THE THIRD TIME'S A CHARM

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ABSTRACT

The City of San Antonio has suffered from numerous floods on the San Antonio River throughout its history. In response to major floods in 1913 and 1921, the City implemented a flood control improvements program. Olmos Dam, finished in 1929 and located six miles north of downtown, was built as a flood retarding concrete gravity dam with no emergency spillway. Flood releases were made through six outlet tunnels regulated by slide gates. In 1974, an engineering study indicated that the dam did not have sufficient discharge capacity to prevent overtopping at PMF and the structure did not meet acceptable safety factors for events larger than the 100-year flood. As a result, modifications began in 1978 to replace 1,500 feet of the non-overflow section with an uncontrolled spillway and to add post-tensioned anchors to the non-overflow sections. As early as 1984, problems were reported with some of the bar anchors. In 1995, post-tensioned strand anchors were added to a select area of the dam to again increase its resistance to sliding and overturning. Subsequent inspection and testing of the anchors at Olmos Dam demonstrated a progressive deterioration of the anchors and a reduction in their capacity to hold the required load. Thus, in 2007 Bexar County embarked on a study to evaluate alternatives to stabilize Olmos Dam. Several alternatives were considered for stabilizing the dam in addition to strand anchors. However, strand anchors were far more cost effective in the final analysis and would have little or no effect on the historic appearance of the dam. Thus, Olmos Dam headed into its third round of anchors.

INTRODUCTION

The City of San Antonio, Texas, has suffered from numerous floods on the San Antonio River throughout its history. In response to two major floods in 1913, the City performed a comprehensive study to develop recommendations for flood control improvements. The study recommended that various projects be developed that would protect the City from a flow on the San Antonio River of up to 22,000 cubic feet per second (cfs) through

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downtown, roughly triple the peak of the 1913 flood. The plan included Olmos Dam and channel improvements to the San Antonio River and its tributaries. Sadly, in September 1921, a flood of almost this magnitude did occur with the loss of 50 lives and millions of dollars in property damage. The City began to implement the recommended plan promptly thereafter.

Olmos Dam in its original form was completed in 1928 and modifications to the river, including the “Great Bend Cutoff” in the downtown area, were finished by 1929. The dam was located on Olmos Creek, about 6 miles upstream from downtown San Antonio. The dam was built in the Edwards Aquifer zone and directly on the Buda Limestone Formation which was in turn found to be directly underlain by the Del Rio Clay Formation. The original Olmos Dam consisted of a concrete gravity structure with no emergency spillway section. It was 1,941 feet long and had a maximum height of 58 feet above grade and about 85 feet above its rock foundation. The crest of the dam was a roadway for vehicular traffic. Flood releases were made through six outlet tunnels regulated by slide gates. The purpose of the dam was to store flood waters and to release them gradually through the tunnels.

Olmos Dam remained virtually unchanged for almost 50 years and served an integral part of the flood protection program for the City. In 1974, an engineering study re-evaluated the potential flooding from Olmos’ 33 square-mile drainage area. The study stated that the dam was acceptably stable up to the 100-year flood, but for potentially much larger floods, the structure would not be stable nor have sufficient discharge capacity to pass the flood without being overtopped. Construction of dam modifications occurred between 1979 and 1981 to address the noted issues. A 1,151-foot long spillway with a crest elevation of 728 feet-mean sea level (msl) was constructed in place of the roadway across the dam. The construction also included 113 post-tensioned anchors on the remaining non-overflow section of the structure.

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. In 2007, Bexar County contracted Freese and Nichols, Inc. (FNI) to further evaluate the anchors at Olmos Dam and consider alternative stabilization options in lieu of the anchors.

Background

Anchors at Olmos Dam have endured a troublesome history. Extensive research was required to develop a better understanding of the issues leading to anchor failures. The research started with historic records of the dam followed with a literature search of anchor technology and its evolution over the years. The following sections provide a brief introduction to anchor technology and its history followed with a summary of the history of anchors at Olmos Dam.

Overview of Anchor Technology

Multi-strand and bar post-tensioned anchors are commonly used throughout the world for anchoring gravity dams [1]. The first post-tensioned anchoring projects on dams occurred in the 1930's, although the widespread use of post-tensioned anchors for stabilizing dams in the USA did not begin until the early 1960's. The United States Army Corps of Engineers (USACE) was an initial proponent of the technology and were soon joined by the United States Bureau of Reclamation (USBR) and certain utility companies. In the United States, permanent post-tensioned anchors in dams have been used in over 400 structures over the past 40 years. Since the mid 1970's, five different versions of recommendations for ground anchor practice have been used as a national guidance document. The first attempt to provide guidance on anchor design came in 1974 though the Post Tensioning Division of the Pre-stressed Concrete Institute (PCI) with a document titled "Tentative Recommendations for Pre-stressed Rock and Soil Anchors". In 1980, the Post-Tensioning Institute (PTI) published "Recommendations for Pre-stressed Rock and Soil Anchors" which was subsequently updated in 1986, 1996, and 2004. Each version built on the previous one with improvements, clarifications, and additional information as time went on. The latest 2004 edition by PTI provides detailed guidance and commentary on materials, site investigation, design, corrosion protection, construction, stressing and testing, recordkeeping, and specifications.

History of Anchors at Olmos Dam

Developing an understanding of the issues with post-tensioned anchors at Olmos Dam was important for assessing their viability versus other stabilization alternatives. 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 series of post-tensioned anchors at Olmos Dam 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 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 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. Post-tensioned 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.

The first report of an anchor problem was written 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 in all 113 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 ultimate strength. 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 holes was performed using a down hole 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.

The 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 [2]. In response, the City conducted additional studies at Olmos Dam in 2006 which included lift-off tests on all the anchors at the dam [3] with the following results:

- three new broken bars (Figure 1),
- two bars still damaged as noted in the past,
- one bar broken during testing, and
- continued loss of load.

Figure 1. 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. Five monoliths were listed as not meeting the design criteria. Monoliths whose anchor cumulative loads were above design criteria showed variations of 105% to 134%. Given the yearly load loss, it was reported that additional monoliths would fail to meet design values in a few years. To address the five unsafe monoliths, it was recommended to install nine new anchors and to re-stress seven existing anchors. These improvements would bring the monolith loads back to design loads. However, this was considered a temporary fix since the anchors would continue to lose load with time. Finding a long term solution to the stability problems at Olmos Dam was ultimately recommended to the City.

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. 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.

Difficult Geology. 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 this 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.

Geotechnical drilling under the direction of FNI included rock strength and geologic assessment and supplemented previous studies by including packer testing and installation of piezometers throughout the site and at various depths. As a whole, the geotechnical investigation confirmed four distinct rock strata below the dam in the following order: 1) tan-white limestone, 2) blue limestone, 3) blue-gray clay shale, and 4) blue-gray limestone. The limestone was very porous as expected for a site located within the Edwards Aquifer zone. The tan-white limestone was more weathered and porous than the blue and blue-gray limestone. The compressive strength of limestone was moderate to high and packer testing revealed a highly permeable formation. The shale compressive strength varied from very low to high with a very low permeability. Piezometer measurements showed a fast response and significant water pressure when the dam impounded water or the aquifer level was high. Water movement was evidenced by steady flows coming from the foundation drains when aquifer levels were high.

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. Given the site conditions and past history of de-bonding failures, FNI recommended a test anchor program prior to construction to aid in the design effort, evaluate groutability of the rock, and assess construction sequencing.

<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 SYSTEM

The new design phase for 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.

Stabilization Alternatives

A stability analysis of Olmos Dam demonstrated that the structure would not be considered stable at high flood loading conditions. The PMF is 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>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.

Added Mass. 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.

Full Anchoring. Despite the short-comings in design and construction, most of the existing anchors have performed satisfactorily for up to 30 years. However, anchor design and construction for Olmos Dam had to be approached with care. With the anchor history at Olmos Dam, careful design and construction was essential to avoid the issues that have affected prior anchoring projects. A Test Anchor Program was performed as part of the anchor design to evaluate bond stresses, installation sequence, stressing and monitoring.

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, we recognized that they would require extra effort during construction due to being located on the inaccessible ogee portion of the crest.

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 a location very near the original anchor 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 2.

Figure 2. 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. 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 3 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.

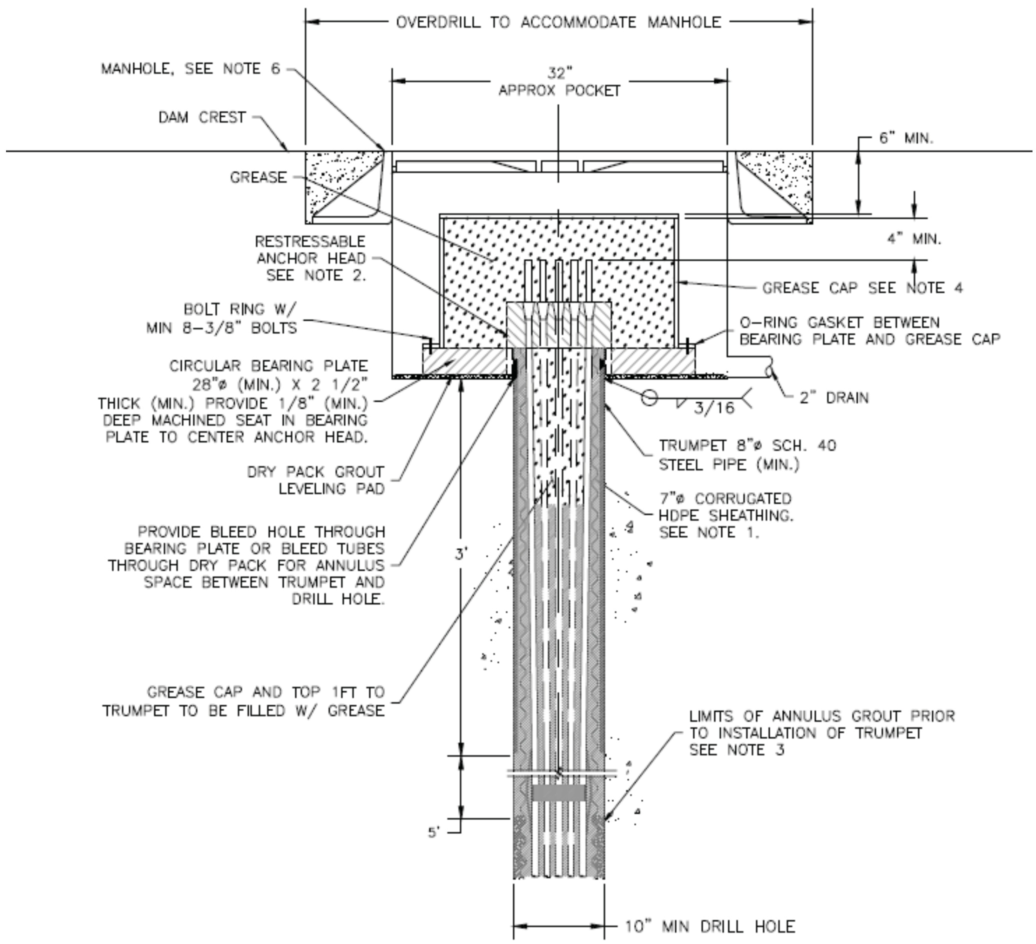

Figure 3. 20 Strand Anchor Vault

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.

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 which are not discussed in this paper. 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 4.

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 5.

Figure 4. Drilling Rig on Left Abutment

Figure 5. 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 6. 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 7 and Figure 8. 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 6. Access to Overflow Section

Figure 7. Moving Platform at Overflow Section

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 9.

Figure 8. Rig on Moving Platform

Figure 9. Rig at Right Non-overflow Section

Environmental Controls. A down hole hammer was used for the anchor hole drilling in both concrete and rock. Significant amounts of steel as seen in Figure 10 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 11. This was accomplished using a deflector bolted to the concrete to divert the discharge into a roll off.

Figure 10. Steel Removed from One Hole

Figure 11. 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 12. 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 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 12. Water Level Probe

Figure 13. Corrugated 12 feet above grade for Leak Test

Corrugated Sheathing. All anchors utilized HDPE corrugated sheaths for the Class I corrosion resistance. 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 13. The leakage test was then simply measuring the water level change 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.

It was easy to see if there was a leak on the surface. Any visible leaks were patched with heat shrink tape. End caps on the sheaths were grouted with neat cement and then heat shrink tape was placed around the HDPE cap. Sheathing with observed leaks was patched with large heat shrink tubing. 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.

<u>Tendons</u>. 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 14. 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 15. 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 14. Installing Tendon with Un-coiler under the bridge

Figure 15. 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 strength and 7 days had elapsed after tendon grouting, the tendons could be stressed.

<u>Stressing.</u> Before setting the large stressing jack seen in Figure 16, 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 16. 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 17. 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 18. 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 17. Grease Cap Installed

Figure 18. Lean Concrete Backfill on Ogee Anchor Holes

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. 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. Thus, “The Third Time’s A Charm.”

REFERENCES

1. Bruce, D.A., and J. Wolfhope (2007). "Rock Anchors for North American Dams: The Development of the National Recommendations (1974-2004)," Institution of Civil Engineers, Ground Anchorages and Anchored Structures in Service, November 26-27, London, England, U.K., 11 p.

2. Givler Engineering, Inc. (2004). “Olmos Dam Engineering Report.”

3. Givler Engineering, Inc. (2006). “Olmos Dam Testing and Inspection.”