elastic and residual movement.

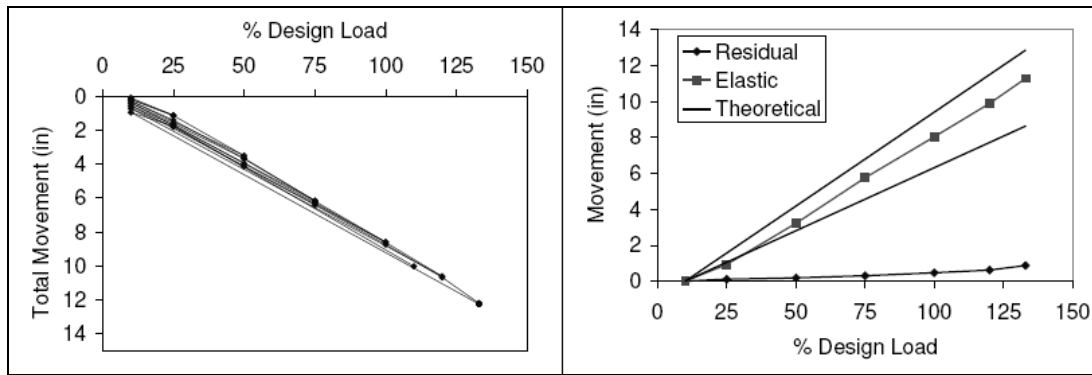


Figure 6. Movements recorded during Performance Testing of Anchor A40, a vertical 58-strand anchor.

One 58-strand inclined anchor had wires on 8 strands break during Performance Testing. The anchor had been successfully loaded to 120% Design Load and experienced strands breaking as loading was cycled to 133% Design Load, at which time stressing was ceased. The cause of failure was most likely either an uneven Alignment Load on individual strands or misalignment of the strands and jack. The remaining undamaged 50 strands were subsequently (and successfully) restressed using a Design Load of 50% GUTS rather than the contract specified 60% GUTS. This lesser Design Load was selected because the total load required for structural stability of the monolith was still satisfied while the maximum load (133% the new Design Load) during restressing was less than the previously achieved load. Prior to restressing, the load cell used during Performance Testing was removed so the strands could be regripped closer to the lock-off plate, thereby eliminating the potential of regripping already pinched strands.

Lift-off Testing

Following locked-off of each anchor, an initial lift-off test was conducted to verify that load was successfully transferred to the anchor bond zone. Each of the anchors was within the contract limits of 5% of the specified lock-off load, which was 110% Design Load. Approximately 30 days after the initial lift-off test, 6 vertical and 4 inclined anchors were subjected to an extended lift-off test. The term “extended” simply refers to the time between the initial and second lift-off tests and does not imply that load was applied and subsequently held for an extended period of time. The lift-off loads were graphed similarly to the “sentinel” anchors (Figure 7). Extrapolation of the data indicates that the anchor load available at the end of the 100-year service life will be between 99% and 112% Design Load.

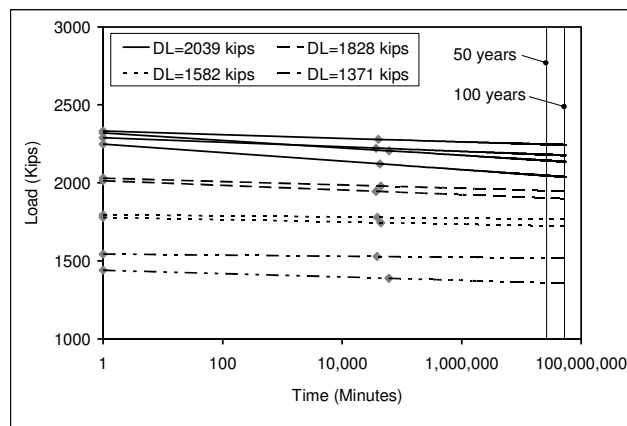


Figure 7. Results of initial and extended lift-off testing of 10 production anchors.

Conclusions

The fast-track interim improvement of Gilboa Dam provided several opportunities to “think outside the box” during rock anchor design and construction. Long-term performance of these anchors was crucial and was verified using design conservatism, extensive testing during construction, and instrumentation and monitoring. The success of the Gilboa Dam anchor project can be attributed to the following factors:

- Design conservatism was beneficial during construction. Since anchors were grouped during design, several strands breaking during stressing allowed an anchor to be restressed at a lesser load while maintaining the minimum factor of safety for stability.
- Pre-production anchors were installed in the same manner as the production anchors to provide insight into the construction procedure, load carrying capacity of the rock, and tendon creep.
- Long-term monitoring of instrumented “sentinel” anchors was utilized to estimate the anchor load for the service life of the dam.
- Corrosion protection of the tendons and anchor heads was assured using thorough testing procedures and inspection at each construction step.
- Performance Testing of all production anchors provided a very high quality of data on anchor behavior. This was especially important since the amount of subsurface information, including rock type at each anchor location and strengths, was limited.
- Extended lift-off tests performed on a small percentage of the production anchors were used to measure the anchor load several weeks after construction and to estimate the available load for the service life of the dam.

Acknowledgements

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Conversion Table

1 foot = 0.3048 meter	1 mile = 1.61 km	1 inch = 2.54 cm
1 gallon = 3.785 liter	1 psi = 6.894 kPa	1 kip = 4.448 kN