Instrumentation of Cumberland Gap Pilot Tunnel

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The Cumberland Gap twin highway tunnels are currently under construction. In 1986 a pilot tunnel was excavated at the crown of the southbound tunnel to investigate the geologic conditions along the 4,200 ft length of the twin tunnels. This paper describes the evolution of the instrumentation program associated with the excavation of the pilot tunnel. The instrumentation program as initially conceived had research as a primary objective. The initial program was reduced to a moderate number of extensometers, convergence points, strain gauges on steel sets, piezometers, and groundwater flow weirs for the bid documents for the pilot tunnel excavation. After about 15 percent of the tunnel had been excavated, the instrumentation program was again reduced substantially because the initial data indicated that the scope of the program could be reduced while still providing sufficient data for designing support of the main tunnels. The paper presents the results of the instrumentation program and concludes with recommendations regarding instrumentation programs for pilot tunnels.

The Cumberland Gap has been a transportation route since before Daniel Boone led settlers through the Gap to Kentucky. Currently, U.S. Highway 25E carries more than 18,000 vehicles per day between Cumberland Gap, Tennessee, and Middlesboro, Kentucky, on a winding alignment that has steep grades. To reduce the accident rate along this section of steep terrain and to allow restoration of the wilderness road in the Cumberland Gap National Historic Park, twin tunnels 4,200 ft long are planned through Cumberland Mountain.

Planning for the tunnels started in the 1950s, but financial restrictions delayed initial design until 1980, when work started in earnest. Investigations for the tunnels have included:

- Geologic literature review (1980–1982);
- Extensive outcrop mapping on the surface and in the nearby railroad tunnel through Cumberland Mountain (1981–1983);
- Extensive core drilling at both portals (1981–1983);
- A 2,000 ft horizontal core boring from the Kentucky portal along the alignment of the southbound tunnel (1983); and
- Excavation of a 10 ft by 10 ft pilot tunnel as a crown drift in the southbound tunnel (1985–1986).

This paper describes the evolution of the instrumentation for the pilot tunnel, the results obtained from the instrumentation program, the implications of the instrumentation results on the design of the main tunnels, and general recommendations for instrumentation of pilot tunnels.

The prime objectives of the pilot tunnel were to investigate the ground conditions along the tunnel alignment, to evaluate how the ground will behave during the construction of the main tunnels, and to expose the geology for first-hand evaluation by main tunnel designers and contractors bidding on the main tunnels (*1*). The importance of understanding ground behavior in tunnel construction is illustrated by the difficulties that were encountered during the excavation of the Eisenhower Tunnel in Colorado (*2*), where ground behavior proved to be much different than expected. Severe squeezing conditions were encountered in a major regional fault zone, resulting in substantial changes in the contractor's planned operations and substantial cost overruns.

GEOLOGY

Cumberland Mountain is an overthrust block near the junction of the Valley and Ridge and Appalachian Plateau provinces. The rocks along the tunnel alignment range from Silurian to Pennsylvanian age formations. Rock conditions vary significantly and range from uniform shales and limestones to interbedded sandstone, shales, and coals. Bedding dips at 35 to 50 degrees toward Kentucky and strikes nearly perpendicular to the tunnel alignment. Figure 1 shows a generalized geologic profile along the tunnel alignment.

The strength, jointing, and engineering characteristics of these units vary significantly. The limestones and some of the sandstone units are massive, without prominent bedding planes and with relatively few joints, while some of the claystones, mudstones, and shale units are weak, with closely spaced bedding partings and numerous slickensided joints. Extensive cave systems and mudfilled solution cavities were encountered in one of the limestone units during excavation of the pilot tunnel. One of these cave systems carries significant quantities of water across the tunnel alignment.

EXPECTED GROUND BEHAVIOR

The behavior of the ground depends on the engineering characteristics of the ground and the construction process. Ground behavior of rock tunnels is often characterized as follows (*3*):

- Structural control or loosening (rock displacing under its own weight along preexisting discontinuities);

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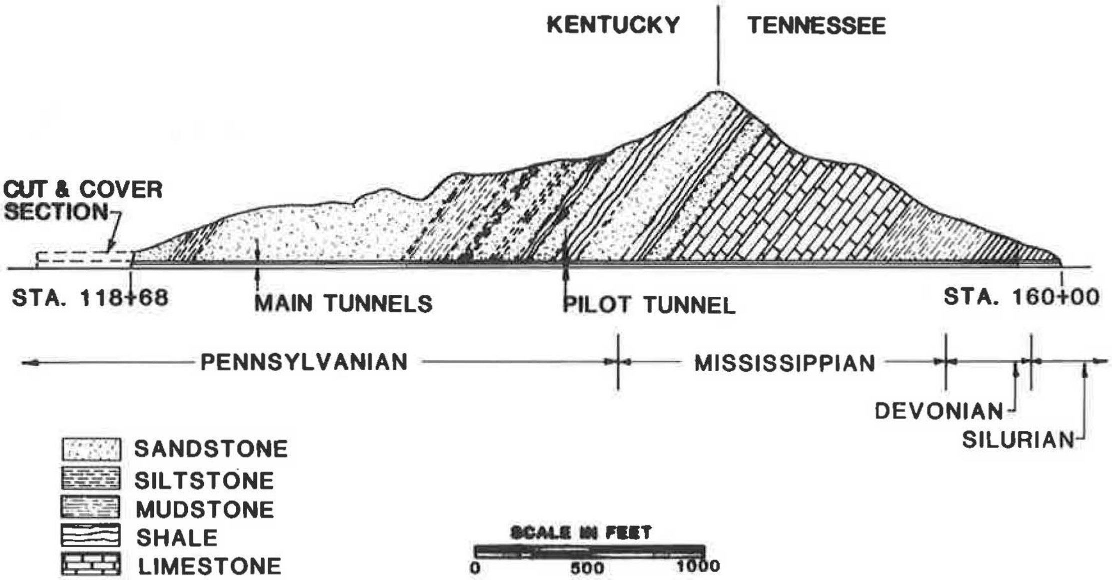

FIGURE 1 **Geologic profile of Cumberland Gap tunnels.**

- Stress slabbing (formation of new fractures and slabs as a result of stress concentrations around the opening);
- Squeezing (creep without volume change due to stresses that exceed the strength of the rock mass);
- Swelling (volume increase); and
- Slaking (volume change and deterioration due to changes in the physical environment, including freeze-thaw cycles and changes in the confining pressure, humidity, and moisture content.

Actual ground behavior often involves a combination of several of the types described above.

The maximum overburden or rock cover above the Cumberland Gap tunnels is approximately 750 feet. Uniaxial compressive strengths of the units in the tunnel were estimated to exceed the in situ stresses by more than a factor of six, the level at which stress-slabbing may begin to develop (*4*). Squeezing was not expected, because even lower strength-to-stress ratios are required to produce squeezing. No rocks with swell-susceptible minerals were expected, but the shales were expected to be susceptible to slaking. Thus, the types of ground behavior of concern on this project were expected to be structurally controlled loosening, due to the prominent bedding and jointing, and slaking in the shale units.

DESIGN OF THE INSTRUMENTATION PROGRAM

The design of the instrumentation program for the pilot tunnel evolved over the period from 1981 through pilot tunnel construction in 1986. Through this period, a number of key personnel changes occurred within the Federal Highway Administration (FHWA) and their consultants. The personal philosophies of the people involved often differed concerning the purposes and benefits of instrumentation on the project. Consequently, the actual instrumentation program differed considerably from the program planned at the beginning of construction, which, in turn, was radically different from that originally conceived in 1981.

At the outset of the design of the pilot tunnel, research into rock mass behavior, ground-support interaction, and advances in tunneling technology was envisioned as one of the primary objectives of instrumentation in the pilot tunnel. However, subsequent funding restrictions eliminated research as an objective in the pilot tunnel instrumentation, and the modified program was redirected to provide data for the design of the main tunnels.

In the modified program, it was decided to monitor ground displacements with respect to heading location and time, loads on steel rib supports, and groundwater flows and pressures. Construction plans for the pilot tunnel called for five instrumented test sections within the tunnel, plus instrumentation to measure crown and sidewall displacements at both portals. Tentative locations of the test sections were indicated in the plans, but the locations could be varied to provide data on the range of ground conditions encountered. Typical instrumentation planned at each test section is shown in Figure 2. In addition to the test sections and portal instrumentation, the plans also included the following devices:

- Convergence points in sets of four, as shown in Figure 2, at about 25 locations;
- Strain gauges to measure loads on steel ribs at 2 locations, with three instrumented ribs at each location;
- Weirs installed at 7 locations in the ditch carrying groundwater from the tunnel; and
- An automatic flow recording device at the portal.

The contractor was paid at the contract unit price for each instrument type and for standby time when tunneling or instrument installation activities were interrupted.

Experience has shown that instrument selection, if left to a low bid process, is often less than satisfactory (*6*). For this

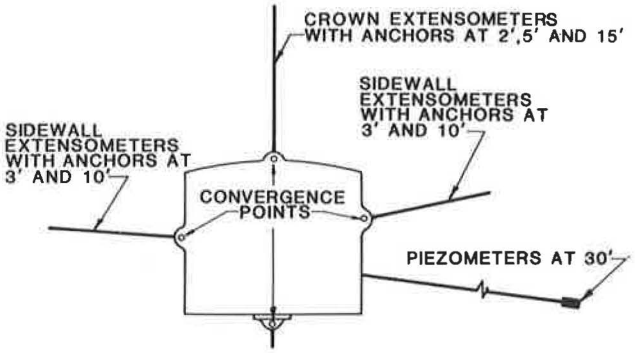

FIGURE 2 Planned instrumented test section.

reason, the procurement and assembly of instruments for this project were done through a geotechnical assistance contract between Golder Associates and the FHWA. With this arrangement, specific hardware with proven accuracy, reliability, and longevity was obtained from preferred sources and furnished to the construction contractor. This procurement process was successful in securing the contractor's cooperation and achieving high-quality installations. Although it was successful here and is recommended by the authors, this procedure requires aggressive management and a liberal interpretation of the acquisition regulations for federal projects.

In addition to the usual language on materials and construction procedures found in most instrumentation specifications, Table 1 was included as part of the contract. This table specifically shows the division of work and responsibility between the contractor and the engineer for each type of instrumentation used on this project. Experience in the pilot tunnel with this specification was satisfactory, and the authors recommend that no matter what procurement procedures are used, a table similar to Table 1 be included in any instrumentation specifications to assure that the division of responsibilities is clearly defined in advance.

TABLE 1 BREAKDOWN OF TUNNEL INSTRUMENTATION WORK

	Responsibilities	
Instrumentation	Contractor	Engineer
Strain gauge	Locate clean steel, and install gauges, sensors and leads Furnish, weld, and paint protective covers to gauges and leads	Furnish gauges, leads, readout unit, and junction box Do final electrical connectors
Stainless steel weir	Prepare site Supply mortar Maintenance cleaning Install weirs	Select location Provide weirs Check calibration
Automatic flow recording device	Furnish and install concrete pipe Install weirs and flow reading devices	Furnish weirs and flow reading devices Calibrate
Convergence measurement	Drill and clean hole Furnish rebar, epoxy, and invert covers Install eyebolts Install invert covers, protective pipe, and caps Furnish and install lockable metal enclosure in tunnel	Furnish eyebolts, protective pipes, and cap; tape extensometers; and calibration bar Indicate location and calibrate Take initial readings
Extensometer	Drill and clean hole Furnish drillers log, core boxes, grout, grout machines, protective covers, and locks Grout assembly in hole	Locate position, log core Furnish complete MPBX units and readout equipment Assemble in borehole Take initial readings
Piezometer	Drill and clean hole Furnish drillers log, core boxes, bentonite grout, and grout machine Grout instrument in hole	Locate position and log core Furnish piezometer, tubing, readout box, and accessories Assemble unit in hole Calibrate and take readings
Settlement inclinometer	Drill and clean hole Furnish drillers log, core boxes, grout, grout machine, protective covers, and locks Grout assembly in hole	Locate position Log cores Furnish complete downhole units and readout equipment Assemble in borehole Take initial readings

PREDICTED INSTRUMENTATION RESULTS

For structurally controlled behavior, displacements measured with extensometers and convergence points were expected to:

- Exceed the theoretical elastic displacement only by the amount required for loosening to mobilize the support capacity (i.e., very small displacements were expected);
- Be less than 0.5 to 1.0 in. at the tunnel periphery, unless a failure occurs (*5*); and
- Stabilize when the heading advances two to three equivalent tunnel diameters (20 to 30 ft) beyond the instrument location (i.e., no creep or long-term displacement, provided the support prevents further loosening).

The need for steel rib support was anticipated only near the portals, where the loading from structurally controlled loosening would be no larger than the overburden pressure. No sidewall pressures were expected.

EARLY PILOT TUNNEL EXPERIENCE

Excavation of the pilot tunnel started in December 1985 from the Kentucky portal. Support through most of the pilot tunnel is shown in Figure 3. Steel ribs (Figure 4) were installed at the portals and in short sections through the tunnel. Ribs were often used along with the rockbolt-shotcrete support shown in Figure 4. In some sections where bolt anchorage was questionable or not available, ribs provided the only ground support. Ribs were blocked to the rock using unseasoned hardwood; in some cases a thin layer of shotcrete (less than 1 in.) was applied to the blocking in the crown. As with most steel rib installations, blocking was irregularly spaced and sometimes loosened by subsequent blasts.

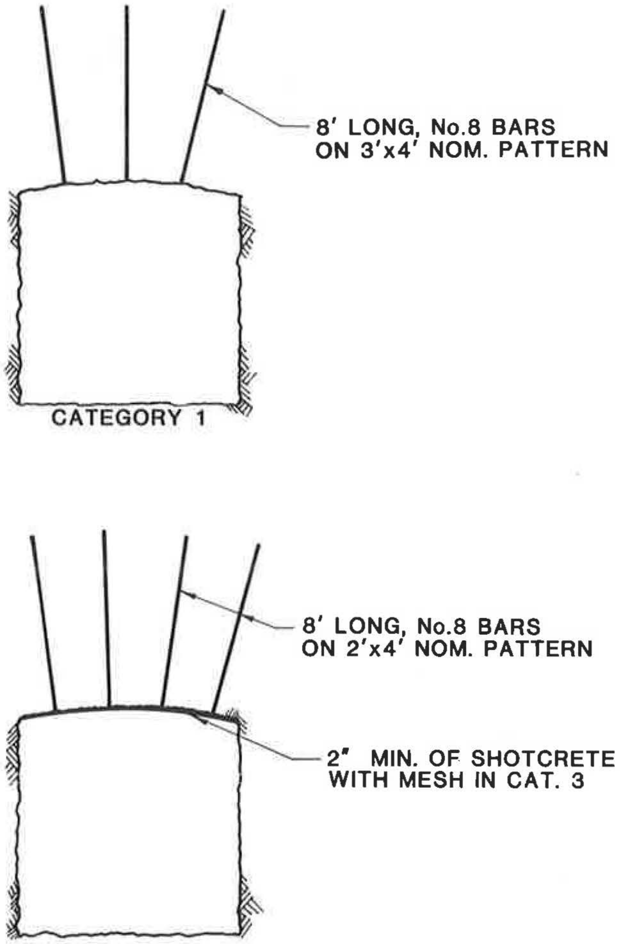

FIGURE 3 **Pilot tunnel support—categories 1, 2, and 3.**

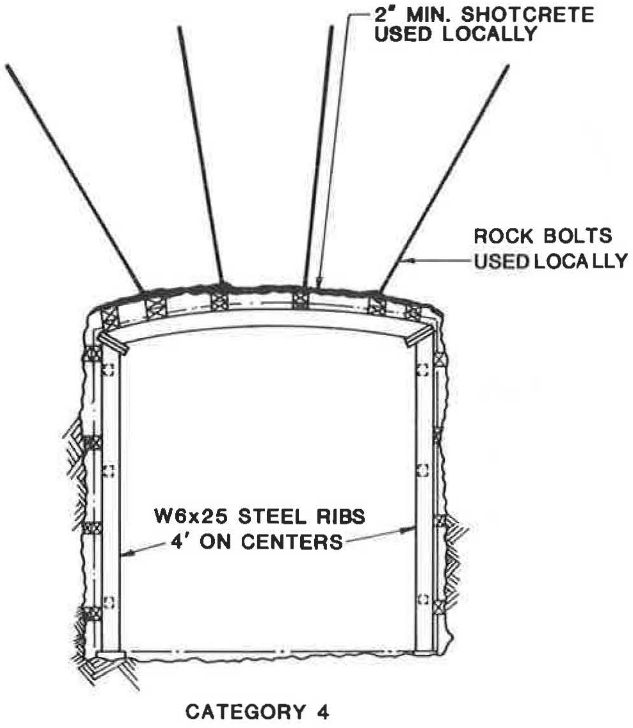

FIGURE 4 **Pilot tunnel support—category 4.**

As the excavation proceeded, the ground behavior was being evaluated regularly by both direct visual observation and instrumentation data. Once the excavation reached a point about 600 ft into the mountain, these data prompted a reevaluation of the instrumentation program. At that point, rock conditions ranging from a weak, sheared claystone to a thin-bedded sandstone to a relatively massive sandstone had been encountered.

Data were available from an extensometer and inclinometer at the portal, three convergence stations, and five instrumented steel ribs. Data on a thin-bedded sandstone from the portal extensometer showed about 0.1 in. of total crown displacement (see Figure 6). The inclinometer in the sidewall of the tunnel at the portal showed just under 0.2 in. of total lateral displacement toward the tunnel. Convergence data showed movements of 0.2 in. or less, except at one location where the convergence point was anchored in a sandstone block in the crown that rotated and settled on the blocking above the steel ribs. Strain gauge data on the steel ribs indicated less than 1 ft of rock load. All the instruments showed the movements or loads stabilized by the time the heading was about 30 ft ahead of the instrument location.

The data indicated that, although some inelastic movements occurred, they were small and stabilized quickly. This confirmed the initial expectation that the ground behavior would be structurally controlled. It was agreed that structurally controlled displacements and loads in the rest of the tunnel were likely to depend on small-scale, local geologic structures, and that instrumentation data would be of little use in quantitatively extrapolating to main tunnel behavior. For the remainder of the tunnel, it was agreed that instrumentation would be used to (1) verify structurally controlled behavior, and (2) monitor stability during construction.

The revised instrumentation program included a crown extensometer in a relatively massive sandstone unit and Tennessee portal instruments, as originally planned; strain gauges on representative steel ribs; and convergence stations in each major rock unit. The remaining crown and sidewall extensometers called for at each instrumented test section were eliminated from the plan. It was believed that the combination of convergence measurements and visual observations would provide sufficient data to evaluate ground behavior. If either of these indicated that the ground behavior might not be structurally controlled, more extensive instrumentation would be installed.

This revised program proved to be adequate for the remainder of the tunnel, except in the Chattanooga shale. Unlike the other rock encountered in the tunnel, the Chattanooga shale is highly fractured and sheared. Because of these factors, a crown extensometer was installed to allow a more thorough evaluation of ground behavior.

As a result of timely on-site interpretations, the total instrumentation cost was substantially reduced. The final instrumentation costs were only about 5 percent of the original estimate. Although much of this cost saving was due to lower than estimated unit prices, a significant portion was due to the lesser quantity of instrumentation installed. Major cost reductions were realized in reduced standby time and cost of installations, as well as reduced time for data collection, evaluation, and reporting of results, most of which would have been only slightly beyond the detectable level of the various instruments.

LOAD MEASUREMENTS ON STEEL RIBS

Geokon Model VSM-4000 vibrating wire strain gauges were used to monitor strain in selected steel ribs. This particular type of strain gauge can be welded to the ribs using conventional welding equipment. It proved to be reliable under the rough handling and adverse conditions of nearby blasting in the tunnel environment. Protective plates were installed, however, to shield the gauges against direct impact by miners' tools and flyrock. Instrumented ribs were installed in groups of two or three to minimize the effects of local variations due to blocking details and overbreak. Strain gauges were attached to the ribs before the ribs were used in the tunnel; instrumented ribs were installed less than 4 ft behind the heading.

The performance of the strain gauges was evaluated by monitoring 12 strain gauges over a two month period on one of the steel ribs while it was stored in the contractor's shop. The purpose of these measurements was to determine how much variation occurred in reading with no change in load. Initial readings were used as a reference value, and six subsequent readings were compared to the reference reading. The data showed an apparently random variation of about 10 microstrains, which corresponds to about one ton of thrust or about 4 in. of average rock load (*7*).

Of 120 gauges installed on 30 steel ribs, only 8 became inoperable during the 12 month monitoring period following installation. On one occasion, an instrumented steel rib was severely damaged by flyrock from the heading advance, rendering 4 of the gauges inoperable.

Approximately twice as many steel ribs were used for pilot tunnel support as anticipated in the original design. Short sections of poor ground conditions were more frequent than expected, and there were obvious differences in support bid prices, which made it advantageous for the contractor to recommend steel ribs rather than shotcrete support whenever possible. The data obtained from the strain gauges were used during the construction to verify that some contractor-installed sets were unnecessary and that the rockbolt-shotcrete support was adequate.

Groups of steel ribs were instrumented at five locations in the tunnel. Groups 1 and 5 were at the Kentucky and Tennessee portals, respectively. The other groups were located in some of the poorest quality rock encountered, to determine whether high loads were developing. Data from the instrumented steel ribs are summarized in Figure 5.

The rock loads shown in Figure 5 were calculated according to the method described by Proctor and White (*7*), by summing the loads measured in the two legs of the steel ribs. Except for one of the ribs in Group 3, all of the measured rock loads were less than 3 ft. Side pressures appeared to be negligible at all instrumented locations, but ravelling of weak rock in the side walls was frequently observed and invert struts were required at two locations to resist lateral pressures from the ravelled material that accumulated behind the ribs.

For comparison, Terzaghi's predicted rock loads (*7*) for a 10 ft by 10 ft tunnel are as follows:

Ground Description	*Road Load (ft)*	*Remarks*
Moderately blocky and seamy	2.5 to 7	No side pressure
Very blocky and seamy	7 to 22	Little to no side pressure
Squeezing rock, moderate depth, swelling rock	42	Heavy side pressure

The magnitude of the measured loads compared to Terzaghi's predicted loads confirm that the behavior was structurally controlled. Much larger rock loads and heavy side pressure would have developed in squeezing or swelling ground.

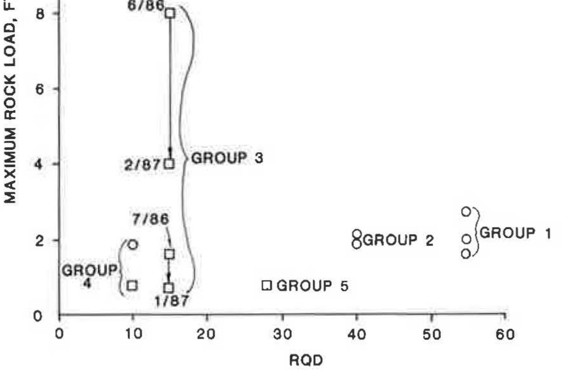

FIGURE 5 Summary of loads on steel ribs.

The loads shown in Figure 5 typically developed over a period of months, indicating structurally controlled blocks in the crown loosened over time and were supported by the ribs. The behavior of Group 3 was different from the other groups. The loads there increased to a peak in June and July 1986 and then decreased. Both the maximum and final measurements are shown in Figure 5. The anomalous behavior suggests the data for this particular group may not be reliable.

DISPLACEMENT MEASUREMENTS

Pilot tunnel convergence was monitored at selected locations using short resin-anchored eyebolts and Sinco Model 518115E/M tape extensometer. A laboratory calibration station was used to verify the accuracy of this measurement device. The attainable precision level of convergence measurements with this equipment in a tunnel construction environment was found to be only about ± 0.03 in.

Crown extensometers, installed from within the pilot tunnel, were used at only two locations. These instruments, Sinco Model 91800127, consisting of stainless steel rods connected to anchors of expandable copper bladders, were installed as close as possible to the heading in a 3 in. cored hole. As discussed previously, the first of these was installed early in the excavation in a relatively massive sandstone unit. The second was installed later in the Chattanooga shale, a highly fractured and sheared fissile shale unit, in an attempt to discern the depth of loosening beyond the excavation limit. Convergence points and extensometers installed from inside the tunnel were all installed less than 4 ft behind the heading.

At both portals, crown extensometers were installed from the surface prior to pilot tunnel excavation using groutable stainless steel rod extensometers (Sinco Model 518118). All extensometers were read from a reference head with a micrometer. Precision levels of ± 0.002 in. were generally attainable with these instruments, even with the physical difficulties of accessing the tunnel crown extensometers. As previously described, the portal extensometer results were useful in supporting the decision to eliminate many of the proposed tunnel crown extensometers. Figure 6 summarizes crown displacement at the Kentucky portal.

An attempt was made to measure both horizontal and vertical movement alongside the pilot tunnel using an inclinometer casing with telescoping couplings and a settlement probe developed by the U.S. Bureau of Reclamation. The precision of the settlement portion of this combined instrument was not high enough to reliably measure the small vertical movements that occurred. The telescoping couplings decreased the precision of the horizontal measurements somewhat, although useful horizontal movement data was obtained.

Displacement data from the pilot tunnel convergence stations and crown extensometers are summarized in Figure 7. For reference, theoretical elastic displacements are also shown in Figure 7. The elastic displacements were computed for a 10 ft diameter tunnel, plane strain conditions, Poisson's ratio of 0.3, and a ratio of horizontal to vertical stress of 1 (*8*). The rock mass modulus of 100,000 psi is estimated to represent a lower bound value for rock in the tunnel; the 1 million psi value is considered representative of the thin-bedded jointed sandstone units encountered. With the exception of the two portals, the reported displacements and convergences were measured after the heading passed the instrumented station. These instruments were installed as soon as possible after excavation and within about 2 ft of the heading. Thus, the reported displacements and convergences do not include movements that occurred prior to the installation (as the heading approached the instrumented section). Consequently, they are, by definition, not directly comparable to the theoretical elastic displacements.

Figure 7 indicates actual tunnel displacements were on the same order as the estimated elastic displacements, except for

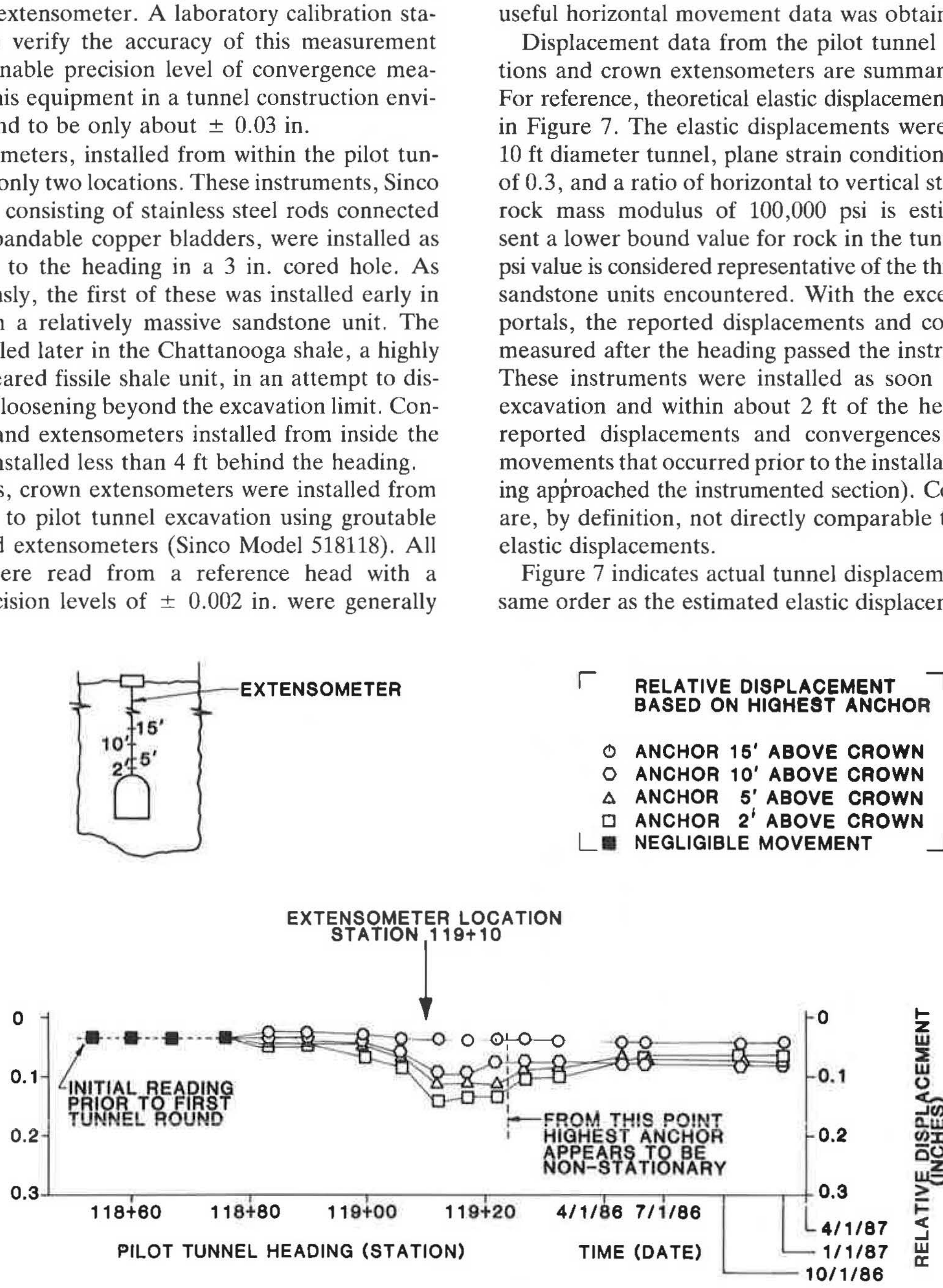

FIGURE 6 Crown displacement at Kentucky portal.

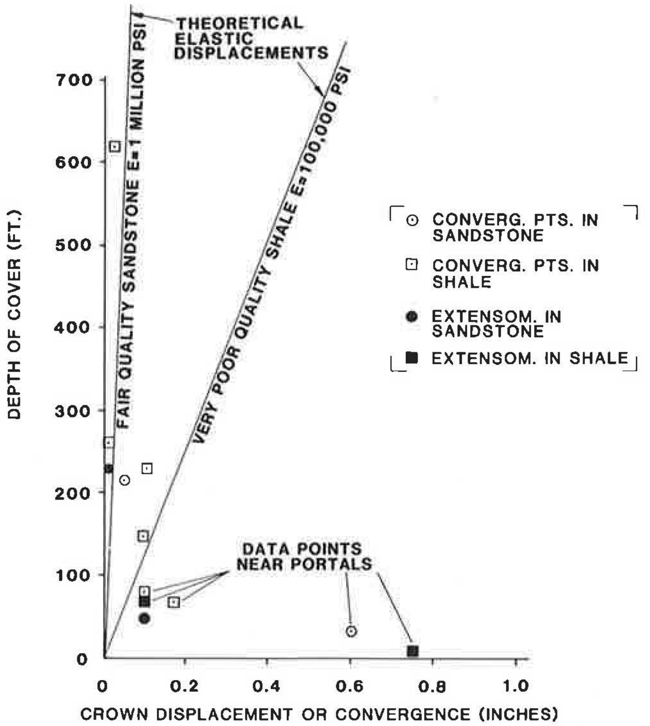

FIGURE 7 Summary of theoretical and measured displacement data for sandstone and shale.

two points at the portals. At the portals, structurally controlled loosening apparently occurred above the rockbolts. With all the data, displacements essentially ceased by the time the heading advanced 30 ft beyond the instrument. Examples of the displacements as the heading advanced are shown in the plot of extensometer data in Figure 6 and an example of the convergence data is shown in Figure 8. The very small measured displacements indicate that little or no loosening occurred in the rock mass surrounding the tunnel. This, coupled with the observation that the displacements stabilized rapidly, further confirms that the ground behavior was structurally controlled.

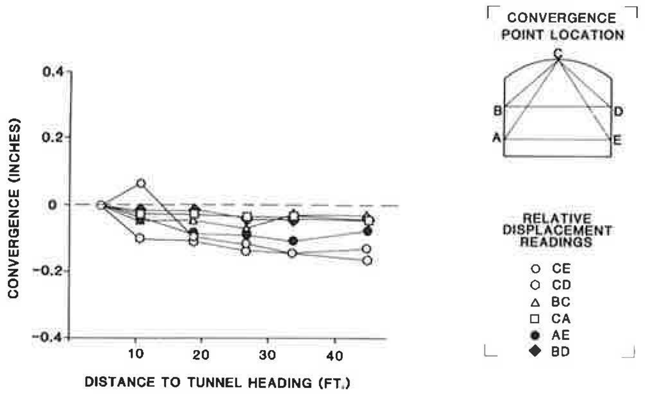

FIGURE 8 Tunnel convergence data for Instrument Station 142+88.5.

GROUNDWATER FLOW AND PRESSURE MEASUREMENTS

Environmental considerations within the National Historical Park did not allow surface access above the tunnel for any significant distance beyond the portal excavations. Yet some assurance was required that drainage into the tunnel would not dramatically affect the overall groundwater regime. Some of the sandstone units exhibited large secondary permeability (fracture flow) resulting in substantial inflow into the pilot tunnel. Some method of predicting the main tunnel inflow was desirable.

A series of V-notch weirs was therefore installed throughout the pilot tunnel to measure the quantity of inflow over various lengths. Data from these simple and inexpensive devices have shown that inflows vary somewhat during local rainfall events, due to direct hydraulic connections to the surface through solution cavities in the limestone. However, the overall inflow has remained surprisingly constant over the 3 years since the tunnel construction was completed. The original contract documents included another V-notch weir to measure immediate inflows relatively close to the advancing heading. This proposed installation proved to be impractical and was deleted shortly after excavation began.

Piezometers (Geokon Model 4500) were installed from the pilot tunnel in two sandstone units in an attempt to discern the pressure distribution in these units within the influence of the pilot tunnel. Relatively constant heads have been measured over the 3 year period since piezometer installation, despite considerable variation in rainfall conditions during the period.

Data from the piezometer installations were used to calibrate flow nets and estimate drawdown around the pilot tunnel. These results and the measured inflow data were used to estimate main tunnel inflows, the impacts of the main tunnels on the groundwater system, and groundwater pressures on the main tunnel lining. From these analyses, it is expected that the main tunnels will have a local effect on the groundwater system, but they will not have an adverse environmental effect.

PLANS FOR MAIN TUNNELS

Excavation of the main tunnels will start in mid-1990. The current design is shown in Figure 9. From the experience gained in the pilot tunnel, the ground behavior is expected to involve structurally controlled loosening and slaking, and the support has been designed accordingly. The instrumentation planned for the main tunnels is minimal and includes (1) extensometers at the four portals, where the cover is relatively low compared to the tunnel diameters, and (2) a few convergence points to monitor the behavior of particular structural blocks in the Chattanooga shale where the rock is highly fractured. Visual observations of shotcrete lining behavior will be used to identify sections requiring additional support.

RECOMMENDATIONS FOR INSTRUMENTATION FOR PILOT TUNNELS

As with any instrumentation program, clear objectives and an understanding of the types of ground behavior that are

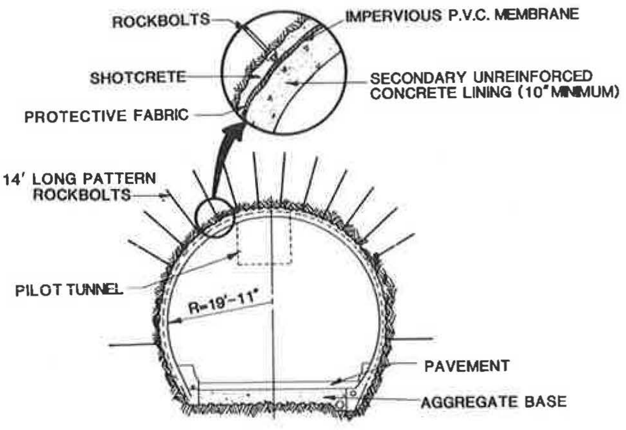

FIGURE 9 Design cross-section of southbound main tunnel.

expected should precede the development of an instrumentation program (*6*). In establishing objectives, available models for extrapolating pilot tunnel measurements to main tunnel ground behavior and support design must be considered.

In general, for the five types of ground control (*3*), the following types of instrumentation programs are considered appropriate:

- *Structural control.*—If structural control is the dominant type of ground behavior, little direct benefit can be gained from an extensive instrumentation program. The authors are unaware of models that would allow displacements and load measurements in a pilot tunnel to be used to quantitatively estimate main tunnel displacements and support requirements. Instrumentation should be limited to providing sufficient data to verify that the ground behavior is structurally controlled and to provide the monitoring needed for construction safety.
- *Stress slabbing, squeezing, and swelling.*—Significant design data can be obtained from instrumenting pilot tunnels where the ground behavior involves stress slabbing, squeezing, or swelling. The distribution of movement in the rock mass and the rates of movement with respect to time and the excavation process are useful in evaluating which type of behavior is dominant. Models are available for translating displacement and load measurements in the pilot tunnel into estimates for the main tunnel (*9,10*) and, therefore, for designing appropriate rock support.
- *Slaking.*—Visual observations of behavior are more appropriate than a tunnel instrumentation program for evaluating slaking.

REFERENCES

1. W. R. Sullivan and R. M. Leary. Benefits of the Pilot Tunnel on the Cumberland Gap Highway Project. *Proc., Rapid Excavation and Tunneling Conference*, New Orleans, La., 1987, pp. 3–20.
2. R. C. Hopper, T. A. Lang, and A. A. Mathews. Construction of Straight Creek Tunnel, Colorado. *Proc., First North American Rapid Excavation and Tunneling Conference*, Chicago, Vol. 1, Society of Mining Engineers, 1972, pp. 501–538.
3. E. J. Cording. State-of-the-Art: Rock Tunneling. *Proc., GEOTECH '84*, Atlanta, ASCE, New York, 1984, pp. 77–106.
4. E. J. Cording and J. W. Mahar. Index Properties and Observations for Design of Chambers in Rock. *Engineering Geology*, No. 12, 1978, pp. 113–142.
5. Shannon & Wilson, Inc. *Geotechnical Instrumentation Applications for Pilot Tunnels*. FHWA, U.S. Department of Transportation, Washington, D.C., 1982.
6. J. Dunnicliff. *Geotechnical Instrumentation for Monitoring Field Performance*. John Wiley & Sons, New York, 1988.
7. R. V. Proctor and T. L. White. *Rock Tunneling with Steel Supports* rev. ed. Commercial Shearing Inc., Youngstown, Ohio, 1977.
8. E. Hoek and E. T. Brown. *Underground Excavations in Rock*. Institute of Mining and Metallurgy, London, 1980.
9. E. T. Brown, J. W. Bray, B. Ladanyi, and E. Hoek. Ground Response Curves for Rock Tunnels. *Journal of Geotechnical Engineering*, Vol. 109, No. 1, 1983, pp. 15–39.
10. R. M. Semple, A. J. Hendron, and G. Mesri. *The Effect of Time-Dependent Properties of Altered Rock on Tunnel Support Requirements*. Federal Railway Administration U.S. Department of Transportation, 1973.

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