REHABILITATION OF A STEEL PENSTOCK

BY: ROBERT A. JAMES, ROGER L. INMAN, MARK S. PETERSON,
THOMAS L. KAHL, AND KEITH MARTIN

ABSTRACT

One of the 29 hydroelectric stations owned and operated by the Tennessee Valley Authority (TVA) is a two-unit plant with vertical Francis turbines. The water passageways for these units consist of two intakes feeding individual rock tunnels and above ground steel penstocks with surge tanks. A butterfly valve is located just downstream of each of the surge tanks.

Although these penstocks had operated reliably for nearly 80 years, TVA's proactive maintenance and inspection program identified an area of concern: the shell thickness readings for the Unit 1 penstock showed steel thinning. Calculations were performed which revealed unacceptable stresses in the steel for some loading conditions.

TVA selected Kleinschmidt, Inc., an energy and water consultant, to perform an independent evaluation of the Unit 1 penstock, including taking ultrasonic thickness readings and performing a stress analysis. This analysis confirmed previous TVA investigations. While the penstock did not pose an immediate safety hazard, the shell was experiencing continued deterioration, and its present condition did not meet normal industry safety factors as outlined in the American Society of Civil Engineers (ASCE) Manuals and Reports on Engineering Practice No. 79, Steel Penstocks. Therefore, Kleinschmidt recommended replacement of the Unit 1 penstock. As a precaution, TVA placed operating limitations on the plant, and the frequency of inspection was increased until the penstock could be replaced.

Kleinschmidt was awarded the task of designing the replacement Unit 1 penstock and supports. This was also an opportune time to address other possible problems with the penstock system.

FIGURE 1 ABOVE GROUND PENSTOCK BEFORE REPLACEMENT

DESCRIPTION OF PENSTOCK SYSTEM

The concrete-lined rock intake tunnels are approximately 490 feet long for each of the units and have approximate diameters of 14 feet for Unit 1 and 16 feet for Unit 2.

The tunnels transition into above ground riveted steel penstocks which originally were roughly 160 feet long and had diameters of 12 feet for Unit 1 and 14 feet for Unit 2. Original steel thicknesses were 3/8 and 7/16 inches for Unit 1 and 1/2 inch for Unit 2. The penstocks were supported by concrete saddles, and a portion of each penstock was buried as it entered the powerhouse. **Figure 1** is looking upstream at Unit 1 from the powerhouse.

The Unit 1 surge tank is a riveted steel structure that is approximately 100-feet tall with a diameter of 20 feet. The original steel thickness was 3/8 inch. The Unit 2 surge tank is constructed of reinforced concrete and has a significant portion embedded in rock.

The butterfly valves located just downstream from the surge tanks are anchored primarily by post-tensioned rock anchors.

PROBLEMS TO ADDRESS

In early May 2004, Kleinschmidt was awarded the task of designing the replacement Unit 1 penstock. Kleinschmidt also was awarded the task of evaluating two other concerns and designing fixes, if required. The two concerns were:

- Evaluate the ability of the Unit 1 and Unit 2 butterfly valve anchorages to resist emergency closure of the valves under flow.
- Assess the ability of the slender Unit 1 surge tank to resist current seismic loads.

DESIGN AND EVALUATION

GEOTECHNICAL INVESTIGATION

Geotechnical design criteria were needed for the design of the Unit 1 penstock and its supports and also for evaluation of the butterfly valve anchorages. As one of the first tasks, Golder Associates, a geotechnical firm, was retained to investigate and characterize subsurface soil, rock and groundwater conditions underlying the project structures. A field program including bedrock surface mapping, rock core sampling, and hydro-geologic testing was performed in June and July 2004.

DESIGN OF REPLACEMENT UNIT 1 PENSTOCK

Because of dramatically rising steel prices, TVA required that the shell material steel grade and thickness be established early in the project. This was accomplished in early June 2004 and allowed TVA to initiate steel procurement and lock in the cost of the steel. The preliminary design also included a revised layout where the new replacement penstock geometry was simplified by using only one vertical bend in the penstock instead of two vertical bends as used in the existing penstock.

The 30-percent design complete meeting was held at the site to allow the design team to obtain measurements and confirm critical dimensions obtained from drawings of the existing facility. These field dimensions revealed some minor dimensional discrepancies and ambiguities, which, while not affecting the fundamental aspects of the replacement penstock design, would have an important impact on the detailed design and construction fit-up. As a result, a detailed survey of pertinent portions of the existing facility affecting the replacement penstock was determined to be needed and was completed by licensed TVA surveyors in early July.

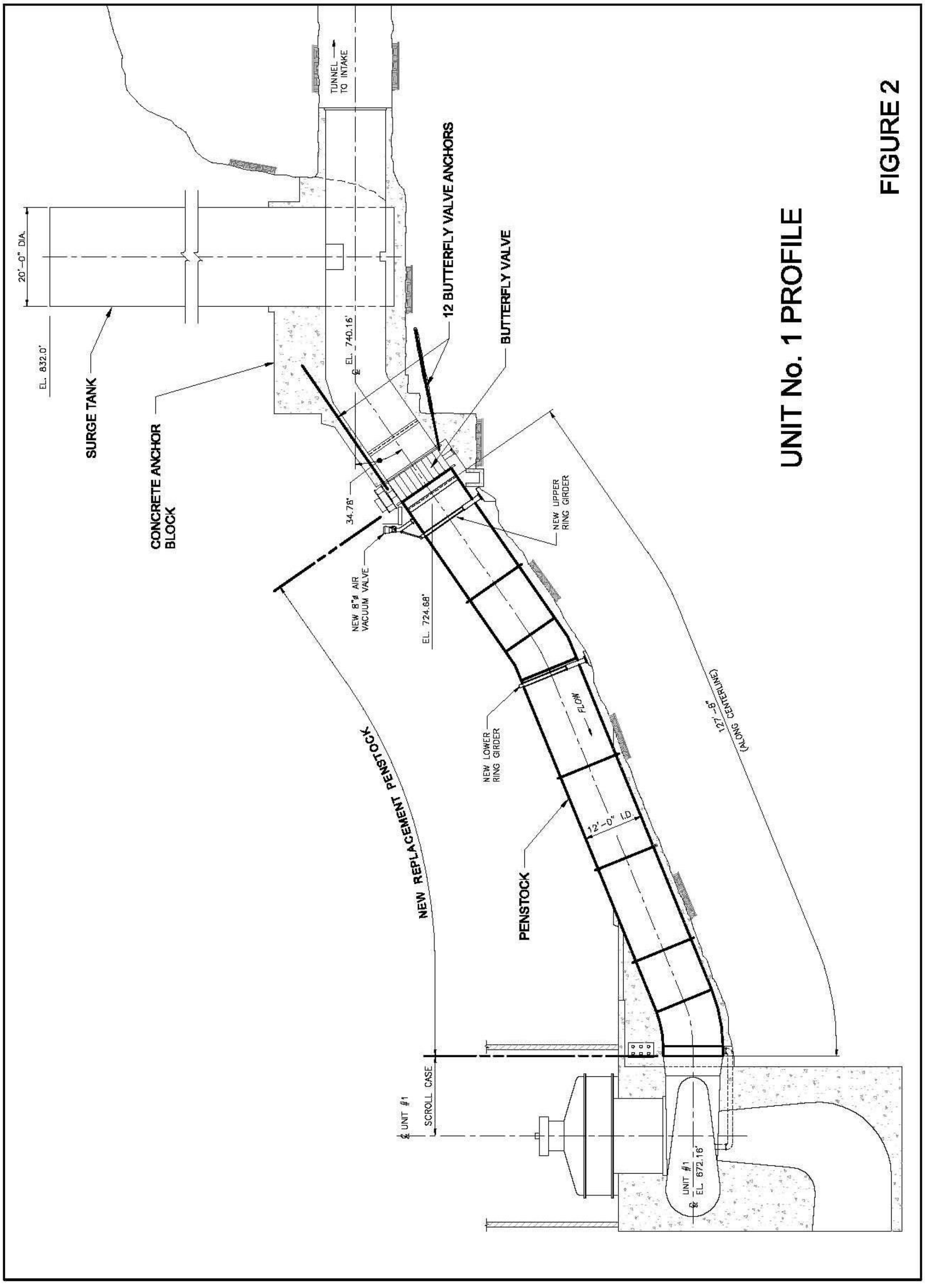
During the 30-percent design complete meeting, TVA and Kleinschmidt engineers were also able to discuss important issues, including construction access, pertinent TVA operational procedures, replacement penstock design options, and project design parameters such as the importance of minimizing station operation disruption during penstock replacement construction.

After completion of a detailed site survey in July and receipt of a preliminary geotechnical design criteria memorandum in early August, the preliminary design of the Unit 1 replacement penstock design was advanced to the 60-percent complete level. This level included the design of the ring girders that TVA had selected in the preliminary phase as the new penstock's above ground supports.

During the valve anchorage analysis, which will be discussed later in this paper, it was realized that rigidly connecting the penstock directly to the downstream face of the butterfly valve could cause the valve reactions to be inadvertently resisted by the stiff downstream penstock shell. Therefore, the completed Unit 1 replacement penstock final design included an expansion coupling located immediately downstream of the butterfly valve. A newer Depend-O-Lok "Victaulic" style expansion coupling that clamps over the ends of the mating pipe, manufactured by Brico Industries, was selected because of the cost savings and increased reliability offered by its simplified construction compared to more traditional multi-bolted ring-type expansion couplings. Large diameter Depend-O-Lok couplings have been successfully used on hydroelectric penstocks since the mid-1990s.

In late November 2004, TVA solicited bids for construction of the project. At that time, it was found that estimated construction costs far exceeded the project budget due to the amount of rock excavation that was required for the new layout using one vertical bend. Therefore, it was decided to revert to the existing configuration using two vertical bends. Although this change was made late in the process, it did not have significant impact on the project. The design thickness of the steel did not change, and fabrication of the new penstock had not begun. **Figure 2** shows the final profile of the Unit 1 replacement penstock and location of the other primary water passage features.

BY: ROBERT A. JAMES, ROGER L. INMAN, MARK S. PETERSON, THOMAS L. KAHL, AND KEITH MARTIN



UNIT No. 1 PROFILE

FIGURE 2

EXISTING SURGE TANK

The existing 1916 vintage surge tank, with a slender 100-foot height to 20-foot diameter aspect ratio, had an unknown resistance to TVA's current seismic criteria. TVA uses site-specific determined seismic loads based on a five-percent damped, probabilistically derived earthquake (5,000-year return period). The imposed seismic loads were a horizontal acceleration of 0.16g corresponding to a spectral acceleration of five Hertz and a vertical acceleration of 2/3 the horizontal acceleration. Based upon the tank's geometry and seismic load magnitude, it was agreed that a practical analysis approach was to utilize the pseudo-dynamic analysis approach incorporated in the American Water Works Association (AWWA) "Welded Steel Tanks for Water Storage" D-100-96, as modified by the NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (FEMA 450). This seismic analysis methodology assumes a rigid tank shell, but takes into account the fundamental period of the convective mode of vibration, or the sloshing of surface liquid against the tank shell.

The surge tank's shell stress analysis was based on TVA's recent 2002 actual surge tank shell thickness ultrasonic readings. This analysis showed that the existing tank shell and bottom anchorage were sufficiently strong to resist TVA's current seismic loads. Although the tank shell is not directly anchored to the surge tank's concrete base through conventional shell anchor bolts, a simplified conservative load-path analysis showed that the surge tank's tight penetration into the concrete foundation effectively transferred the shell reactions into the tank's concrete foundation without any shell overstress. Also, the existing foundation mass provided sufficient stability to prevent tank sliding and overturning.

During the surge tank analysis, an unexpected result was discovered. When the shell's longitudinal rivet efficiency of 73 percent was taken into account, three of the lower five-foot high courses of the existing tank shell did not meet the ASCE Manual No. 79 factor of safety for normal internal pressures.

The lowest of the three overstressed shell courses is encased by mass concrete with an eight-inch high by 12-inch thick concrete curb for a portion of its circumference and by a 12-inch thick reinforced concrete wall for the remainder of its circumference. The stresses in the shell course were determined to be acceptable if confinement by the mass concrete and reinforced concrete was considered. However, investigation by TVA with a rebar detector determined that there was no circumferential reinforcement in the top portion of the wall. Therefore, the repair for this shell course involved installing a steel clamping plate around the outside circumference of the eight-inch high curb and the top of the wall.

The repair for the two higher shell courses above the surge tank's foundation involved welding reinforcing plates to the shell at the riveted longitudinal joints. The reinforcing plates were designed to have holes drilled to clear the existing rivet heads so that the plates could fit tightly against the tank exterior.

EXISTING BUTTERFLY ANCHORAGE EVALUATION

This task included evaluation of the existing anchorages for the large diameter butterfly valves for gravity and hydrodynamic loads. The 1988 vintage butterfly valves had been retrofit into the penstocks to provide a means to rapidly close off the penstocks without having to dewater the tunnels. The valve's outward thrust is resisted by circumferential post-tensioned rock anchors set at various angles and lengths into the supporting concrete and rock mass.

This evaluation began with an evaluation of the butterfly valve reactions, especially during emergency closure when the turbine is at overspeed. Previously completed TVA computer dynamic waterhammer penstock and valve reaction analyses were reviewed and found to reasonably match traditional simplified hand calculations. One of TVA's primary concerns for the butterfly anchorages was the effect of an overall moment on the existing valve anchorage created by unbalanced disk pressures during valve closure. After a review of the reactions, including consultation with TVA's in-house hydraulic specialists and valve manufacturer design engineers, it was concluded that the lateral valve disk pressures are balanced when considering the overall valve freebody, and so the valve anchorages only need to resist the overall valve axial reaction. The calculation for anchorage reactions based on the maximum pressure with closure during overspeed when the turbine is tripped off-line and incorporating the non-symmetrical rock anchor arrangement resulted in individual anchor reactions ranging from 63 to 104 kips.

To appreciate how the rock anchor capacities were evaluated, an understanding of the complex anchor arrangement and bedrock conditions is required. Twelve post-tensioned anchors were installed during the 1988 retrofit construction in an equidistant circumferential pattern around the perimeter of each penstock, as shown in **Figure 2**. The anchors consist of grouted high strength Dywidag bars installed at vertical orientations above horizontal at angles varying from nine degrees to 34 degrees. At Unit 1, the four anchors installed in the invert area of the penstock were drilled into the bedrock mass behind the concrete structure containing the butterfly valve. The eight other anchors at Unit 1 were installed into the mass concrete structure. At Unit 2, all 12 anchors were installed into the bedrock. According to the drawing notes, all rock anchors were installed at least 19 feet into the bedrock face, with a 14-foot grouted bond zone at the end of each anchor. The free stressing length and total anchor length varied for different anchors depending on the thickness of concrete that had to be penetrated to reach the rock face. Total anchor lengths vary from 24 to 42 feet.

The capacity of the rock anchors depends on the strength of the steel bar, the strength of the grout in the annular space between the bar and the rock, the bond strength for bar-to-grout and grout-to-rock, and the characteristics of the rock mass engaged by the grouted bond zone. Adequate strength of the bar steel, grout, and bar-to-grout bond were confirmed following conventional hand calculation methodology and guidelines prepared by the Post Tensioning Institute (Ref. 1). To assess grout-to-rock bond capacity, shear strength of the rock mass, and the geometry of the engaged rock mass within the anchor bond zone surrounding each penstock, an evaluation of bedrock structure, rock shear strength and anchor resistance mechanisms was completed.

The bedrock lithology at the site consists of crystalline and siliceous limestone with shaley limestone interbeds. In the vicinity of the butterfly anchorages, the findings from test borings and bedrock mapping indicated a transition occurred near the base of the butterfly valves (about elevation 715 feet) from a more competent lower part of the bedrock formation to a more fractured upper zone with extensive horizontal and vertical joints, soil-filled cavities and voids. Most of the anchor-bond zones were interpreted to be located in the upper more fractured and less competent limestone; only the lower four rock anchors beneath the invert of the Unit 2 penstock were assumed to be located within the more competent lower limestone formation. Based on comparing field and lab tests to applicable ranges of published grout-to-rock bond stresses (Ref. 1, 2, 3), it was concluded that all anchors had satisfactory grout-to-rock bond capacity even when accounting for the possible presence of bedrock voids in the bond zone.

To assess the affects of rock mass strength on the capacity of the anchors, it was necessary to make an approximation of the shape of the engaged rock mass providing resistance for the anchors and to assign a shear strength value for the rock mass acting on the surface of the engaged mass. The factor of safety of the anchor group was calculated as the ratio of the rock strength acting on the cone surface to the sum of the tension load on the anchor and the gravity load of the rock mass. Normally, the gravity load of the rock opposes the load in the anchor. However, since all anchors are inclined upward, the weight of the engaged rock mass reduces capacity.

Because the anchors are closely spaced, the theoretical cone of engaged rock for each anchor overlaps with cones from adjacent anchors creating a group-action effect. The aggregate total volume and surface area of the engaged rock mass for all anchors in rock at a penstock is less than the sum of individual cones for single anchors.

Golder used the three-dimensional solid modeling capabilities of AutoCAD 2004 to develop interpreted configurations for engaged rock mass shapes. A three-dimensional model was created for each butterfly valve anchorage that included the arrangement of the radial anchor pattern, the variable anchor orientation both vertically and horizontally, the close proximity of the penstock and concrete structures supporting the butterfly valve and surge tank, and rock slope face geometry. The apex angle of the cone at each anchor was assumed to be 90 degrees based on the tabular characteristics of the bedded limestone formations at the site. **Figures 3** and **4** show profile and oblique views generated by AutoCAD for the interpreted engaged rock mass at each butterfly valve. The volume and surface area of the engaged rock mass at each unit was calculated by AutoCAD.

On the basis of the interpretation of bedding and joint plane geometries defining the geologic structure within the anchor zones, Golder calculated a tensile strength for fractured rock using the Hoek-Brown failure criterion (Ref. 4). The rock mass tensile strength value was determined based on bedrock characteristics determined from surface mapping, test borings, and field and laboratory tests completed on rock core samples. A tensile strength value of 40 pounds per square inch (psi) was determined for the rock mass representing the upper less competent limestone, and a tensile strength of 210 psi was determined for the lower more competent formation below elevation 715 feet.

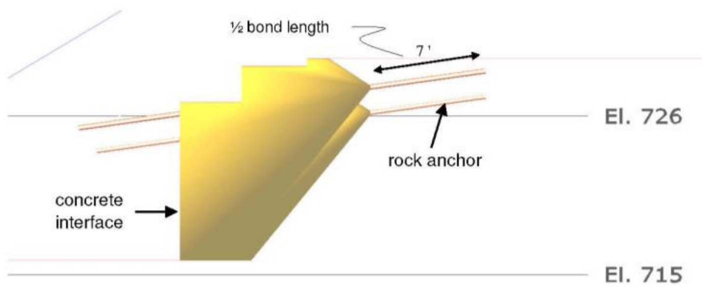


Image 1 - Profile view of Unit 1 rock anchorage, showing engaged rock mass cones developed by the four rock anchors

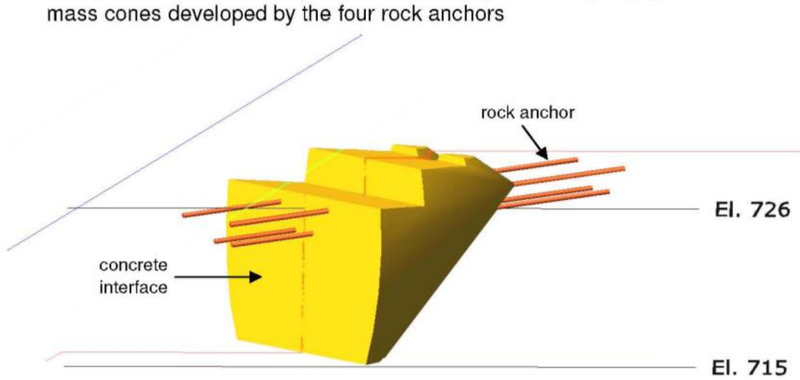


Image 2 - Oblique front view of Unit 1 engaged rock mass anchorage

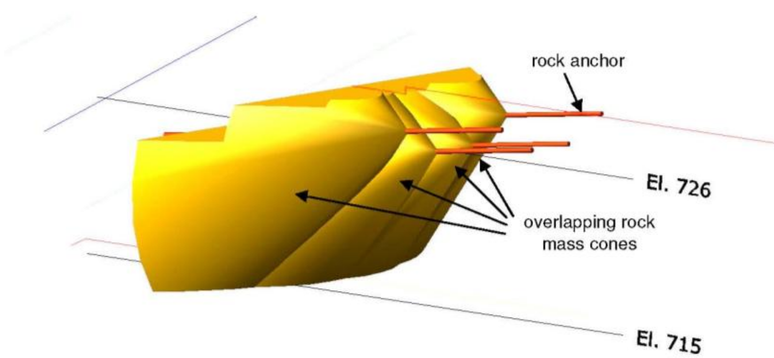


Image 3 - Oblique rear view of Unit 1 engaged rock mass anchorage

FIGURE 3
INTERPRETED UNIT 1 BUTTERFLY VALVE ROCK MASS ANCHORAGE

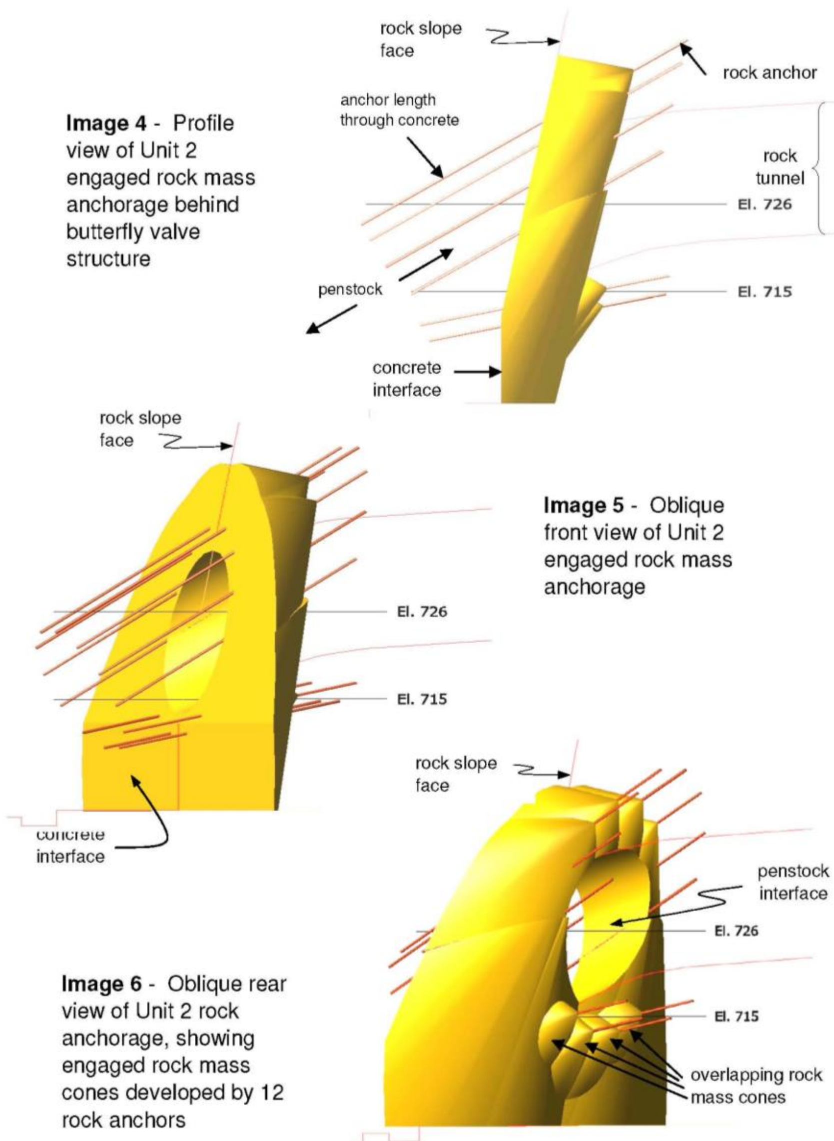


FIGURE 4
INTERPRETED UNIT 2 BUTTERFLY VALVE ROCK MASS ANCHORAGE

The results of the evaluation indicated a factor of safety against rock mass anchor pullout of 12.4 for the Unit 1 anchors, and 6.9 for the Unit 2 anchors. Both values exceeded the minimum recommended safety factor of 3.0.

CONSTRUCTION

SCHEDULE

The construction schedule was developed utilizing data that showed the value of the electricity produced at the plant for each month of the year. The construction and the required outage were then scheduled for August through October 2005 when the lost revenue would be the least.

UNIT 1 PENSTOCK REPLACEMENT

FIGURE 5 FIRST FIT-UP SLIT CUT

Existing Fill Concrete – Although the plant drawings showed backfill around the below grade portion of the existing Unit 1 penstock, it was suspected that fill concrete had actually been placed to encase the penstock. Therefore, some preliminary excavation was performed prior to construction mobilization, and it was determined that the penstock was indeed encased in concrete. Because of the effort required to excavate the concrete, consideration was given to the possibility of keeping the encased penstock and making the connection of the new penstock to the old penstock just outside the concrete instead of at the face of the powerhouse as had originally been planned. This would have involved cutting off the new penstock that had already been fabricated and would have probably involved modification of the unreinforced fill concrete and the encased penstock. Ultimately it was decided to excavate the concrete and replace the penstock all the way to the face of the powerhouse. To decrease the outage time for penstock replacement, extra shifts had to be added for concrete excavation.

Old Penstock Removal – After some test cuts to determine if the interior penstock coating would catch on fire, a plasma arc was used to cut the old penstock into pieces that could be lifted out with a crane. The entire top half of the above-grade portion of penstock was removed first. Next, the bottom half was removed, which proved to be a much slower process due to stiffener ribs around the penstock that were embedded in a concrete spine that ran between the concrete saddle supports. Removal of the below-grade portion of penstock was an even slower process as concrete encasing the penstock had to be removed before pieces of the penstock could be cut away and removed.

Fit-up at Powerhouse – Fit-up between the new penstock and the old penstock at the face of the powerhouse was a problem when the circumferences did not align perfectly. To resolve this problem, Kleinschmidt recommended cutting longitudinal slits in the new penstock to allow the circumference of the new penstock to be adjusted to match the circumference of the old penstock. The slits were then welded back together with full penetration welds. Five slits were cut, and the alignment method worked well. **Figure 5** shows the first fit-up slit cut.

Fit-up Between Penstock Sections – The installation was intended to use fit-up strap plates which would cover the gap at the joints between penstock sections. The constructor requested that the strap be made in two pieces with one piece being shop-welded to the bottom of one penstock section and with the other piece of the strap plate shop-welded to the top half of the adjacent section that would join with it. This would allow the strap to serve as a support for the adjacent penstock section. These straps had dog ears that could be bolted together until the field welds were made that would join the penstock sections together via the straps. This worked adequately as long as the gap between penstock sections remained as designed. But when the gap could not be accurately obtained because of field adjustment tolerances, the straps and their bolt holes were no longer in alignment. Therefore, the top piece of the shop-welded strap had to be removed and realigned. There also was a problem with welding the straps together at the equator of the penstock. The straps did not quite meet, and a small piece had to be welded between the ends of the straps to fill the gap. **Figures 6** and **7** show penstock installation and completion.

FIGURE 6 PENSTOCK INSTALLATION

FIGURE 7 COMPLETED PENSTOCK

Kleinschmidt

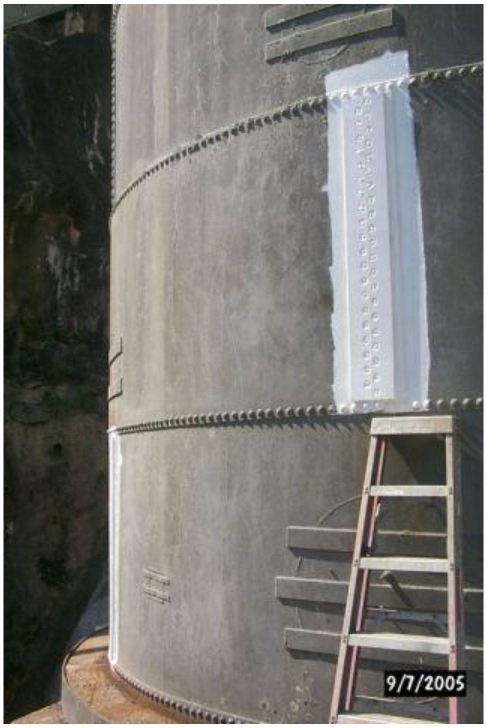


FIGURE 8 SURGE TANK REPAIR

UNIT 1 SURGE TANK REPAIR

Prior to fabrication of the surge tank repair plates, a paper template of the rivet reinforcement plate was prepared to verify that the holes in the plate would fit over the rivet heads. There was concern that the rivets may not have been uniformly placed and the rivet heads would prevent the plate from fitting correctly. The template showed no problems, and the final plates did fit well during construction. However the strap plate that was added to confine the concrete around the surge tank did not fit tightly. Sika Dur Lo Mod LV cement was used to fill the gaps between the strap and concrete and provide a uniform contact surface. **Figure 8** shows the reinforcing plate. The strap plate also can be seen in the bottom left of the photo.

RECOMMENDATIONS AND LESSONS LEARNED

Based on our experience with this project, we offer the following recommendations for similar rehabilitation projects.

- Even though steel penstocks typically have long reliable service lives, a systematic condition monitoring program should be an important aspect of any aging penstock's maintenance program.

- A comprehensive field investigation in the preliminary design phase is advisable to confirm that the rehabilitation work scope is complete. Look for the unexpected, particularly on all water passage components.
- It is very important to have accurate existing dimensional information from constructible baselines. Rehabilitation projects inevitably need to utilize a combination of information and dimensions from existing drawings of unknown accuracy. These dimensions should be treated as assumptions and confirmed as much as possible with accurate independent field measurements in the preliminary design phase.
- It is advantageous to be open to design alternatives that may offer project savings without decreasing the project's quality or performance. During construction bidding on this project, for example, the initial single-bend simplified penstock layout was unexpectedly more costly than the more complicated multi-bend layout that eliminated some rock excavation and the design was modified for the less costly alternative.
- Cost-effective, three-dimensional analytical methods are now available to evaluate unusual rock mechanics geometries such as those presented by the butterfly valve anchorages for this project. Resolution of the rock mass anchor resistance for this project reduced concern for an issue that was initially considered to present significant risk.

REFERENCES

1. Post Tensioning Institute, "Recommendations for Prestressed Rock and Soil Anchors," 3rd Edition, 1996.
2. Littlejohn, G.S. and Bruce, D.A., "Rock Anchors –Design and Quality Control," Design Methods in Rock Mechanics, proceedings from the Sixteenth Symposium on Rock Mechanics, University of Minnesota, Sept.1975.
3. Wylie, Duncan C., Foundations on Rock, E&FN Spon, New York, 2nd Edition, 1999.
4. Hoek, E., Carranza-Torres, C., and Corkum, B., "Hoek – Brown Failure Criterion – 2002 Edition," 5th North American Rock Mechanics Symposium and 17th Tunneling Association of Canada Conference, NARMS-TAC, 2002, p. 267-271.

AUTHORS

Robert A. James, P.E., is a Principal Engineer in the Dam Safety and Civil Engineering Department of the Tennessee Valley Authority in Chattanooga, Tennessee.
Roger L. Inman, P.E., is a Senior Civil Engineer in the Dam Safety and Civil Engineering Department of the Tennessee Valley Authority in Chattanooga, Tennessee.
Mark S. Peterson, P.E., is a Senior Consultant with Golder Associates Inc. in Brunswick, Maine.
Thomas L. Kahl, P.E., is a Kleinschmidt Associate senior engineer and the Vice President of Hydro Engineering in Pittsfield, Maine.
Keith A. Martin, is a Kleinschmidt Associates structural engineer in Pittsfield, Maine.