The instrumentation and stabilization of a major excavation: Point Marion Lock, Pennsylvania

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ABSTRACT: In 1961 at Wheeler Lock, AL, the Tennessee Valley Authority attempted to incorporate an existing lock wall into a new cofferdam. A major portion of the land wall slid about 30 ft (9 m) into the dewatered excavation killing several people. Reportedly, sliding occurred on an undetected weak clay seam in the foundation bedrock. No movement detection systems or stabilization measures had been implemented. About 30 years later, a similar project was undertaken at Point Marion Lock, PA to replace the 68 year old navigation lock on the Monongahela River. The designers were the Pittsburgh District of the U.S. Army Corps of Engineers. The paper describes the design and operation of a $1 million structural and geotechnical instrumentation program. Elastic finite element analyses were performed during project design as a tool for predicting the magnitude of cofferdam movements during excavation. Instrumentation data were monitored throughout the foundation excavation and construction of the new 84 ft (25 m) by 720 ft (220 m) lock, in real time through on- and off-site computers. Over 470 high capacity prestressed rock anchors were installed: the vertical anchors to increase resistance to overturning of the old lock wall, now being used as the river side of the excavation, and the inclined anchors to resist sliding of this structure and the adjacent coffercells. Throughout the excavation process, cofferdam deflections were closely monitored. Analysis of instrumentation data indicated that horizontal displacements, within the bedrock below the base of the cofferdam, exceeded that predicted from finite element analyses by a factor of 2.5.

1. INTRODUCTION

Point Marion Lock and Dam is located on the Monongahela River on the Pennsylvania-West Virginia border, about 75 miles south of Pittsburgh. At this site, the Pittsburgh Corps of Engineers has recently constructed a new navigation lock to replace an existing lock built in 1926. The age, advanced concrete deterioration and marginal structural stability of the existing 56 ft by 360 ft (17 m by 110 m) lock chamber, coupled with heavy river traffic, led to the decision to construct a larger 84 ft by 720 ft (25 m by 220 m) replacement lock to ensure safe, dependable and more efficient navigation. The existing gated dam, built in 1959, was rehabilitated in 1988 with repair work involving the installation of prestressed rock anchors to improve stability of the structure.

The physical location for construction of the new lock was restricted by several factors including retention of the rehabilitated dam, minimizing the volume of excavation required for approach cuts, and avoiding costly impacts to and possible relocation of an adjacent state road and active railroad line. Because of these constraints, the new lock was designed to be built immediately landward of the existing Point Marion Lock. Excavation for the new structure was as close as 8 ft (2.4 m) from the landward edge of the existing lock and extended to a maximum depth of about 13 ft (4.0 m) in rock below the foundation of the existing land wall. The intent of the project design was to construct the replacement lock while continuing to operate the existing navigation lock throughout the construction period.

Construction of the new lock required a complex cofferdam arrangement which incorporated the existing land wall as the river arm of the cofferdam (Figure 1). Nearly 500 high capacity prestressed rock anchors were installed in three rows to assure the required stability of the existing land wall cofferdam and adjacent sheet pile cells. Excavation proceeded in stages and was closely tied to the installation and stressing of each row of anchors. Careful monitoring of the entire cofferdam was accomplished by an extensive instrumentation program developed for the project.

The contract for construction of the Point Marion replacement lock was awarded in April, 1990 and the project was completed in late 1994.

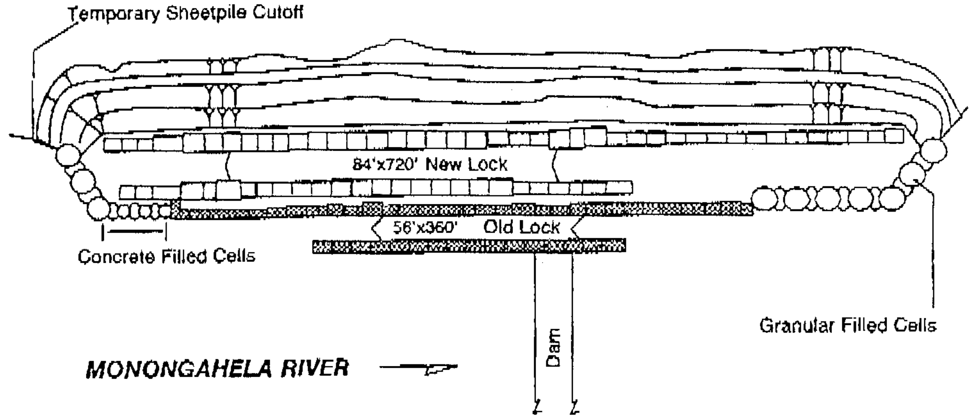

Figure 1 Plan of Cofferdam.

2. WHEELER LOCK FAILURE

Point Marion Lock was not the first time that an existing, operational lock chamber was used as a part of a cofferdam for a new lock chamber that was being constructed adjacent and landward of an existing lock. On June 2, 1961, a major portion of the land wall of General Joe Wheeler Lock and Dam on the Tennessee River moved about 30 ft (9m) into the dewatered excavation which was being used to construct an adjoining lock. Contemporary accounts of the failure of Wheeler Lock in American Society of Civil Engineers (1961) and Terzaghi (1962) indicate that this catastrophic event resulted in the loss of two lives and suspension of navigation on a reach of the Tennessee River while lock reconstruction proceeded. The reported cause of failure was sliding of the existing land wall on an undetected weak clay seam in the foundation rock (Kiersch and James, 1991). No stabilization measures or instrumentation systems were used at Wheeler Lock.

The design and construction aspects of Point Marion Lock were in many ways quite similar to Wheeler Lock. Therefore, in developing the Point Marion project, much attention was given to circumstances surrounding the 1961 lock wall failure.

3. SITE GEOLOGY

The geology of the site is fairly simple consisting of Quaternary alluvium overlying Middle Pennsylvania age, flat-lying sedimentary rocks. Bedrock is typical of Coal Measure stratigraphy, and consists of a series of claystones, indurated clays, siltstones, sandstones, and a prominent coal seam, the Bakerstown Coal. All rock units found locally are part of the Middle Pennsylvanian age Glenshaw Formation. Indurated clay is a rock name used locally within the Ohio Valley region to describe a weak, very soft, highly slickensided claystone. In coal mining, the term underclay is often used to describe this material, though not all indurated clays occur below coal seams.

As depicted in Figure 2, the individual rock units are numbered 1 through 7 for correlation purposes with the Bakerstown Coal serving as a prominent stratigraphic marker bed. The geologic section above the coal seam is highly variable and consists of indurated clay, claystone, siltstone, and occasional discontinuous lenses of sandstone. In general, the bedrock in this part of the section is fine grained, soft to moderately hard, variably slickensided, and highly fractured. These units also contain very soft clay seams and limestone nodules. Directly below the coal seam, there is an indurated clay layer approximately 3 to 5 ft (1 to 1.5 m thick. Below the indurated clay the rock units are more competent, consisting of moderately hard, argillaceous siltstone grading downward into a hard, fine grained sandstone. Although stratigraphically simple, site bedrock has been altered by the effects of valley stress relief which takes the form of intensely sheared and broken zones with the rock mass (Ferguson, 1967). Since the existing landwall was to serve as the main arm of the cofferdam for the new lock excavation, conditions beneath this structure were critical to project design. The existing landwall monoliths are founded on Unit 1 indurated clay or Unit 2 siltstone.

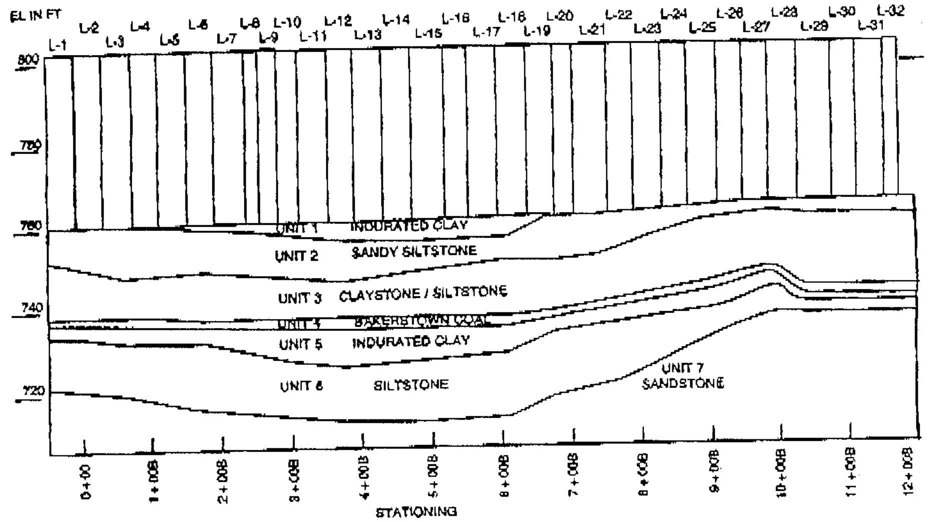

Figure 2 Geologic Profile along Existing Landwall Cofferdam.

4. COFFERDAM STABILIZATION DESIGN

4.1 Design Rock Strength Parameters

In designing the replacement lock, a priority was placed on developing reasonable foundation strength parameters from the accumulated laboratory test data for both the new lock and cofferdam. Since the landwall of the existing lock was not designed to withstand the cofferdam loading condition, assuring its stability throughout the excavation phase of the project was of paramount importance. In particular, the shear strength of the weak Unit 1 indurated clay that underlies a major portion of the existing land wall was critical to the analysis of cofferdam stability.

Direct shear testing was performed at three different Corps of Engineers laboratories. As evidenced by Table 1, different test results were reported by each of the three laboratories.

With regard to residual strength, both Missouri River Division Laboratory and Waterways Experiment Station yielded shear strengths that are considered similar as far as the variability of rock discontinuity surfaces are concerned. Results from the Ohio River Division Laboratory are significantly higher than the other two labs. According to Nicholson (1994), the prime suspected cause for the observed variation in test results can be attributed to differences in the test equipment used.

Selection of appropriate foundation strength parameters is based on both the analysis of laboratory test data and engineering geologic judgment. The qualitative descriptions of geologic site conditions have to be merged with the quantitative values derived from rock mechanics testing to arrive at a shear strength for use in a stability analysis. Generally, the available shear strength of a rock mass lies somewhere between the peak and residual strength of all its component parts along some failure surface through that rock mass (Simmons and Swartz, 1988). The orientation of the principal discontinuities within the rock mass, the rock mass shear strength and the direction of loading all play a major role in determining the location of a potential failure surface.

Table 1. Shear strength test results of Unit 1 indurated clay.

Corps of Engineers	Peak		Residual	
Laboratory	φ	c(psi)	φ	c(psi)
Ohio River Division Lab	47	15	38	0
Missouri River Division Lab	24	8	15	0
Waterways Experiment Station	32	56	18	0

Design strength parameters: $\varphi = 15$, $c = 0$

At Point Marion, approximately one-half the existing land wall is founded on Unit 1 indurated clay. Because of the high density of slickensides and broken zones within the indurated clay unit over the relatively narrow width of the monolith base (about 22 ft [6.7 m]), it was assumed that a continuous failure plane could develop and daylight into the excavation. Based on careful evaluation of all available data, a residual shear strength with a friction angle of 15° and zero cohesion was adopted for the stability analysis. These strength parameters represented the lowest residual values from all the rock mechanics test data accumulated for the project and were felt to be appropriate based on a review of test results on similar rock from other projects.

It was established during the design process that the bond zone for the rock anchors would be within the more competent Unit 6 siltstone and Unit 7 sandstone (Figure 3). Based on pull-out test results of these rock units, a bond strength of 70 psi (0.48 MPa) was used for design purposes. Factoring this data, bond lengths were determined to be 20 ft (6 m) for the vertical anchors and 24 ft (7.3 m) for the inclined anchors.

4.2 Stability Analyses

Sliding and overturning stability analyses were performed for each cofferdam loading condition. The total number of prestressed rock anchors needed to stabilize the cofferdam was then determined. Both shallow and deep-seated sliding analyses were performed using Spencer's Method. For sliding stability, a factor of safety of 1.5 was required with one notable exception: because of the existence of the culvert in the land wall, the upper row of inclined anchors had to be positioned to pass beneath it (Figure 3). This required over 30 ft (9m) of soil excavation immediately behind the land wall. A temporary factor of safety of 1.25 was accepted for this condition since the time between excavation and anchor stressing would be limited.

In order to utilize the existing land wall as the main arm of the cofferdam, three rows of rock anchors were required to ensure stability against cofferdam failure. A total of 139 vertical, 12-strand anchors with working loads of 422 kips (1.88MN) were installed to prevent overturning. After the stressing of the vertical row of anchors was completed, soil excavation behind the land wall could proceed down to the upper row of inclined anchors (Figure 3). A total of 157 inclined, 14-strand anchors with working loads of 492 kips (2.19MN) were then installed to resist sliding of the land wall monoliths along the top of rock. Once this row was stressed, soil excavation could be completed. A lower row of inclined anchors was then required to prevent a deep-seated sliding failure into the adjacent excavation for the new river wall. A total of 129 inclined, 14-strand anchors were installed for this purpose. Once stressing was completed, rock excavation could proceed down to the pre-determined founding elevations of the new river wall monoliths. Additional anchors were also employed to stabilize the concrete and granular filled coffercells, resulting in a total of 471 prestressed rock anchors being used to stabilize the entire cofferdam.

4.3 Finite Element Modeling

Elastic finite element analyses were performed for each stage of excavation and each application of anchor load. Values for the in-situ rock mass modulus were estimated using the formula by Serafim and Pereira (1983):

$$E = 10^{(RMR-10)/40} \text{ (in GPa)}$$

where RMR = rock mass rating in accordance with the Geomechanics Classification. The calculated values of rock mass rating and rock mass modulus are presented in Table 2.

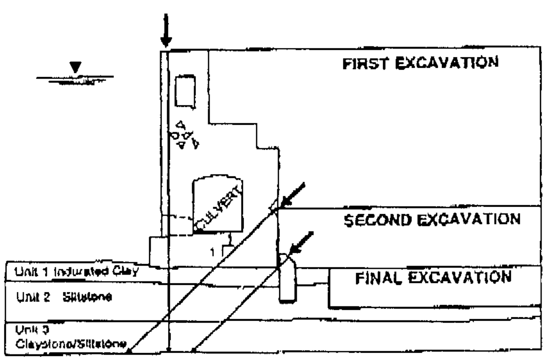

Figure 3 Typical Cross-section of an Anchored Monolith.

Table 2. Rock Mass Modulus Predictions for Point Marion Lock Foundation (1.0 GPa = 145 Ksi)

Unit No.	RMR Average	E Average (Ksi)
1	18	230
2	29	433
3	24	325
4	25	344
5	24	325
6	51	1536
7	63	3065

In order to verify the accuracy of the predicted modulus values, horizontal deflection measurements of the top of the land wall were surveyed while the pool in the lock chamber was raised and lowered. The modulus values estimated from modeling these conditions were in close agreement with those values predicted by the Serafim and Pereira formula.

The analyses for each stage of excavation and anchor loading revealed that the most critical phase of construction was after the first stage of excavation and prior to the installation of the upper row of inclined anchors. Horizontal displacements at the bases of monoliths founded on indurated clay were predicted to be 0.12 in (3 mm). This was well below the estimated allowable peak shear strain of 0.1% (Barton, 1982), which corresponds to a horizontal displacement of 0.25 in. (6 mm).

5. ROCK ANCHOR INSTALLATION AND PERFORMANCE

5.1 Rock Anchor Construction

Prior to the start of any rock anchor work in a monolith, one hole was first drilled using a six-inch (15 cm) double tube core barrel. This was 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 an eight-inch diameter down-the-hole hammer.

Prior to rock anchor installation, all holes were pregrouted in order to reduce seepage in the foundation beneath the land wall and into the adjacent excavation. Grouting procedures generally followed those given by the Water Resources Commission of Australia (1981). For a rock mass permeability of approximately 2 x 10^{-4} cm/sec, a starting grout mix having a water-cement ratio of 2:1 by volume was selected. Because of the weak and fractured nature of the rock mass, a maximum allowable grouting pressure of 1 psi/ft of depth was used. Grouting was successful in creating an effective curtain as very little seepage was later observed in the excavation for the new river wall.

All tendons were of the double corrosion protection type with each 15 mm diameter strand coated with corrosion inhibitor grease and encased in a sheath along its free length. The individually sheathed strands allow for single stage grouting of each anchor. High early strength (Type III) cement was used for anchor grout which allowed for stressing in as little as three days.

5.2 Stressing and Testing

The vertical and upper inclined anchors were stressed against a steel pipe casing grouted into the borehole in order to distribute 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. 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, 36-in (914 mm) diameter drilled shafts filled with concrete which transferred the vertical component of the anchor load to more competent rock below (Figure 3).

All anchors were either proof or performance tested according to Post-Tensioning Institute (1986) recommendations for prestressed rock anchors. In addition, one anchor per monolith for each of the vertical and upper inclined rows was creep tested. Out of a total of 471 anchors, only three were not successfully stressed. These three anchors were accepted at reduced working loads.

6. COFFERDAM INSTRUMENTATION

6.1 Instrumentation Layout

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 (24 m) 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 (Figure 4).

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.

6.2 Movement During Excavation

The most critical phase in the construction of

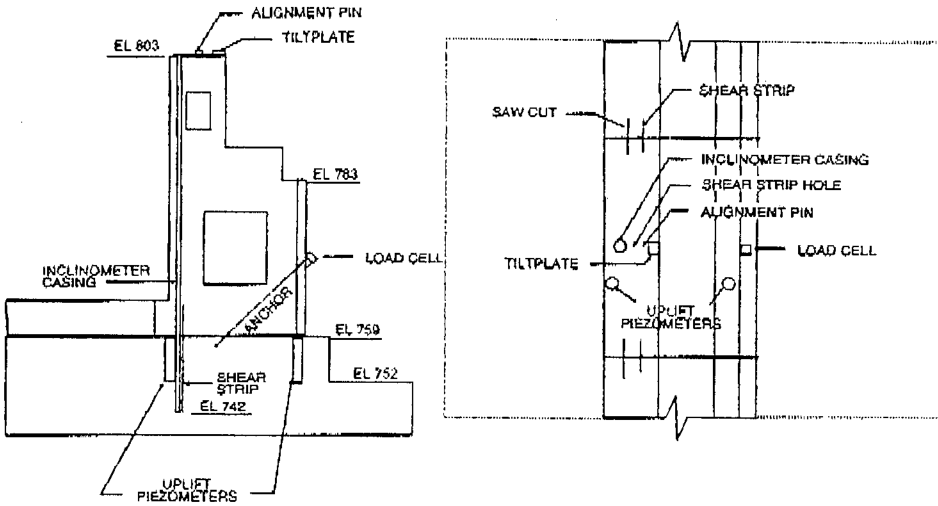

Figure 4 Instrumentation Layout, Existing Concrete Monoliths.

the new lock chamber was during the installation of the upper row of inclined anchors when backfill behind the wall was removed to the upper inclined anchor head elevation. At this stage of excavation, only the vertical anchors had been installed and stressed. 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 ft (234 m) lengthwise 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 by the instrumentation. Land wall monoliths 14 and 15 experienced rapid movements in the foundation, approach 0.30 in. (8 mm), when the backfill in the trench was removed. Movement at the base of land wall monolith 14 is shown plotted as movement versus time (Figure 5). This immediately raised concern since the reported movements had reached the shear strains estimated to mobilize peak strength.

The contractor was immediately directed to replace backfill behind the entire length of the land wall and to make localized excavations just large enough to install and stress two upper inclined anchors. After two anchors per monolith were stressed, a continuous excavation was made and the remaining upper inclined anchors installed. Once the entire row of anchors was 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

(Figure 5). A review of foundation conditions beneath the land wall monoliths reveals 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 are entirely due to mass deformability of the indurated clay or that the joint shear strength and stiffness of the indurated clay is lowest where it is thickest.

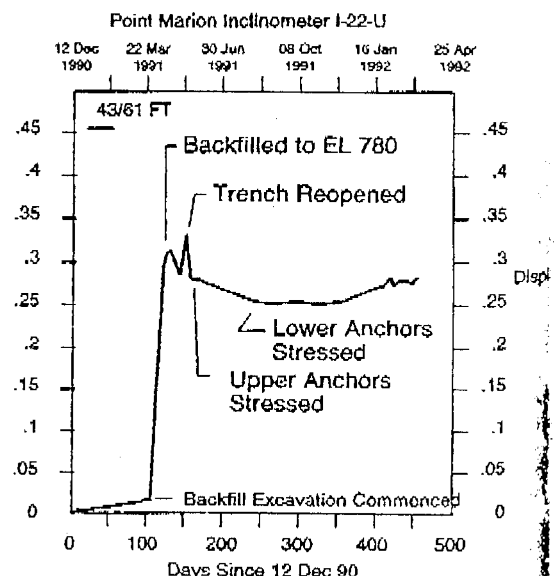

Figure 5 Time-history Plot of Movement at Base of Landwall Monolith 14.

6.3 Instrumentation Performance

The shear strip system did not operate as expected in that the alarm system falsely sounded several times without any significant cofferdam movements occurring. This was later determined to be due to electrical interference and was corrected by increasing the resistance of the circuitry. A more serious problem occurred when the shear strips did not break despite movements in the foundation in excess of 0.10 in (2.5 mm) as measured by the inclinometers. This led to the load cells being connected to the alarm system as a supplement to the shear strips.

A comparison of the movement measuring systems revealed that the inclinometer readings determined for the top of the cofferdam were in general agreement with those determined by the alignment surveys. The tiltmeter readings did not agree with the inclinometer and alignment survey readings but this is probably due to the fact that only a component of the total movement was due to tilting of the structure. The magnitude of tilt was so small, considering the height of the structure, that it was most likely not within the accuracy of the measuring system. Based on information from the manufacturer, the tiltmeters are accurate to approximately - 1.5 min of arc. A typical displacement of 0.3 in (8 mm) at the base and 0.6 in. (15 mm) at the top of a monolith is equivalent to a rotation of 0.03° or about 2 min of arc.

7. CONCLUSIONS

Stabilization of the excavation required to construct the new Point Marion Lock required detailed geotechnical investigations and characterization of rock mass properties in order to design an anchored cofferdam for the project. The only previous attempt in the United States to incorporate a lock wall as part of a cofferdam for a replacement lock resulted in failure of the structure and loss of life. With the circumstances of the Wheeler Lock failure in mind, the stabilization scheme utilized at Point Marion was keyed to both the adverse geological conditions present at the site and the progressive stages of excavation and loading required to construct the new lock. Rock anchors were installed sequentially with each row stressed prior to advancing to the next stage of excavation.

In the design of excavation sequences and stabilization measures for the Point Marion Lock cofferdam, elastic finite element analyses were conducted. Input values of rock modulus required for the finite element model were estimated using a formula developed by Serafim and Pereira (1983). This yielded a predicted horizontal displacement for cofferdam structural elements, founded on weak rock, of 0.12 in. (3 mm).

Of significant importance to this project was an extensive instrumentation program which permitted real-time monitoring of cofferdam performance throughout the excavation process. Movements of the cofferdam were rapidly detected and analyzed to enable the project design team to modify both the rate and scope of excavation as well as the sequence of rock anchor stressing. Subsequent analysis of the instrumentation data indicates that horizontal displacements within weak bedrock below the base of the cofferdam exceeded that predicted from a finite element model prepared during project design by a factor of 2.5. It is concluded from these data that the use of the Serafim and Pereira formula for slickesided indurated clay (claystone) rock formations yields values for rock mass modulus that are higher than is probably the true case.

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