THIRD POST-TENSIONED ANCHORS STABILIZATION AT OLMOS DAM

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Abstract

Olmos Dam is a concrete gravity dam founded on the Buda Limestone Formation and the Edwards Aquifer, the primary water supply for the City of San Antonio, Texas. The City of San Antonio constructed the dam in 1929 to reduce flooding. In 1974, an engineering study indicated that the dam did not meet acceptable safety factors for events larger than the 100-year flood. An uncontrolled ogee spillway was constructed on 1,150 feet of the non-overflow section. Post-tensioned bar anchors were installed in 1978 to stabilize the remaining portion of the non-overflow sections of the dam to increase resistance to sliding and overturning.

Starting in 1984, the City of San Antonio discovered several broken bar anchors. Lift-off tests also revealed significant loss of pre-stressing load on several bar anchors. In 1994, post-tensioned strand anchors were added to compensate for the load lost from some of the bar anchors. This work encountered difficulties during drilling, installation, and stressing. Subsequent inspection and testing of the bar and strand anchors at Olmos Dam demonstrated a progressive deterioration of the anchors and their capacity to hold the required load.

In 2007, Bexar County and the City of San Antonio studied alternatives to stabilize Olmos Dam for the third time. Several alternatives were considered for stabilizing the dam in addition to post-tensioned anchors. The study evaluated the likely cause of failure of the previous anchors and included on-site testing to demonstrate satisfactory performance of anchors built in accordance with current standards. The study determined that anchors could be successfully installed in this foundation. In 2010 and 2011, 68 multi-strand anchors were installed at Olmos Dam along with piezometers, extensometers and load cells to monitor the anchor performance. This paper presents the history of past projects and the design challenges and construction of 68 anchors successfully installed on Olmos Dam in 2010 and 2011 as well as the instrumentation data obtained to date.

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Introduction

Olmos Dam, in its original form, was completed in 1928. It was built as a flood retarding structure to alleviate the flooding of downtown San Antonio. Throughout the years, the City of San Antonio, Texas, had suffered from numerous floods on the San Antonio River. The dam was built in response to two major floods in 1913 and a third in 1921 which killed over fifty people. The dam is located on Olmos Creek, about 6 miles upstream from the heart of San Antonio. The dam was a concrete gravity structure with no emergency spillway section. It was 1,941 feet long and had a maximum height of 58 feet. Height to the rock foundation interface was about 85 feet. The crest of the dam was a public roadway. Flood releases were made through six outlet tunnels regulated by slide gates. The dam would store flood waters and release them gradually through the tunnels thereby reducing the peak flows into the city.

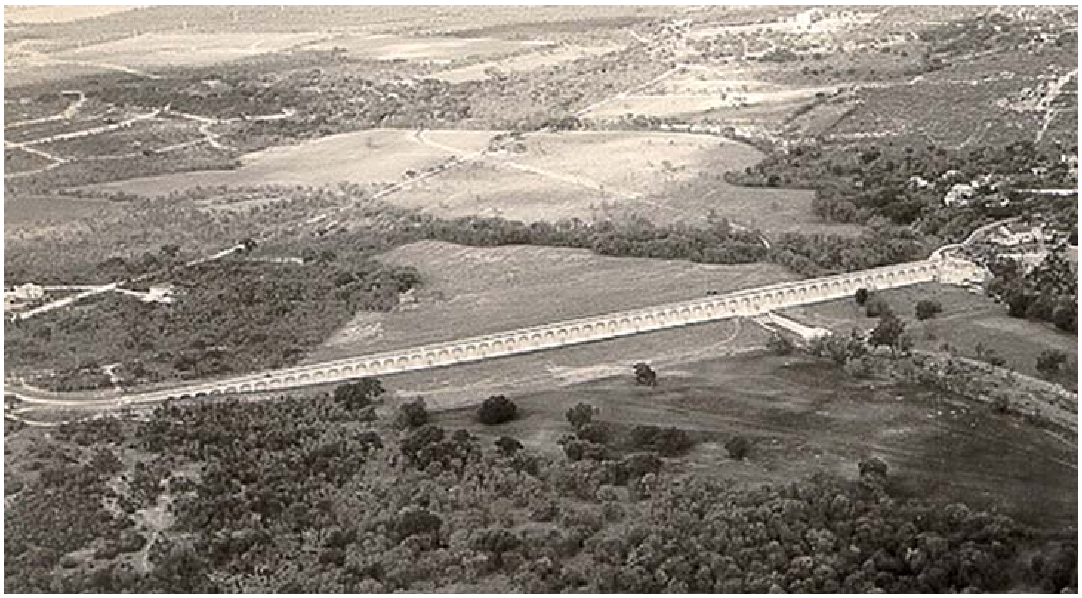

Figure 1. Original Olmos Dam

In 1974, an engineering study evaluated the potential flooding from Olmos' 33 square-mile drainage area. The study indicated that the dam was acceptably stable up to the 100-year flood, but not stable for potentially much larger floods. Not only would the structure not be stable, it would not have sufficient discharge capacity to pass the flood without being overtopped. The ensuing construction modifications to address these issues occurred between 1979 and 1981 and introduced Olmos Dam to its first use of post-tensioned anchors. The roadway was removed and a 1,151-foot long spillway was formed in the dam to increase discharge capacity. The remaining non-overflow section of the dam (790 feet) was slated for installation of 61 post-tensioned anchors.

Figure 2. Current Olmos Dam

Anchor History

The 61 post-tensioned anchors mentioned above, were originally planned to be strand anchors. This first series of post-tensioned anchors was designed between 1974 and 1979. The purpose of the anchors was to provide adequate resistance to overturning and sliding during the Probable Maximum Flood (PMF). Design drawings from this project show the sixty-one multi-strand post-tensioned anchors with 0.50-inch diameter, 270 kips per square inch (ksi), seven wire strand. The number of strands varied from 4 to 17 and the corresponding bond lengths varied from 3 to 10 feet. However, these anchors were never constructed. Instead, prior to construction, 113 bar anchors with 1.375-inch diameter and 150 ksi steel were substituted for the strand tendons. The free length of the bar anchor was covered with a polyethylene (PE) tube with grease for protection against corrosion. The anchor was grouted in one stage from the bottom of the hole to a few feet below the bearing plate. These post-tensioned bar anchors were installed through the non-overflow sections and the outlet works structure. The bar anchors were installed between 1980 and 1981. Problems were encountered with five anchors during installation and they had to be replaced. Three of the five failed anchors slipped at the bond zone and the problem was attributed to poor grouting. Another bar broke due to inadequate installation of couplings, while the fifth anchor failed when honeycomb concrete crushed under the bearing plate. As quality control, the specifications required that certain anchors be subjected to “lift-off” tests six months after they were initially post-tensioned. The lift-off tests were to check that any loss of post-tensioning force did not exceed design assumptions. From available records, it appears that only fifty-eight (58) anchors received lift-off tests. All lift-off tests were reported acceptable and the project was declared complete.

As early as 1984, problems were reported with some of the pre-stressed bar anchors. The anchors were found in poor condition with significant corrosion damage and loss of load.

In 1995, the second round of anchors was installed. Post-tensioned strand anchors were added to a select area of the dam to supplement the failing bar anchors and re-establish Olmos' resistance to sliding and overturning. This work encountered difficulties during drilling, installation, and stressing. Between 2001 and 2006, subsequent inspection and testing of the anchors at Olmos Dam demonstrated a progressive deterioration of the anchors and their capacity to hold the required load.

Stabilization Alternatives

The City of San Antonio partnered with Bexar County to develop measures to again stabilize Olmos Dam. Freese and Nichols (FNI) was selected as the engineer to perform the analysis and final design. FNI recognized the frustration from Bexar County's perspective of anchors as a future stabilization method for Olmos Dam and undertook a serious look at other potential stabilization methods. A stability analysis of Olmos Dam was performed and indicated that the structure would not be considered stable at high flood loading conditions. The PMF was the most extreme, critical load on each portion of the structure. The stability analysis showed that the Non-overflow Sections and the Outlet Structure would all be considered unstable during this event if the existing anchors are not taken into consideration. The overflow section was barely stable at PMF. If the existing anchors were accounted for using the 2006 lift-off tests as their current pre-stressed level, then both the Right Non-overflow and the Outlet Structure would be considered overstressed at the PMF level, but not at the other lower flood levels. The project goal was to find a long-term solution to the stability problems at the dam. Three alternatives were considered for the project.

<u>Added Mass.</u> Adding mass to increase the weight of the structure was considered as an alternative to stabilize the dam. Concrete and RCC were evaluated as potential construction materials. The amount of concrete equivalent to the anchor loads was considerable and extensive amounts of earth excavation would be required to prepare the site. This alternative would require a USACE 404 permit in those areas where work would extend into and through the current creek boundary. Finally, the added mass would dramatically change the aesthetics of the dam on the upstream face, and such change would require coordination with the historic commission. This solution was short lived and also provided similar reasoning for abandoning further evaluation of stabilization methods which would change the dam's footprint or historical appearance.

<u>Partial Anchoring.</u> This alternative consisted of adding only a few anchors at select locations where the load losses from previous anchors were excessive and resulted in unacceptable factors of safety. It was assumed that new anchors would be sized to provide the loading needed assuming that the existing anchors would have an extrapolated additional 20 years of deterioration. This alternative would require testing of all the anchoring every five years and likely the addition of additional anchors at multiple times in the future. This option was ruled out because the existing anchors fail to meet current design standards for permanent installations.

<u>Full Anchoring.</u> This alternative consisted of adding new post-tensioned strand anchors throughout the entire length of the dam. Existing anchors were discounted from providing any future stability forces. Thus, the new anchors would be independently capable of stabilizing Olmos Dam. Despite the short-comings in design and construction, most of the existing anchors have performed satisfactorily for up to 30 years. However, rather than speculate on

future contribution from the bar anchors, a complete independent anchor stabilization system was recommended to Bexar County. This alternative provided a cost effective solution for stabilizing the dam.

FNI recognized that having anchors recommended for a third time would understandably invoke apprehension from Bexar County. Thus, the burden of proof was upon the design team to demonstrate that anchors were now a different breed of quality than they were 30 plus years ago.

Anchor Forensics

Failure Investigations

Developing an understanding of the issues encountered previously with post-tensioned anchors at Olmos Dam was very important in development of installation details for future anchors. Bexar County and the City of San Antonio made numerous documents available for review. The following sections provide a summary of findings related to the first and second installation and performance of the post-tensioned anchors at Olmos Dam.

The first report of an anchor problem was issued in January 1984. The report documented a large spall over one of the post-tensioned anchors on the downstream face of the outlet structure. The spalling was later repaired, yet similar conditions were noted in other anchors by City staff. Significant investigations and actions by the City of San Antonio with respect to the anchors began in 1992. During a visual inspection in the summer of 1992, the concrete cap over the head of one anchor was noted cracked and displaced. Other anchors were soon found to have similar cracking and displacement at the concrete cap. The City explored the problems further by removing cracked/displaced caps and conducting lift-off tests on the exposed anchor bars. Lift-off tests were conducted on all 113 bar anchors, which found the following:

- nine broken bars with two of them failing next to a coupling,
- two deficient bond zones,
- one bar broken during testing,
- one de-stressed anchor with untightened nut, and
- two bars giving erratic elongation results during stressing.

Some of the failed or deficient anchors had originally passed six-month lift-off tests. The recommendations that resulted from the 1992-1993 examination led to the re-stressing of the remaining bar anchors at the outlet works structure and replacement of the long anchors at the non-overflow sections.

The second set of anchors at Olmos were installed from 1994-1996 as a follow up to the 1992-1993 findings. The City proceeded to install three multi-strand post-tensioned anchors at the outlet structure to make up for the load loss caused by the failed bars. No as-built drawings were available for the multi-strand anchors, and some documents made note of 12-strand (0.6-inch diameter) anchors with partially-sheathed tendons (free length) and an uncoated length (bond length) grouted with 3,000 psi grout. The design load was reported at 422 kips per anchor or 60% of guaranteed ultimate tensile strength (GUTS). The anchors were planned to be stressed to 44% of design load so that they could be stressed to a higher load if more existing bar anchors were reported unserviceable.

Construction of these anchors had its share of problems. Drilling 6.5-inch diameter holes was performed using a down hole percussion hammer. The first two anchor holes were drilled and left open for more than 24 hours until the third hole was drilled. These initial two holes experienced sloughing in the limestone or shale which was further complicated with water accumulation from the aquifer. Two days passed while trying to address the water and sloughing. These two anchor holes were ultimately grouted and re-drilled, and it was noted that the cleaning of the holes prior to grouting was questionable. No water takes were measured after re-drilling, and no special treatment was done to the third hole since it did not have contamination problems and remained open. The multi-strand tendons were installed and grouted. The primary grout was to extend 55 feet from the bottom of the hole with the bond zone reportedly 20 feet long. The bond zone was to be set in the shale formation below the limestone. The tendons were encased in greased sheaths along the free-length to allow future re-tensioning and to be protected from corrosion.

During stressing, only the anchor whose hole did not have sloughing problems held the design load. The other two anchors did not perform as designed and only held 40 to 60 percent of design load before failure. The one anchor holding load was stressed to 100% of design load (422 kips) to make up for the loss of the other two anchors.

The anchor installation problems were attributed to deficient bond zones causing de-bonding during stressing. It was concluded that the one working multi-strand anchor provided sufficient capacity to replace 76% of the anchor load lost by failure of the 5 bar anchors installed in 1980. The factor of safety at PMF was considered acceptable when the single multi-strand anchor was successfully installed and the remaining anchors were taken into consideration. However, no additional capacity was left in the anchor system, as originally intended, to allow further stressing in the event of failure of other existing bar anchors. Monitoring of the acceptable strand anchor was recommended every 5 years.

A dam safety inspection performed in 2002 noted that heads for the multi-strand anchors located within the outlet works were found exposed and the cavity was holding water. Several other inspection reports also noted anchor deficiencies [1]. In response, the City conducted additional studies at Olmos Dam in 2006 which included lift-off tests on all the anchors at the dam [2] with the following results:

- three new broken bars (Figure 3),
- two bars still damaged as noted in the past,
- one bar broken during testing, and
- continued loss of load.

Figure 3. Typical Broken Bar

Since the previous lift-off tests and anchor re-stressing, the anchors were reported to have lost from 0.3% to 2.7% of their load every year with an average of 1.5% a year.

Anchor Failure Analysis

The historic information of the anchors at Olmos Dam reveals a troubled and aging system. The anchor problems cannot be attributed to a single issue, but rather to a combination of factors that related to site conditions and level of knowledge for design and construction guidelines at the time. Post-Tensioning Institute (PTI) publishes "Recommendations for Pre-stressed Rock and Soil Anchors" which are best practices for anchor design. The recommendations were most recently updated in 2004. The previous work at the dam has been compared to current PTI standards and current knowledge of post-tensioned anchor technology. The following sections present a summary of findings in an effort to better understand the reasons behind the anchor failures.

<u>Difficult Geology.</u> Significant geotechnical investigations have been performed over the years at the Olmos Dam site. Most of the geotechnical information suggested by the 2004 PTI manual was collected during the 1970's investigation. These early geotechnical studies focused on geologic characterization and rock strength with less emphasis on permeability and groutability of the formation. No records of a pre-production anchor test program or additional geotechnical investigation by the Contractor during construction were found. The design in previous projects developed anchor bond zones in both limestone and shale using published data and engineering judgment based on rock compressive strength. The first anchor project intended to set the bond zones in the shale but later changed course and set them in limestone. The second anchor project set the anchors in the shale. Anchor failures were noted in both limestone and shale. Some of the anchor failures at Olmos Dam can be related to the construction difficulties that arise with a foundation that is highly permeable and holes susceptible to sloughing.

<u>Other Shortcomings of Previous Anchors.</u> The other potential items which could have adversely affected the performance of the Olmos anchors are the following:

- Inadequate assumed bond strengths for the rock
- Inadequate bond lengths for shale type material
- Bond stresses near or exceeding current recommended PTI values for limestone
- Design parameters reflecting ultimate bond stresses close to or exceeding the recommended PTI values
- Bond lengths on bar anchors less than the 10-feet minimum currently in PTI
- Drilling and grouting with open holes too closely spaced
- Drilling and grouting sequence not producing clean and "waterproofed" holes
- Inferior corrosion protection by present day standards for permanent anchors

Design of New Anchor System

The design phase for new anchors at Olmos Dam extended from 2008 to 2009 and included a conceptual design phase and a final design. During conceptual design, geotechnical investigations, hydraulic and hydrologic modeling, stability analysis and evaluation alternatives were performed. Upon selection of the stabilization alternative, the final design phase focused in redefining the anchor concept through implementation of a test anchor program, and concluded with plans and specifications for construction.

The information gleaned from previous geotechnical investigations, new geotechnical investigations by FNI and findings from the failure analyses were utilized to develop a test anchor program. The test program was intended to demonstrate proper drilling, grouting, and installation of multi strand anchors could be achieved at Olmos Dam.

Test Anchor Program

A test anchor program was conducted in 2009, prior to the final development of plans and specifications for anchors at Olmos Dam. The test anchor program was performed on the concrete apron above the outfall channel. Four anchor holes were located directly above the splitter walls of the outlet conduits. Geotechnical borings were performed at each test anchor hole with small diameter core drills. The materials were logged and tested. The design team used this information to formulate the test program for the test anchors.

Bond lengths of 5, 10, 15, and 20 feet were selected for the four test anchors. It was anticipated that the five-foot bond length would fail and thus provide ultimate bond data for future use in final design of the Olmos anchors. The anchors were intended to have their entire bond zone in the Buda Limestone and not in the Del Rio clay, if at all possible. FNI then developed project specifications for the test anchors. The specifications included drilling, grouting, water testing, instrumentation, performance testing, and lift-off requirements. The test program included load cells to monitor the anchor performance for an extended period after lock-off.

All four test anchors were 14 strand, 0.60-inch diameter, 270 ksi, low relaxation uncoated seven wire strand meeting ASTM A-416. The test program revealed that the foundation could be expected to perform well when properly drilled, grouted, and stressed. The ultimate bond strength determined was 218 psi. Thus, the final design proceeded based upon the positive results demonstrated by the test anchor program.

Production Anchors

Anchor Locations. Anchor location selection proved difficult during the design process. The previous anchoring projects had essentially utilized most of the preferred anchor locations. The anchors on the non-overflow sections were selected to not interfere with previous anchors. The anchor locations on the overflow section were easier to pick because there were no previous anchors installed on the crest. However, FNI recognized that they would require extra effort during construction due to being located on the inaccessible ogee portion of the crest.

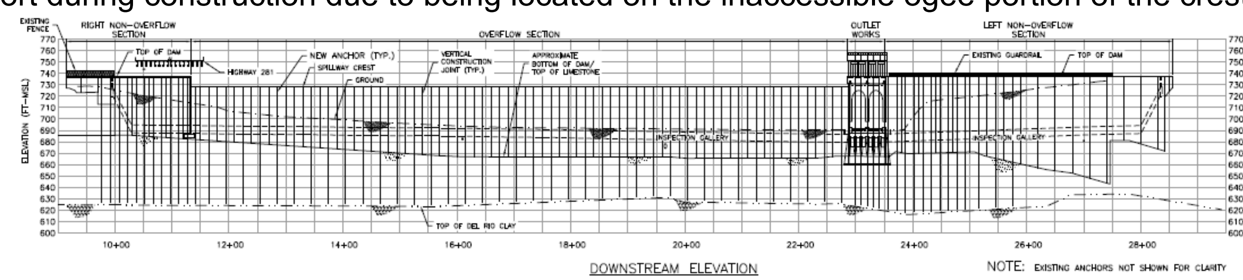

Figure 4. Downstream Elevation View of Olmos Dam with Proposed Anchors

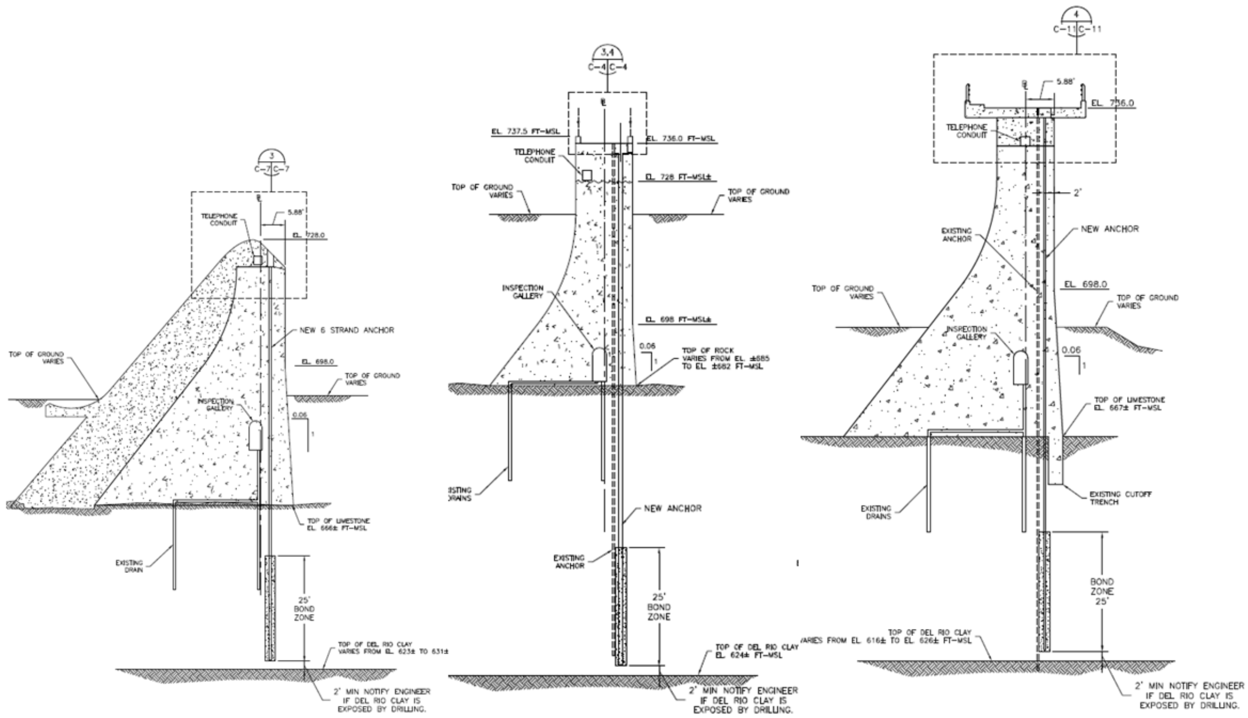

Figure 5. Anchor Configurations for the Overflow, Right Non-Overflow and Left Non-Overflow Sections

The outlet tower anchors proved to be the most challenging. The only available locations for the five anchors in the outlet tower were within the narrow splitter walls of the outlet conduits. Anchors had already been installed in this same location. Gate operating equipment prohibited locating the anchors upstream or downstream by any significant amount. Thus, FNI selected locations very near the original anchors and required that the existing bar anchors be de-stressed before drilling the new holes. The new pockets overlapped the bearing plate area of the old bar anchors, as shown in Figure 6.

Figure 6. Pocket Overlap with Existing Anchors

Corrosion Protection. The corrosion protection was selected to be Class I as described by PTI. It is the highest level of corrosion protection and is mandated when anchors are intended for permanent use rather than temporary use. Previous anchors at Olmos had not received this level of protection since it was not yet developed. The corrosion system consists of the entire anchor length below the trumpet being encapsulated within a 60 mil high density polyethylene (HDPE) corrugated sheath and with the entire free length of the anchor being protected by grease filled polyethylene tubing. The anchor is fully grouted inside and outside the HDPE sheathing with neat cement grout. Also, the entire head assembly is encapsulated beneath a grease filled galvanized steel cover complete with rubber gaskets. Figure 7 presents the details associated with the new Olmos anchors.

Grouting. The Olmos foundation materials were not ideal for installing post-tensioned anchors. This was obvious from geotechnical borings and some of the historical records from previous anchor projects on Olmos. The foundation was highly fractured limestone and gravels in a rather shallow aquifer. The water table was typically only 50 feet below the crest of the non-overflow segments of the dam and approximately 30 feet below the overflow crest. This high water table complicated the drilling and grouting processes. The design team decided to approach this problem by requiring all drill holes to be pre-grouted with a sanded grout mixture upon completion of the first drilling. This pre-grouting would essentially seal the holes and improve the likelihood of producing holes which would result in good corrosion protection in conjunction with the Class I materials. If the pre-grouting did not result in an acceptably tight hole, i.e. one that lost less than 2.5 gallons of water in a ten minute period under 5 psi of differential head, then the hole was grouted with neat cement and re-drilled. This neat cement grouting and re-drilling continued until the hole passed the water test. Only then was the corrugated sheathing and anchor assembly to be approved for installation.

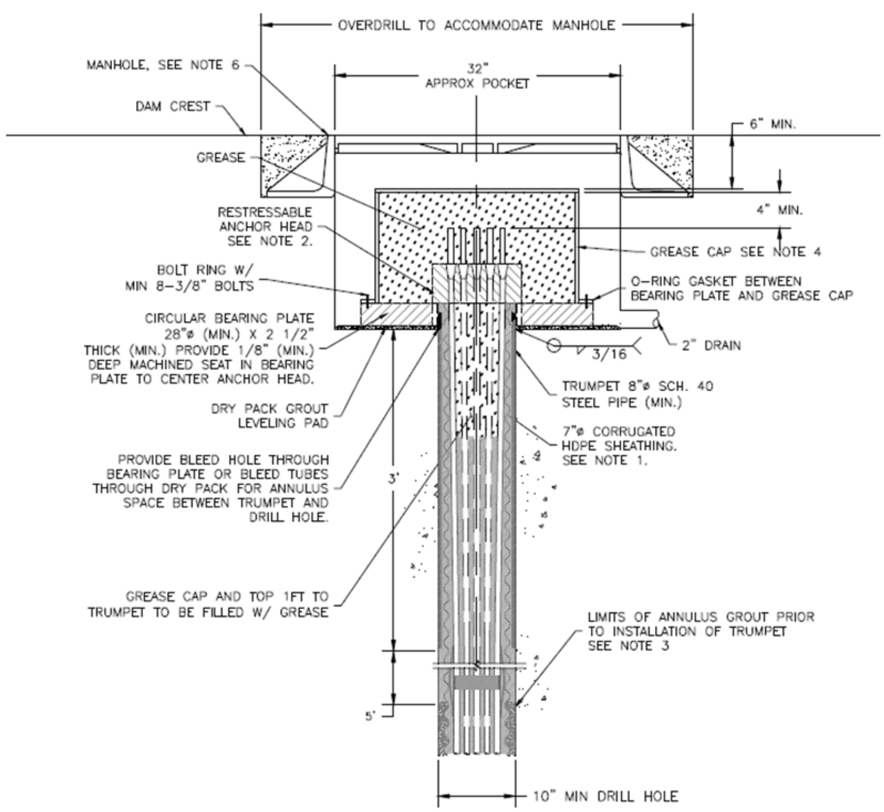

Figure 7. 20 Strand Anchor Vault

Stressing. The stressing program for Olmos was chosen to provide as much information about the performance of the new anchors as possible. Thus, Proof Tests which are typical on many anchor projects were abandoned, and every new anchor was Performance Tested. This would not have been the project approach to stressing had it not been for the failures over the years from the two previous projects.

Re-stressable. All anchor heads were developed with external threads so that a re-stressing device could be used in future lift testing without the need for pulling on the strands directly. This feature will allow shimming if necessary to increase the load on any particular anchor.

Instrumentation. Several anchors were selected to receive load cells for monitoring their long term performance. This was done to provide a level of confidence that the system is working as designed and alleviate the need for lift-off tests for several years. A later segment of this paper deals extensively with all of the instrumentation developed for this project.

Construction

Eight contractors presented bids for the project. The Engineer's estimate was $4,549,100 and the average from all bids was $4,916,675. The low bidder was Archer-Western Contractors with a total bid of $4,092,150. In addition to the anchors, this bid included instrumentation and repairs to the spillway walls. The project duration was established in the project documents as 300 calendar days. The notice to proceed was issued on Wednesday, September 8, 2010, and substantial completion was achieved on June 5, 2011. Archer-Western subcontracted the anchor portion of the project to The Judy Company, Inc.

Drilling

Access Constraints. Installation of the anchors had many challenges. The most significant was access to the work areas: there were four distinct work areas and three of the four areas were severely limited in accessibility. The left abutment was easily accessible as this non-overflow portion was a 20-feet wide concrete driveway to the outlet works gatehouse. The area is accessed from a public street (Olmos Drive), and the 13 anchor holes in this area were drilled in the concrete drive near the upstream parapet using a conventional crawler drill as shown in Figure 8.

In the next section, known as the outlet works, there were five anchors inside the gatehouse. The anchors were located between the gates in order to fit within the splitter walls of the outlet conduits. They were so close to the existing bar anchors that the bar anchors had to be de-stressed prior to drilling the new anchor holes. It was necessary to work around the gate actuators and electrical control panels as well as beneath the roof of the gatehouse. The work area was very restricted. A limited access drill with a width of 30 inches was used to work in the highly congested area as shown in Figure 9.

Figure 8. Drilling Rig on Left Abutment

Figure 9. Limited Access Drill in Gatehouse

The least accessible work area was also the longest section of the work. Thirty-nine anchors were located along the 1,500-feet long ogee crest of the overflow section as seen in Figure 10. Normally dams have water upstream and access to dam crests can be obtained with barges. However, Olmos is a flood retarding structure and normally the reservoir is empty. Thus, the anchors were up to 40 feet above the reservoir floor and inaccessible to normal construction. On this project the contractor decided to construct a rolling platform for drilling the anchor holes. The platform was supported on a rail bolted to the upstream concrete face and on the concrete crest of the dam as seen in Figure 11 and Figure 12. The platform was 17 feet wide by 29 feet long providing room for a full-size crawler drill and adequate work area for crews. Wheels driven hydraulically from the drill moved the platform along the dam. The platform was built in two elevations to provide access to the tendons and pockets. Man lifts and two small rolling platforms were used to core the pockets, install and grout the tendons, and perform post-tensioning. The drill had to be lifted by a crane onto the platform, and the drill remained on the platform until all 39 crest anchor holes were drilled. A crane then removed it from the platform.

Figure 10. Access to Overflow Section

Figure 11. Moving Platform at Overflow

The third work section with severe access constraints was the right abutment. This segment had the tightest vertical limitations where six of the eleven required anchors were located directly beneath the US Highway 281 bridge. The bottom of the beams was only eight feet above the concrete crest. The anchors were located between the beams to gain approximately three more feet of clearance and the contractor drilled the holes using a crawler drill with a shortened mast as seen in Figure 13.

Figure 12. Rig on Moving Platform

Figure 13. Rig at Right Non-overflow Section

Environmental Controls. A down hole percussion hammer was used for the anchor hole drilling in both concrete and rock. Significant amounts of steel as seen in Figure 14 were encountered in the concrete, particularly in the left abutment non-overflow section and the gatehouse. The rock was relatively soft and drilled easily. The site was environmentally sensitive making it necessary to collect all the drill cuttings as seen in Figure 15. This was accomplished using a deflector bolted to the concrete to divert the discharge into a roll off.

Figure 14. Steel Removed from One Hole

Figure 15. Steel Roll Off Tank for Cut Drilling Waste

Water Testing and Grouting. After each hole was drilled, a water test was performed to check the hole for potential leakage. This project was unique in that there was no reservoir water upstream to establish the static surface above which leakage testing would be performed. A falling head test was used as the testing method for evaluating the leakage. After the initial drilling, the static water elevation was checked with a water level probe as seen in Figure 16. Then 12 feet of water was added to the hole above the static water level, and the water surface was monitored for 10 minutes. The water take into the formation was calculated based on the drop in water level. Holes leaking more than 2.5 gallons in 10 minutes were grouted with sanded Portland cement grout on the first grouting. The grout was allowed to harden for at least sixteen hours and then the hole re-drilled. The hole was then retested. Those failing were re-grouted with neat Portland cement grout as many times as necessary until passing the test. The number of times a hole required grouting was variable depending on location. Only a few holes passed without an initial grouting. Some holes required grouting four times before passing the falling head test.

Figure 16. Water Level Probe

Figure 17. Corrugated 12 feet above grade for Leak Test

Corrugated Sheathing. All anchors utilized HDPE corrugated sheaths for the Class I corrosion protection. When the drilled hole passed the water test, it was deemed ready for installation of the corrugated sheathing. There was also a requirement for water testing the corrugated sheaths. Getting the sheath to pass was one of the more difficult and elusive tasks on the project. The 20 strand tendons used 6-inch diameter sheathing and the 6 strand used 4-inch diameter. The HDPE sheathing was 60 mils thick. The specifications required testing the sheathing for leakage prior to and after installation in the drill hole. On the surface testing, the open end of the sheath was lifted 12 feet above the lower end cap and filled with water, as shown in Figure 17. The leakage test was then simply measuring the water loss in the sheath over a ten minute time period. It was difficult to get all the air out of the sheathing as air would become trapped in the corrugations and only slowly work its way to the top. Filling with a hose inserted to the bottom worked best.

End caps on the sheaths were grouted with neat cement and then heat shrink tape was placed around the HDPE cap. Any visible leaks were patched with heat shrink tape. Sheathing with significant damage was discarded. No splicing was allowed on the project.

Once the sheathing passed the leakage test, it was inserted into the drill hole. The static water level was approximately 50' below the top of the dam. It was necessary to fill the sheathing with water to sink it into place. The sheathing was very sensitive to unbalanced water levels. The sheathing would crush from excessive water on the outside or burst from excessive pressure on the inside. It was necessary to match the water levels inside and out within 20 feet to prevent damage. Once installed, the sheathing was again water tested to assure that the insertion process had not damaged the sheathing and caused leakage. Once the corrugated sheathing passed the water test, it was to be grouted prior to the tendon insertion. However, numerous collapses of the corrugated sheaths occurred during the grouting stages. The practice of grouting the sheathing was abandoned in favor of inserting the tendon in the un-grouted sheath and then simultaneously grouting inside and outside the sheathing.

Tendons. The tendons were of conventional construction with the free length greased and sheathed with extruded polyethylene over the strand. The 25-foot bond length was bare strand. The tendon was assembled at the manufacturer and shipped in a coil. The tendon was lowered into the hole using a un-coiler provided by the supplier as shown in Figure 18. The tendon was inspected as it was inserted into the hole. Any cuts in the sheathing were carefully patched.

Difficulties were encountered while inserting the tendons. In some instances, the corrugation was damaged during insertion, and in some cases the tendon became wedged and could not be lowered into the hole. After several failed insertion attempts, one of the tendons was uncoiled on the surface. It was found that because of the difference in the inside and outside radius of the coils the extra length would accumulate making a large basket that would expand in the hole, as shown in Figure 19. It would damage the corrugation and wedge inside the hole. Once the problem was discovered, the bands used for assembly were cut as the tendon was inserted and the extra material worked to the top so the strands could even out. This was a slow and tedious process.

After the tendon was fully inserted, it was suspended from a frame or tied to the un-coiler to keep it off the bottom prior to being grouted. During grouting it was necessary to keep the pressure balanced inside and out. It was more sensitive than the water testing due to the greater density of the grout. Grouting in stages was attempted. It was found that in some instances a 20 foot lift would damage the tendon, so lifts were confined to 10 foot stages. This was also slow and tedious. It was necessary to make test cubes of each lift and keep track of what elevations were grouted with a specific batch.

Figure 18. Installing Tendon with Un-coiler under the bridge

Figure 19. Strand Tendon Opening up During Uncoiling

A process of using magnetic flow meters to balance the grout between the inside and the outside and pumping in one lift was tested and was successful. The flow from the grout pump was routed to a header that could control the flow to two flow meters. One meter was connected to the grout tube extending inside the corrugated tube and the other to the tube on the outside. The flow rate was balanced based on the ratio of the areas by controlling valves on the header. With some experience, the crews were able to keep the level balanced within a few feet. This greatly increased production and reliability.

After the grouting had been completed, the tendon was now ready for the bearing plate. To assure full contact with the concrete, high strength, non-shrink grout was used to make a leveling pad for the bearing plate. This grout has very interesting thixotropic properties. It appeared to be very dry and mortar like. If mixed it would return to a more liquid state. With about ¾ inch of leveling material in place, the bearing plate was positioned on the uncured grout. It was checked for alignment and levelness and then tamped or repositioned as needed.

Grout cubes were made for tendon and bearing plate grout. Once the grout cubes attained adequate strength and 7 days had elapsed after tendon grouting, the tendons could be stressed.

Stressing. Before setting the large stressing jack seen in Figure 20, each individual strand was stressed with a mono-strand jack to 10% of the lock off load and the wedges installed. Each tendon was Performance Tested including Creep Test. Hydraulic jack pressures and load cell readings were taken at each load. Dual dial gauges measuring to the nearest 1/1000 inch were used to measure strand elongations. After Performance Testing, the wedges were restrained, the lock off load was applied, and the wedges seated. Immediately after lock off, a lift off test was performed. The tolerance was -2% to +5%. Shims were added as needed if the lock off load was lower than the tolerance.

Figure 20. Performance Testing a 20 Strand Tendon

A total of sixty eight anchors were installed on this project: 27 twenty-strand and 41 six-strand. There were no failures or excessive elongations during the testing of the tendons. The data from each tendon were plotted and analyzed prior to cutting the tails and completing the tendons by grouting the free length, installing caps, and filling the caps with grease as seen in Figure 21. Load cells were installed on ten selected anchors for long term performance monitoring. All anchors have re-stressable heads for re-tensioning the strands if necessary. Anchors on the overflow section were backfilled with lean concrete as seen in Figure 22. All other anchors are installed beneath removable manhole covers. Data collected through January 2012 from available load cells show anchors responding satisfactorily and in accordance with design parameters.

Figure 21. Grease Cap Installed

Figure 22. Lean Concrete Backfill on Ogee Anchor Holes

Instrumentation

Due to the history of poor anchor performance at the dam, instrumentation was installed to monitor the anchor performance without the need for lift-off tests. Load cells were installed to monitor the anchor load, extensometers were installed to monitor for movement of the dam and piezometers were installed to measure uplift pressures on the dam foundation. The data has been collected for nearly one year and shows the anchors are performing as anticipated.

Load Cells

Ten load cells were installed to monitor the anchor load. Four load cells were installed in the overflow section on the six strand anchors while six load cells were installed in the non-overflow sections on the twenty strand anchors. Load cell readings were taken immediately after stressing the anchor and at the 72-hour lift off test to compare with the jack reading. The load cells readings were generally within 10% of the jack readings. The load cell readings were adjusted so that the jack readings and load cell readings at the 72-hour lift off tests were equal. The first load cell was installed in February 2011 and additional load cells were installed periodically until the end of May 2011. The load cells were read on average three times after installation until June 2, 2011, when the automated data collection system was functional. At that point, readings were taken every six hours.

A review of the load cell data reveals that the load cell readings are clearly related to temperature. Figure 23 shows the 20-strand load cell readings versus temperature. The temperature effect may be due to the load cell or both real load changes caused by the expansion and contraction of anchor tendons, anchor head and dam concrete. In order to observe anchor performance independent of temperature, the linear trends observed in Figure 23 were used to adjust the remaining figures to 100ºF.

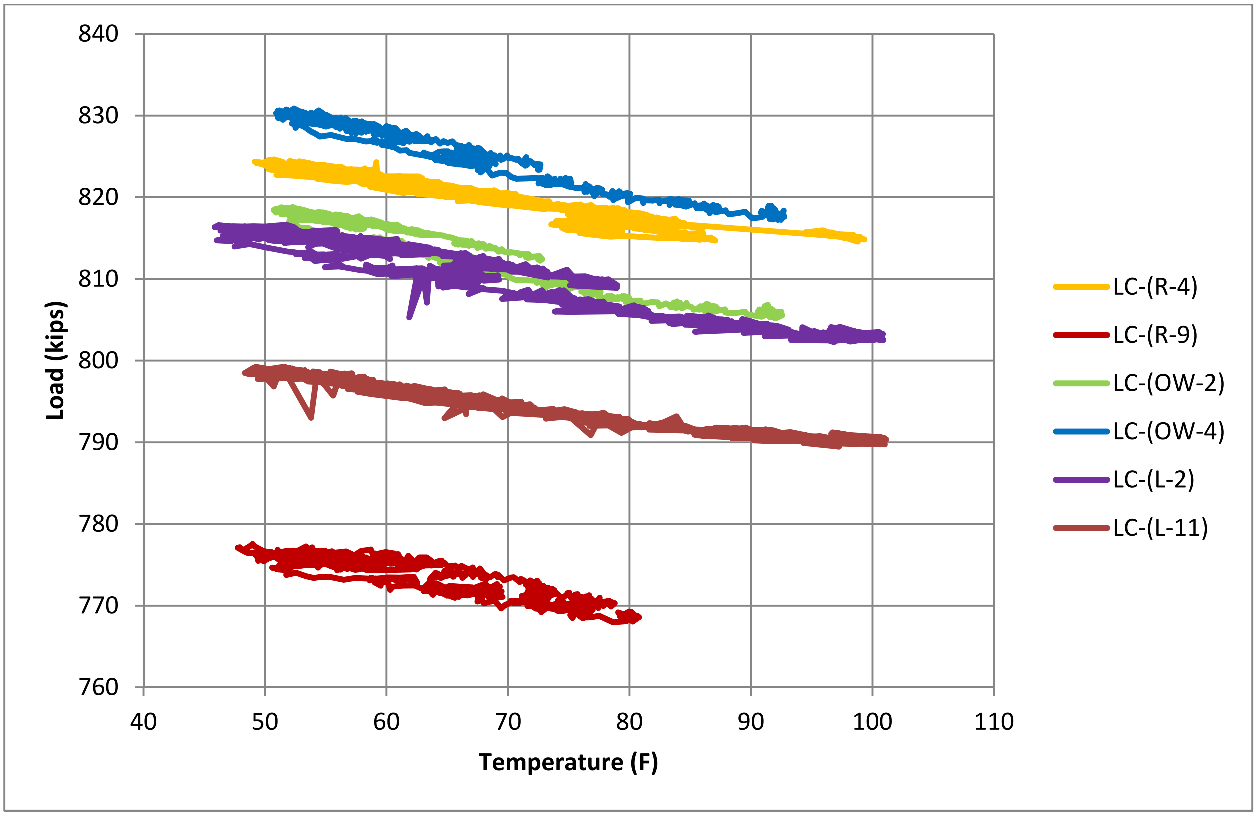

Figure 23. 20-strand load cell readings related to temperature

Figure 24 shows the temperature adjusted 20-strand load cell readings versus date and Figure 25 shows the same readings versus logarithmic installed time. The readings show an initial load loss and then relatively constant readings. The design line on Figure 25 shows the anticipated anchor load for the 50-year design life. The 20-strand anchors are reporting loads greater than design and the slope of the observed loads is less than the design slope, indicating that the anchors are holding the load better than design expectations.

The temperature adjusted readings for the 6-strand load cells are shown in Figure 26 and Figure 27. The readings are similar to the 20-strand readings in that there is some initial load loss during the first few months after installation and then the readings become relatively constant. Of interest are sudden load reductions of approximately two kips in load cells O-26 and O-38. The readings indicate that the anchors are providing the design load and that if the current trend continues, they will have more than the design load at the end of 50 years. Future readings will be monitored to verify that the trend continues. No correlations have been observed between lake level and load cell reading to date.

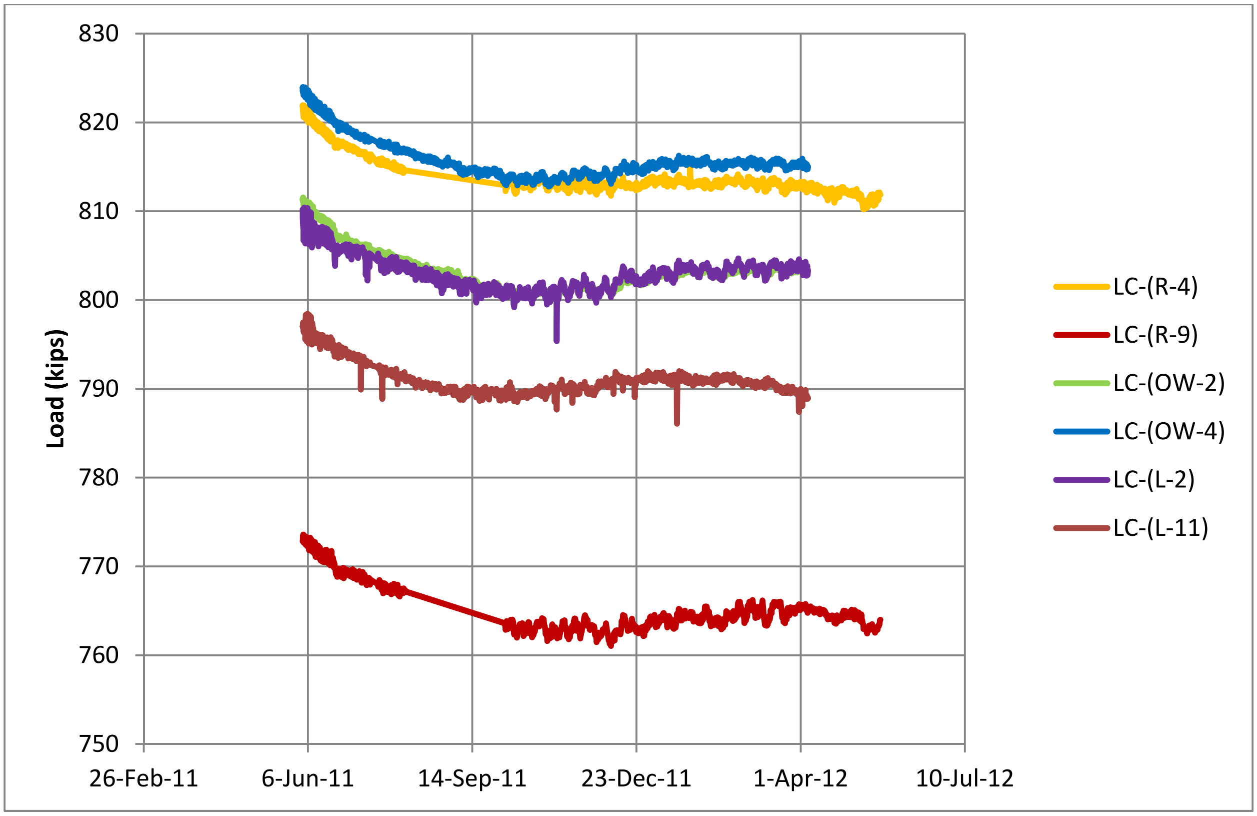

Figure 24. Temperature adjusted 20-strand load cell readings versus date

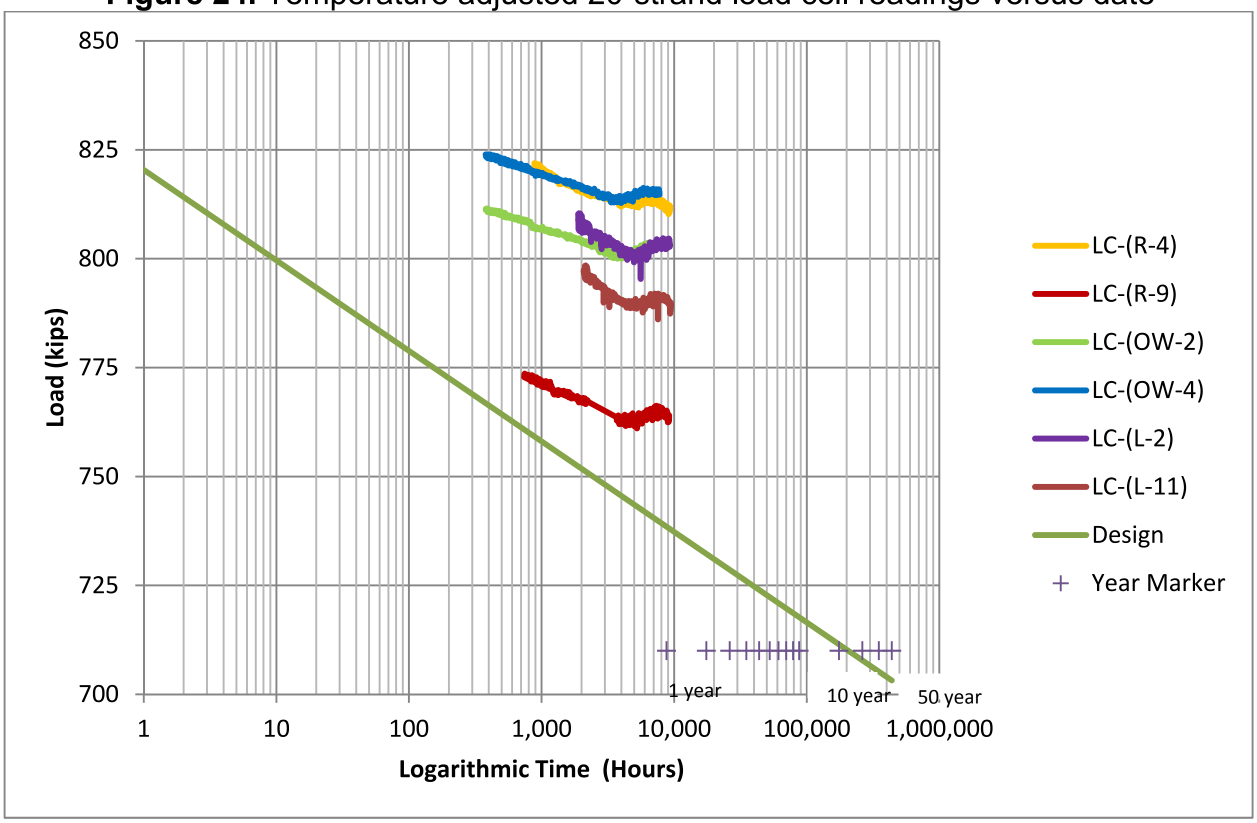

Figure 25. Temperature adjusted 20-strand load cell readings versus installed time.

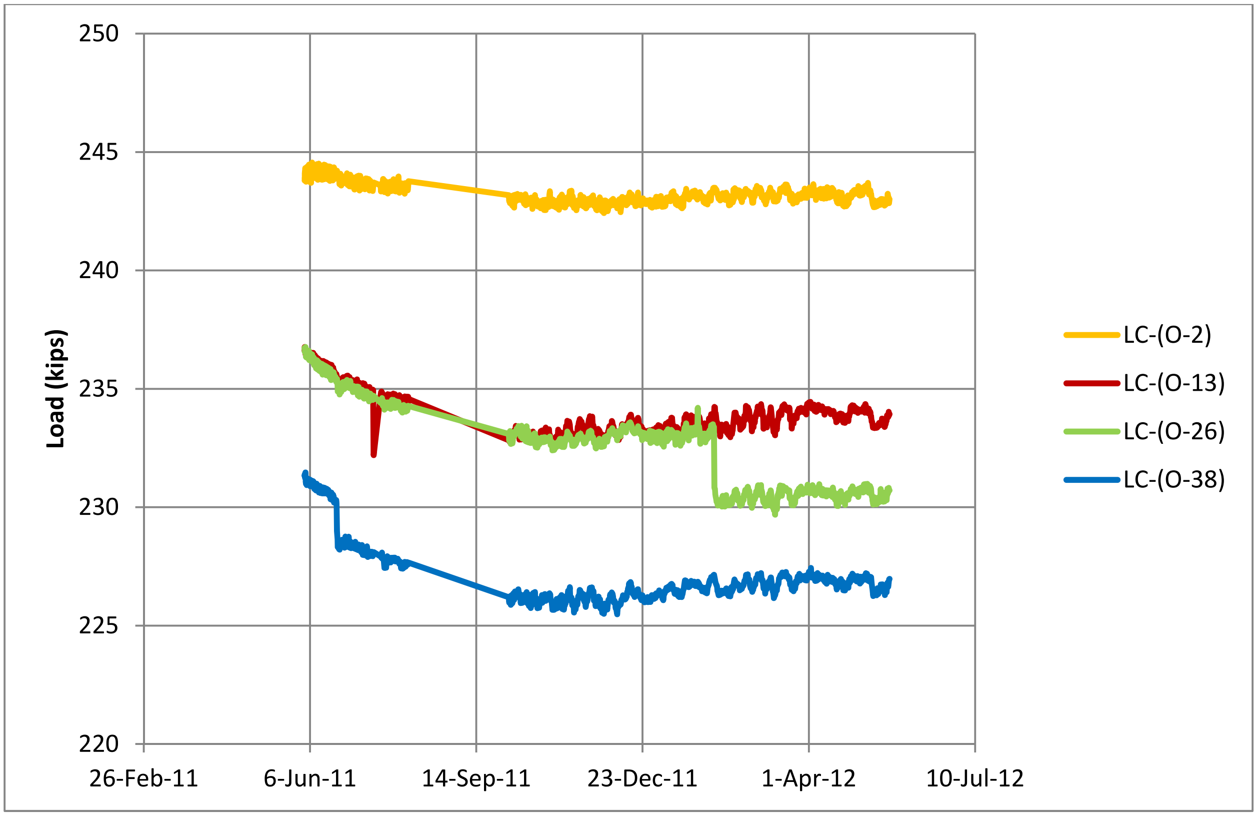

Figure 26. Temperature adjusted 6-strand load cell readings versus date

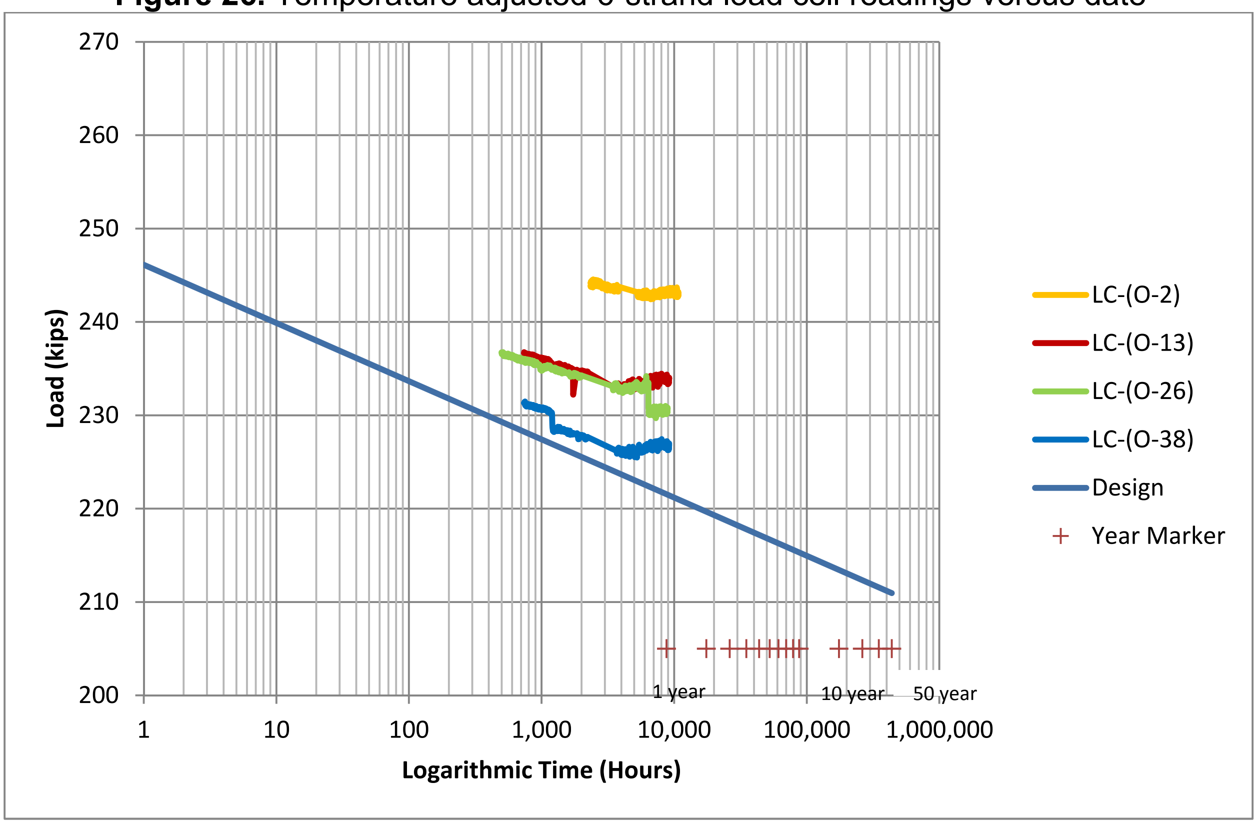

Figure 27. Temperature adjusted 6-strand load cell readings versus installed time

Extensometers

Four extensometers were installed to monitor movement of the dam crest relative to the foundation. Movements of approximately 0.1 inch have been observed. Similar to the load cells, the readings in the left non-overflow are correlated to temperature. As the weather cooled, the extensometers reported contraction between the foundation and the crest and as the weather warmed, the displacement grew. The extensometer in the right non-over flow has reported minimal movement. Figure 28 and Figure 29 show the observed movement. No correlations have been observed between lake level and extensometer reading to date.

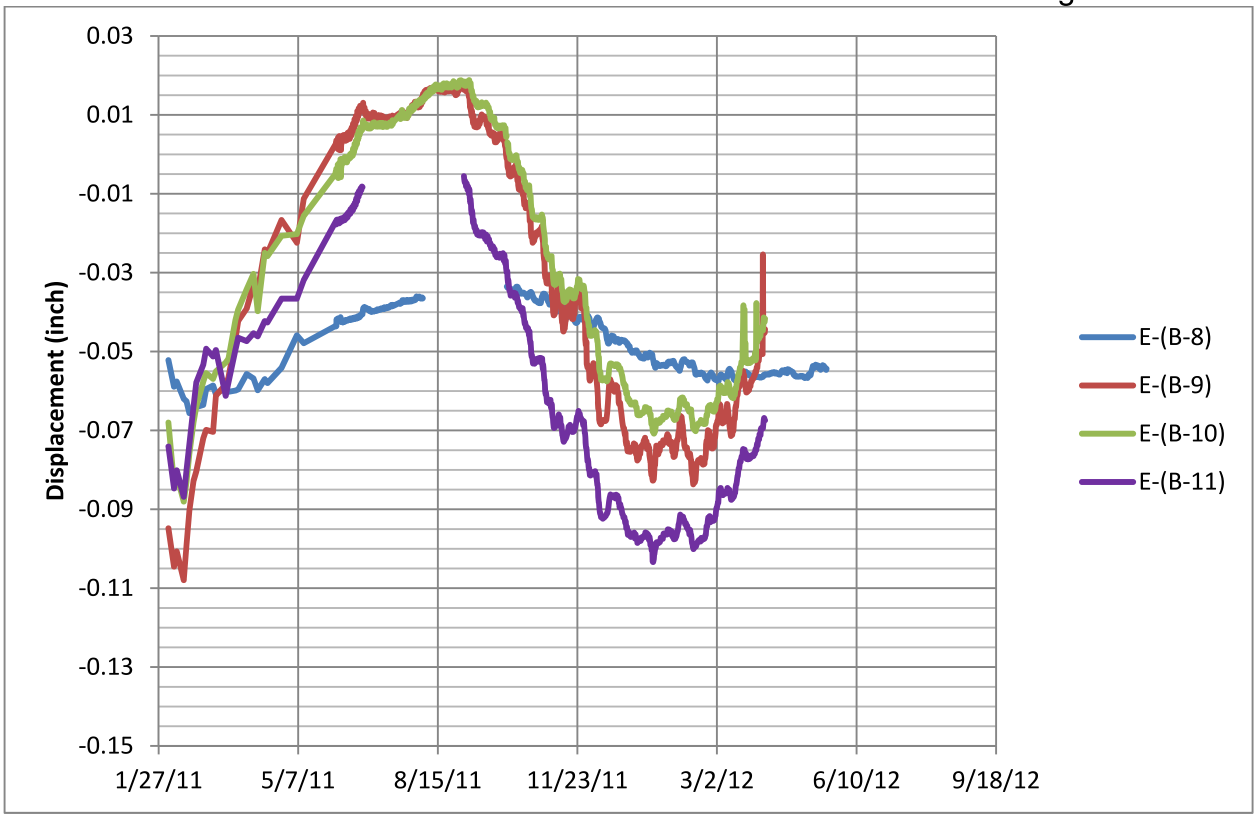

Figure 28. Extensometer readings versus date

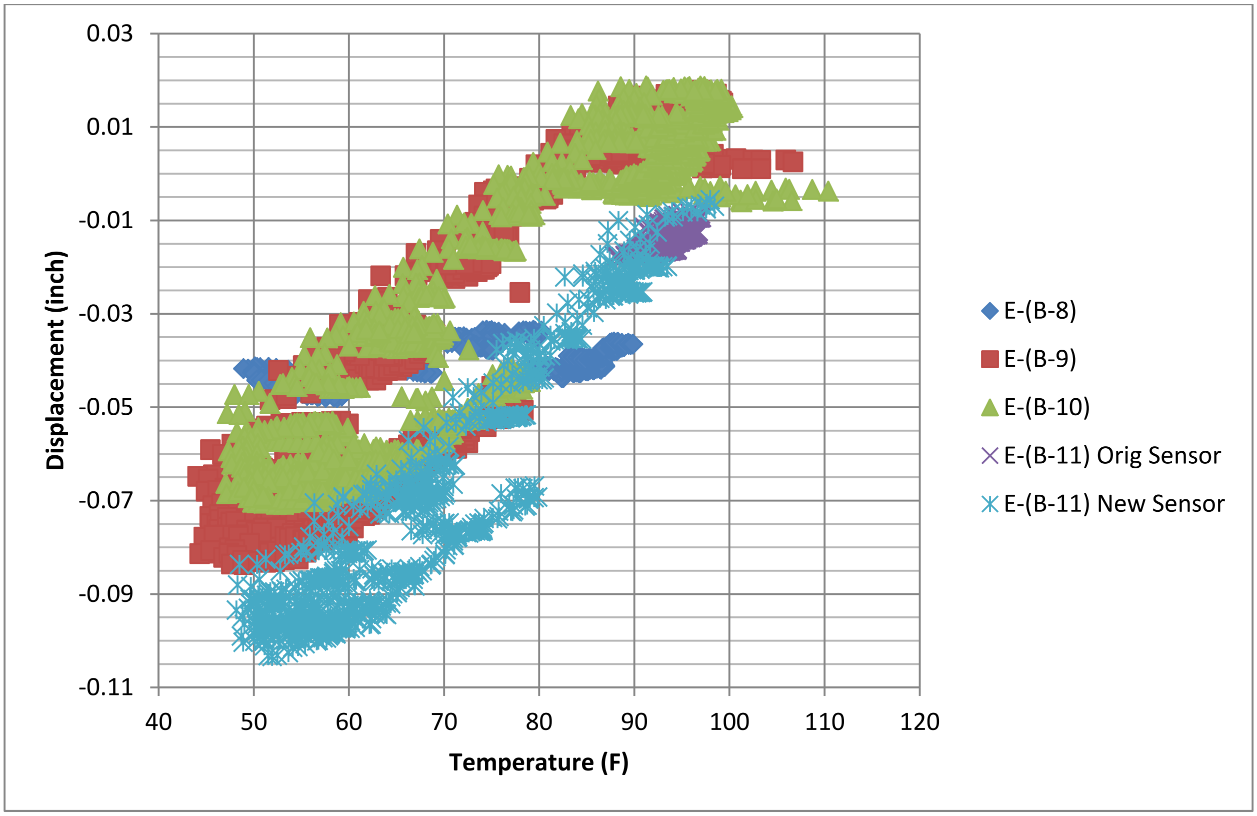

Figure 29. Extensometer readings versus temperature

Conclusion

There may well have been significant apprehension on the part of the Owner of Olmos Dam when approached with yet another set of anchors for Olmos Dam stabilization. However, a successful test anchor program and development of special construction requirements were instrumental in designing and constructing 68 new post-tensioned strand anchors without a single failure. Including change order, Archer-Western completed the project on schedule and $71,455 below the bid price. Instrumentation load cells indicate that after almost one full year of service all instrumented anchors are performing above our design expectations. The design and construction provided a viable solution to the dam stability concerns without significant change to the structural appearance of the dam and at a reasonable cost. We trust that the anchors will provide a very satisfactory stabilization to Olmos Dam for another 50 years.

References

1. Givler Engineering, Inc. (2004). “Olmos Dam Engineering Report.”
2. Givler Engineering, Inc. (2006). “Olmos Dam Testing and Inspection.”