Case Study

Slurry Wall Construction for a Cut-and-Cover Tunnel

The use of slurry walls is not a common practice. However, they may pose satisfactory alternatives and consequences.

PHILIP BONANNO, DONALD T. GOLDBERG & AMOL R. MEHTA

A SLURRY WALL is a reinforced concrete wall that is constructed using the slurry trench method. Figure 1 shows the construction sequence of a slurry wall. A trench is dug in the ground alongside a completed wall panel. The trench's width is equal to the width of the slurry wall. The length of the wall panel is generally between 12 and 20 feet depending on the soil conditions at the site. The trench is excavated down to the required depth of the wall, and filled with bentonite slurry to its full height in order to prevent the vertical sides of the trench from collapsing. A prefabricated reinforcing steel cage is lowered into the slurry trench. Tremie concrete is pumped into the trench replacing the bentonite slurry. The slurry walls are then interlocked to form a continuous wall as normally done for the exterior walls of a tunnel.

The Massachusetts Bay Transportation Authority's (MBTA) Northwest Extension of its Red Line rapid transit service into the cities of Cambridge and Somerville represents the first time in the U.S. that slurry walls were used as a permanent part of a cut-and-cover structure. A total of approximately 600,000 square feet of slurry walls were incorporated in the tunnel structure between Harvard Square and Alewife Stations where the extension ends. The greatest amount of slurry wall construction, almost 440,000 square feet, was used as exterior structural walls for a two- and three-track tunnel structure constructed using the cut-and-cover method from the Davis Square to the Alewife Stations (see Figure 2). A breakdown of the amounts of slurry wall construction used for the Northwest Extension project are shown in Table 1.

The alignment of the cut-and-cover tunnel from Davis Square Station continued through Somerville within the right-of-way of the Boston & Maine Railroad, passing under Cameron Avenue and Massachusetts Avenue in Cambridge. From that point it travelled under Russell Field to Alewife Brook Parkway, passing under the Little River and under the Route 2 bridge. Figure 3 presents a view of the bridge.
The four-lane Route 2 bridge was required by the Massachusetts Department of Public Works to be kept in service for vehicular traffic for its entire width during the construction of the tunnel directly under the railroad track. To enable the tunnel construction, the existing bridge embankment, abutments and piers required support and underpinning. Very deep slurry walls in clay were utilized to provide support for the existing bridge piers. These slurry walls also served as the permanent walls of the tunnel. The existing bridge pier loads were transferred onto the tunnel roof prior to excavation. Excavation and bracing then continued under the roof until the construction of the invert slab was completed. Extensive monitoring of both the slurry wall construction and the existing bridge structure was performed throughout the construction period. Construction for the tunnel beneath the Route 2 bridge began in 1980 and proceeded in three stages — con-
struction of tunnel roof slab, excavation under the roof slab and construction of tunnel invert slab and intermediate walls.

Design Considerations

The Route 2 bridge is a three-span continuous structure that was built in 1933. It carries Route 2 over a now abandoned Boston & Maine Railroad track, and is supported by two spill through abutments and two intermediate piers. The span lengths are approximately 48, 43 and 48 feet, and the total width of the bridge is about 82 feet. The railroad track is located between the two piers. The superstructure consists of steel stringers with center span stringers encased in concrete. The substructure consists of approximately 33-foot high spill-through abutments and two 28-foot high piers. Each pier has a 3-foot concrete pier cap supported by ten 2.5-foot square columns spaced at 11.5 feet on centers. The pier columns are supported by a 6.5-foot wide by 115-foot long continuous footing.

The bridge is constructed on footing foundations that bear on fine-to-medium sand. The sand, in turn, is underlain by a very deep deposit of compressible clay. Since its construction in the 1930s, the bridge has experienced approximately 2 feet of settlement as a result of the consolidation of this clay under the weight of the 25-foot high earth embankment above the original grade.

The cut-and-cover tunnel section consisted of a 50.5-foot wide three-box tunnel located directly beneath the bridge piers (see Figure 4). The exterior walls of the tunnel were 3-foot thick concrete slurry walls extending 37 feet below the bottom of the invert slab. The roof slab and the invert slab were 4 feet thick. The overall depth of the tunnel section between the top of the roof slab and the bottom of the invert slab was approximately 26 feet. The tunnel section consisted of two rows of interior walls supporting the tunnel roof. The center lines of bridge piers were located about 8.5 feet and 7 feet from the center lines of the exterior walls of the tunnel.

Figure 4 shows that 34 feet of fill and fine-to-medium sand overlie the clay, extending to a depth of approximately 140 feet. Glacial till underlies the clay. The clay is stiff near its upper horizon, but is of medium consistency with blow counts of four or less below the stiff upper crust. While deep consolidation tests are lacking for this particular area, this formation is typical of Boston clays that exhibit normally consolidated characteristics below depths of about 80 to 90 feet.

The shear strengths of the clay, as determined from two field vane borings, averaged about 1,400 psf within the depth range of the slurry wall (about 75 feet). The shear strength of the clay between the depths of approximately 75 and 100 feet averaged about 1,100 psf.

Figure 4 indicates that an approximately 40-foot deep excavation lies adjacent to a 25-foot highway embankment for a total depth from the pavement to the bottom of the excavation of approximately 65 feet which requires lateral support.

The requirement that Route 2 be kept in

<table>
<thead>
<tr>
<th>TABLE 1</th>
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<tbody>
<tr>
<td>Use of Slurry Walls as Part of Permanent Structure</td>
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<tr>
<td>Location</td>
</tr>
<tr>
<td>Harvard Square Station</td>
</tr>
<tr>
<td>Cut-and-Cover Tunnel</td>
</tr>
<tr>
<td>from Davis Square to North of Route 2</td>
</tr>
<tr>
<td>Alewife Station</td>
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</table>
FIGURE 4. A cross-section of the tunnel beneath the Route 2 bridge.

service for vehicular traffic for its entire width during the construction of the tunnel posed several geotechnical challenges:

- development of an underpinning scheme
- maintaining embankment stability
- prevention of excess settlement

The major structural factors that had to be considered during design were:

- the three-box tunnel section had to be capable of supporting the two bridge piers
- the tunnel section had to be capable of withstanding lateral pressures from abutment surcharge in addition to soil and ground water pressures

The major issues to be considered during the construction of the tunnel and the underpinning of the bridge were:

- there was limited vertical clearance and narrow horizontal width available under the bridge
- performing the actual construction without interfering with traffic on the bridge

Geotechnical Considerations
The principal geotechnical issues were further defined as:

- determining the depth requirement of the slurry wall for underpinning
- containing lateral earth pressure against
Underpinning. An early task was to evaluate the site using the slurry wall as the underpinning element rather than piles extending into the till at great depth. To a large measure, the feasibility of this alternative depended on whether or not the slurry wall had to extend to the glacial till (about 140 feet deep) or whether the load applied to the wall could be transferred to the surrounding soil by a combination of adhesion (side friction) and end bearing. The vertical reactions on the slurry wall due to supporting the bridge, the soil cover over the tunnel roof and the weight of the tunnel roof, as well as all of the lateral pressures acting on the wall, were calculated.

Analytical studies demonstrated that the slurry wall did not have to extend to the till and that the load could be transferred to the surrounding soil.

The geotechnical analysis considered two conditions. The first was the short-term condition using undrained shear strength parameters of the clay. The second used effective stress parameters (drained analysis) following pore pressure dissipation. The parameters used in this analysis are given in Table 2.

The rationale for the reduction in soil parameters from the in-situ condition to the in-contact-with-slurry wall condition was derived from published data. Some of the reasons for this reduction are due to the remolding of the clay during excavation and the existence of bentonite mudcake along the sand interface. The results of the analysis yielded a factor of safety of 2.0 with a 35-foot slurry wall extension below the bottom of the excavation.

Lateral Pressure. The pressure diagram shown in Figure 5 presents the net lateral earth pressure against the slurry wall (the}
difference between the pressure on the outside of the wall and the passive pressure on the inside of the wall). It includes the sum of loading from earth, water, the embankment and the bridge abutment.

Control of Load on Bracing System. The wall depth requirement was dictated by the adhesion requirements for vertical load transfer. As shown in Figure 5a, extension of the wall below the bottom of the excavation produces greater loads because of the relative stiffness of the wall compared to that of the soil, and the weakness of soil. This load not only imposes large bending moments on the wall but also high forces in the bracing system, particularly at the bottom strut level. This situation led to the concept of designing a hinge into the wall by crossing reinforcing steel into an X-shaped configuration approximately 10 feet below the bottom of the excavation. The rationale for this scheme was that the hinge would relieve moment and, thus, horizontal forces on the wall and the strut system. This design would require a thinner slurry wall and would lower the amount of reinforcing steel required in the wall than would be necessary if the wall were designed to resist the cantilever moment acting on the 35-foot length of the wall below

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**TABLE 2**

<table>
<thead>
<tr>
<th>Soil Parameters (Average)</th>
<th>In-Situ</th>
<th>Assumed in Contact With Shary Wall</th>
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<tbody>
<tr>
<td>Clay Shear Strength</td>
<td>1,400 psi</td>
<td>900 psi</td>
</tr>
<tr>
<td>Friction</td>
<td>0.70 x 1.400 = 900 psi</td>
<td></td>
</tr>
<tr>
<td>Friction</td>
<td>28°</td>
<td>2/3 x 28° = 18.5°</td>
</tr>
<tr>
<td>Cohesion Intercept</td>
<td>0.70 x 1.400 = 900 psi</td>
<td></td>
</tr>
<tr>
<td>Friction</td>
<td>2/3 x 150 = 100 psi</td>
<td></td>
</tr>
<tr>
<td>Friction</td>
<td>26°</td>
<td>1/2 x 26° = 13°</td>
</tr>
</tbody>
</table>

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FIGURE 6. Modes of potential shear displacement: (1) represents conventional base instability; and (2 & 3) “Swedish” circular arcs.
the invert slab (see Figure 5b).

**Overall Stability.** Figure 6 presents the modes of potential shear displacement that were studied as part of the geotechnical analysis. Mode 1 was a conventional analysis using bearing capacity relationships. In this case, the pressure difference between the inside and outside of the wall at the full depth of excavation was determined. Dividing the ultimate bearing capacity by the pressure difference yielded the factor of safety. Modes 2 and 3 were conventional "Swedish" circular arcs analyzed by the method of slices. These analyses furnished a factor of safety of approximately 2.0.

**Field Performance.** Bridge settlement was measured at the underpinned intermediate piers and at the adjacent non-underpinned bridge abutments. Figure 7 shows the settlement of the underpinned intermediate piers. The data reveal approximately 2 inches of settlement. Approximately half of this settlement occurred during the period of slurry wall excavation and roof slab placement. The remaining settlement was in response to the excavation. Approximately 0.5 inch of this response settlement occurred between the placement of B-level struts (elevation 90) and the C-level struts (elevation 82). The ultimate depth of excavation was to elevation 75.

Figure 8 shows approximately 3 inches of settlement at the bridge abutments. As in the case of the intermediate piers, approximately 1 inch of settlement occurred during the placement of the slurry wall and the roof slab. The remainder was in response to the excavation.

Measurements of the lateral displacement of the slurry wall were performed using inclinometer instrumentation located immediately outside the slurry walls. The data
shown in Figure 9 reveal an inward rotation of the wall in response to the sequential stages of tunnel excavation. Of particular interest are: the point of discontinuity at the location of the hinge; the relatively smaller movement at the lowest extremity of the wall; and essentially zero movement at the clay below the bottom of the wall. The greatest inward displacement was approximately 1.7 inches at the hinge point, confirming the design assumption that rotation at the hinge point would relieve wall moments due to lateral pressure acting on the wall below the hinge.

**Structural Design Considerations**

A typical three-box cut-and-cover tunnel section for the Red Line Northwest Extension was analyzed with the slurry wall acting as an earth-retaining structural system during the construction of the tunnel box, and then with the slurry walls and roof slab acting as a structural rigid frame resisting all lateral and vertical loads existing on the tunnel invert slab and end bearing of the bottom of the slurry walls. The connections between the invert slab and slurry walls were designed for shear transfer.

For the underpinning design, the tunnel roof slab was used as the underpinning beam to support the bridge piers. The roof slab of the tunnel needed to be constructed first between the slurry walls in order to support the piers while the excavation beneath the roof slab could be carried out to complete the tunnel section. This method of cut-and-cover tunnel construction is commonly known as the Milan method. Thus, the tunnel section was analyzed for various construction conditions and for its completed condition.

The total reaction at each pier from dead
FIGURE 9. Inclinometer data showing lateral displacement.

and live bridge loads was calculated to be 20.5 k/lf. The soil cover over the tunnel roof slab imposed a dead load of 1.4 k/lf. The thickness of the roof slab was 4 feet and the invert slab was 4 feet thick. The tunnel section required 3-foot thick slurry walls. The railroad track
over the tunnel roof was abandoned and was not a factor in the design. However, the tunnel was designed to support a live load of 0.6 k/ft. All of the lateral pressures acting on the tunnel structure were determined by geotechnical analysis.

The structural members of the tunnel were then designed for a worst loading condition. The design was analyzed for each stage of construction:

**Stage 1.** The tunnel roof slab was analyzed as a fixed slab between the slurry walls supporting the bridge pier reactions.

**Stage 2.** After completion of the excavation under the roof slab between the tunnel exterior walls, the structural frame consisting of the slurry walls and the roof slab was analyzed for loads from pier reactions, soil cover on the roof slab, live load and the lateral pressure from soil, and groundwater and surcharge loads from the bridge abutments. The contractor was required to analyze the exterior slurry walls to determine the location of temporary struts required for bracing during construction.

**Stage 3.** After construction of the tunnel invert slab and intermediate walls, the tunnel section was analyzed for all vertical and lateral loads including the reactions from the bridge.

The loads from the bridge piers are supported by the tunnel roof slab, and are transferred to the slurry walls through moment connections between the roof and walls and then transferred to the ground by friction between the soil and the slurry walls. This analysis required that the depths of the slurry walls be approximately 36 feet below the bottom of the tunnel invert slab. Since it was impractical to design such a long slurry wall below the invert slab in order to resist moments due to lateral loads, the walls were designed with hinges at a depth of 10 feet below the invert slab. The lateral loads acting on the walls below the hinges were transferred to the wall portion above the hinges as shear forces. The loading diagrams for the various construction stages are shown in Figure 10.

**Construction Considerations**

The actual construction of the tunnel section under the Route 2 bridge was modified during construction. The roof slab was raised by approximately 4 feet and was cast just above the bridge pier footings (see Figure 11). The pier reactions were transferred to the tunnel roof slab by shear friction by inserting dowels between the pier columns and the roof slab. After completing the construction of the tunnel roof to underpin the bridge piers, the construction of the remaining tunnel section was performed beneath the roof slab (see Figure 12). Limited low vertical clearances of approximately 21 feet under the bridge and the required slurry wall panel depths of approximately 71 feet further affected construction and necessitated additional construction modifications.

**Equipment.** In order to accommodate the low clearance, a special low head boom was fabricated to fit a 60-ton crawler crane. The excavating bucket was designed and fabricated so that the maximum height in the closed position was only 12 feet (see Figures 13, 14 & 15). The bucket height of 12 feet enabled the bucket to be dumped into a front end loader bucket and then subsequently deposited into a truck for final disposal.

**Slurry Wall Panels.** The panel widths were limited to 9 feet for secondary panels and 14 feet for the primary panel. Since the primary panel had to be excavated 18 feet to accommodate the placement of the end stops, the 9-foot multiple worked very well as the secondary panel width.

**Reinforcing Cages.** Since the head room available was only 21 feet, the reinforcing steel cage for the slurry wall could not be built at full depth. Instead, the cages were built in place in approximately 15-foot sections. Each section was connected to the previous section using bar grip compressed couplers and then lowered into the wall cavity (see Figure 16).

**End Stops.** The end stops, or bulkheads, also had to be set in approximately 15-foot sections due to limited clearance. The bulkheads were 36 WF 150 beams. These sections were bolted to each other and lowered into
FIGURE 10. Loading diagrams for the tunnel's three construction stages.
FIGURE 11. Pier footing and reinforcing steel for the roof slab and slurry wall connection.

the wall cavity in the same manner as the reinforcing steel cages. Crushed stone was then placed on the outside of the end stop to prevent any movement of these members as the concrete was placed.

Concreting. Concrete was placed using a tremie pipe consisting of 8-foot sections. Two tremies were used in the 14-foot slurry wall panel and one pipe was used in the 9-foot panel.

Installation Techniques. Installing slurry walls in areas of restricted head room requires certain modifications to the normal slurry wall installation techniques. Since the restricted

FIGURE 12. Tunnel construction under the roof slab. Note the pier footings supported by the roof slab. Reinforcing bars hanging below the roof slab are for tunnel interior walls; steel beams are for temporary bracing.

FIGURE 13. Low head boom on crane.

FIGURE 14. Limited clearance under the bridge.
FIGURE 15. Excavation of the slurry trench for the slurry wall.

Excavation of the slurry trench for the slurry wall. Head room forces the excavating machine to have a short boom, vertical alignment of the excavated hole is very difficult to maintain. This situation was especially true for the deep panels that were required for the Route 2 bridge. The main load line was so short that any slight misalignment resulted in a panel that was not vertical. The installation of both end stops and the reinforcing steel cage in non-vertical panel excavation was unachievable. The excavating bucket used possessed three digging teeth on one leaf of the bucket and two teeth on the other half. This slight variation along with the short load line caused the side of the panel excavation with three teeth to drift slightly toward the side with two teeth. This drift was not discovered until the test template was inserted into the panel hole. This particular panel had to be re-excavated. The corrective action taken on subsequent panels was to rotate the digging bucket as the panel was being excavated. The resulting alternation of the three-teeth and two-teeth leaves prevented the drift that had previously occurred. All other panels were excavated with no problems.

Conclusions

The construction of the cut-and-cover tunnel under the Route 2 bridge provided an interesting design and construction challenge due to such factors as the underpinning of the existing three-span continuous bridge, limited vertical and horizontal clearances, extension of clay to about 140 feet below the ground surface, and support of lateral pressures due to the bridge abutment and embankment.

The geotechnical analysis determined the depth of the slurry wall and the lateral pressures for maintaining the base stability. The structural analysis of the tunnel section was carried out based on the geotechnical parameters and also to support the bridge reactions to underpinning. The construction techniques were modified to adapt to the restricted clearances, and vertical settlements...
and lateral ground movements were monitored during construction. The project was successfully completed in 1984, fulfilling the stipulation that traffic over the bridge not be impeded, and for a bid price of $52 per square foot for the slurry wall construction.

PHILIP BONANNO graduated from the Columbia School of Engineering in 1954 and received his Masters Degree in Civil Engineering from Columbia in 1964. He has been in the contracting business since 1955 when he started as a field engineer with the Arthur A. Johnson Corp., a subway contractor in the city of New York. Since 1968 he has served as President of the J.F. White Contracting Co. He is a member of the American Society of Civil Engineers and is a registered professional engineer in Massachusetts, New York and Pennsylvania.

DONALD T. GOLDBERG was a co-founder of Goldberg-Zaino & Associates, Inc., in 1964. His educational experience includes a B.A. from Tufts University in 1948, as well as a B.S., in 1954, and M.S., in 1957, from the Massachusetts Institute of Technology where he specialized in soil mechanics and foundation engineering. He has been Principal-in-Charge of most of Goldberg-Zaino’s work with the MBTA, including the Red Line Northwest Extension from Davis Square to Alewife, and the Southwest Corridor project from Section 3 to Forest Hills. His extensive experience in cut-and-cover tunneling includes being one of the authors of a reference guide, Lateral Support Systems and Underpinning, prepared for the Federal Highway Administration.

AMOL R. MEHTA is a Project Manager with Sverdrup Corp’s New England Division. He received his B.S. in Civil Engineering from Gujarat University in 1966, an M.S. in structural engineering from the University of California at Berkeley in 1967, and an M.B.A. from Babson College in 1977. He has worked on a number of transit stations and cut-and-cover tunnel projects. He was the structural group leader for the cut-and-cover tunnel design for the MBTA’s Red Line Extension from Davis Square to Alewife. He is a member of the American Society of Civil Engineers and BSCES.

REFERENCES
1. Major participants in this project were Sverdrup Corporation, Consulting Engineer; Goldberg-Zaino and Associates, Inc., Geotechnical Consultant; and J.F. White Contracting Co., General Contractor.