BURLINGTON TUNNEL EMERGENCY REPAIRS: A CASE HISTORY

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ABSTRACT: The Burlington Tunnel is a 104-m- (340-ft-) long 4°curved tunnel constructed in the 1860s below a busy city street in Burlington, Vermont. A portion of the tunnel's brick lining had experienced significant distress due to age and weathering and required relining with 150 mm (6 in.) of shotcrete. During lowering of the tracks to compensate for the loss of clearance from the relining, serious tunnel lining stability problems developed. The lower portion of the tunnel wall rotated inward and significant longitudinal cracks developed as the railroad's contractor worked to lower the tunnel invert. The railroad quickly took emergency action to stabilize the tunnel with a ready-mix concrete brace slab at the invert to provide temporary stability until a permanent repair could be made. Subsequently, ECI Rail Constructors was engaged by RailAmerica to assess the tunnel, and design and construct the permanent repairs. This case history will review the project details including information regarding problems experienced during the original 1860's construction, summarize the design parameters for the recent permanent repair, describe the lining movements of the tunnel, and summarize the lessons learned.

INTRODUCTION

The Burlington Tunnel, built in the 1860's, is 104 m (340 ft.) long with a 4° horizontal curve and is considered to be of national historic significance because it is one of the earliest railroad tunnels in the United States (DMJM Harris, 2002). As indicated in Figure 1, the tunnel is aligned approximately east-west and passes under North Avenue. The tunnel is currently owned and maintained by the New England Central Railroad (NECR), a short-line railroad owned by RailAmerica, and it serves as a critical rail link between the NECR and the neighboring Vermont Railway. Rail traffic is typically two or more freight trains per day. The tunnel is located at Mile Post 1.15 on NECR's Burlington Branch. The tunnel clearance is rated for a Plate C car.

The tunnel is brick-lined with about 18.0 m (60 ft.) of overburden. The tunnel configuration is elliptical horseshoe shape as depicted in the cross-section shown in Figure 1. This original brick lining is six wythes (layers) thick and constructed with mortared joints. The brick lining is supported on cut limestone blocks (henceforth referred to as wall footings) and prior to the 2008 shotcrete work, a cobblestone floor served as horizontal bracing between the footings supporting the walls. The ballast and wood cross-tie track structure had been supported on the cobblestone floor. The portals consist of mortared cut stone with wing walls of similar construction. Figure 2 shows a photo of the west portal.

In recent years, a portion of the tunnel's brick lining had experienced significant distress due to age, weathering, surface runoff seepage, and cyclic freeze/thaw effects. Several large areas (up to 100 square ft., or more) of the walls and ceiling had lost up to three wythes of the brick lining. Figure 3 shows a photo of one of the distressed areas in 2007. The distress appears to have developed from the weathering of the mortared joints in the brick lining which caused the cement between bricks to deteriorate, leach and erode away, leaving behind sand, or voids in the joints. Eventually, the bricks fell out. The distressed areas were

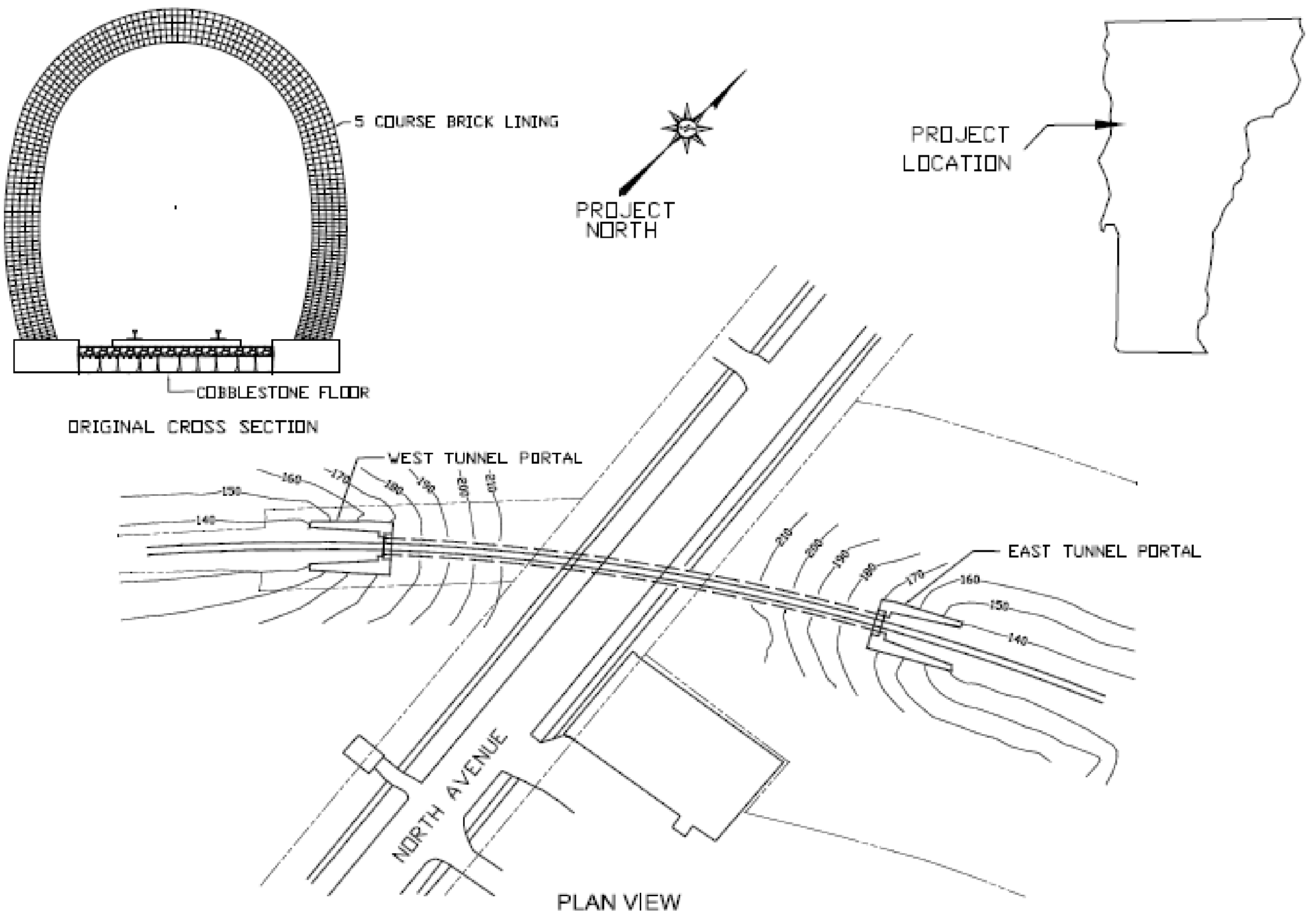

Figure 1. Project location, plan view and original cross-section

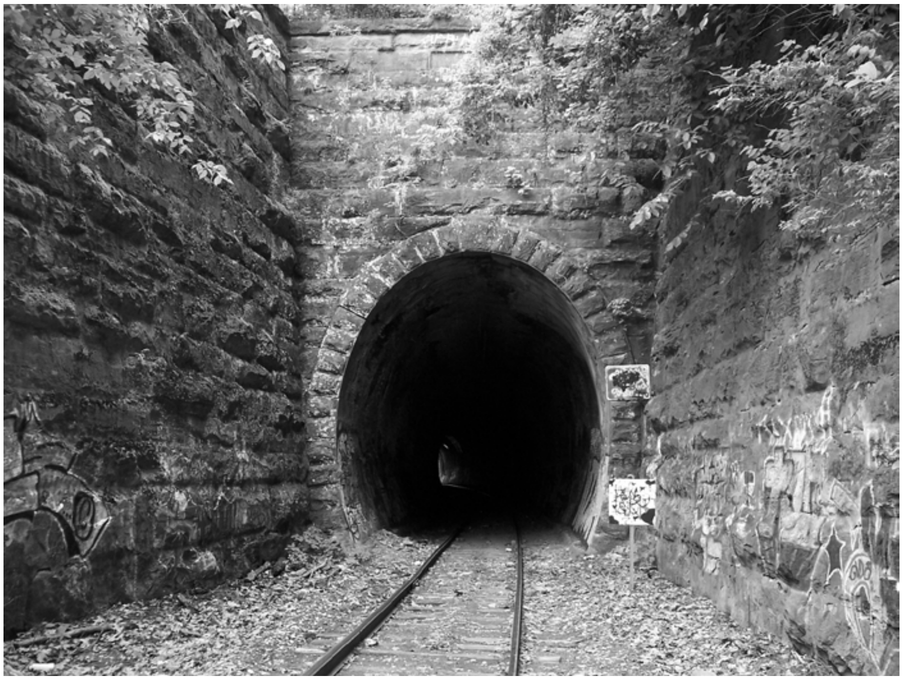

Figure 2. West portal in 2007

Figure 3. Deteriorated brick lining in 2007

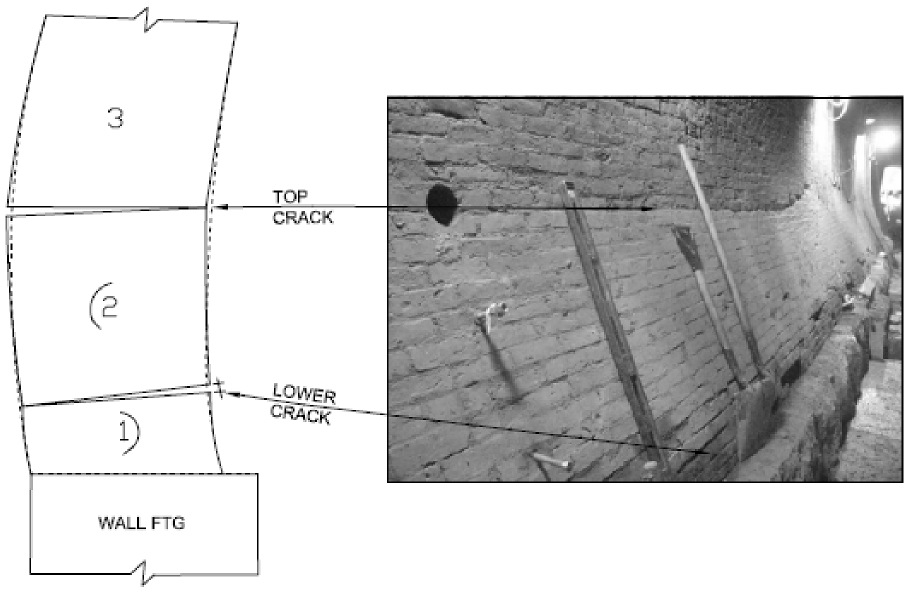

Figure 4. South wall cracking

concentrated near the portals, approximately the first 7.5 to 15 m (25 to 50 ft.), apparently related to the greater number of freeze-thaw cycles and amount of surface runoff related seepage that tends to occur at the ends of the tunnel. The lining temperatures further inside the tunnel vary less in response to the daily external ambient temperature fluctuations and solar warming compared to the lining temperature near the portals.

Other than the loss of bricks from the lining, there were no signs of significant movement of the tunnel lining, portals, or wing walls. Previous repairs to the lining included: insertion of concrete masonry unit blocks, bolting, gunite and mortar patches, and thin coatings of mortar had been performed in past years. Some of these previous repairs have failed or experienced significant distress.

In 2008, RailAmerica began to consider making repairs to the most severely spalled areas of the tunnel lining. Upon securing public funding, the scope was expanded to a permanent fix with 150 mm (6 in.) of shotcrete for the entire tunnel lining. However, adding a 150-mm- (6-in.-) thick layer of shotcrete also required that the tracks be lowered to compensate for the loss of clearance from the relining. The only engineering documents of the tunnel construction were a 1913 Valuation Sketch of the tunnel which indicated there might be a paved floor. Subsequent field investigation and potholing found minor stones which by size and nature were assumed to be construction debris from the original construction. Accordingly the decision was made to proceed with the track lowering and shotcrete lining. As the work progressed, the contractor encountered what we now know to be the brace floor which had been buried under the more than 150 mm (6 in.) of ballast under the ties. In addition, while the overburden is generally sand in nature, the contractor encountered zone of expansive clay that was not identified during the exploratory excavations. Serious stability problems for the foundation stones soon developed as the contractor operated equipment over this section of exposed subgrade and caused significant cracking of the brick lining.

As indicated in Figure 4, the lower portion of the tunnel wall fractured in two locations and each section rotated in opposite directions forming two distinct longitudinal cracks. The upper crack (about 1.07 m (42 in.) above the wall footing) was a pinching condition at the lining surface while the lower crack (about 230 to 300 mm (9 to 12 in.) above the wall footing) opened up to about 10 mm (3/8 in.) wide at the surface of the lining. The cracks extended longitudinally though the middle third of the tunnel (where the subgrade was also disturbed) and on the south side only (inside curve). One hundred-mm- (4-in.-) diameter cores into the lining confirmed that the cracks extended through the entire lining depth.

We surmise that the wall movements were likely the combined result of: 1) the removal of the cobblestone floor which acted as a brace at the tunnel invert, 2) the disturbance and vibration of the saturated subgrade from heavy equipment which reduced the soil/footing interface friction and passive soil resistance below the floor that helped to stabilize the tunnel, and 3) heaving and lateral displacement of the disturbed soil below the wall footing as the equipment moved through the tunnel multiple times. Cause #3 most likely resulted in observed rotation of the intact lining sections and crack arrangements (Figure 4), which cannot be explained by lateral thrust movements alone.

The railroad undertook immediate emergency action to stabilize the tunnel with an unreinforced ready-mix concrete brace slab between the wall footings at the tunnel invert to provide temporary stability until a permanent repair could be designed and implemented. The emergency slab was poured only over the middle third of the tunnel where the crack and disturbed subgrade was observed. Subsequently, ECI Rail Constructors was engaged by RailAmerica to assess the tunnel and design and construct the permanent repairs. Boscardin Consulting was hired by ECI to assist in the assessment and to provide technical recommendations. In true railroad expedited fashion, the entire repair including the shotcrete operation was completed in 2 months. The timeline of events is summarized in Table 1.

GEOLOGY

The Burlington Tunnel, also known as the North Avenue Tunnel, is located on the north side of Burlington about 370 m (1,200 ft.) from the eastern shore of Lake Champlain in the physiographic province known as the Champlain Lowlands (Doolan, 1996). The tunnel passes through a 30-m- (100-ft.-) high ridge composed of loose, fine to medium wind-blown sand associated with deltaic deposits that developed in the marine environment of the Champlain Sea (Wright et al., 2009). The Champlain Sea was a saline precursor to the current freshwater Lake Champlain (Wright, 2003). The sand resulted a running ground condition that was reportedly encountered during the original construction (Chittenden County Historical Society Bulletin, 1996). The zone of clay encountered during the repairs is likely varved clay associated with Glacial Lake Champlain.

Table 1 Timeline of Events

Date	Event
15 September 2008	Railroad installed emergency concrete brace installed to stabilize the tunnel
15-22 September 2008	ECI assessed tunnel condition, test cored lining, installed survey
20 September 2008	ECI milled up to 460 mm (18 in.) off emergency concrete brace to elevation suitable to pass equipment
Mid September –Mid October 2008	Johnson Western installed shotcrete
14-27 October 2008	ECI removed emergency concrete brace in segments and installed new concrete base slab
Late October 2008	ECI installed an 5.5-m- (18-ft-) long by 250-mm- (10-in-) thick approach slab at both portals
27 October – 3 November 2008	ECI constructed rigid track: set steel cross-ties, set rebar, installed rail, aligned track, dowelled existing footings, formed curb
3 November 2008	ECI ran test car for clearance check
6 November 2008	ECI poured concrete for rigid track
10 November 2008	Railroad passed test train
10-13 November 2008	ECI finalized track alignment and surfacing at approaches, turned over to railroad
14-21 November 2008	ECI installed grout tubes and grouted cracks

HISTORY OF ORIGINAL CONSTRUCTION

Original tunnel construction began on 1 November 1860 and was completed on 17 May 1861. When the excavation of the tunnel began, it was reported that the ground was found to be like “quicksand, whereby for each shovelful of sand removed, two shovelfuls slide in.” In order to address these difficult ground conditions a vertical wooden shield was constructed and the top heading and bench method of excavation was used. The holes along a semi-circle were drilled at the top of the shield and wood poles (forepoling) were driven through the holes to form a wood arch to support the sand ahead of the shield (Chittenden County Historical Society Bulletin, 1996). The sand was then excavated beneath the arch. The initial lining comprised of contiguous 300-mm (12-in.) by 300-mm (12-in.) timbers was used to support the top heading. The space between the timbers and the soil was reportedly packed with cordwood (Faucett, 1984). The process was performed repeatedly to construct the top heading. As soon as the top heading for the tunnel was completed, the sides and bench were excavated and again supported with contiguous timbers and timber invert struts at 1.5-m (5-ft.) horizontal intervals to complete the horseshoe-shaped excavation. Approximately 700,000 board feet of lumber was used for the initial timber lining and related temporary tunnel excavation support. The outside excavated dimensions of the horseshoe were approximately 7.3 to 7.6 m (24 to 25 ft.) high by 6.4 to 6.7 m (21 to 22 ft.) wide.

As soon as there was sufficient room, the brick final lining was constructed, with an arch about 0.6 m (2 ft.) thick and side walls up to 1.2 m (4 ft.) thick. (Note, horizontal coring of the sidewalls during the 2008 repairs only encountered a 0.6-m- (2-ft.-) thick lining.) The volume between the outside of the brick lining and the initial timber lining was filled with concrete packing. The interior dimensions of the final tunnel as originally constructed were: approximately 5.8 m (19 ft.) from top of invert to bottom of crown; 4 m (13 ft-4 in.) wide at the bottom of the brick lining; and 4.9 m (16 ft.) wide at approximately mid-height of the tunnel. The tunnel work was performed by about 85 men with half working days and half working nights. The average rate of tunnel construction was 0.9 m (3 ft.) per day.

The portals consisted of limestone masonry headwalls and wing walls. The headwalls about 12 m (39 ft.) high and tapering from about 1.7 m (5 ft.-6 in.) thick at the bottom to 1.2 m (4 ft.) thick at the top. The wing walls were each about 14 m (46 ft.) long and at the highest section the thickness ranges from 5.5 m (18 ft.) at the bottom to 1.2 m (4 ft.) at the top.

TECHNICAL APPROACH TO PERMANENT REPAIRS

The tunnel movement that occurred when the invert cobblestone floor was removed had made everyone keenly aware of the critical condition of the tunnel. Once the emergency concrete brace slab had cured, survey monitoring points at 3-m (10-ft.) stations and four crack monitors were installed and used to confirm that movements had been arrested so that the remainder of the repair work and shotcrete work could be performed safely. The survey monitoring points were measured with a surveyor’s grade rod positioned between PK nails on opposite wall footings. This simple measurement method has a precision of about 1.5 mm (0.005 ft. or 1/16 in.). Monitoring was performed daily until a stable condition was confirmed and then weekly thereafter. Daily monitoring was again performed at critical times such as when the emergency brace slab was removed and replaced with the permanent bracing.

Once the emergency bracing slab was in place and the tunnel stability was verified, the shotcrete contractor started their work. The 3-week duration of the shotcrete operation gave ECI time to assess the tunnel conditions, evaluate remedial options and design permanent repairs. The design objectives for the permanent tunnel repairs included: 1) re-establish a permanent lateral brace between the wall footings throughout the length of the tunnel, 2) maintain tunnel stability while removing the existing temporary emergency slab, 3) provide an adequate bearing surface for the track structure through the middle third of the tunnel where the disturbed subgrade was observed, 4) provide an acceptable clearance envelope with due consideration for the shotcrete reinforcement and other previous defects/repairs in the tunnel geometry, 5) complete the repairs as quickly as possible and allow for the shotcrete contractor to complete their work, 6) repair the longitudinal cracks as necessary, and 7) be as economical as possible.

The resulting design, as shown in Figure 5, consisted of a rigid track structure to serve as both the track support and as the lateral brace. The design included a 250-mm- (10-in.-) thick reinforced concrete base slab (for improved bearing on the disturbed subgrade); and a 230-mm- (9-in.-) thick layer of crushed stone (to replace some of the disturbed soil). The base slab was only installed in the middle section of tunnel (section that had experienced the movements and cracking) and was constructed in 1.8-m- (6-ft-) long segments to maintain the capacity of the emergency brace slab until the new base slab and temporary shoring was installed.

The rigid track structure was designed for 416 kN/m (28.5 kips/ft.) brace load and consists of concrete encased steel cross-ties. The steel cross-ties are Narstco H12 which are 2.6-m (8-ft-6 in.) ties with a total weight of 91 kg (201 lbs.) each (about 0.02 kN/m or 23.6 lb/ft). The steel thickness in the cross-ties is 12 mm (0.472 in.) and the cross-ties are curled down at the ends. We installed headed studs at the curled down ends to assure adequate stress transfer to the steel member (refer to Fig 5). The ties were attached together with a series of steel reinforcing bars welded to the cross-ties. The rail consists of jointed, 45 kg (100 lbs.) RA and is continuously anchored to the cross-ties with rail clips.

The rigid track structure had special installation considerations because once the steel cross-ties were concrete encased, the track alignment was essentially permanent. The alignment of the new track within the existing 4° horizontally curved tunnel needed to consider both the reduced side and ceiling clearances from the shotcrete and occasional irregularities of the tunnel. In practice, this complexity was made even more difficult because the rails were superelevated 25 mm (1 in.) throughout the curve and because the longer rail cars encroach even further on the limits of a clearance envelope (imagine chords on a horizontal curve). These factors made it necessary to: 1) assemble the entire track, 2) align the track, and 3) verify clearances with an actual rail car – all prior to casting the concrete of the rigid track structure. In order to meet this objective, the rigid track structure (the permanent brace) needed to be installed after the base slab, rather than segmentally which would have been the best method to maintain continuous bracing during removal of the emergency brace slab. Installing the complete base slab first provided a stable base to work from and assemble the steel cross-ties and rail (as shown in photo in Figure 6). However, it did not provide a direct lateral brace to the tunnel wall footings since there was not enough remaining height of the footings to engage with the base slab. It was anticipated that some additional temporary lateral shoring posts would be necessary throughout the middle section until the rigid track structure could be installed.

As construction proceeded, the wall footing depth was greater than expected along the middle third of the tunnel so that the base slab was providing some limited lateral bracing. This condition, along with the stable conditions verified with the survey monitoring points, allowed for the decision to proceed without the cumbersome temporary shoring braces. The observed stable condition also indicated that the longitudinal cracks were more of a result of disturbing and heaving the subgrade soils below the tunnel floor and wall footings rather than the immediate horizontal loading on the tunnel lining when the cobblestone floor was removed. This was a direct indication that the soil/footing interface friction at the bottom of the wall footing was sufficient to resist the lateral earth pressures once the disturbed soil had relaxed (excess pore pressures had dissipated) and provided a confirmation that the 416 kN/m (28.5 kip/ft.) design brace load was adequately conservative.

The final phase of the repair was the grouting of the tunnel lining at the crack locations to fill the crack voids thereby restore full bearing across the thickness of the lining and limiting the potential for stress concentrations and related spalling. This work (shown in Figure 7) was done after the tunnel was put back into railroad service. The grouting program consisted of two rows of 12-mm- (1/2in.-) diameter grout holes at a 600-mm- (2-ft.-) horizontal spacing. Each row of grout holes was drilled into the brick lining at a 45° angle above each crack so to intercept the cracks about midway through the lining. The grout was pumped with a Kendrich GP-1 hand operated pump to better control the grout pressure to the 28 kPa (4 psi) maximum criterion. The grout was hand-mixed and consisted of deNeef MC500 Microfine Cement with NS200 Dispersant. Grout holes were injected once in a zoned primary/secondary sequence. The grout takes ranged from 0 to 12.3 l (416 fl. oz.) per hole with an aggregate total of 118 l (4000 oz.).

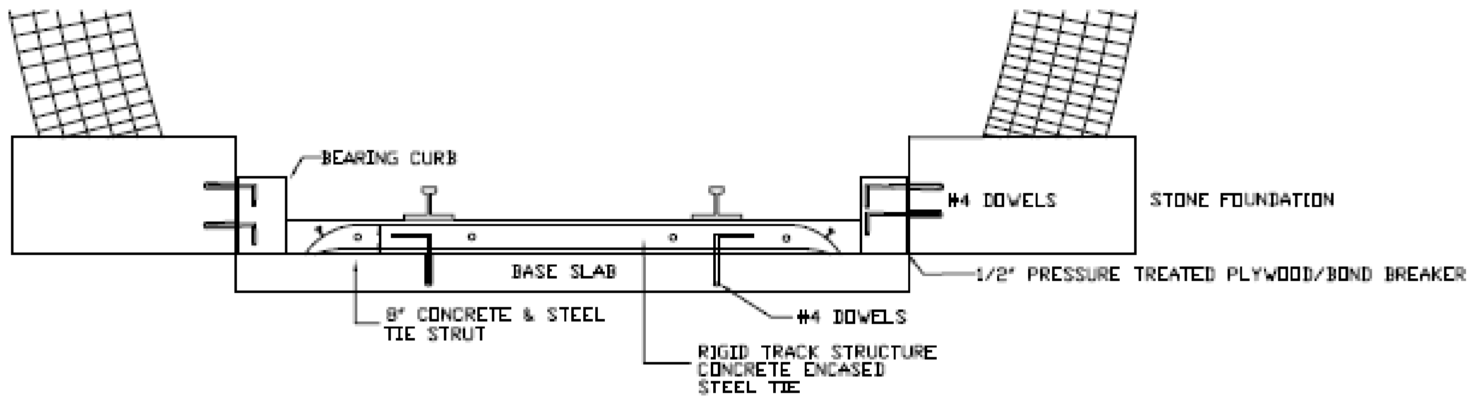

Figure 5. Repair section at center of tunnel

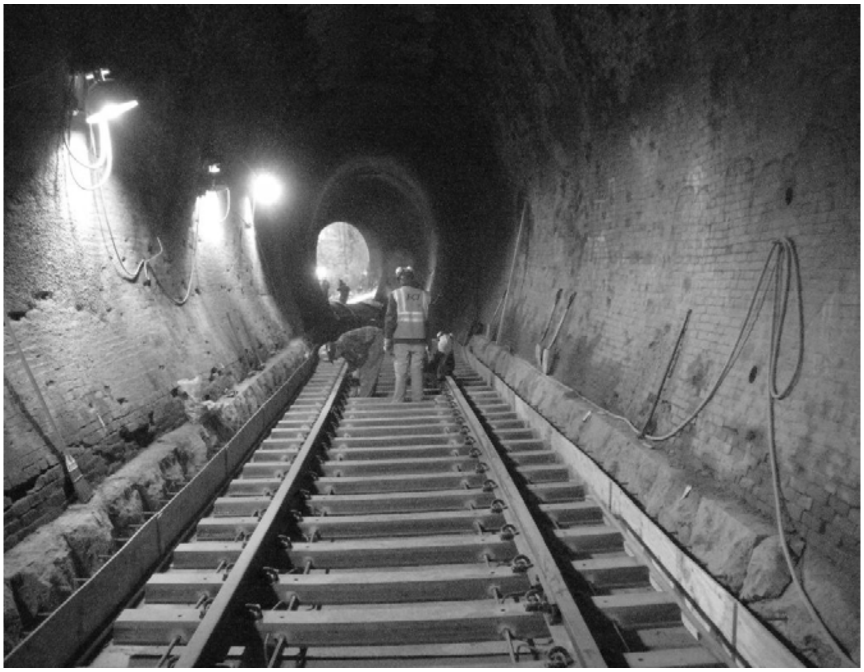

Figure 6. Rigid track structure being assembled

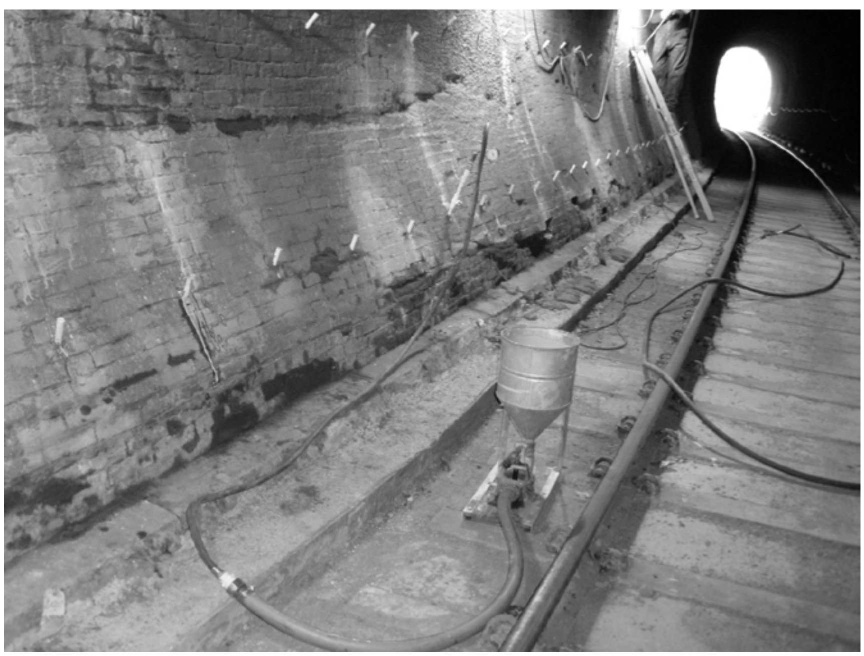

Figure 7. Grouting cracked lining

REPAIRED LINING PERFORMANCE

ECI visited the tunnel in November 2010 to observe the condition of the tunnel lining and to obtain additional data regarding the convergence of the tunnel lining and crack monitor readings. The tunnel, including the shotcrete walls, the unshotcreted walls, the portals, and rigid track structure appeared in excellent condition. Visual inspection of the lining did not detect additional cracking or other indications of continued movement of the lining walls. Similarly, the crack monitor data did not indicate movements or changes. Since the end of the repairs in mid November 2008, the measured convergence ranged from 0 to 6 mm (1/4 in.) with most readings in the 3 mm (1/8 in.) range. It is estimated that the shrinkage of the concrete bracing slab since the end of construction would in the range of 3 mm (1/8 in.); therefore the bracing slab has been successful in stopping convergence of the tunnel sidewalls. As part of the 2008 grouting repairs, ECI mudded the cracks with a neat cement grout. This also provided a tell-tale for future crack changes. In our 2010 visit, we did not observe any fracturing of that new surface which indicates that the cracks have not opened further since 2008.

SUMMARY & CONCLUSIONS

The permanent repair required extensive labor effort, attention to details, and careful coordination, but with such care, was very successful. All of the design objectives were met or exceeded. That first evening when the cracks appeared, it was anticipated that the shotcrete project would likely experience significant delay. Instead with careful planning and innovative approaches the tunnel was repaired and back in service within 2 months. Just as amazing, the repairs would be performed for under $300,000 (excluding the temporary concrete brace, the steel ties, and rail).

Looking back upon the situation, it seems likely that the tunnel floor could have been removed without causing the large cracks in the tunnel lining if the original contractor had used low ground pressure track equipment rather than a heavy wheeled loader and limited vibrations. However, without a plan to install a permanent brace, it is likely that the crack would have eventually occurred and probably extended even further, perhaps damaging the newly installed shotcrete reinforcing.

ECI performed an informal follow up inspection and re-measured the survey monitoring points in November 2010. A very slight .005 ft inward deflection was measured in the new survey. We surmise that the measured change was either because of differences in the measuring equipment or that concrete shrinkage had occurred which allowed a small amount of convergence to occur.

REFERENCES

The Chittenden County Historical Society Bulletin. 1969. Vol. 5, No. 1, September 1969.

Faucett, T.J. 1984. *Letter with miscellaneous drawings and notes from archives regarding Burlington Tunnel construction*. St. Albans, VT: Central Vermont Railway, Inc.

Doolan, B. 1996. The Geology of Vermont. *Rocks and Minerals, Vermont Issue.* Washington, D.C.: Heldref Publications, pp. 216-225.

DMJM Harris. 2002. *Burlington-Essex Rail Project, Burlington Rail Tunnel Assessment*. Boston, MA: DMJM Harris.

Wright, S.F. 2003. *Glacial Geology of the Burlington and Colchester 7.5" Quadrangles, Northern Vermont.* Waterbury, VT: Vermont Geological Survey.

Wright, S., Fuller, S., Jones, S., McKinney, A., Rupard, S., and Shaw, D. 2009. *Surficial Geologic Map of the Burlington, Vermont 7.5 Minute Quadrangle*. Open File Report VG09-1. Waterbury, VT: Vermont Geological Survey.