WATER OPERATION AND MAINTENANCE

BULLETIN NO. 172

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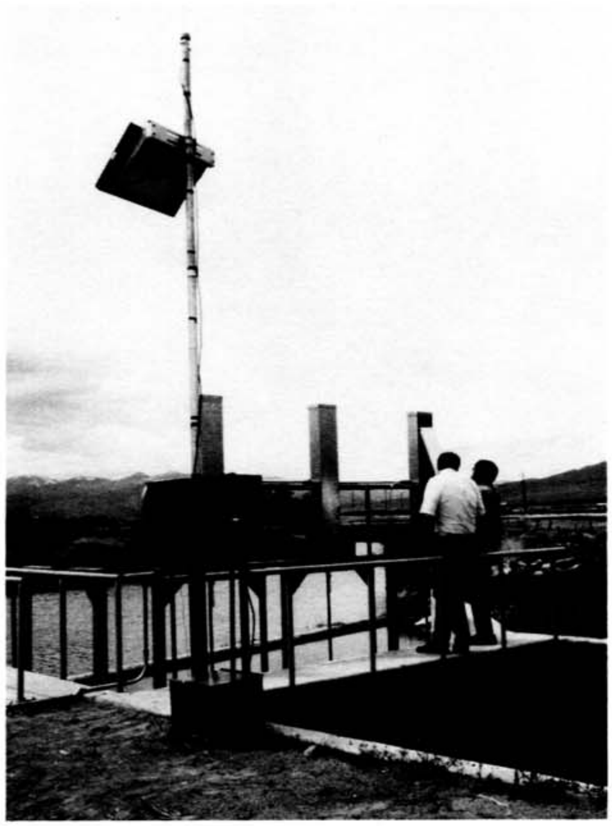

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UNITED STATES DEPARTMENT OF THE INTERIOR
Bureau of Reclamation

The Water Operation and Maintenance Bulletin is published quarterly for the benefit of water supply system operators. Its principal purpose is to serve as a medium to exchange information for use by Reclamation personnel and water user groups for operating and maintaining project facilities.

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REPAIR AND REHABILITATION OF PRESTRESSED CONCRETE PIPE AT THE CENTRAL ARIZONA PROJECT

by Michael T. Peabody

INTRODUCTION

The Bureau of Reclamation (Reclamation) has designed and constructed thousands of miles of pipelines of various types, diameters, and pressure heads. Although no type of pipe can be expected to last indefinitely, the agency has recently experienced costly, premature problems with prestressed concrete pipe.

In August 1984, Reclamation experienced an abrupt failure of a 66-inch-diameter prestressed concrete cylinder pipe unit on Reach 3 of Jordan Aqueduct near Salt Lake City, Utah. The catastrophic rupture occurred in the prestressed section of the cathodically protected line and discharged an estimated 5 million gallons of water into the residential neighborhood adjacent to the break, essentially emptying the line. The subsequent flood covered an area of one city block. Extensive investigations were undertaken to determine the cause(s) and extent of the distress and to evaluate future pipeline serviceability. Final remedial measures involved lining all 2.3 miles of the aqueduct with an internal steel liner at a cost of about $5 million.

In January 1990, Reclamation surveys indicated potential corrosion problems and the need to initiate excavations along six 252-inch-diameter Hayden-Rhodes prestressed concrete siphons of the Central Arizona Project near Phoenix, Arizona. Widespread pipe distress, much of it severe, was confirmed at some of the suspected sites and at many additional sites as well. Large-diameter steel pipe sections were therefore fabricated, which could be used as a means of repair in the event of a rupture. In addition, a contractor was placed on "standby" status to minimize the mobilization time requirement should a rupture occur. An emergency program was also undertaken to further locate and repair distressed units. Because the distress was only partially relatable to the survey indications, essentially any nondestructive inspection method that offered any hope of reliably detecting distress was attempted, most of them unsuccessfully. Of the 223 units exposed to springline, 91 (41 percent) were distressed and required repair. Of that 41 percent, 10 percent (23 units) were so severely distressed that complete replacement of the prestressing wire was required on the entire pipe unit or portions of a pipe unit. Three new and unique types of repairs were developed and implemented. The costs associated with the emergency repairs approached $2 million and required a 45-day outage. Extensive investigations, costing an additional $2 million, were also undertaken to determine the cause(s) and extent of the distress and to evaluate future serviceability. Complete remedial measures will entail totally replacing two of the siphons, partially replacing and relining a third siphon, partially relining and repairing a fourth siphon, and repairing a fifth and sixth siphon on an as-needed basis. The cost of implementing these recommendations is estimated at $117 million. To date, expenses incurred for replacements of the Salt River and New River siphons have totaled $25.7 and $9.5 million, respectively.

Prestressed concrete pipe problems have also been experienced in the public, private, and international communities. In the fall of 1990, the Denver Water Department hosted a meeting of concerned prestressed concrete pipe users. Forty-six representatives from 18 agencies/utilities across North America were in attendance. Collectively, the represented organizations reported 41 failures of prestressed concrete pipe units that were produced by 6 different manufacturers. The meeting resulted in the formulation of a Prestressed Concrete Pipe User's Group, which has continued to meet regularly (generally on a yearly or biyearly basis).

As the preceding indicates, prestressed concrete pipe problems have occurred well before the expiration of the expected design life and have lead to interruptions in service and necessitated unanticipated and costly repairs. When failure has occurred, life has been threatened, property destroyed, and litigation pursued. As a result of these problems, the Bureau of Reclamation invoked a moratorium in March 1990 negating the future use of prestressed concrete pipe on Reclamation projects. Because of similar problems, other utilities have also imposed moratoriums on the use of this type of pipe as well. The Reclamation moratorium remains in effect and will continue until the causes for the durability problems are identified and design criteria and manufacturing processes are defined which adequately address these causes. To address industry concerns from a user's perspective (i.e., to protect the investment), Reclamation and the American Water Works Association Research Foundation (AWWARF) embarked on a collaborative research program that was designed to assess, repair, and protect prestressed concrete pipe already in service. A research report is being prepared for AWWARF publication detailing findings to date. The report will likely be published in 1996. The report summarizes 16 research reports prepared by Reclamation engineers and consultants. To date, 13 of these reports are available upon request. This paper will address one critical phase of the overall program (namely the repair and rehabilitation phase) and one topic from that phase (namely the research, development, implementation, and monitoring of the new and unique repairs that were performed on the Central Arizona Project siphons). Although these repairs were performed on large-diameter pipe, they would be applicable to smaller sizes as well.

BACKGROUND

Central Arizona Project Siphon Construction

The largest circular precast structures ever built were manufactured and installed on Reclamation's Central Arizona Project from 1975 to 1980. Although the conveyance system is primarily open channel, water flows through inverted siphons at points of major cross drainage. Six of the siphons, totaling about 6.5 miles in length, consist of 21-foot-diameter prestressed concrete pipe. These siphons include Centennial Wash, Jackrabbit Wash, Hassayampa River, Agua Fria River, New River, and Salt River siphons. The siphons have been operational since 1980.

To fabricate such large pipe, strategically located manufacturing plants were positioned in the desert. The plants were disassembled upon project completion. Two types of prestressed concrete pipe were fabricated—embedded cylinder and noncylinder. Embedded cylinder pipe differed from noncylinder pipe in that it contained a 1/16-inch-thick steel cylinder embedded within the concrete core. It was also installed at locations where the pressure head exceeded 100 feet. Both types of pipe had a 19-1/2-inch-thick concrete core and a capacity of 57,000 gallons. After the core cured to strength, high carbon, high strength (ASTM A 648) steel prestressing wire (ranging in size from 0.162 to 1/4 inch in diameter) was helically wound on the surface at a mean stress approaching 75 percent of its specified minimum tensile strength. The prestressing placed the concrete core in compression such that the pipe could withstand high inservice pressure heads. Pipe was fabricated with one, two, or three layers of prestressing wire depending on the location of installation and the expected service load. During wrapping, a slurry coat was placed beneath the wire, and after wrapping, a cement mortar coating about 3/4 inch thick was shotcreted over the wire. The slurry and mortar coating were designed to protect the wire from handling damage and corrosion effects. The 225-ton, 22-foot-long pipe was maneuvered through various stages of fabrication (figure 1) and was transported and joined in the pipe trench with large-scale equipment that was specially designed by the pipe manufacturer (figure 2). The maximum head and earth cover are about 250 feet and 20 feet, respectively. Rubber gaskets were used at pipe joints. The pipe was not fabricated with bonding straps or joint bonds.

Figure 1.—To fabricate such large pipe, strategically located manufacturing plants were positioned in the desert.

Figure 2.—The pipe was joined in the pipe trench with large-scale equipment that was specially designed by the pipe manufacturer.

Central Arizona Project Siphon Condition Assessment

Several excavations to springline were performed at locations identified by corrosion surveys as well as other random sites along the six siphons. Other nondestructive diagnostic techniques such as underwater remotely operated vehicles, infrared imagery, ground-probing radar, impact-echo, impulse response, and pulse-echo acoustics also served as supplemental tools for locating additional siphon distress. None of these methods proved very beneficial at that time, but some appeared promising with further research. Reclamation is currently evaluating some of the more promising techniques.

Pipe distress in the form of degradation of the mortar coating and the prestressing wire (i.e., corrosion) ranged in intensity from minor to extreme (figure 3). In many cases, the mortar coating had disbonded to expose the underlying prestressing wire. In other cases, sounding of the pipe by hammer tapping revealed hollow or drummy areas where the mortar coating had delaminated but had not completely disbonded. Visible coating cracking was sometimes apparent. Drummy areas commonly continued to expand or grow with additional exposure. Although inspection by hammer tapping proved to be fairly successful in pin-pointing delaminated coating, mortar coating continued to disbond in certain instances, even in areas that had been previously inspected and thought to be sound.

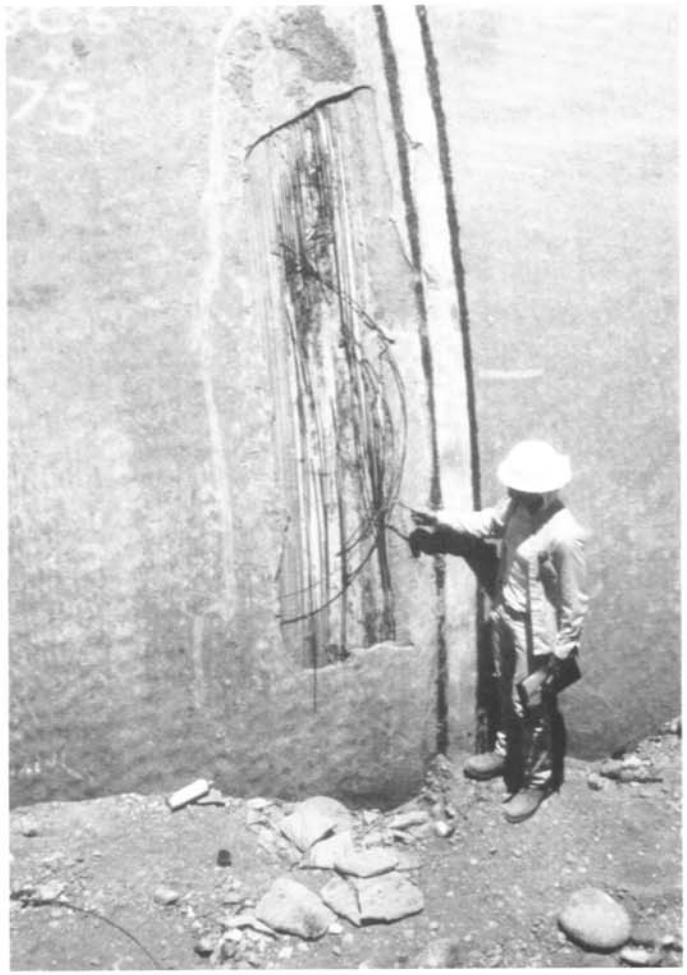


Figure 3.—The pipe distress ranged in intensity from minor to extreme.

When the pipe was inspected internally, hairline circumferential cracking of the core was observed, but longitudinal cracking corresponding to observed external distress was not apparent.

METHODS AND EXPERIMENTAL DESIGN

Central Arizona Project Siphon Repair

The repair of 21-foot-diameter pipe presented a unique problem. Because the pipe was so massive, distressed sections simply could not be replaced because new units could not be readily manufactured or transported. The pipe, therefore, had to be repaired in place. In addition, irrigation demands required that water deliveries continue until winter, when a 45-day outage period would be allowed to perform the necessary repairs. If distressed pipe units were found after the outage period, however, repairs would also have to be implemented when the siphons were flowing full. A 6-month period was therefore available to research, develop, and test new repairs which had never been attempted and to successfully implement them during and after the outage period. Both internal (a steel liner) and external methods of repair were evaluated, but efforts were primarily directed toward an external repair because of the desire to repair while the pipe was in use.

Initial Repair Research

Research was performed to develop three types of repairs that could be performed during and after the scheduled outage. The basic repair procedure for all three types was essentially the same and called for surface excavation of the pipe to a point above springline, excavation beneath the pipe invert near the repair area, restoration of prestressing, backfilling of the annulus, and replacement of the overburden. Two of the repairs used epoxy-coated ASTM A 416, grade 270, seven-wire strands (tendons) that were covered with corrosion-preventive and friction-reducing grease and encased in a seamless polyethylene sheath; a third repair used 6-gage and 1/4-inch prestressing wire.

To completely replace the prestressing of the original pipe with tendons, it was estimated that about six 0.6-inch-diameter strands were required per foot of pipe with an average centerline-to-centerline spacing of 2 inches. These calculations, however, were based on assumptions for the friction loss and relaxation of the strand around the pipe perimeter. To determine actual values and to evaluate the slippage potential of anchoring systems which bit through epoxy coating, testing was initially conducted on an instrumented strand that was wrapped around exemplar pipe simulating the dimensions and mortar surface texture of the Central Arizona Project siphons.

The strand was instrumented with three strain gages and one thermocouple every 90 degrees and wrapped around the pipe such that the overlapping ends were secured in an epoxy-coated anchorage assembly. The strands were instrumented with donut-shaped load cells that butted up against steel plates which rested against the anchorage assembly. The anchorage assembly was designed to contain two strands with a centerline-to-centerline spacing of 2 inches on the dead end and 4 inches on the stress end. The strands were locked in place by wedges contained in tapered cylindrical anchor heads. The anchor heads butted up against the anchorage assembly. Wedges were hand seated on the dead end and mechanically seated by use of a hydraulic ram on the stress end. Strands were tensioned simultaneously on the stress end of the anchorage assembly to 80 percent of their minimum specified tensile strength and anchored at about 70 percent after seating losses. The strand was subjected to several cycles of tensioning and relaxation before final anchoring. Systems from two different companies, namely Dywidag and VSL, were evaluated. The friction and seating losses, as well as elongation, were measured after anchoring. The slippage potential and strand relaxation were also determined after monitoring for about 1,000 hours. The testing indicated that a repair method using tendons was viable because friction and relaxation factors were minimal, and Reclamation proceeded with plans to implement the proven technique. The potential for anchor slippage, however, was apparent as wedges from one of the proposed proprietary anchoring systems pulled out (slipped) during laboratory tensile testing of a strand.

Type IA Repair

Finite element studies were performed to determine radial, circumferential, and longitudinal stresses which could be incurred during loading of distressed units. Concrete stresses caused by placing tendons around the pipe perimeter at prestressing levels equivalent to initial prestressing values were calculated. Modeling and analysis indicated a potential problem with the implementation of the planned repair because a bending moment or hinging effect was created on the concrete core at locations where the prestressing wire was severed. This effect developed as the severed wires re-established bond with the core away from the wire fractures and resulted in an unbalanced loading condition which tended to create a moment in the pipe wall. When this moment was combined with springline flexural moments, potential cracking of the outer concrete core at pipe springline could occur. The analysis therefore demanded that damaged wire and the overlying mortar coating be completely removed from the pipe for a predetermined circumferential distance (10 feet) above and below springline before the tendon repair could be initiated. Under these circumstances, the siphons had to be dewatered prior to implementation of the repair.

In practice, however, the removal of wire for limited distances above and below springline was difficult. Specifications for the first type of repair therefore required that the wire and mortar coating be completely removed from the pipe perimeter to expose the concrete core. This procedure became known as a type 1A repair.

To implement the repair, the area below springline was hand excavated such that the outside circumference of a pipe was exposed. To provide adequate pipe support, excavations under the invert were limited to longitudinal lengths of 6 feet or less. The depth of excavations below the pipe invert was limited to 2 feet. The mortar coating and prestressing wire were completely removed from the pipe perimeter, and tendons were wrapped around the core such that the overlapping ends anchored near the crown in epoxy-coated anchorage assemblies (figure 4). The tendons were placed so they extended at least 5 feet beyond the anchoring heads to allow for future restressing if that became necessary.

Figure 4.—During the siphon outage, the mortar coating and prestressing wire were completely removed from the pipe perimeter, tendons were wrapped around the core, and the overlapping ends anchored near the crown in epoxy-coated anchorage assemblies.

The overhang was also placed in a protective sheath. The anchorage assemblies were staggered over the entire length of the repair. To allow uniform stress transition from a repaired to a nonrepaired area, one set of tendons was also placed around intact mortar coating at the terminus of each repair zone (figure 5).

Figure 5.—To allow uniform stress transition from a repaired to a nonrepaired area, one set of tendons was placed around intact mortar coating at the terminus of each repair zone.

To enhance corrosion protection, the anchorage assemblies were filled with corrosion-resistant grease. The tapered cylindrical anchor heads and their internal wedges were also coated with protective tape. Lead wires, which extended to corrosion-monitoring stations at the soil surface, were attached to the anchorage assemblies. Continued monitoring will determine whether cathodic protection using sacrificial anodes is indicated.

After tendon installation, the excavated area below the 90-degree bedding angle was filled with soil-cement. The soil-cement was mixed at a local batch plant and hauled to the site by transit mixers. It was designed to be poured into place from one side of the pipe such that it flowed beneath the pipe and filled both haunches. The soil-cement typically contained a water to cement ratio of less than 3.5 to 1 and a cement content of 6 to 12 percent of the dry weight of the soil to provide a design strength of 50 to 100 pounds per square inch in 7 days. Other requirements included limiting the amount passing the #200 (75-micrometer) sieve and the #100 (150-micrometer) sieve to less than 30 and 50 percent by weight, respectively. In addition, particles greater than 3/8 inch and clay balls greater than 1/2 inch in diameter were not allowed. Ninety-five percent of the materials also had to pass a #4 (4.75-millimeter) screen, and clay balls could not exceed 10 percent of the total wet weight. Although the soil-cement generally flowed underneath the haunches of the pipe, sometimes it did not reach the 90-degree bedding angle on the side opposite the pour. In these instances, additional soil-cement was placed on the opposite side by bucket from a crane. After hardening, the soil-cement exhibited very little cracking or drying shrinkage (figure 6).

Shotcrete was applied over the installed tendons above the 90-degree bedding angle. Because a saturated-surface-dry condition was required, a 10,000-pound-per-square-inch water jet was used to moisten the surface prior to application. The shotcrete was mixed at a local batch plant and was delivered by truck. It was proportioned so that it contained a cement to water ratio of 1 bag of cement

Figure 6.—After hardening, the soil-cement exhibited very little cracking or drying shrinkage.

to 8 gallons of water to provide a design strength of 3,200 pounds per square inch in 3 days. The shotcrete was applied to a minimum thickness of 1 inch over the tendons and anchorage assemblies and at least 3 feet beyond the end of each repair area. The process consisted of applying shotcrete from the soil-cement interface to the ends of the tendon overhang (figure 7). Thereafter, the anchorage assemblies and tendon overhangs were also encased.

Select material (excavated between the springline and below the pipe invert) was reprocessed, regraded, redeposited (in horizontal layers), compacted to a relative density of at least 70 percent, and placed between the springline and the soil-cement. The select material contained a maximum particle size of 1-1/2 inches. Select, well-graded material containing a maximum particle size of 3 inches was also placed within 2 feet of the outside of the siphon pipe above springline. Uncompacted backfill was placed from the pipe springline to the original ground surface.

Reclamation anticipated anchor slippage problems might occur because the supplied wedges exhibited features similar to those which pulled out during laboratory testing. Representatives were therefore on hand to instrument and monitor the first tendon repair. Slippage occurred as the wedges failed to completely penetrate the epoxy coating and pulled out shortly after anchoring. New wedges were fabricated in accordance with a proven pattern specifically designed for epoxy-coated strands. They were longer and exhibited a much coarser tooth pattern than previous versions. Minimal slippage problems were experienced thereafter. Additional testing was conducted on other type 1A repairs as tendons were instrumented and the load was monitored during and after strand tensioning and during restoration of flow to the siphons. A typical load versus age curve is shown in figure 8. The testing was performed on pipe 2012 of New River siphon.

Over 550 tendons were used to successfully repair 34 areas on the Central Arizona Project siphons using the type IA repair technique. Type IA repairs were performed on 18 different pipe units, and in some cases, more than one repair was required on the same unit. The repair was limited, however, in that it could only be implemented after siphon dewatering.

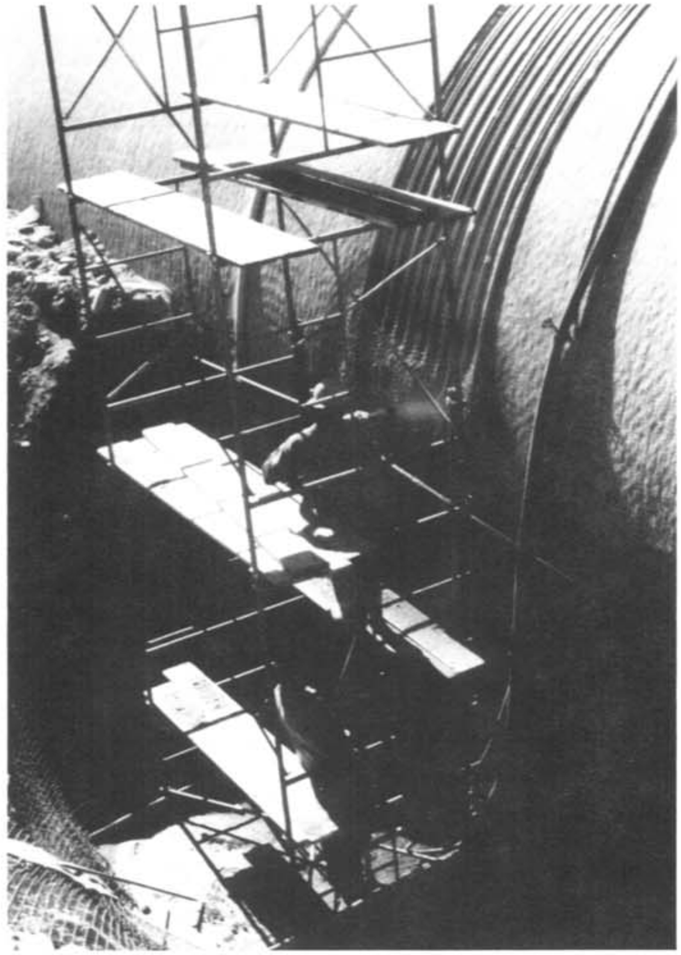


Figure 7.—Shotcrete application over the tendons.

Type II Repair

A repair which could be performed safely while the damaged pipe remained in service, allowing water deliveries to continue, was also developed. The repair differed from the previous repair in that it called for the tendons to be wrapped directly over the mortar coating and underlying prestressing wire. As previously mentioned, computer modeling had determined that such a repair was feasible provided the prestressing wire of the deteriorated pipe had not fractured near springline. The development of this repair, however, also required another consideration. Pipe in service must remain supported, and excavation is generally tolerable only to springline. Minimal excavation through a narrow annulus beneath the pipe therefore had to be performed before tendons could be installed below springline. Excavation techniques (such as vacuum extraction or water jetting) which had not been used for this application had to be fine tuned and analyzed for their effectiveness. Companies specializing in this type of excavation were contracted, their equipment modified, and their techniques evaluated.

Repairs were implemented as shotcrete was applied to areas where loose mortar coating had been removed. The shotcrete was allowed to cure for at least 3 days prior to tendon installation. The specialized technique used to excavate below springline consisted of placing a channel, which conformed to the outside diameter of the siphon, on the crown of pipe adjacent to the area being excavated. The channel was designed to contain a second inverted channel on which a chain was welded which extended the entire circumferential length. The second channel also contained protruding teeth on one end as well as water jets and a return hose. The chain sockets of the second channel were placed into the teeth of a worm gear. The worm gear was fastened to a steel frame that was stabilized by concrete thrust blocks. Excavation occurred as the worm gear thrust the channel up and down and water jetting freed the select material below springline. Vacuum extraction was used to remove the loosened debris (figure 9). Tendons were installed through the excavated annulus (figure 10), stressed, and staggered as previously outlined for the type 1A repairs. The annulus was

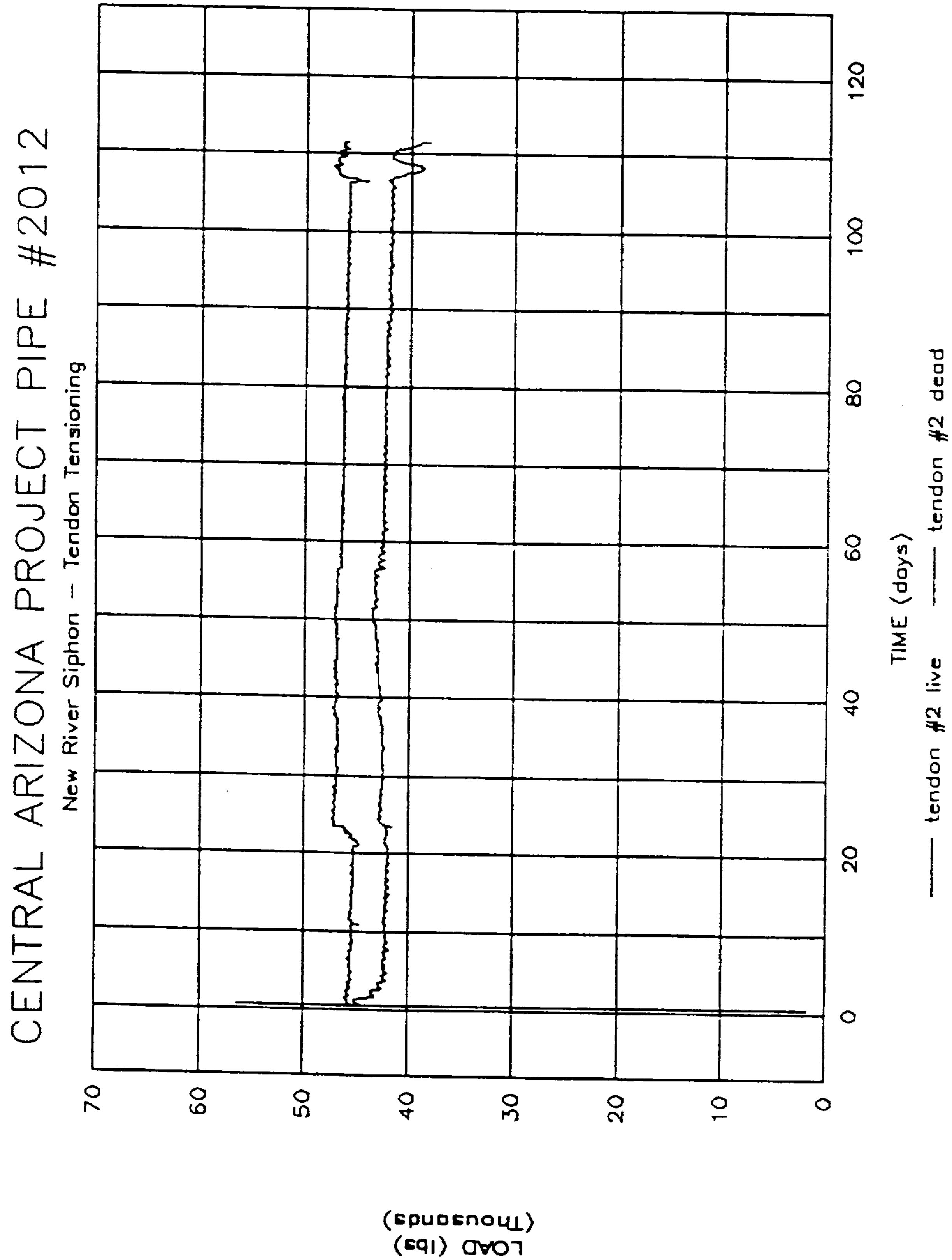


Figure 8.—A typical load versus age curve for a type 1A repair on pipe 2012 of New River siphon.

Figure 9.—The specialized excavation equipment required for the type II repair.

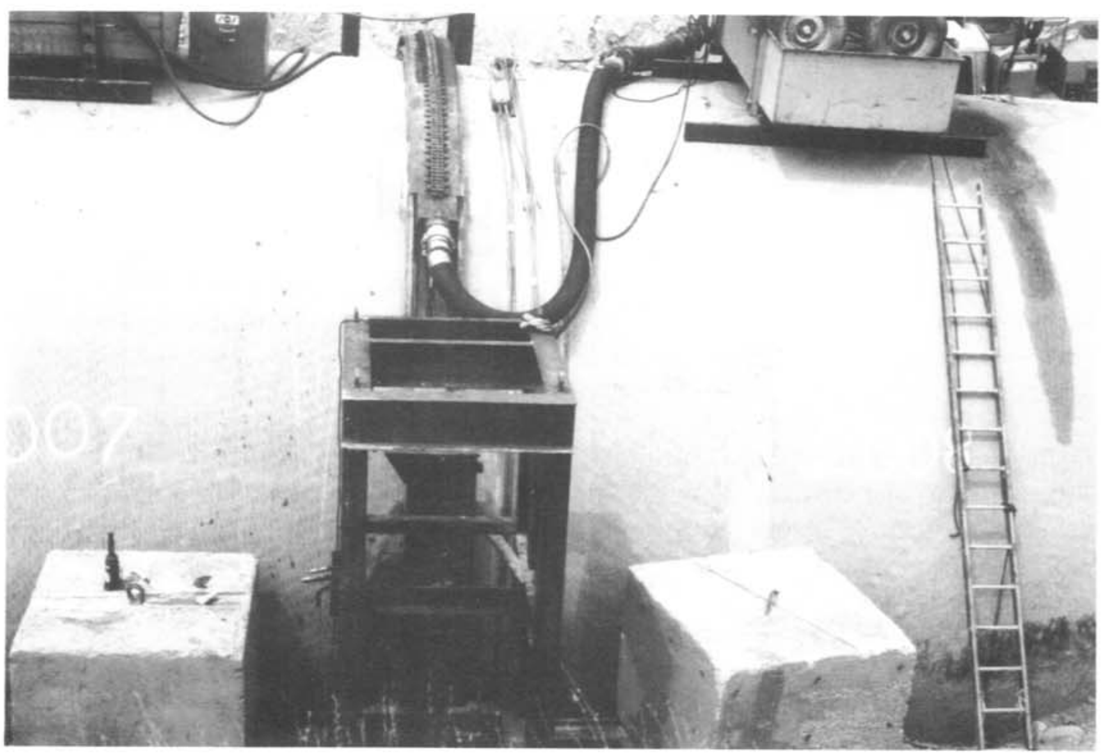


Figure 10.—When pipe was repaired in service, tendons were installed over mortar coating and corroded and broken prestressing wire through the narrow annulus beneath springline.

filled with soil-cement while shotcrete was applied over the exposed tendons above springline. The soil-cement and shotcrete mixture proportions and strength designs were the same as those used in the type 1A repairs. The pipe was also backfilled above springline as previously outlined.

Testing was again conducted on actual siphon repairs. Several tendons were instrumented, and load was monitored both during and after strand tensioning. The testing was performed on pipe 2007 of New River siphon. A typical load versus age curve is shown on figure 11. Over 125 tendons were used to successfully repair 8 areas on the Central Arizona Project siphons using this technique. Type II repairs were performed on five different pipe units, and in some cases, more than one repair was required on the same unit. Although the type II repair could be implemented while the siphons were flowing full, it was limited to those units where the wire of the deteriorated pipe had not fractured near springline.

Further research is being conducted to ascertain whether the type II tendon repair may be implemented when pipelines are flowing full and the wire has fractured near springline. Testing is being conducted in the laboratory to check the validity of assumptions used in the Reclamation finite element model. Exemplar pipe has been instrumented and the prestressing wire severed so that the magnitude of the hinging effect could be measured. In addition, tendon repairs will be performed to measure the magnitude of the longitudinal and circumferential stresses that are induced during tendon tensioning. The empirical data will be compared to theoretical projections predicted by the Reclamation finite element analysis. The evaluation will ultimately determine whether the implementation of the tendon repair should be governed by the location of the fractured prestressing wire.

Type III Repair

A third type of repair was also developed. The repair was used to rehabilitate select localized areas along the Central Arizona Project siphons when the pipe was flowing full or dewatered. Repairs of this type performed during the 45-day outage were limited by specification to damaged areas measuring 12 inches or less along the longitudinal axis of the pipeline but potentially could be used for larger areas after flow was restored to the siphons, particularly if wire fracturing was found near springline. The repair procedure called for removal of broken or corroded prestressing wire, installation of swage block anchors to the exposed wire, attachment of new wire to connect the broken strands, tensioning of wire with hydraulic rams to restore the prestressing, and application of shotcrete over the repair area. The concept for this repair was developed by Ameron. Testing for adequacy was conducted by Reclamation and Ameron.

Research was performed to study the stress distribution along the wire length relative to the distance from the stressing point. Initial estimates indicated that stresses corresponding to 60 percent of the minimum ultimate tensile strength of the wire would be transferred to 1/4-inch, class III prestressing wire if loads were applied through a specially designed hydraulic ram at pressures approaching 3,500 pounds per square inch. These estimates were based on assumptions for friction loss and stress distribution along the circumferential wire length. Two tests were performed to study the stress distribution along the wire length with respect to the distance from the stressing point. In this manner, maximum limits for the circumferential splice length were established as well as optimum hydraulic ram pressures necessary for adequate stress transfer to the spliced wire. Testing was conducted as wire was instrumented and spliced to existing wire on one of the distressed areas of the Central

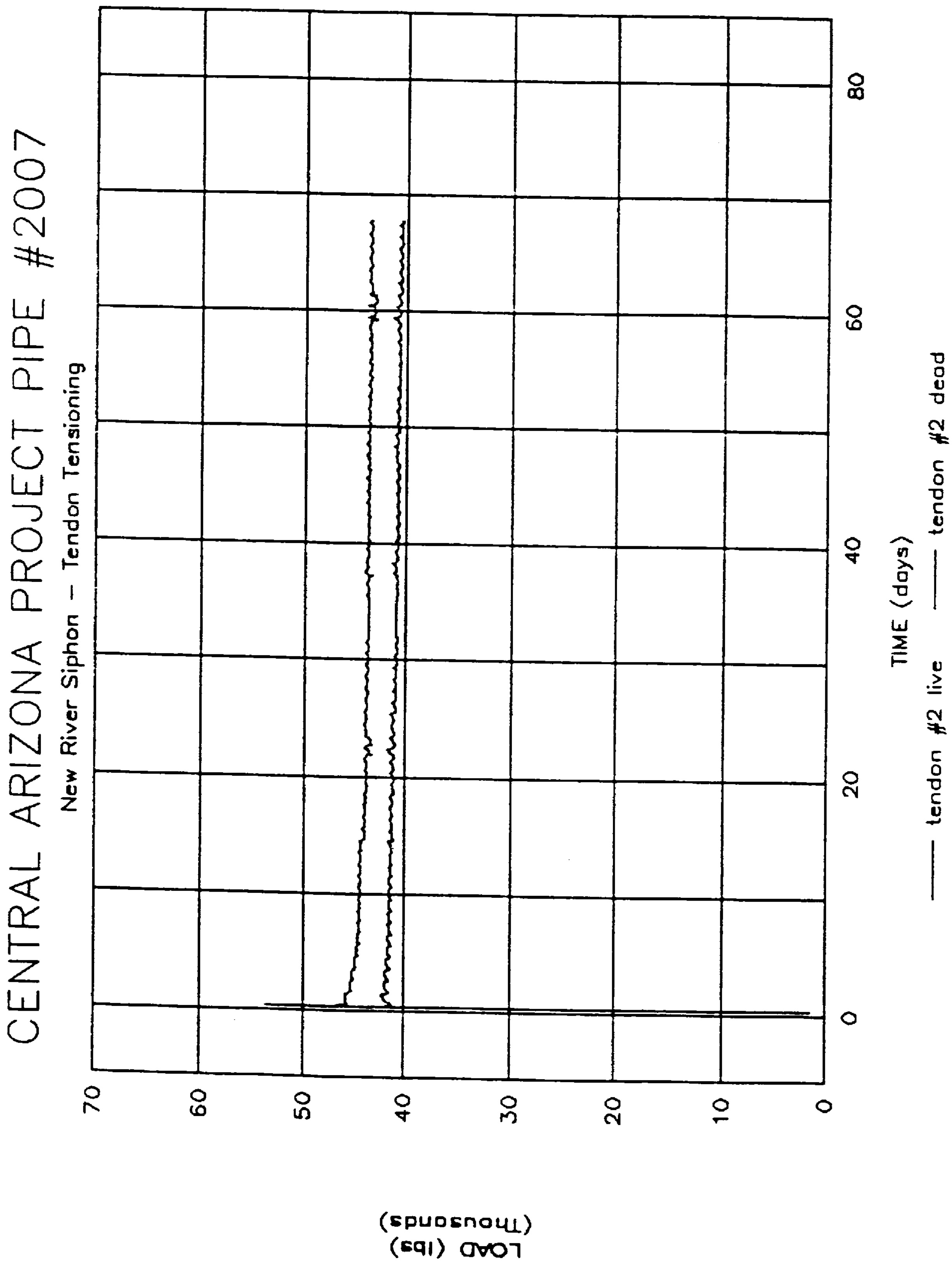


Figure 11.—A typical load versus age curve for a type II repair on pipe 2007 of New River siphon.

Arizona Project siphons. Testing was also performed as splices of various lengths were stressed around a large exemplar test section simulating the dimensions and mortar surface texture of the Central Arizona Project siphons.

Testing determined that the type III repair could be used if the circumferential length of the splice was 8 feet or less when stressing from two connecting anchor points or 4 feet or less when stressing from one connecting anchor point. To adequately transfer stresses approaching magnitudes of 70 percent of the minimum ultimate tensile strength of the wire, testing also determined that the hydraulic ram pressure must be at least 5,000 pounds per square inch when splicing with 1/4-inch, class III prestressing wire. After anchoring, losses occur which reduce the tensile stress to about 60 percent of the minimum ultimate tensile strength.

To implement the repair, damaged wire was removed until the remaining wire was relatively free from corrosion or pitting. Irregularities on the surface of the repair area were removed to provide a smooth surface for splicing the new wires. All new wire was mechanically straightened, and a splice length of at least 8 inches was required to make an adequate connection. Because anchor blocks were secured to both ends of the splicing wire and the in situ wire, about 5 inches of exposed in situ wire were required at the juncture of each splice to allow sufficient room for installation of anchors. The swage block anchors contained teeth within a semicircular groove slightly larger than a wire diameter. The teethed groove extended along the longitudinal length of the anchor block. Anchor blocks were fastened by hammering onto the prestressing wire at splice juncture points. The anchor blocks contained set screws as well as holes and threads for connecting bolts (figure 12). Prior to tensioning, a gap was set between two adjoining anchor blocks by adjusting the screw. Adjacent anchor blocks were then clamped together by bolting. After tensioning to the desired stress level, bolts were torqued to connect the adjacent anchor blocks, thereby completing the stress transfer (figures 13 and 14). The overall success of the connection was highly dependent on cutting a wire to the proper splice length.

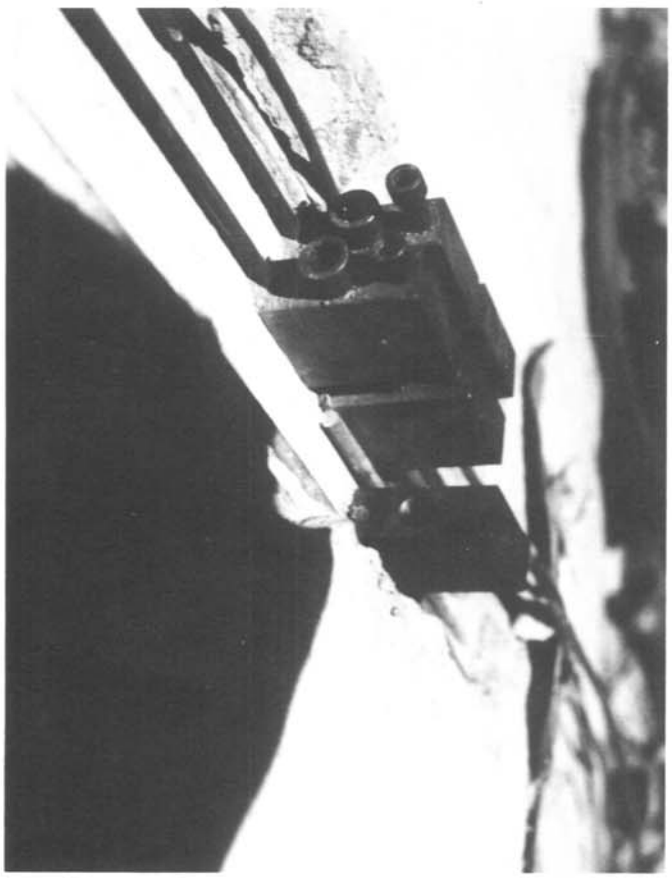


Figure 12.—Anchor blocks of a type III repair were fastened by hammering their teethed grooves onto the prestressing wire at splice juncture points. The anchor blocks also contained set screws as well as holes and threads for connecting bolts.

Figure 13.—The splice was tensioned to the desired stress level.

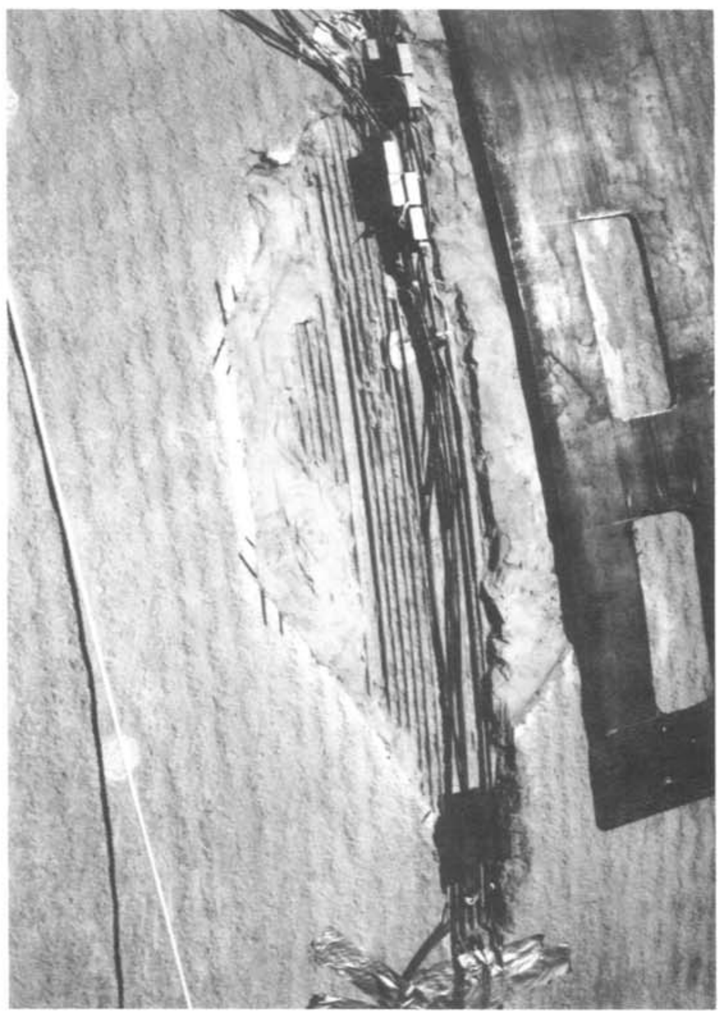


Figure 14.—A type III repair.

If a wire was too long, the gap between abutting ends of anchor blocks could be closed as the wire elongated during tensioning. If the gap closed prior to obtaining the required stress, a new splice would be necessary.

Splices were staggered so that an adequate buildup of shotcrete could be applied around the anchor blocks. A minimum shotcrete thickness of 1 inch was also placed over the wires and steel anchors. A total of 31 areas was successfully repaired on the Central Arizona Project siphons using this method. Type III repairs were performed on 21 different pipe units, and in some cases, more than one repair was required on the same unit.

SUMMARY OF RESULTS

Initial research performed on strand wrapped around large-diameter exemplar pipe substantiated that the tendon friction loss around the pipe circumference was minimal. A friction factor of 5 percent was suggested for use in design calculations. The negligible friction effect was also verified by strand elongation measurements, which consistently equaled or approached the elongation predicted by theory. Seating losses of about 6,000 pounds, minimal strand relaxation (below 3 percent), and no apparent anchor slippage were measured after about 1,000 hours of monitoring. Tests were conducted on two different proprietary systems.

Over 550 tendons were used to successfully repair 34 areas on 18 different pipe units along the Central Arizona Project during the 45-day outage. The type IA repair technique required that the corroded prestressing wire and the mortar coating be completely removed from the concrete core prior to tendon installation. The type 1A repair was therefore limited, in that repairs could only be implemented after siphon dewatering. Longitudinal repair lengths varied from 2 feet to an entire pipe section (22 feet). Instrumented tendons were monitored, during and after strand tensioning and restoration of flow to the siphons, for a period of about 100 days. Friction losses of 5 to 10 percent, as well as strand elongations between 5-15/16 and 6-15/16 inches, were measured. Seating losses between 5,000 and 7,000 pounds were also obtained, and no anchor slippage was evident.

Over 125 tendons were used to successfully repair 8 areas on 5 different pipe units along the Central Arizona Project after flow was restored to the siphons. The type II repair technique required that tendons be wrapped directly around distressed pipe units containing corroded and broken wires, and this repair could be implemented when the pipe was in use. Computer modeling of distressed units determined that repairs should be limited, however, to areas where wire had not fractured near springline. Special excavation techniques using vacuum extraction methods allowed tendons to be successfully installed through a narrow annulus below springline. Instrumented repairs were monitored during and after strand tensioning. Friction losses approaching 5 percent, as well as strand elongations between 5-15/16 and 6-15/16 inches, were measured. Seating losses between 5,000 and 7,000 pounds were obtained, and no apparent anchor slippage had occurred.

A total of 31 areas was successfully repaired using the type III technique on 21 different pipe units along the Central Arizona Project during the outage and after flow was restored to the siphons. The method used spliced wire to restore prestressing. Repairs of this type performed during the 45-day outage were limited to distressed areas measuring 12 inches or less along the longitudinal axis of the pipe but could potentially be used for larger areas after flow was restored to the siphons, particularly if wire fracturing was found near springline. Reclamation testing determined that the effective circumferential length of the repair should be 8 feet or less when stressing from two connecting anchor points or 4 feet or less when stressing from one connecting anchor point. Testing also determined the hydraulic ram should be pressurized to 5,000 pounds per square inch when splicing with 1/4-inch, class III prestressing wire. At this pressure, tensile stresses approaching 70 percent of the minimum ultimate tensile strength of the wire were transferred to the splice. After anchoring, stress losses occurred that reduced the stress to about 60 percent of the minimum ultimate tensile strength.

Research is currently under way to determine whether future tendon repairs may be implemented in service under any circumstance, regardless of the wire fracturing location.

ACKNOWLEDGEMENTS

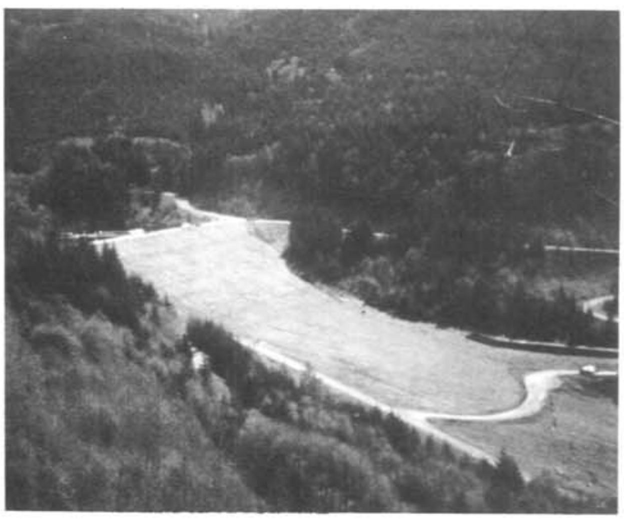
The majority of methods described herein represent the efforts of teams of individuals, both retired and employed, from the Technical Service Center and the Phoenix Area Office within Reclamation. The technical contributions of Fred Travers, Lowell Pimley, John Thurston, Fred Tan, Jim Keith, Christi Young, Jim Hartwell, Pete Klein, Thom Fisher, and Rick Pepin from the Denver Office and Randy Randolph, Tom Bortak, Will Worthington, Marty Sullivan, Brian Yoshida, and Tom Stendar from the Phoenix Office are gratefully acknowledged. Special acknowledgement is also due to Harry Uyeda, who in addition to serving as a technical specialist on the aforementioned design team, also served as co-principal investigator of the Reclamation/AWWARF collaborative research prior to his retirement in 1994.

TOLT DAM ADVANCE WARNING SYSTEM

by Barry Myers, Woodward-Clyde Consultants, Seattle, Washington;
Jill Marilley, Seattle Water Department, Seattle, Washington;
and Jack O'Rourke, Woodward-Clyde Consultants, Oakland, California

As a key component in the Seattle Water Department (SWD), Tolt Dam provides more than 35 percent of the drinking water to more than 1 million customers in the greater Seattle area. The dam is located on the South Fork Tolt River, northeast of Seattle, about 26 kilometers (16 miles) upstream from the rural community of Carnation, Washington. The earth embankment dam is about 60 meters (197 feet) high and was constructed in the early 1960's. The dam is considered to be in a remote location and is normally unattended. The closest SWD personnel are located in the water treatment plant 8 kilometers (5 miles) from the dam.

An advanced warning system was installed in 1979 in response to the public's general safety concerns and perceptions associated with living downstream from such a large dam. The system installed was comprised of the most reliable basic monitoring and emergency warning equipment available at the time. The system components included:



Tolt Dam.

- An electronic water level sensor about 1.6 kilometers (1 mile) below the dam
- A radio transmission link from the gauging station to the water treatment plant 8 kilometers (5 miles) farther downstream
- Seven public warning sirens (four in Carnation and three along the river between Carnation and the dam) connected by radio transmission links to the water treatment plant

In addition to the warning system, SWD's formal emergency and warning plan was updated to include several more local officials.

In the unlikely event of a catastrophic failure, dam failure flood inundation arrival time and peak elevation time at Carnation are predicted at 0.8 hour and 1.3 hours, respectively, with predicted flood inundation depths up to about 4 meters (13 feet). Residents along the river closer to the dam are subject to shorter flood arrival times.

The original system operated well for a number of years. However, in 1985, the system began triggering a series of false alarms. During this period, the electronic level sensors at the river gauging station malfunctioned and transmitted false emergency signals to the plant operators. In addition, the sirens themselves had malfunctioned and sounded randomly at various hours of the night and day. Extensive repairs were necessary to return the system to an acceptable level of performance and reliability. The public's faith in the system had been seriously undermined.

Therefore, in 1988, SWD established more stringent goals and objectives for the Tolt Advanced Warning System, including the desire for a system that provided a more interactive and reliable environment for both the operators and those who were monitoring and maintaining the dam. To achieve these goals, SWD retained Woodward-Clyde Consultants (WCC) to design the new system.

SYSTEM DESIGN

The design criteria adopted for upgrading the failure warning system included the following:

- Furnish and install an Automatic Data Acquisition System (ADAS) to regularly monitor strategically located instrumentation points and to provide early warning of potential failure conditions
- Retrofit and/or supplement existing instrumentation points with additional instrumentation points to increase reliability and effectiveness of the failure warning system
- Develop new alarm logic
- Upgrade failure warning sirens
- Install a closed-circuit television (CCTV) surveillance system at the dam

Typical seepage gauge installation.

The previously existing monitoring capabilities at Tolt Dam covered most of the complete range recommended by the International Commission on Large Dams (1982) for a dam of this height and setting. All observation wells, piezometers, and seepage monitors appeared to be providing acceptable data, adequate to demonstrate the safety of the dam in several analyses performed in 1978, 1981, and 1989. Therefore, the new system did not need to establish a large number of new instrumentation and monitoring points.

Typical piezometer installation, including solar panels and measurement control unit.

The existing instrumentation selected for incorporation into an ADAS-controlled warning system included nine piezometers and three seepage gauges. These instruments can provide early warning of deteriorating safety conditions. In addition, several new instrument installations were designed to supplement the capability of the existing downstream river gauging station in providing direct monitoring of a dam failure event. The installations included a dam crest failure monitor, a pool level gauge, and another river gauging station closer to the dam.

Seepage performance was recognized as probably one of the most sensitive early warning indicators of any change in stability of the dam. Electric piezometers were installed in existing, open observation wells located in a section along the centerline of the dam and in an upstream-downstream section through the dam. In addition, seepage flow gauges were installed at three strategic locations. The design of the automated piezometer and seepage gauge installations included the use of electric, vibrating wire sensors. This type of sensor historically has shown great success, particularly in geotechnical projects where it is subjected to environmental extremes.

The instruments were connected to local Measurement Control Units (MCU's), which are microprocessor-controlled data acquisition units. The MCU houses signal conditioning equipment to operate up to six vibrating wire instruments. The MCU operates off a solar-charged battery power pack (or a-c power, available at two locations) and has radio transmission equipment to communicate with other MCU's, or to send data and alarms, via the "Gateway" MCU No. 1, back to the Network Monitoring Station (NMS), located about 8 kilometers (5 miles) away in the water treatment plant. A total of eight MCU's were installed to serve the widely spread array of ADAS instrumentation at the dam and along the river. See figure 15 for a plan of the ADAS instrumentation layout.

The NMS is the operator-system instrument interface and consists of an IBM-compatible personal computer equipped with the system software and an alarm signal console. The menu-driven program supplied with the ADAS allows easy modification of instrument numbers, types, calibrations,

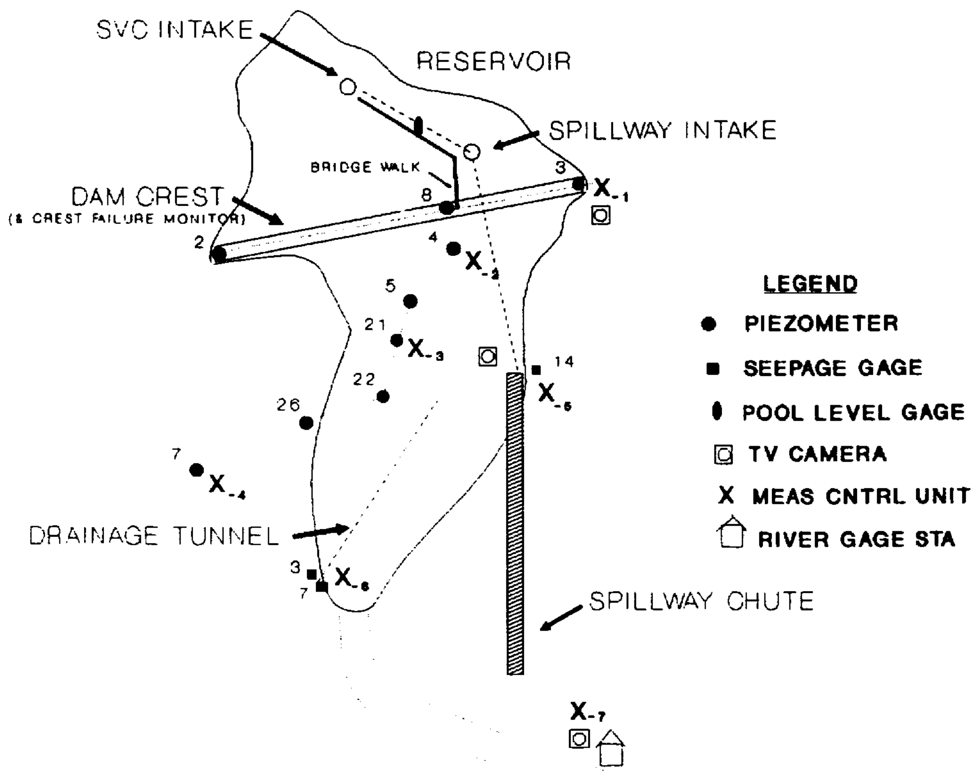


Figure 15.—Plan of instrumentation layout.

monitoring frequency, and other operational parameters in a changing system. In addition, personnel can access this system by modem from offices in Seattle for regular or emergency data analysis.

As mentioned, in addition to retrofitting the existing instrument locations, three new instrument installations were installed at or near the dam. The new instrument installations at the dam included:

- A dam crest failure monitoring instrument, installed in a shallow trench across the dam crest.
- An electric pressure transducer, installed in a deep standpipe within the reservoir pool, to monitor pool level.
- A new river gauging station, installed just downstream from the dam. This station would provide early, high-river-stage data as well as a failure backup to the existing river gauging station located further downstream.

The alarm criteria developed for the Tolt system attempts to balance the need to provide early warning to the public with the need to detect the possibility of instrument failure and spurious alarm conditions. The alarm signal console, located with the NMS in the water treatment plant, is shown in figure 16. Low and mid-level alarms 1, 2, 4, and 5 indicate developing problems or hazardous situations. Alarms 1 and 2 identify situations dealing with the river downstream from the dam, and alarms 4 and 5 alert operators to situations with the instruments on the dam face. When these lights illuminate, a quick reference to the NMS computer will indicate the exact problem. If the upstream or downstream river gauge exceeds its established maximum threshold limits, the crest monitor instrument shows a failure condition, or the pool level shows a greater than allowable pool drop, then

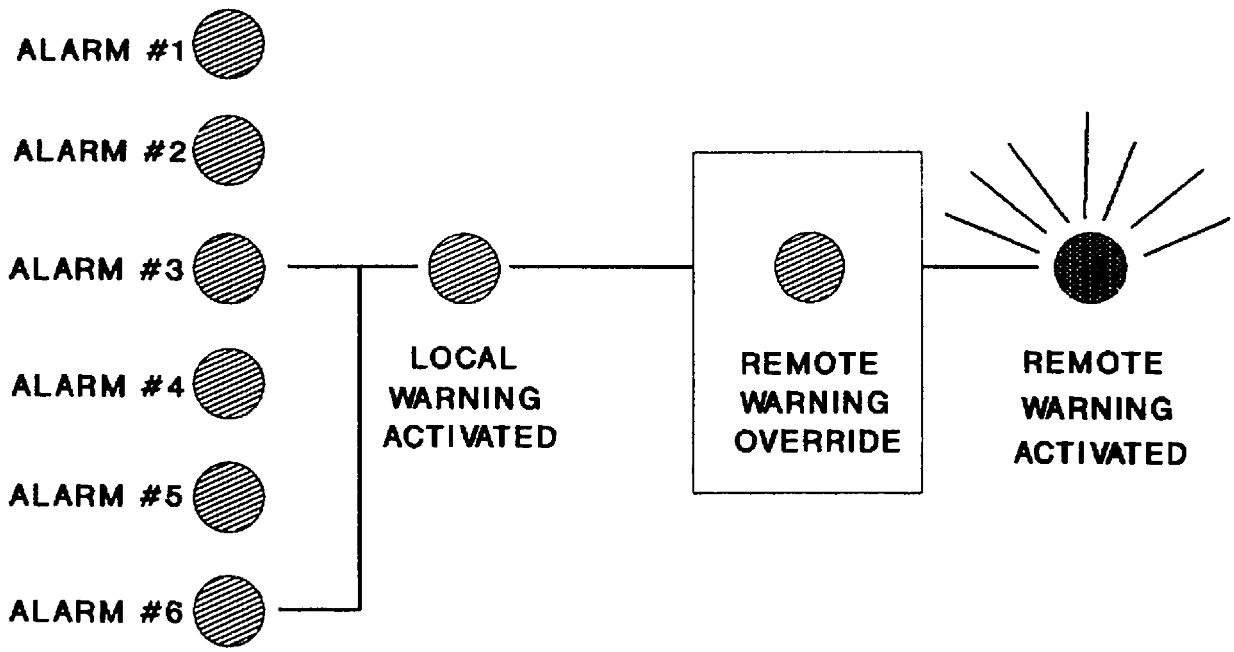


Figure 16.—Alarm console diagram.

high-level alarms 3 or 6 are triggered. Local warning sirens are immediately sounded at various locations around the nearby water treatment plant and a time-delay switch starts closing, leading to activation of the public warning sirens.

During the time-delay period of a failure warning event, the ADAS continually checks for a dam failure condition through programmed, established failure criteria. If these criteria are met, the ADAS activates the sirens in Carnation, and the town follows evacuation procedures. Also during this time, the operator, alerted by local sirens, comes to the NMS to check the television cameras and computer to identify the actual problem. The operator can then either override the system and not allow the computer to activate the sirens in Carnation or sound the sirens sooner.

The upgrade of the sirens included the ability to send and receive radio transmissions to acknowledge startup and diagnostic checking. The community also requested the sirens have a pre-recorded voice announcing the reason for the alarms sounding, (i.e., testing, alert to failure, community emergency, or an all-clear message). The new sirens are the nonrotating electronic type and have proven to be very effective and reliable.

Finally, a closed circuit television surveillance system with three cameras at key points was installed at the dam to allow visual checking of the dam and river conditions by the water treatment operator (see figure 15). Although various safeguards are built into the automated warning system, television inspection provides a final opportunity to observe conditions at the dam. The CCTV has some limitations during extreme weather conditions but is considered a strong complement to the ADAS system.

OPERATIONAL PERFORMANCE

The installation of the new advanced warning system was completed in July 1993 and has been operating satisfactorily for the past 12 months. Although the system has performed as designed, some operational problems have occurred that required design modifications and replacement of some of the units. The authors feel that it is important to present our experiences so that they can be used by designers of similar systems in the future. Specifically, the solar power units have not provided adequate power to the MCU's during Seattle's rainy months of late winter and spring. In addition, two vibrating wire pressure transducers and one vibrating wire float gauge have had to be replaced.

MCU Power Supply Upgrades

The MCU's can be powered by either a 120-volt a-c or 12-volt d-c supply. For the Tolt ADAS, the 120-volt a-c supply for the television cameras was used to power three nearby MCU's. The additional five MCU's were all configured with a standard Geomation, Inc., battery power system that delivers 12 volts d-c power and is capable of stand-alone operation with solar recharge. The battery power system consisted of a 10-ampere-hour battery recharged by an 18-watt solar panel. The power systems were provided with the five MCU's, and the solar panels were sized for the site conditions by Geomation, Inc.

All of the units that are supplied by 120-volt a-c power have performed well, but the battery power systems for the other five MCU's have all performed poorly during the rainy months of late winter and spring. In all cases, the 10-ampere-hour batteries did not have the capacity to provide the power consumed by the MCU's during the periods of limited sunlight. The MCU's would shut down operation when the battery voltage dropped below the minimum operating level until the sun returned to recharge the battery. In installations where more than one instrument was being monitored, or the frequency of monitoring was greater than once per hour, the process of the solar recharge was also inadequate. For these installations, the 18-watt solar panels could not replace the power consumed by the MCU for the previous period of cloudy days. The solar recharge during those limited sunny days could not keep up with the drain on the battery, and the system could not overcome the power deficit without replacing the battery. Replacement of the batteries at regular intervals was not an acceptable solution.

The temporary solution was to place the units into power miser or "sleep mode" during the period of time between the hourly readings. The power miser mode uses software commands within the MCU programming to "turn off" the MCU power switch for a preprogrammed amount of time and can save a significant amount of power consumption. Therefore, all of the units except the downstream gauging station were programmed to "sleep" for 50 minutes of every hour and to "wake up" and evaluate the instruments once an hour for 10 minutes. Because the downstream gauging station was required to evaluate the river stage every 5 minutes, it could not be put to sleep. The interim fix for the gauging station was to use large lead-acid batteries to power the MCU.

These temporary solutions worked adequately for all of the five units except for one. Even using the power miser mode, this unit could not maintain continued operation. The only difference between this unit and others was that it monitored four vibrating wire instruments instead of two. Apparently, the power consumed from monitoring these additional two instruments was enough to continue to draw down the battery between the sunny day events. The downstream gauging station performed adequately; however, exchanging the lead-acid batteries involved a 366-meter (400-yard) hike down and up a 91-meter (300-foot-) deep canyon every 2 weeks, carrying an 18-kilogram (40-pound) battery. This procedure was not acceptable as a permanent solution. Although the power miser mode works well, the MCU cannot evaluate the instruments that it is monitoring until it wakes up. During an emergency situation, the MCU would not be able to evaluate the instruments until the subsequent 10-minute "wake up" cycle. It was decided that communications should be maintained at all times with all nodes in the system and, therefore, the power miser mode could not be used as a permanent solution.

The permanent solution for all five units required upgrading the solar-recharged battery power supplies. The downstream gauging station was upgraded with a 110-ampere-hour gel-cell battery and two 50-watt solar panels. The upgrading for the remaining units consisted of a 60-ampere-hour gel-cell battery and a 50-watt solar panel for one MCU, a 40-ampere-hour battery and two 18-watt solar panels for a second MCU, and a 30-ampere-hour battery and two 18-watt solar panels for two more MCU's.

Vibrating Wire Instrument Performance

Thirteen vibrating wire pressure transducers and three vibrating wire float gauges were installed for the Tolt ADAS. Of these instruments, two pressure transducers and one float gauge had to be replaced because they were not performing adequately. Detailed reasons for the failure of these three instruments were not investigated, and the purpose of this discussion is to describe how the system as a whole performed. The philosophy of the owner and designer is that if the installation remains accessible when an instrument malfunctions and begins to report misleading information, it should be returned to the manufacturer for a diagnostics check, troubleshooting, and repairs or replacement. For this project, the two manufacturers of the vibrating wire instruments provided replacements for the malfunctioning units, and all have performed well since.

ACKNOWLEDGEMENTS

WCC performed the overall upgrading concept, the geotechnical instrumentation, and ADAS system design and installation. The new public warning sirens were procured and installed by SWD forces according to specifications developed by WCC's subconsultant, Leedshill Herkenhoff, Albuquerque, New Mexico, who also designed and prepared the plans and specifications for the CCTV system. The CCTV system was advertised, bid, furnished, and installed by Cochran, Inc., with resident engineer services and inspection provided by SWD.

SWD engineering and support staff are currently being trained in use of the system, and SWD engineers can communicate with the NMS and obtain complete data and system status by means of a telephone modem from their offices in Seattle. The Seattle Water Department now feels much more confident in its ability to protect the citizens living immediately downstream from Tolt Dam, and the city of Carnation is much more trusting of the system's ability to give advance warning.

GATE AUTOMATION UPGRADE

A Solar-Powered Gate Operator

by David G. Ehler[1]

The Water Resources Research Laboratory (formerly known as the Hydraulics Branch, Division of Research) in the Denver Technical Service Center is developing low-cost automation products for canal automation. A simple, low-cost gate control system for remote sites has recently been developed and field tested in conjunction with the Provo Project Office. In addition to providing conventional telemetry (remote monitoring) of water levels, flows, and gate positions, gates even at the most remote site can now be monitored and controlled. Automatic gate control is also possible at these sites at a relatively low cost.

To paraphrase Russel Anderson at the Richfield site: It is really a great help to be able to control the canal gates from my home. I want all our canal gates set up this way (figure 17).

Figure 17.—Automated solar-powered gates at Richfield, Utah.

This system provides telemetry and gate operation by using a gate operator with the following operating criteria:

- Low power
- Solar/battery operation
- Use with rising stem gates or rising stem valves
- Low cost

COMMERCIAL GATE OPERATORS

A number of commercial gate operators are available. However, these operators are expensive and often require 3-phase power or additional expense to modify for single phase or d-c operation. These systems are too power hungry for battery and/or solar operation.

[1] David G. Ehler is an Electronics Engineer in the Water Resources Laboratory, Bureau of Reclamation, Denver, Colorado.

LOW-COST GATE OPERATORS

To overcome these problems, a d-c gate operator was designed by Technical Service Center personnel, with help from the Provo Project Office. Design requirements included low-cost, low-power, 12- or 24-volt operation using deep cycle marine type batteries, solar panel battery charging, and suitability for use with rising stem gates and valves.

The system was designed around a fractional horsepower, 24-volt, d-c gear motor and an adapter plate used to fasten a gear to the gate hand wheel. A number of gear motors are available in the 12- and 24-volt range rated at 1/15 and 1/8 horsepower. Typical are the Dayton d-c gear motors available from Grainger and other suppliers. These motors match up well with one or two 12-volt deep cycle batteries and one or two 40-watt solar panels with chargers. The 1/8-horsepower motors are used for larger gates and higher head submerged gates. The 24-volt systems are used for gates with high duty cycles. Figure 18 shows a typical fractional horsepower gear motor. The gate position can be derived from a fabricated string transducer attached to the gate stem. Open and closed limit switches are activated by the gate stem; the limit switches are installed to stop the gates at the maximum and minimum positions. These switches can be adjusted to set operational limits such as minimum flow and gate normal maximum.

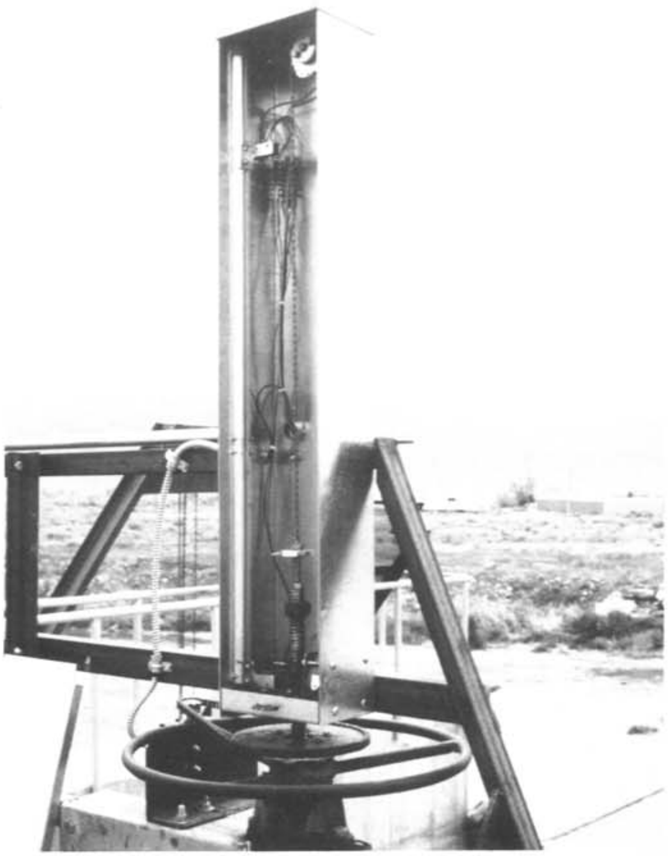


Figure 18.—Drive installation with position and limit switches.

DESIGN OF THE GATE DRIVE

Low horsepower d-c motors can be used for these applications because most gate movements are small and do not require full cycle or fast operation. The motors are geared to move slowly, allowing the use of fractional horsepower motors. An occasional full cycle operation for canal shutdown or an emergency is possible.

A spacer plate, as shown in figure 19, is used to attach a gear to the gate handwheel. The spacer plate also aligns and centers the gear on the handwheel. Additionally, the adapter provides spacing between the handwheel and gear for clearance between the chain and handwheel. Figure 20 is a typical rising stem slide gate installation. The gear motor is attached to the gate frame with a bracket that allows adjustment of the chain. Idler sprockets should not be used.

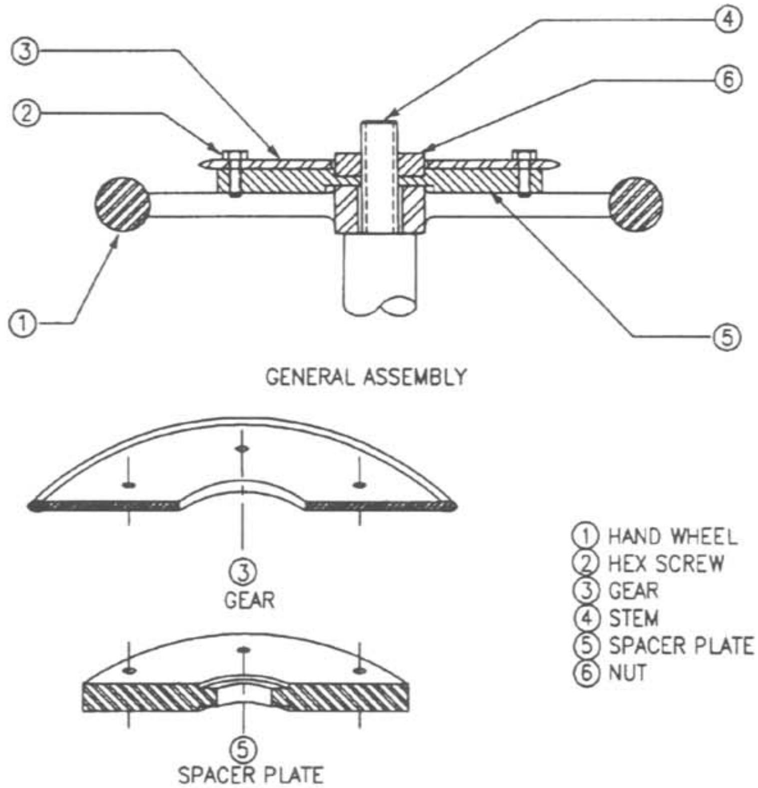


Figure 19.—Gate handwheel adapter assembly.

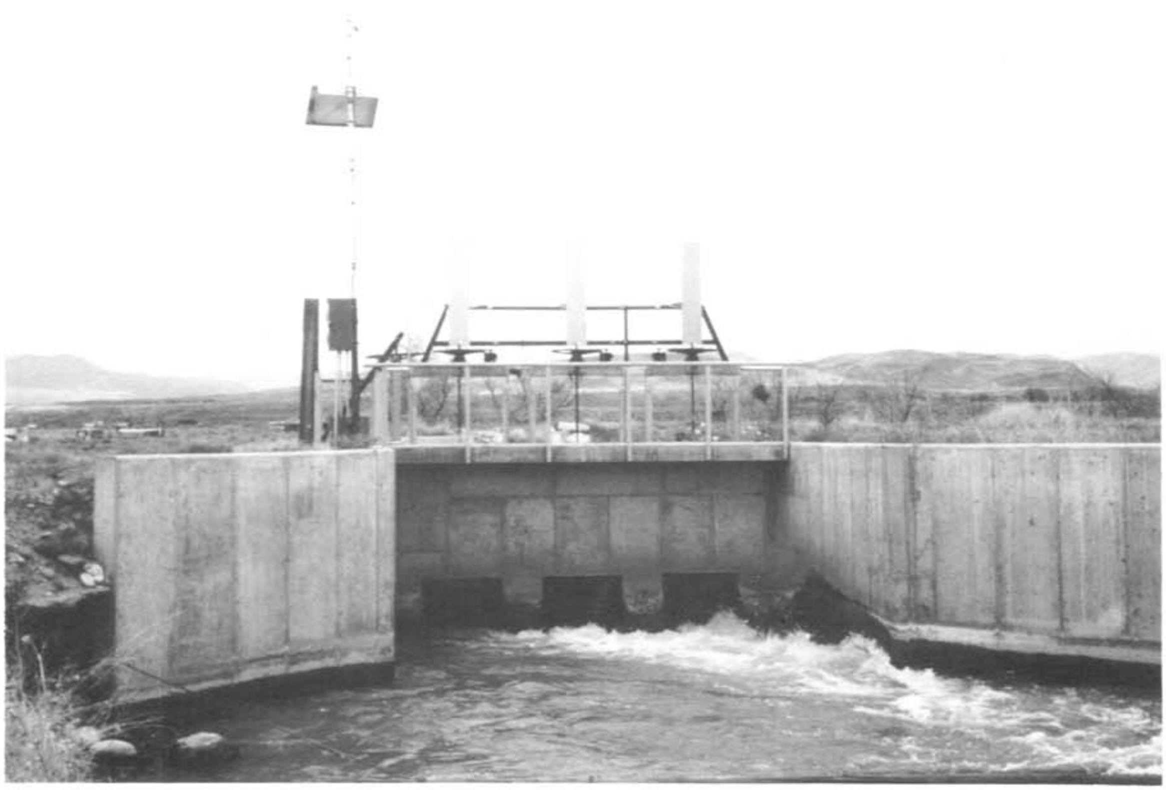


Figure 20.—Typical installation.

A reversing motor controller is made with two double-pole, double-throw relays. The relays have 12- or 24-volt coils to match the gate hoist motors and 10- to 15-amp contacts to match the motor ratings. The relays are wired to provide an interlock to stop the motor should both relays activate at the same time. A fuse is installed in the motor circuit as a secondary safety device. The fuse is sized to blow should the motor stall or draw heavy current. These problems could occur if the gate is jammed with debris, the hoist is jammed, or a limit switch fails. Figure 21 is a schematic of the gate controller. The controller can be driven by outputs from controllers such as the Campbell Scientific CR10 or the Geomation 2300 series.

EQUIPMENT LIST AND COST ESTIMATE

Equipment and parts for a rising stem gate modification:

Hoist		
Gear motor, 1/15 horsepower, 20 rotations per minute, 12- or 24-volt d-c		$250
Large sprocket, flat, 72 teeth for handwheel		30
Small sprocket, bushed bore for motor, 10 teeth		7
Chain		25
Fabricated handwheel adapter plate		
Fabricated motor bracket		
Fabricated chain/sprocket guard		
	Subtotal	$312
Control		
Relays, DPDT 12- or 24-volt coil, 2 each at $25		$50
Inline fuse holder and fuse (with spares)		5
Limit switches, 2 each at $5		10
Position device (string transducer)		
Fabricated limit switch/position device housing		
	Subtotal	$65
Power (for a 12-volt system, double for 24-volt system)		
Solar panel(s) for 12- or 24-volt system		$250
Regulator/charger		45
Battery(s) for 12- or 24-volt operation		65
	Subtotal	$360
Total (not including fabricated parts and labor)		$737

ACKNOWLEDGEMENTS

Arlen Hilton and Frank Woodard from the Provo Area Office installed the gate operator, solar equipment, controls, and communications equipment at various sites in Utah. Frank Woodard assisted in building the prototype operator in Denver. For more information about Reclamation's automation projects in Utah, contact Roger Hansen at the Provo Area Office (801) 379-1170.

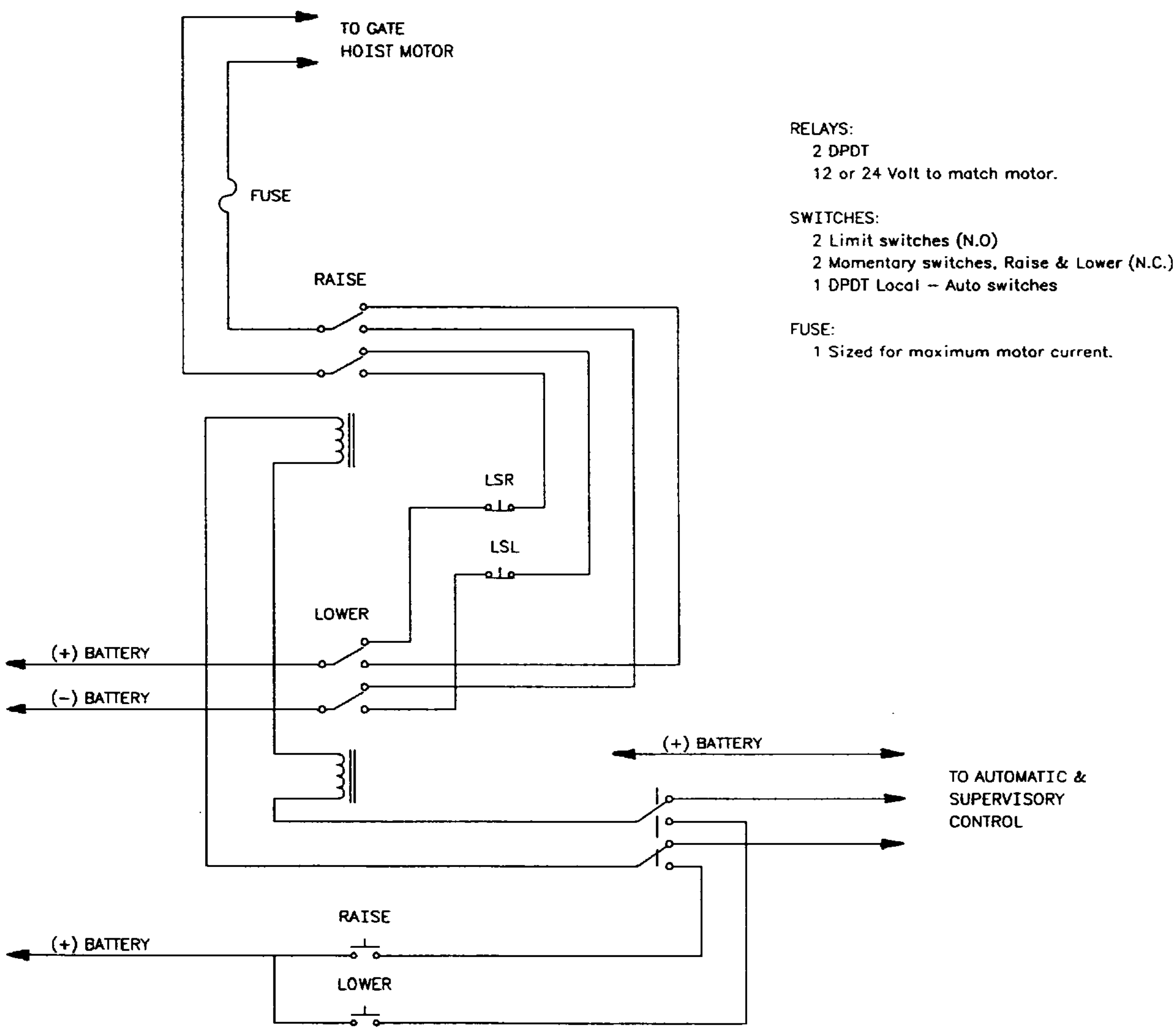


Figure 21.—Motor controller schematic.

PANEL WALL HEATERS[1]

by F. Donald Haynes, U.S. Army Cold Regions Research and Engineering Laboratory

The use of panel water heaters to control and minimize icing in miter gate recess areas is an effective solution to the icing problem at Starved Rock Lock and Dam on the Illinois River. A panel heater was installed at this location in December 1993, just in time for one of the worst winters of the decade. The panel was bolted onto the concrete wall in the miter gate recess area (figure 22) and was very successful in keeping the wall ice free (figure 23).

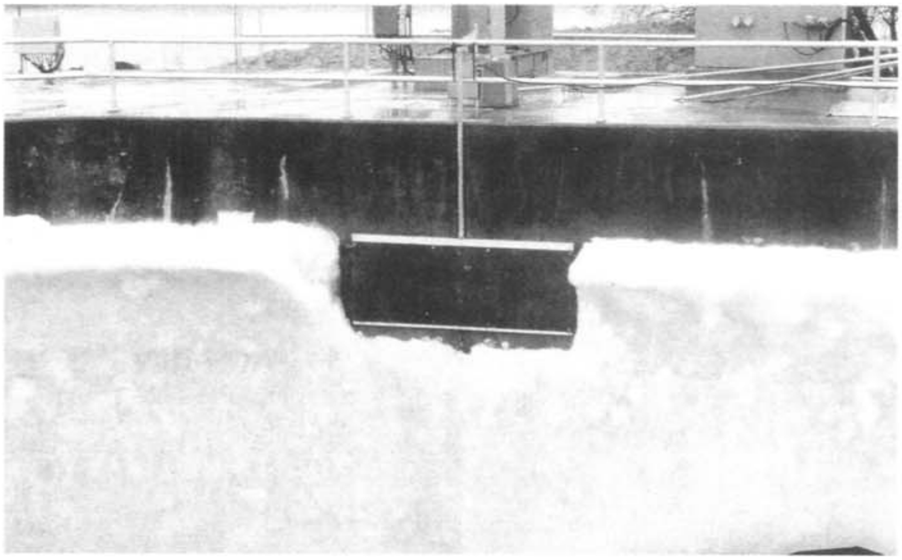


Figure 22.—Closeup of the panel.

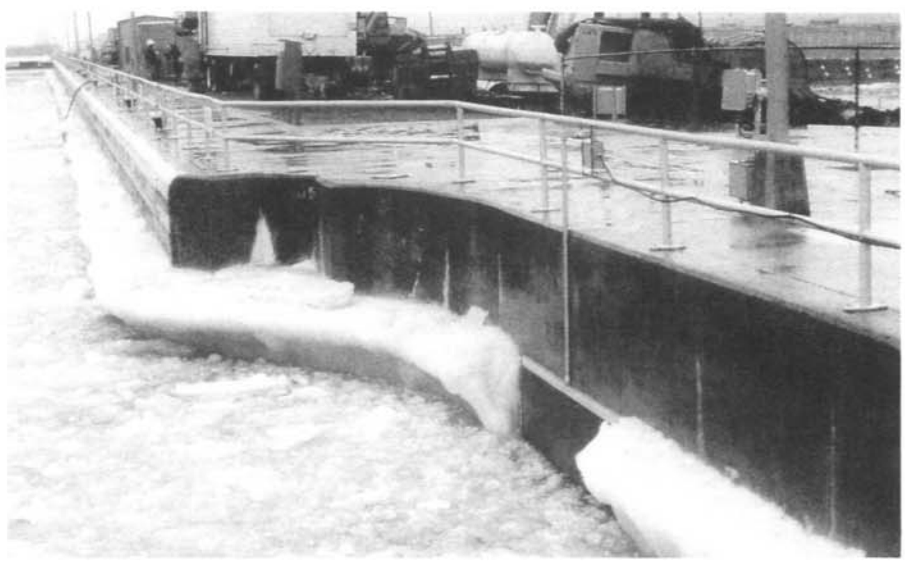


Figure 23.—Longer perspective shot of the panel from the side.

[1] Reprinted with permission from author, F. Donald Haynes, P.E., U.S. Army Cold Regions Research and Engineering Laboratory, Corps of Engineers, Hanover, New Hampshire 03755-1290.

Some heating effect occurred even beyond the perimeter of the panel, which contains 48 feet of self-regulating heat cable. The heat cable is rated at 40 watts per foot at 240 volts, for a total power of 1,920 watts. One advantage of the self-regulating heat cable is that its power requirement decreases as it heats up, eliminating the need for a thermostat. These panels have been recommended for placement on all miter gate recesses at Dresden Island and Marseille Lock and Dams during scheduled 1995 rehabilitations.

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