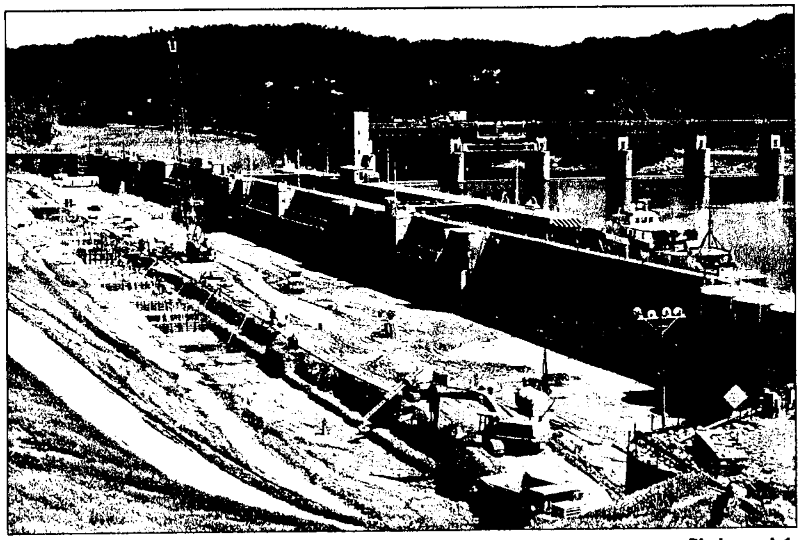
Unique cofferdam construction: Pt Marion Lock and Dam, Pennsylvania



Photograph 1.

by Donald A Bruce, Brian Greene and Andrew Schaffer †.

When it was first attempted to incorporate an existing lockwall into a new cofferdam, in 1961 at Wheeler Lock, Alabama, the result was a failure. A major portion of the land wall slid about 9m into the dewatered excavation which was being used to construct an adjoining lock, and there was loss of human life. The reported cause of the failure was sliding on an undetected weak clay seam in the foundation rock. No stabilisation measures or instrumentation systems had been implemented.

So, when a similar project was recently conceived to replace an existing navigation lock that had been in service on the Monongahela River in Southwest Pennsylvania since 1926, the Pittsburgh District of the US Army Corps of Engineers was clearly determined to avoid a repetition of the same problem. The solution involved nearly 500 high capacity prestressed rock anchors, constituting one of the largest single uses of prestressing strand in North America, and a structural and geotechnical instrumentation program reckoned by the suppliers to be one of the most intense ever installed in their long experience. The data from the instrumentation were monitored in real time through on site and off site computers to help optimise ongoing construction activities by eliminating potential problems at the earliest stage.

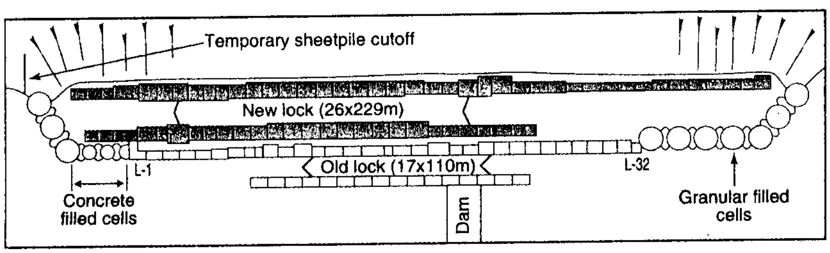


Figure 1. Plan of cofferdam incorporating the existing land wall of the older lock as the river arm of the new cofferdam.

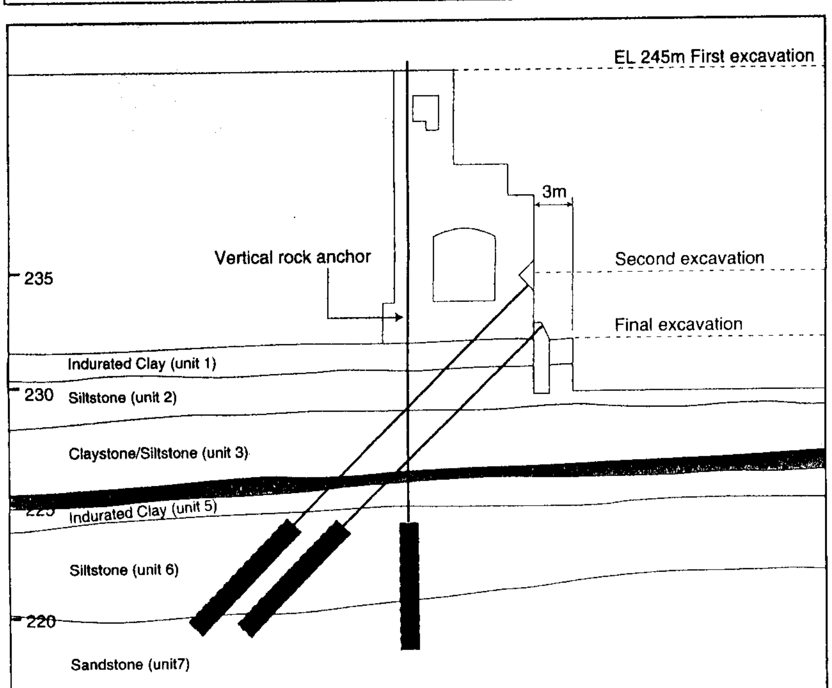


Figure 2. General layout of rock anchors as related to the typical geological sequence.

The site

Point Marion Lock and Dam is situated on the Monongahela River on the Pennsylvania-West Virginia border, almost 120km south of Pittsburgh. The age, advanced concrete deterioration and marginal structural stability of the existing 17m x 110m lock chamber, coupled with a heavy river traffic, demanded the construction of a larger, 26m x 220m replacement lock to ensure safe, dependable and more efficient navigation. The physical location for this reconstruction was restricted by several factors including retention of the existing (rehabilitated) gated dam, the minimisation of the volume of excavation required for approach cuts, and the need to avoid costly impacts to adjacent road and rail routes.

The new lock was therefore positioned immediately landward of the existing lock (**Figure 1** and **Photograph 1**). The new excavation was to be as close as 2.5m to the landward edge of the existing lock and to extend to a maximum depth of about 4m below the rock foundation of the existing landwall. The intent of the project

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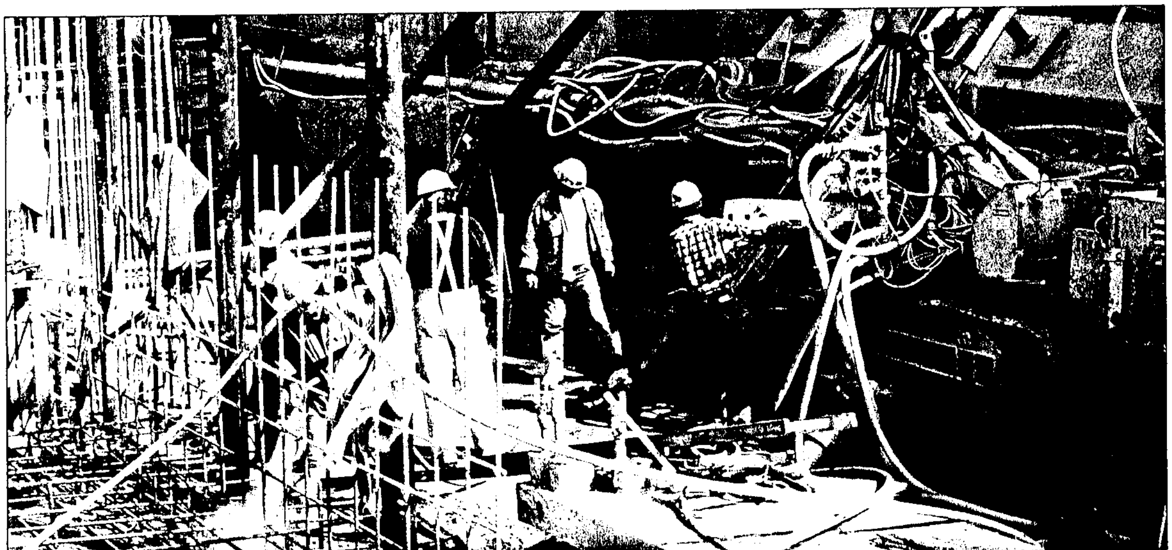


Photograph 2.

design was to construct the replacement lock while continuing to operate the existing navigation lock throughout the construction period. Upon completion of the new lock, the existing lock would be removed and a fixed weir section constructed to tie the new lock into the existing dam.

Geology

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 claystones, argillaceous limestones, siltstones, sandstones and coal seams from early Carboniferous to Permian age. At the site, the sequence of the Middle Pennsylvanian age Glenshaw Formation rocks is shown in **Figure 2**.

Groundwater levels at the site appear to be generally influenced by regional groundwater flow, draining off the adjacent hillside, and locally by the effects of upper and lower navigation pools. An intense phase of preconstruction site investigation was conducted, featuring over 150 core holes, laboratory testing, field groundwater testing, and the placement of an extensive network of piezometers.

Rock anchor design considerations

The central half of the existing landwall structure is founded on Unit 1 indurated clay with the remainder resting on the Unit 2 sandy siltstones (**Figure 3**). Because of the high density of slickensides and broken zones in Unit 1 over the relatively narrow width of the monolith base (about 7m), it was assumed that a continuous failure plane could develop and daylight into the excavation. Conservative strength parameters were selected ($\phi = 15^o$; C = 0). The details of the anchor retention system were then finalised. Based on an allowable working bond stress for the rock/grout interface of about 0.5N/mm^2, bond lengths of 6m for the vertical anchors, and 7.2m for the inclined anchors were selected. No prestressed support was needed for the new lock wall foundations founded on Units 2 and 3.

A total of 139 vertical 12-strand anchors with a design working load (DWL) of 1880kN were first installed and stressed to prevent overturning. Following soil excavation behind the land wall, the upper row of 157, 14-strand anchors (DWL = 2190kN) was designed to resist sliding of the monoliths along the top of rock. Once this row was stressed, soil excavation could be completed and a lower row of 129, similar anchors installed to prevent a potential deep seated sliding failure, before excavation to final level. An additional 46 anchors were also used to stabilise the concrete and granular filled coffercells.

Rock anchor construction

Prior to the start of any rock anchor work in a monolith, one hole was first drilled using a 150mm diameter double tube core barrel. This was

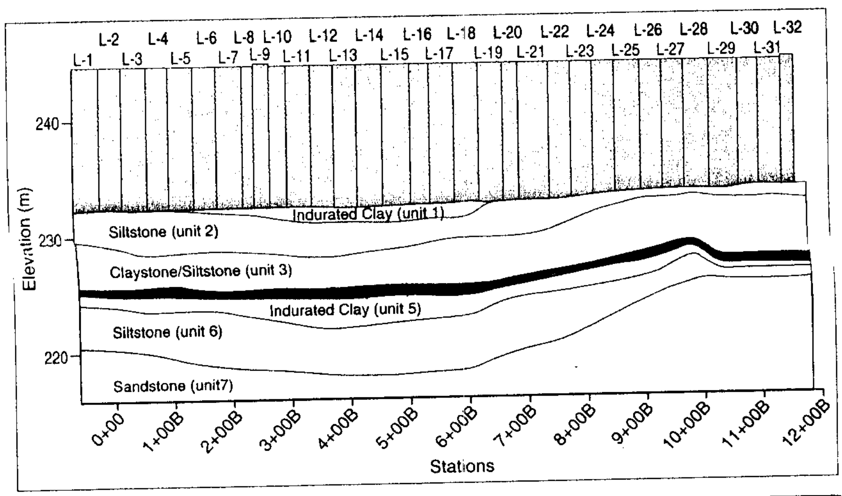


Figure 3. Geologic profile along existing landwall cofferdam.

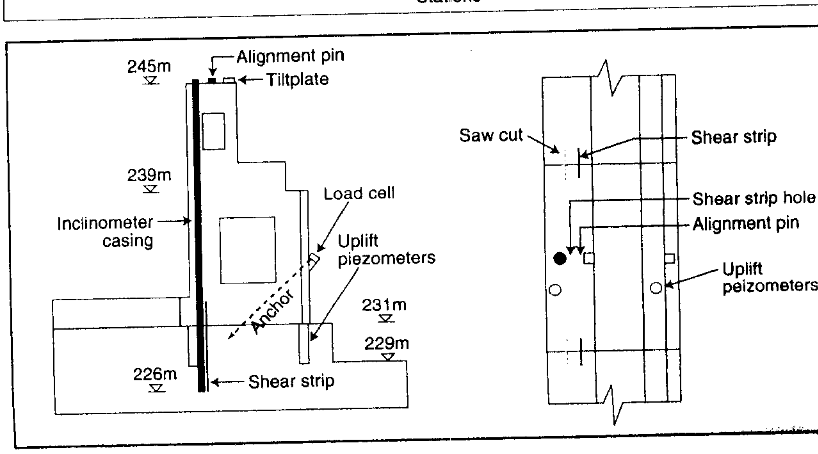


Figure 4. Instrumentation layout, existing concrete monoliths.

done to better define the depth and quality of the rock in the bond zone. Once final anchor lengths were determined, shop-fabricated tendons could be ordered and the remaining holes in the monolith drilled using a 200mm diameter down-the-hole hammer.

Anchor holes were pregrouted before tendon installation in order to both treat the rock mass to reduce seepage in the foundation beneath the land wall and into the adjacent excavation, and to thereafter seal each hole against loss of subsequent anchor grout. All tendons were of the double corrosion protection type with each 15.2mm diameter strand coated with corrosion inhibitor grease and encased in a sheath along its free length. They were supplied by Dywidag Systems International, USA, Inc. The individually sheathed strands allowed for single stage grouting of each anchor. High early strength (Type III) cement was used for anchor grout, and this allowed stressing in as little as three days after placement.

The vertical and upper inclined anchors were stressed against a steel pipe casing grouted into the borehole in order to distribute more evenly the anchor load from the anchor head to the concrete within the wall. The lower inclined anchors distributed their loads directly against thrust blocks at the toe of the wall as shown in **Photograph 2**. 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. Therefore, thrust blocks were cast on short, 915mm diameter drilled shafts filled with concrete which transferred the vertical component of the anchor load to more competent rock below.

All anchors were either proof or performance tested (**Photograph 3**) according to Post-Tensioning Institute recommendations (1986) for prestressed rock anchors to a maximum test load of 133% DWL. In addition, one anchor per monolith for each of the vertical and upper inclined rows was creep tested. No abnormal anchor performance characteristics were recorded.

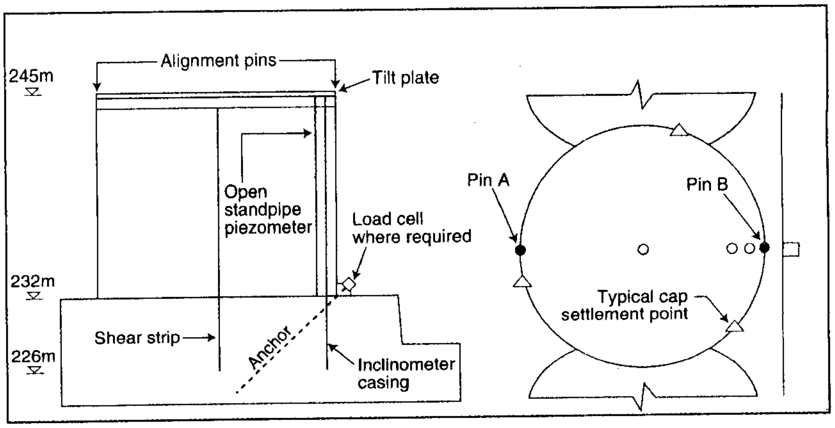


Figure 5. Instrumentation layout, granular filled cells.

Cofferdam instrumentation

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

As illustrated in **Figures 4** and **5**, 40 inclinometers were installed to depths of 24m below the top of the cofferdam. Survey alignment pins were installed along the top of the entire cofferdam perimeter (for horizontal and vertical movements), while tilt plates were similarly installed to monitor rotation. Open standpipe piezometers were installed in five of the granular filled coffercells to define their saturation level. Uplift piezometers were installed at two locations in each of three existing landwall monoliths and in one concrete filled coffercell. Shear strips were used to give an immediate indication of stressing displacement and of differential movement between the cofferdam elements. Both vertical and horizontal strips were positioned, and their resistances were continuously read by computers and connected to an automatic alarm system. Vibrating wire load cells were placed under 32 anchor heads and read automatically. These cells were also connected to the alarm system which would be triggered if loads increased or decreased beyond certain preset limits, judged indicative of either structural or anchor failure, respectively.

Photograph 3.

Cofferdam performance

The most critical phase in the construction was calculated as being just prior to the stressing of the upper row of inclined anchors. During the installation and prior to the stressing of the inclined anchors, each monolith and coffercell experienced varying amounts of movement into the excavation. Soil backfill was first removed to Elevation 780 lengthways behind the land wall monoliths and then a trench sloping downward to the anchor head elevation was excavated. As excavation proceeded, movements beneath the base of the land wall monoliths as well as at the top of the structure were recorded. Land wall monoliths 14 and 15 experienced rapid movements in the foundation, approaching 7.5mm, when the backfill in the trench was removed. Movement at the base of land wall monolith 14 is shown in **Figure 6**. This immediately raised concerns since the reported movements were approaching the shear strains estimated to mobilise peak strength in the Unit 1 indurated clay.

The contractor was immediately directed to replace backfill behind the entire length of the land wall and to make localised excavations just large enough to install and stress two upper inclined anchors per monolith. After these anchors were stressed, a continuous excavation was made and the remaining upper inclined anchors installed. Once the entire row of anchors had been stressed, backfill behind the land wall monoliths was removed to the top of 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 at this time. 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 (**Figure 7**). It is not clear whether these movements were entirely due to rebond of the rock mass caused by stress relief or that the shear strength of the clay was lowest where it was thickest.

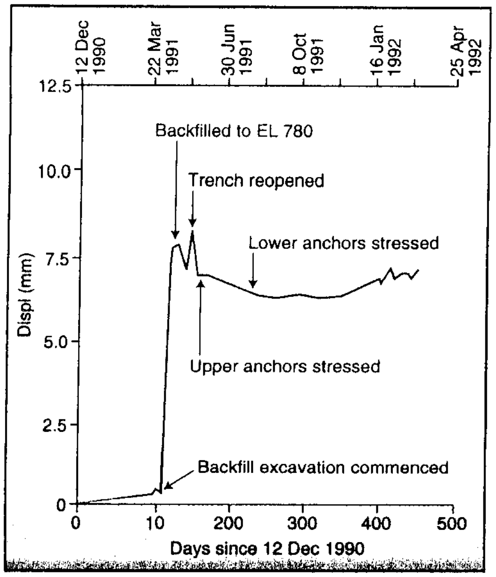
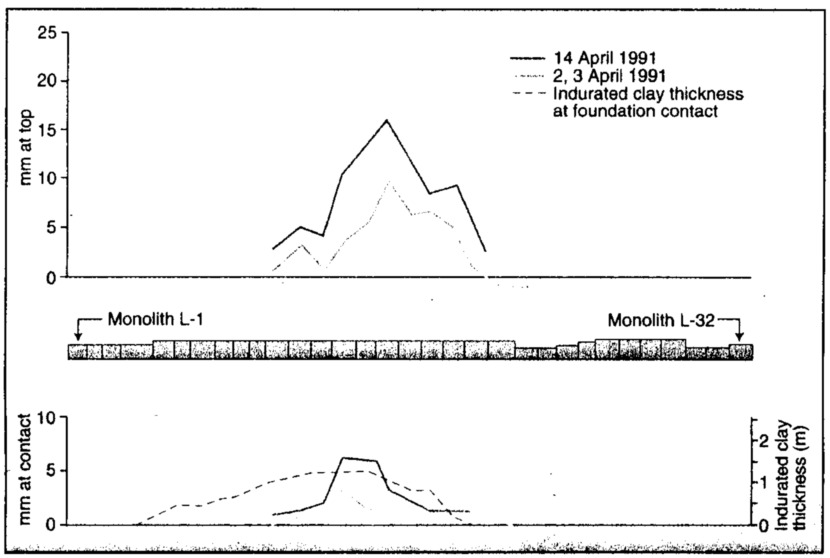
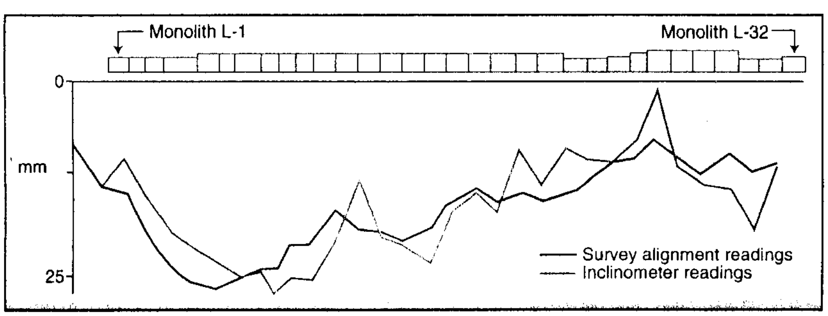


Figure 6. Time-history plot of movement at the base of landwall monolith 14.

BELOW: Figure 7. Movement at top and foundation contact of landwall monoliths.

BOTTOM: Figure 8. Inclinometer -v- survey alignment readings, taken January 1992.





Instrumentation performance

The shear strip system proved relatively unsatisfactory mainly due to electrical interference reasons. However, the inclinometer and survey alignment readings gave good correspondence (**Figure 8**) although the tilt meter readings were not so confirmatory, probably due to the fact that only a very small component of the total movement was due to tilting and was beyond the accuracy of the instrument.

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 action included sealing, drainage and additional anchoring. The uplift piezometers confirmed that there were no excess uplift pressures in the foundation and highlighted the efficiency of the earlier rock grouting program.

Final remarks

With the tragic circumstances of Wheeler Lock in recent memory, the similar work projected at Point Marion was conducted with a cautious realism and all necessary attention to detail. This ranged from a close understanding of site geology and accurate characterisation of rock strength properties to the use of intensive real time instrumentation monitoring and the extensive application of rock anchors during progressive, strictly controlled excavation sequences.

As a consequence, potential problems were nipped in the bud, and the replacement lock was constructed in a cost effective and safe manner with minimal interruptions to commercial navigation traffic.

Acknowledgments

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Anchor sub contractor Nicholson Construction Company, Bridgeville, PA;
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Marshall Fausold was Chief of Geotechnical Branch, Pittsburgh District, during this project, and **John A Gerlach** was responsible for instrumentation analysis for the US Army Corps of Engineers throughout the project construction.