LOCKING INTO SUCCESS

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Taking lessons from a 30-year-old failure, designers used hundreds of prestressed rock anchors and help from an extensive instrumentation system to incorporate an existing navigation lock into the cofferdam for a new lock and a 50% cost savings. The navigation lock on the Monongahela River in southwest Pennsylvania began operating last December.

In 1961, at Wheeler Lock in Alabama, the Tennessee Valley Authority tried to incorporate an existing lock wall into a new cofferdam. A major portion of the land wall slid about 30 ft into the dewatered excavation being used to construct the adjoining lock, killing several people. Reportedly, sliding occurred on an undetected weak clay seam in the foundation rock. No stabilization measures or instrumentation systems had been implemented.

Now 30 years later, designers with the U.S. Army Corps of Engineers' Pittsburgh District have installed a similar project at Point Marion Lock in Pennsylvania to replace the 68-year-old navigation lock on the Monongahela River in southwest Pennsylvania. This time, designers called for 471 high-capacity prestressed rock anchors—one of the largest single uses of prestressing strand in North America—to avoid repeating the Wheeler Lock failure. The project cost $88 million. Had a new site been selected that required the construction of both a new lock and dam, the project would have cost approximately 50% more. Designers created an extensive $1 million structural and geotechnical instrumentation program for this project. Engineers monitored the instrumentation data in real time through on- and off-site computers to help eliminate potential problems at the earliest stage and keep construction functioning.

The age, advanced concrete deterioration and marginal structural stability of the existing 56 ft by 360 ft lock chamber, coupled with heavy river traffic, required construction of a larger 84 ft by 720 ft replacement lock to ensure safe, dependable and more efficient navigation. Several factors restricted the location of this reconstruction: the need to retain the existing rehabilitated dam, reduce the volume of excavation required for approach cuts, and avoid costly impacts to adjacent road and rail routes.

Consequently, the new lock was positioned immediately landward of the existing lock (Fig. 1). Excavation would occur within 8 ft from the landward edge of the existing lock and extend to a depth of about 13 ft below the rock foundation of the existing land wall. The project was designed to allow construction on the replacement lock while existing navigation lock operations continued. Upon completion of the new lock, crews would remove the existing one and construct a fixed weir section to tie the new lock into the existing dam.

IN 1961, WORKERS TRIED TO INCORPORATE AN EXISTING LOCK WALL INTO A NEW COFFERDAM. A MAJOR PORTION OF THE LAND WALL SLID INTO DEWATERED EXCAVATION KILLING SEVERAL PEOPLE.

The valley of the Monongahela River is entrenched in flat-lying sedimentary rocks of the Kanawha section of the Appalachian Plateau. Local bedrock includes a variable series of clay stones/indurated clays, argillaceous limestones, siltstones, sandstones and coal seams from early Carboniferous to Permian time. At the site, there is a sequence of the Middle Pennsylvanian time Glenshaw Formation rocks (Fig. 2). Ground-water levels appear to be influenced by regional ground-water flow, draining off the adjacent hillside, and local effects of upper and lower navigation pools. A comprehensive preconstruction site investigation was conducted that featured more than 150 core holes, laboratory testing, field ground-water testing and the placement of piezometers.

ANCHOR DESIGN AND CONSTRUCTION

The central half of the existing land-wall structure is founded on indurated clay (unit 1) with the remainder resting on the sandy siltstone (unit 2). The unit 1 rock has a high density of slickensides (smooth failure surfaces movement during compaction) and broken zones over the relatively narrow width (about 22 ft) of the monolith base. Because of this, designers assumed that a continuous failure plane might develop and daylight into the excavation. Strength parameters were selected (15 deg. friction and zero cohesion) and details of the anchor retention system finalized. Based on an allowable working bond stress for the rock-grout interface of 70 psi, designers chose bond lengths of 20 ft for the vertical an-

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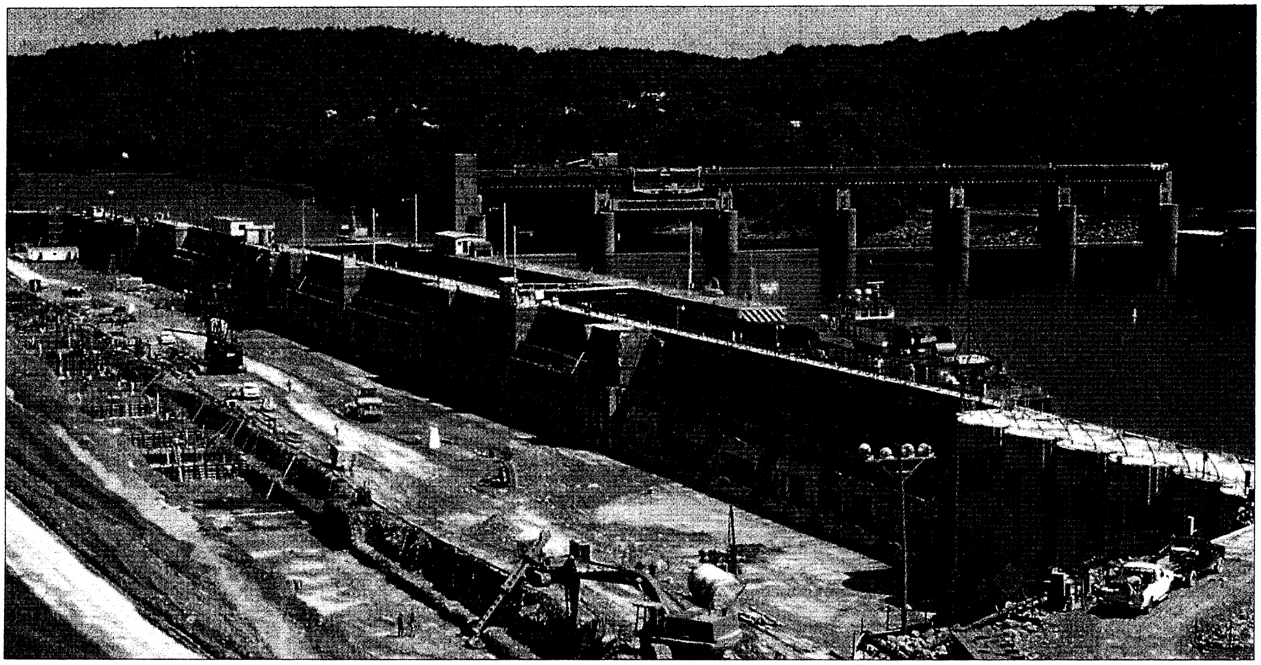


DETERIORATION AND MARGINAL STRUCTURAL STABILITY OF AN EXISTING LOCK CHAMBER, COUPLED WITH HEAVY RIVER TRAFFIC, REQUIRED THE CONSTRUCTION OF A LARGER REPLACEMENT LOCK TO ENSURE SAFE, DEPENDABLE AND MORE EFFICIENT NAVIGATION.

chors and 24 ft for the inclined anchors. Prestressed support was unnecessary for the new lock wall foundations founded on units 2 and 3, the interbedded silty clay stone and siltstone.

Crews from Nicholson Construction Co., Bridgeville, Pa., installed 139 vertical 12-strand anchors with a design working load (DWL) of 422 kips and stressed them to prevent overturning. Following soil excavation behind the land wall, they installed the upper row of 157 14-strand anchors (with a DWL of 492 kips) to resist sliding of the monoliths along the top of rock. Once this row was stressed, soil excavation was completed and a lower row of 129 anchors installed to prevent a potential deep-seated sliding failure during excavation to final grade. In addition, 46 anchors were used to stabilize the concrete and granular-filled coffercells.

Prior to the start of any rock anchor work in a monolith, crews would drill a 6 in. diameter core hole using a double-tube core barrel to better define the depth and quality of the rock in the bond zone. Once anchor lengths were determined, shop-fabricated tendons were ordered and the remaining holes in the monolith drilled using an 8 in. diameter down-the-hole hammer.

Workers pregrouted anchor holes before tendon installation to waterproof each hole and treat the rock mass to reduce seepage in the foundation beneath the land wall and into the adjacent excavation. All tendons have double corrosion protection. Each 0.6 in. diameter strand is coated with corrosion-inhibitor grease and encased in a sheath along its free length. Dywidag Systems International, U.S.A., Inc., Lemont, Ill., supplied the tendons.

The individually sheathed strands allowed for single-stage grouting of each anchor. Because high-early-strength (type 3) cement was used for anchor grout, workers were able to stress the tendons in as little as three days. They stressed the vertical and upper inclined anchors against a steel pipe casing grouted into the borehole to distribute more evenly the anchor load from the anchor head to the concrete within the wall. The lower inclined anchors distributed loads directly against thrust blocks at the toe of the wall. Where unit 2 siltstone existed, the thrust blocks were cast on top of rock. Where unit 1 indurated clay existed, the bearing capacity of the rock was not high enough to take the applied pressures of the thrust blocks. Consequently, workers cast thrust blocks on short, 36 in. diameter drilled shafts filled with concrete that transferred the vertical component of the anchor load to more competent rock below.

All anchors were either proof or performance tested, according to Post-Tensioning Institute 1986 recommendations for prestressed rock anchors, to a maximum test load of 133% design working load. In addition, one anchor per monolith for each of the vertical and upper inclined rows was creep tested.

COFFERDAM INSTRUMENTATION

All instrumentation had to be installed and operational prior to any excavation in the coffered area. Each existing land-wall monolith and sheet-pile coffercell was closely monitored. Data from the instruments were automatically read, recorded and transmitted via modem to the Corps' Pittsburgh District office to aid the coordination between the design team and field personnel.

Forty inclinometers were placed to depths up to 80 ft below the top of the cofferdam. Crews installed survey alignment pins along the top of the entire cofferdam perimeter for horizontal and vertical movements. Similarly, tilt plates were installed to monitor rotation.

Open standpipe piezometers were set in five of the granular-filled coffercells to define their saturation level. Uplift piezometers were installed at two locations in each of three existing land-wall monoliths and in one concrete-filled coffercell. Shear strips gave an immediate indication of horizontal displacement and of differential movement between the cofferdam elements. Both vertical and horizontal strips were installed, and their resistances read continuously by computers connected to an automatic alarm system. Vibrating wire-load cells were placed under 37 anchor heads and read automatically. These cells were connected to an alarm system to alert crews if loads increased or decreased beyond certain preset limits, signaling either structural or anchor failure.

The most critical construction phase was just prior to the stressing of the upper row of inclined anchors. During the installation and prior to stressing the inclined anchor, each monolith and coffercell experienced varying amounts of movement. Workers first removed soil backfill to elevation 780 ft lengthwise behind the land-wall monoliths, then excavated a trench sloping downward to the anchor-head elevation. As excavation proceeded, movements beneath the base of the land-wall monoliths as well as at the top of the structure were recorded. When the backfill in the trench was removed, two land-wall monoliths experienced rapid movements of almost 0.30 in. in the foundation, almost at a level of shear strain estimated to mobilize peak strength in the unit 1 indurated clay.

The contractor immediately replaced backfill behind the entire length of the land wall and excavated areas just large enough to install and stress two upper inclined anchors per monolith. After the anchors were stressed, workers made a continuous excavation and installed the remaining upper inclined anchors.

Once they had stressed the entire row of anchors, crews removed the backfill behind the land-wall monoliths to the top of the rock, permitting the lower row of inclined anchors to be installed and stressed. All movements of the land wall at the top of rock ceased. A later review of foundation conditions beneath the land-wall monoliths revealed that the zone of weak indurated clay was thickest directly beneath those monoliths that experienced the largest magnitude of movement. It is not clear whether these movements were due entirely to rebound of the rock mass caused by stress relief, or that the shear strength of the clay was lowest where the clay was thickest.

The instrumentation program was, in general, very successful. Although the shear strip system proved unsatisfactory mainly due to electrical interference, the inclinometer and survey alignment readings were in close agreement. The tilt meter readings were not so helpful, probably because only a very small component of the total movement involved tilting.

The piezometers in the granular-filled cells performed well and, at one point, detected a potential stability problem due to leakage through a sheet-pile interlock. Remedial actions included sealing, draining and increasing the anchoring. The uplift piezometers confirmed absence of excess uplift pressures in the foundation and highlighted the efficiency of the earlier rock-grouting program.

With the tragic circumstances of Alabama's Wheeler Lock in mind, similar work at the Point Marion Lock was conducted with all necessary attention to detail. This ranged from a close understanding of site geology and accurate characterization of rock strength properties to the use of real-time instrumentation monitoring and extensive application of rock anchors during progressive, strictly controlled excavation sequences. As a consequence, potential problems were averted early on, and the replacement lock was constructed in a cost-effective and safe manner with minimal interruptions to commercial navigation traffic. Construction began in April 1990 and was completed in December 1993.

The general contractor for this project was J.A. Jones Construction Co., Charlotte, N.C.; the anchor subcontractor was Nicholson Construction; and the instrumentation subcontractors were D'Appolonia, Pittsburgh, and CTL, Morgantown, W.Va.

FIG. 1. LOCATION OF NEW LOCK

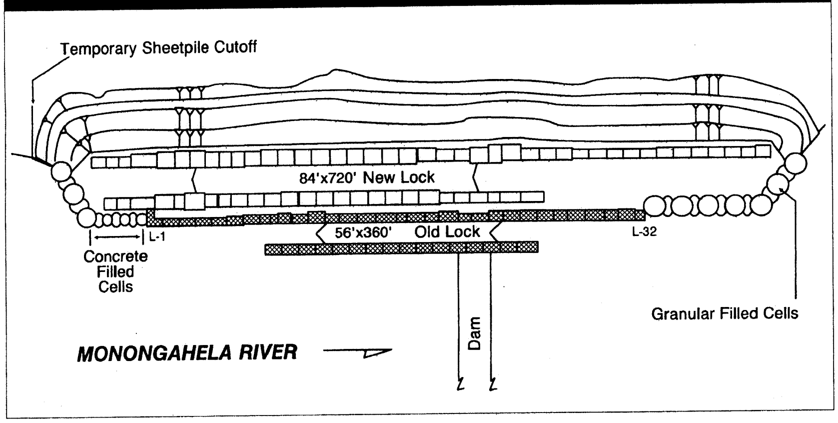
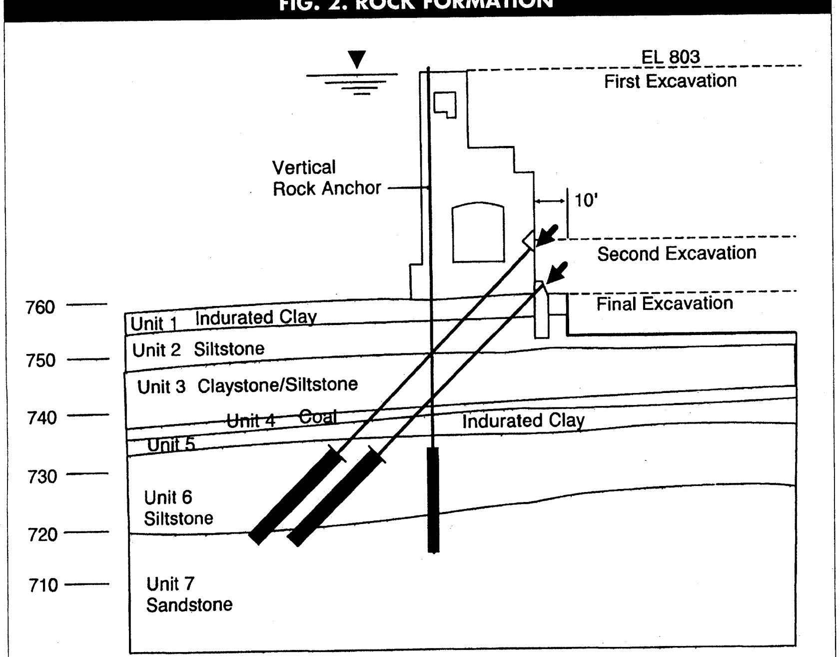


FIG. 2. ROCK FORMATION



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