Solway Tunnel rehabilitation

La récupération du tunnel de Solway

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SYNOPSIS The severely distressed concrete lining of a railroad tunnel in eastern Tennessee was stabilized using steel struts, soil and rock tiebacks, mini-piles and deadman anchors, while daily traffic was maintained on the heavy tonnage mainline. The invert of the horseshoe-shaped tunnel had been lowered about twenty years ago and this disturbance resulted in settlement of half of the lining cross-section, thus creating a failure in the concrete along the springline. Over the years, the lining experienced almost 30cm of convergence at the springline. The repairs were designed using the observational approach whereby the design stresses were computed based on the loads necessary to cause the tunnel lining to move inward and actual in-situ stresses were measured based on the tieback loads necessary to reverse the convergence. The stabilization system is providing a long-term support mechanism at a substantial cost savings to the railroad, as compared to other alternatives that were examined.

INTRODUCTION

The Solway Tunnel is an 805m long, concrete-lined, horseshoe-shaped tunnel constructed north of Knoxville, Tennessee in the late 1800's. As was typical for that era, the original tunnel lining consisted of timber ribs and lagging. During the early 1900's after the railroad began generating revenue, a 0.6m thick concrete lining was installed inside the original timber lining.

The tunnel is located in the Valley and Ridge Physiographic Province, specifically in the Copper Ridge Formation. The bedrock in the tunnel area is soluble, massive dolomite containing chert nodules. It weathers to produce a thick, residual soil cover composed of highly plastic, cherty, silty clay over a very irregular bedrock surface. Generally, the residual soil is overconsolidated near the ground surface, but has increasing moisture content and decreasing shear strength with depth. Locally, the overburden can consist of loose, angular chert fragments with wet, soft clay in the voids.

More than twenty years ago, the tunnel invert was lowered about 30cm to provide increased clearance for larger railroad cars. Soon thereafter, the foundation of a 26m long section of the western half of the tunnel lining settled and the resulting decrease in horizontal stress on the sidewalls caused flexural stress cracks to develop at the springline. This cracking reduced the shearing resistance of the concrete lining and, ultimately, created a shear failure in the concrete along the springline. Further, the settlement resulted in a loss of support at the bottom of the arch and the arch settled at essentially the same rate as the foundation.

After the shear failure, the western sidewall converged and the earth pressures were temporarily reduced, thereby averting collapse. However, considering the soil's plastic nature, long-term creep resulted in continued reloading of the sidewall. As the convergence of the sidewall continued, the arch eventually became wedged behind the sidewall. The western sidewall converged more than 30cm, with a maximum offset at the springline of about 20cm as shown by Figure 1. The eastern wall of the tunnel lining did not indicate any convergence.

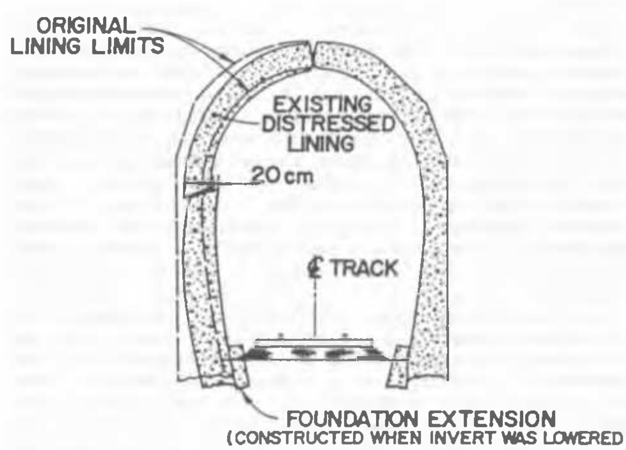

Figure 1. Existing Conditions - Looking North

The tunnel was located on an active mainline (sometimes in excess of ten trains per day). Moreover, any detour would require a several hundred kilometer bypass. Therefore, the tunnel had to be repaired under traffic and the integrity of the lining had to be assured at all times. The distressed portion of the tunnel was in a deeply-weathered, soil-filled slot within the ridge. The earth pressures on the tunnel lining, as computed by conventional methods, were very high. The typical repair method (removing the lining in sections and replacing it with curved, structural steel struts) was not only very expensive but impractical. The large, wide-flanged members which were required could not be rolled into a curved shape; a series of short chords would be necessary around the arch. Clearly, a unique and innovative technique was warranted to effect the required stabilization. This paper discusses how the earth pressures were analyzed and the in-situ stabilization of the structural support system was developed.

GEOTECHNICAL DESIGN CONSIDERATIONS

Review of literature regarding soft ground tunnelling indicated that the at rest lateral earth pressure exerted by plastic soils should range between 60 and 90 percent of the vertical pressure (Terzaghi, 1943). Considering the creep phenomenon, the lateral earth pressure of the plastic soil could be as great as the vertical pressure (Tschebotarioff, 1973). However, structural analyses indicated that if the lateral earth pressure coefficient had indeed been close to one, the lining would have been over-stressed in bending and would have collapsed. A computer model of the tunnel lining was developed to back-calculate lateral earth pressure coefficients for assumed safety factors against bending failure. The earth pressure coefficient was calculated to be no greater than 0.5 for a safety factor of one.

These back-calculated earth pressure coefficients were significantly lower than the minimum earth pressure coefficients typically recommended for underground construction in these types of soils. However, the tunnel lining had performed satisfactorily for over 50 years, thereby justifying the back-calculated values. The initial repair design was based on the lower earth pressure coefficient, thus resulting in a large design economy. Further, it was important to make the repairs as quickly as possible in order to minimize the disruptions to train traffic. Accordingly, the authors believed that the observational design approach (Terzaghi and Peck, 1948) was appropriate for the final design of the repairs.

It was originally planned to increase the structural capacity of the tunnel lining by 50 percent, thus providing a safety factor of 1.5. However, during the design phase, the philosophy was adopted that if the tunnel was quasi-stable in its deteriorated condition, a less conservative safety factor might be in order. Therefore, a monitoring program was instituted during the investigative phase which allowed for a more detailed assessment of the lining performance. It was believed that the monitoring data coupled with stress-strain information on the stabilization system would allow accurate ongoing monitoring of the tunnel lining performance. Accordingly, the initial repair design was arbitrarily reduced to 60 percent, with the provision that additional stabilization measures could be instituted, if necessary.

INVESTIGATION AND MONITORING PROGRAM

The investigation consisted of coring through the tunnel lining to verify the concrete thickness and strength, as well as sampling of the materials behind the lining. During this investigation, a monitoring program was developed to establish the existing lining convergence rates. Convergence points were installed at locations within the distressed area and at control locations beyond either end of the repair area. As shown by Figure 2, the convergence points were recessed into the lining to avoid damage by passing trains.

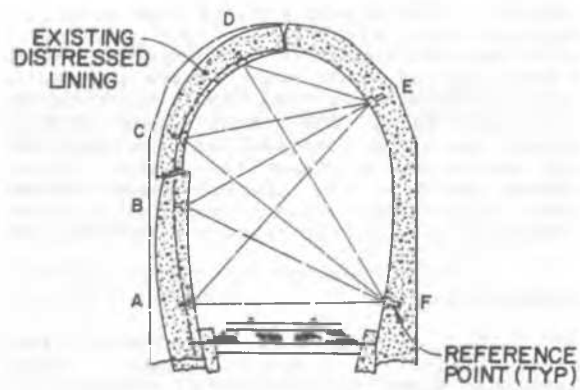

Figure 2. Convergence Points Configuration

A computer program was developed to reduce the triangulation measurements between the points to vertical and horizontal components of movement. The lining performance could be better assessed as it related to settlement or lateral movement. Moreover, the monitoring program served as an early warning system and helped to avert a potentially dangerous situation during construction. The changes in the convergence rates coincided with construction milestones, as indicated in Figure 3.

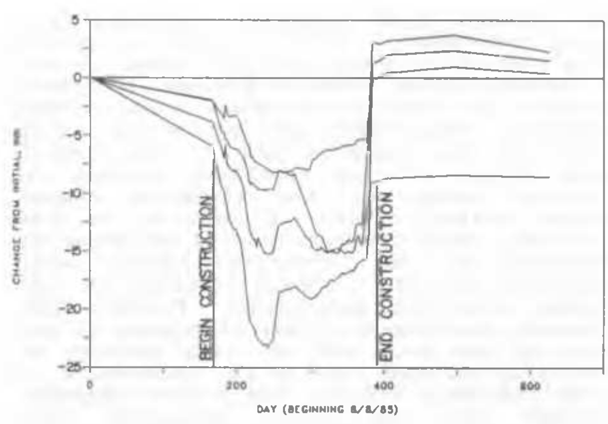

Figure 3. Lateral Movement of Tunnel Lining

CONSTRUCTION REPAIRS

Structural Steel Struts

The most straightforward portion of the repair program consisted of installation of structural steel struts across the bottom of the tunnel. When the distress had first become evident, railroad personnel installed timber struts beneath the cross-ties as a stop-gap measure to retard the convergence. In order to provide lateral support at the base of the sidewalls, railroad forces replaced the timber struts with specially designed structural steel struts. The struts were designed with eccentric end-bearing plates to bear directly against the base of the lining, as shown in Figure 4.

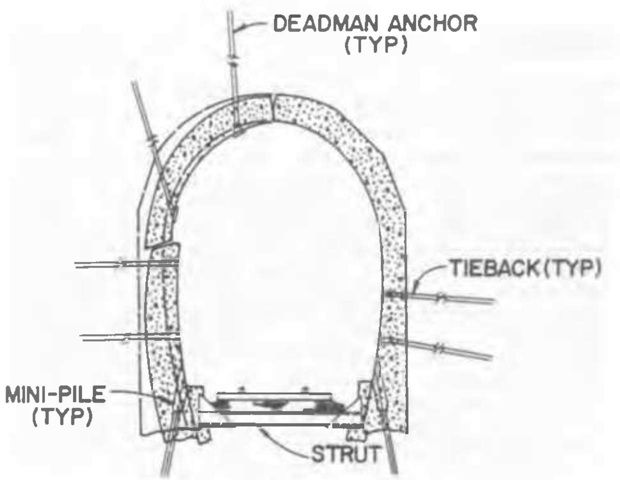

Figure 4. Typical Repair Scheme

Soil and Rock Tiebacks

The sidewalls of the tunnel lining were restrained with fifty-seven soil and rock tiebacks, with design loads of 267, 400 and 480kN (Figure 4). The typical tieback bearing plate was 248 MN/m² steel and measured 30cm by 40cm by 0.3cm. However, at some locations, the deteriorated lining experienced a punching shear failure. Field decisions were necessary to redesign the plate dimensions and to utilize 90cm by 90cm by 0.25cm bearing plates in these areas.

Many of the soil tiebacks were bonded in firm to stiff clays with varying amounts of chert fragments, in which a design bond capacity of 69 to 138 kN/m² was reasonable (Schnabel, 1982). Where practicable, the tiebacks were bonded in bedrock. The ultimate bond stress in the weathered dolomite was 689 to 1724 kN/m² (Littlejohn and Bruce, 1977). Therefore, using an average value for bond stress and a safety factor of two, a reasonable design bond capacity of 620 kN/m² was used. This resulted in a bond length of 3m for a 480kN anchor.

The highly variable subsurface conditions made selecting a drilling method difficult. The bedrock formations were riddled with clay-filled solution cavities and where bedrock was not encountered, chert boulders were present in the overburden. Initially, the lining concrete was cored at the anchor locations. Thereafter, a rotary drill rig mounted on an adjustable working platform advanced a 15cm diameter augered hole to refusal. If the hole was too short to install a satisfactory soil anchor, an 8cm diameter hole was drilled an additional 3m into bedrock.

Upon completion of the drilling, an epoxy-coated, 3cm diameter, 1034 MN/m² steel bar was inserted into the hole in 3m sections with couplings. The bar in the unbonded zone of the anchor was covered with a polyethylene sheathing filled with grease. Also, special coupling housings were used to insure the free movement of the bar in the 6m to 9m unbonded lengths. Since the ground water table in the area was well below the tunnel invert, water-tightness grouting and redrilling was unnecessary.

Three percent of the tiebacks were performance tested to 150 percent of the design load. The initial load test included cyclic loading to check the behavior of the anchors within the unbonded zone. The final load was held for two hours or until the creep measured was less than 0.8 mm/hour. Proof tests were performed on all of the anchors to 120 percent of the design load.

"Lift-off" tests were performed on the tiebacks at about six months after construction repairs were completed. Less than 20 percent of the tiebacks required re-tensioning; the majority of these tiebacks were bonded in soft soils. In fact, the anchor with the lowest load was about 70 percent of the original "lift-off" load. The magnitude of the lining movements indicated that a majority of the unloading was due to the reversed convergence of the tunnel lining.

Deadman Anchors

The vertical arch support of the tunnel lining was to be provided by upward-angled soil anchors installed from inside the tunnel. However, an alternate proposed by the contractor was approved whereby 400 and 667 kN anchors were drilled and installed from the ground surface approximately 27m above the tunnel crown (Figure 4). The deadman anchors were easier to install than the tiebacks and eliminated the potential for creep in the anchors which would have otherwise been bonded in the plastic soils overlying the tunnel. At each location, an epoxy-coated, 3cm diameter, 1034 MN/m² steel bar was lowered down a cased hole extending from the deadman footing constructed at the ground surface to the tunnel lining arch. Thereafter, bearing plates were attached and the bar was stressed from the ground surface. To evenly distribute the loads and span local discontinuities in the lining concrete, 30cm wide steel straps were used between adjacent deadman anchors. The total bar length was unbonded and after stressing, the casing was filled with grout.

The contractor planned to install the cased holes at the beginning of the job. These holes were to be used as access from the ground surface to the tunnel for electric, air, radio and grout lines. Some time after the job commenced, the deadman anchors would be installed and stressed. However, the lining convergence rates were accelerated during the drilling of the access holes and that created concern regarding the immediate stability of the tunnel. To reduce the movement, the drilling was halted and temporary bars and bearing plates were installed in the access holes and the stress crack along the springline was reinforced with rock bolts. Thereafter, cased holes were advanced at the remaining deadman anchor locations to be utilized for access.

Mini-Piles

A total of 33 mini-piles with design loads of 490 and 534kN were installed to provide vertical support to the tunnel walls (Figure 4). At least one third of the piles were installed before any tieback stressing or grouting of voids behind the lining was performed. This precaution was taken to prevent additional tunnel settlement that could occur as a result of the anchor loads or grout loads.

Initially, the lining concrete was cored near the base of the sidewall at each pile location. Afterwards, a rotary percussion drill rig was used to advance a heavy-duty, (Schedule 120) 13cm outside diameter steel pipe a minimum of 0.6m into bedrock. Thereafter, an 8cm diameter hole was advanced 4.5 m beyond the pipe casing. A 3cm diameter, 1034 MN/m² steel bar was lowered to the bottom of the hole and the hole was filled with neat cement grout to within a few centimeters of the tunnel wall. The pipe casing was cut off below the tunnel wall and a dowel was placed in the upper portion of the casing, extending up into the tunnel wall. Thereafter, the cored portion of the hole was filled with non-shrink grout. Selected mini-piles were tested to 175 percent of the design load prior to grouting the cored portion of the hole.

In a portion of the tunnel, the mini-piles did not encounter a competent bearing stratum within a reasonable distance. In this area, the deadman anchors were evaluated and were determined to provide sufficient vertical load-carrying capabilities.

SPECIAL CONSTRUCTION CONDITIONS

As previously discussed, frequent train traffic was maintained on the active mainline track while the tunnel was being repaired. Each morning, the railroad flagman informed the contractor of the scheduled track time for that day, thereby establishing the curfew. The railroad attempted to provide the contractor with a minimum of four continuous hours of track time per day. In order to minimize disturbances to train traffic, it was of extreme importance that the track be cleared by the scheduled time each day.

At the beginning of curfew each day, a crane placed the equipment on the track at the staging area near the tunnel entrance. Railroad push cars carried the equipment approximately 300m into the tunnel to the distressed area. The tandem push car assembly occupied the majority of the 26m long work area. Careful planning was required to keep the work areas separated for each rig.

The ongoing convergence had reduced the train clearance within the tunnel to the extent that trains would scrape the tunnel walls within the distressed area. Therefore, nothing could extend beyond the tunnel lining at the end of curfew. Partially-drilled holes had to be abandoned and all protruding drill tools or anchor bars had to be removed prior to leaving the tunnel. In addition, these clearance restrictions required that some of the anchor bearing plates be recessed into the tunnel lining. Extensive chipping of the lining concrete was required at the bearing plate locations to provide the recess.

CONCLUSION

This project is a good example of the observational method of construction. There are several essential elements to the successful stabilization using this design approach. First, the owner must be willing to recognize the risk associated with this approach from a performance and financial standpoint, in exchange for the substantial cost savings that can accrue when repairs are completed in accordance with the design plans. Second, the consultants must be innovative and non-conservative, notwithstanding the potential liability which could accrue. Finally, a knowledgeable specialty contractor must be selected who is willing and able to adapt to field changes.

The maximum pre-construction convergence rates approached 13mm per year. Construction disturbances accelerated these rates and in some instances, an additional convergence in excess of 25 mm was measured. However, the monitoring program indicates that the repaired lining has experienced a reversed convergence and the current convergence is negligible. Therefore, the tieback system is providing an excellent long-term support mechanism at a significant cost savings to the railroad, compared to the conventional engineering approach involving segmental removal and replacement of the lining. Additionally, the tiebacks may be used as a temporary bracing system at some later date should the railroad elect to remove and replace the lining to increase clearance.

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