Replacement of the Cranston Viaducts Using Spliced Bulb-Tee Girder Technology

The application of bulb-tee girder technology provided economy and durability for this project, and its construction provided valuable experience.

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The Rhode Island Department of Transportation (RIDOT) retained an engineering firm in 1995 to perform environmental studies, preliminary engineering and final design for its Freight Rail Improvement Project (FRIP). This project is an undertaking by RIDOT, in conjunction with the Rhode Island Economic Development Corporation, to enhance the infrastructure within the state. These enhancements will enable the development of the Quonset Point/Davisville Industrial Park as a major port container facility in the northeast United States. When completed, this project will provide a freight dedicated track (FDT) capable of transporting double-stacked containers and triple-stacked car carriers within the Northeast Corridor (NEC) right of way (ROW) and parallel to the existing Amtrak electrified, high-speed mainline. In addition to the construction of the new FDT, the project will require other new construction as well as modifications of many bridges, slopes and utilities within and adjacent to Amtrak's ROW. Although replacement of the Cranston Twin Viaducts — northbound (NB) and southbound (SB) — was not an objective of FRIP, the geometry (see Figure 1) and condition of the existing Cranston Viaducts led the designer to recommend to RIDOT that the viaducts be replaced.

The viaducts are located 0.62 miles south of the Rhode Island Route 10/U.S. Route 6 interchange on the Cranston/Providence city line. They carry Rhode Island Route 10 over two existing Amtrak rail lines, Cranston Street and Huntington Avenue.

Built in 1959, the old viaducts consisted of seven- and eight-span, simply supported,

FIGURE 1. The center span for the old viaducts.

composite steel stringers with a concrete deck. The piers generally consisted of multiple columns bearing on massive pier walls and spread footings. Pier walls were generally placed parallel to the railroad track and skewed to the centerline of the viaducts. Each viaduct carried two lanes of traffic over Cranston Street, the Amtrak NEC ROW and Huntington Avenue. The viaducts were separated by a 1.5-inch-wide longitudinal joint, and each were approximately 500 feet long and 50 feet wide. Spans varied in length with a maximum span of 90 feet. Each structure carried a 2-foot, 10-inch wide concrete safety walk with a bridge railing, a 3-foot-wide right shoulder, 12- and 14-foot-wide travel ways, and an 18-foot-wide paved, sand-filled median (see Figure 2). It was assumed that the sand-filled median was provided so that a third lane could be added to each viaduct in the future. This feature facilitated the construction staging during the replacement.

Feasibility & Type Study Phases

Replacement Decision. Placement of the FDT under the existing viaducts in a manner that minimized ROW requirements would have required the relocation of the massive bridge substructure elements, which proved to be impractical. Maintaining the viaducts would have required placing the FDT 50 feet away from the nearest existing track, significantly increasing the railroad corridor ROW. Implementation of this scheme would have required extensive retaining walls and property acquisition along the tracks. It would have resulted in two new piers and spans for the old viaducts, but would have left a large portion of the viaducts in generally poor to fair condition. Therefore, rehabilitation of the viaducts was necessary.

Full replacement of the aging viaducts in their entirety was another alternative. At the request of RIDOT, a feasibility study was

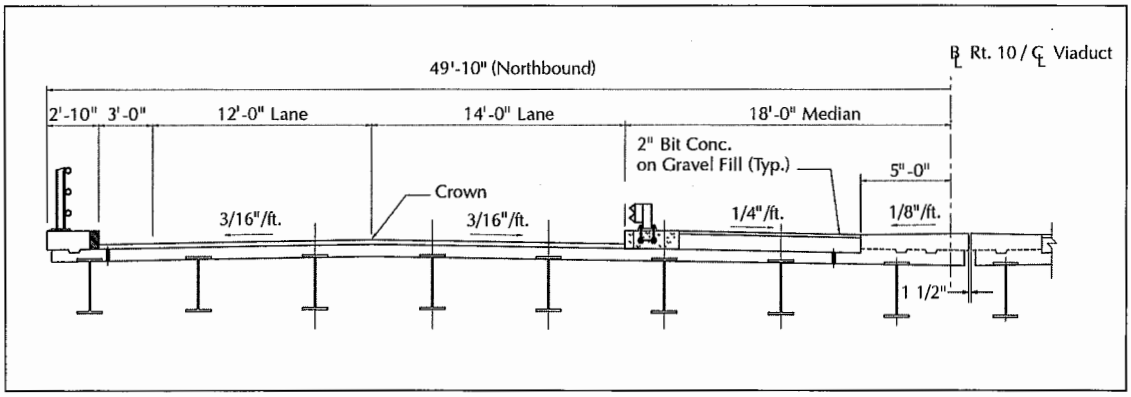

FIGURE 2. The old northbound viaduct.

performed to arrive at the best course of action by comparing the two alternatives. Although the initial construction cost for the full replacement alternative was more, it was found to be the most desirable. Implementation of this alternative would simplify the FDT construction, reduce property acquisition, eliminate vertical and horizontal track alignment deficiencies, andresult in two new viaducts upgraded to carry HS-25 loading. Moreover, when future maintenance costs were considered, the cost differential was even less. RIDOT concurred with the replacement recommendation, and directed the designer to further develop the replacement concept through a type study report.

Maintenance of Traffic. Before a replacement alternative could be considered, the designer developed solutions to ensure that both rail and vehicular traffic could be maintained during various stages of the construction. A key issue was the viability of placing four lanes of H20-44 loading on one viaduct during the construction. After examining various deck modification alternatives, the design team recommended that the NB viaduct (see Figure 2), which was slightly wider than the SB viaduct, be modified as shown in Figure 3 to temporarily carry four lanes of traffic.

In this scheme, low-density cellular concrete (LDCC) weighing 60 pcf was used to replace the existing sand-filled void in the median of the roadway. The superstructure and the substructure of the old NB viaduct were checked for the modified dead load and the additional two lanes of H20 loading. The analysis concluded that the superstructure was capable of carrying four lanes of traffic, and that only minimal shoring of the piers would be necessary while the new SB viaduct was under construction.

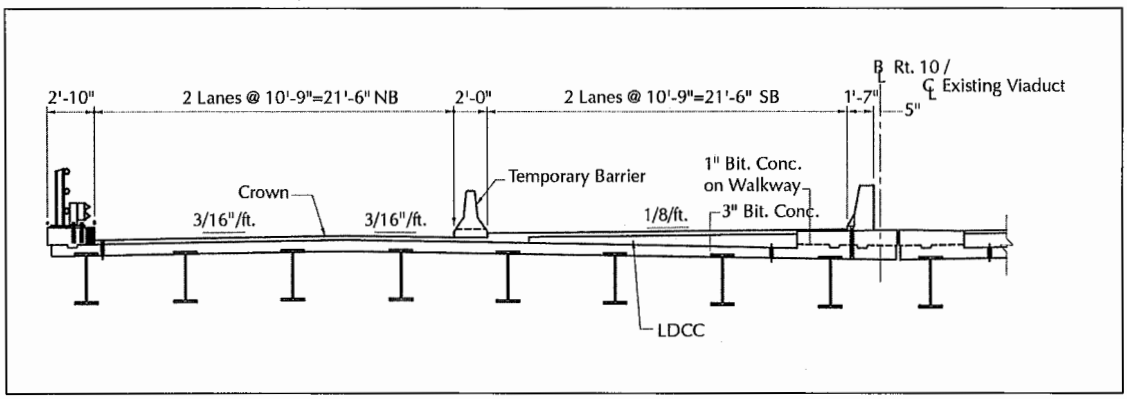

FIGURE 3. Modified northbound viaduct.

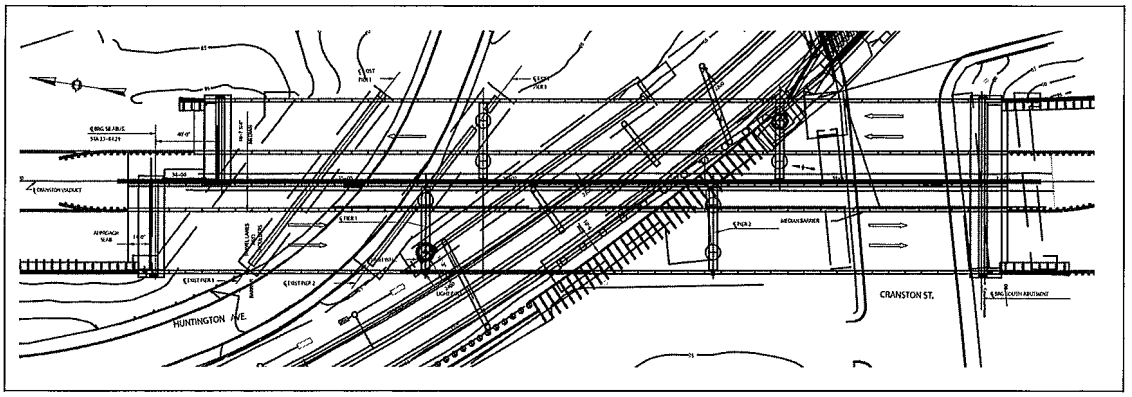

FIGURE 4. Plan for the new viaducts layout (old viaduct piers and abutments shown underneath.

Type Study Process. A type study was required to resolve several issues. Layout alternatives, superstructure alternatives and foundation alternatives were among the issues that were explored.

Layout Alternatives. For the replacement viaducts, two options were evaluated: a "skewed layout" and a "staggered square layout." In the skewed layout, the piers were placed parallel to the proposed railroad corridor and the abutments were also kept parallel to the piers; the spans of the two viaducts were therefore equal. Although this option offered some advantages, the staggered square layout was found to be simpler and less expensive overall. In this layout, the viaducts are each three-span continuous as follows:

- SB viaduct: 163-, 175- and 163-foot spans; and,
- NB viaduct: 158-, 180- and 123-foot spans.

Figure 4 shows this layout superimposed on top of old viaducts' piers and railroad tracks, and Figure 5 depicts the elevation view of the new SB viaduct. The out-to-out widths are 53.5 and 49.75 feet, respectively. The wider SB viaduct was chosen to temporarily carry four 11-foot-wide lanes (two lanes in each direction) during the construction.

Superstructure Alternatives. The following four alternatives were considered for the superstructure:

- Steel plate girders with cast-in-place (CIP) composite concrete deck;
- Precast concrete New England bulb-tee girders with CIP composite concrete deck;
- Steel trapezoidal box girders with CIP composite concrete deck; and,
- Precast segmental concrete trapezoidal box girders

These alternatives were evaluated for the fol-

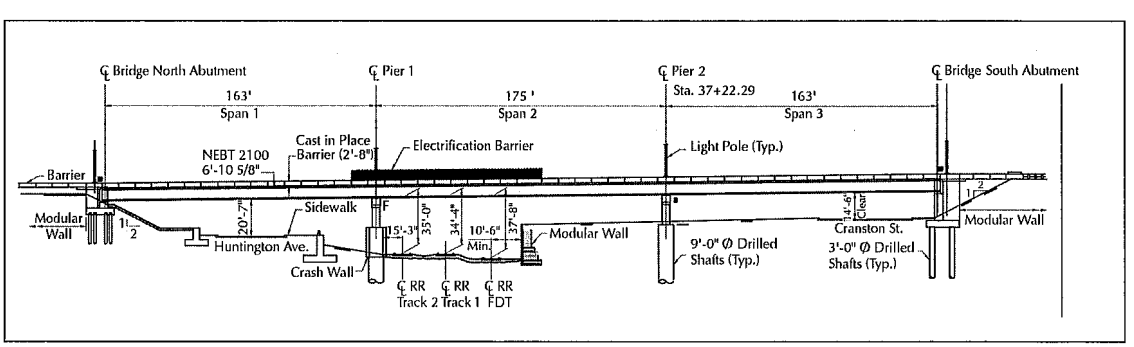

FIGURE 5. New southbound viaduct elevation.

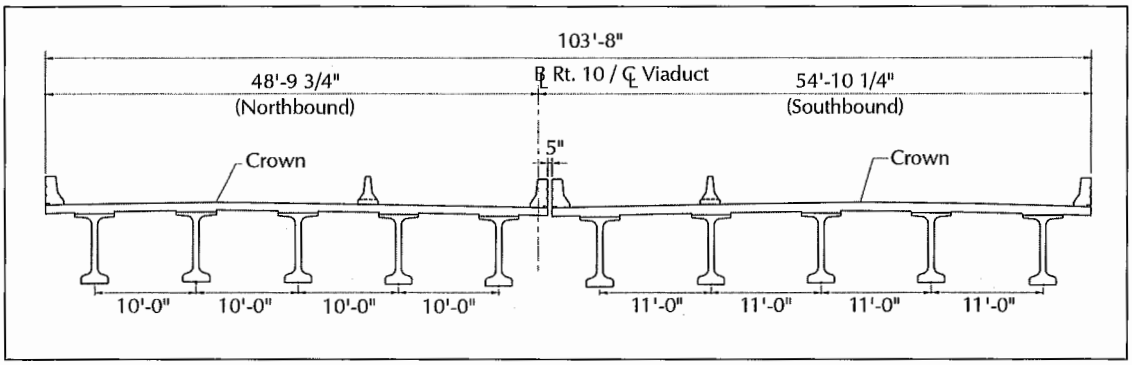

FIGURE 6. New southbound and northbound cross-sections.

lowing criteria: construction cost and schedule, constructability, aesthetics, functionality, maintenance and protection of traffic, and future maintenance cost. The second alternative was found to be the most advantageous, primarily for its lower initial and future maintenance costs (see Figure 6).

Foundation Alternatives. In addition to roadways and the NEC ROW, physical constraints (such as existing foundations) limited the placement of new piers and abutments for both viaducts (see Figure 4). In addition, due to the large axial and lateral loads of the new viaducts, it was clear that deep foundations were necessary in lieu of spread footings. Therefore, the foundation types were limited to either piles or drilled shafts. A pile system would have required large pile caps and excessive excavation supports, interfering with existing features beneath the viaducts. Therefore, because of their small footprint and their ability to penetrate obstructions, drilled shafts were found to be an economical foundation system.

Design & Construction of the New Viaducts

Drilled Shafts. During the final design, the shaft sizes and lengths were finalized as follows: for viaduct piers, 9-foot-diameter mono-shafts extending to a depth of 130 feet; for the abutments, a cluster of 3-foot-diameter shafts extending to a depth of 75 feet. Since the bedrock layer is very deep in this area, the shafts were not extended to the bedrock. The shafts were placed entirely in cohesionless soil and the loads were resisted both by side friction and end bearing. Prior to the construction of the production shafts, a 9-foot-diameter trial shaft was constructed and extensively tested using Osterberg load cells and multiple sets of strain gauges. (Further details regarding the design, construction and load testing of the Cranston Viaduct drilled shafts are provided in Reference 1.)

New England Bulb-Tee (NEBT) Girders. NEBT sections were developed in the late 1980s in response to a growing demand for more efficient precast pretensioned sections than the widely used AASHTO precast sections.[2] In order for NEBT sections to be competitive for longer spans, they had to be spliced and post-tensioned. The Cranston Viaducts presented the opportunity to demonstrate, for the first time, the potential of the NEBT sections for continuous girders in the New England region.

The span lengths are 163, 175 and 163 feet for the SB viaduct and 158, 180 and 123 feet for the NB viaduct, with girder spacing of 11 and 10 feet, respectively, as shown in Figure 6. The viaducts are designed for four lanes of HS-25 loading. During preliminary design, variable depth (haunched) girders were considered. However, precast producers generally prefer constant depth sections; therefore, a NEBT 2100-millimeter section made of 7000 psi concrete was selected for both viaducts.

The design incorporating NEBT sections was evaluated using a computer program specifically suited to analyze a post-tensioned concrete structure by accounting for the construction staging loads and by performing time-dependent analysis to estimate the creep

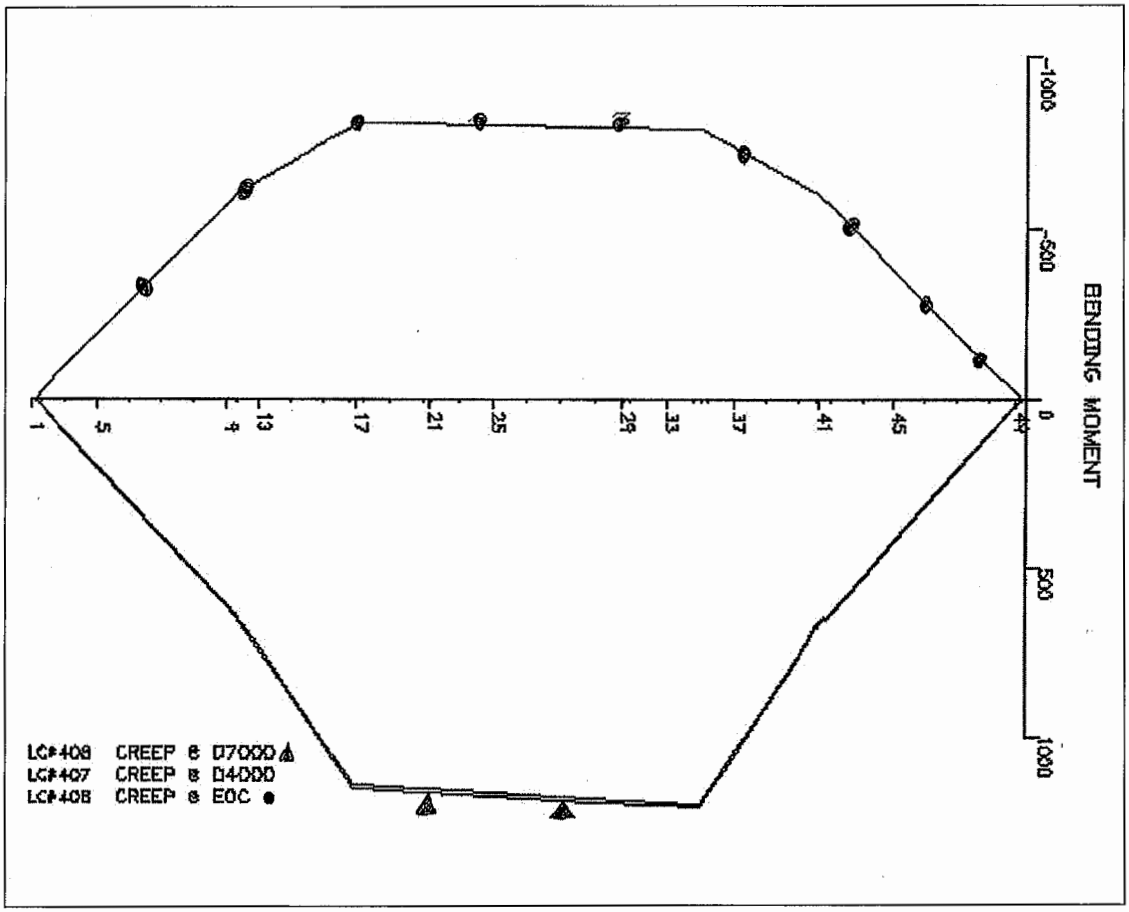

FIGURE 7. Creep-induced moments at various times.

and shrinkage of concrete. Such an analysis provides for the adjustment of prestressing forces over time. The forces in the precast girders were determined at each stage of construction and stresses were checked at all construction stages starting from the precast yard, on to each stage of erection, placement of the deck and ten years after the completion of construction. Stresses at the top and bottom of the girder and top of the deck were compared with the allowable stresses specified in AASHTO specifications for highway bridges and for segmental concrete bridges.[3,4]

The girders were designed based on the service loads and checked for the ultimate loads. Four 12.6-inch tendons were provided (two were stressed after the girders were spliced and two after placement of the CIP deck). For the service loads, the critical sections were the negative moment regions at interior supports. Forces induced by the creep of concrete and secondary post-tensioning moments resulted in positive moments at the negative moment regions and made the long spans viable (see Figures 7 and 8). The same sections were also found to be critical for the ultimate load check. In fact, in order to satisfy the ductility requirements, mild reinforcement had to be placed in the bottom flange (bulb) of the pier girders. This reinforcement consisted of twelve 24-foot-long No. 11 bars centered about the points of maximum moment locations. Apart from the critical bending sections described above, the shear stresses also approached allowable limits at the pier locations. In fact, subsequent load rating analysis performed by the engineering consultant for the viaducts indicated that load ratings were controlled by the shear stresses at these locations.

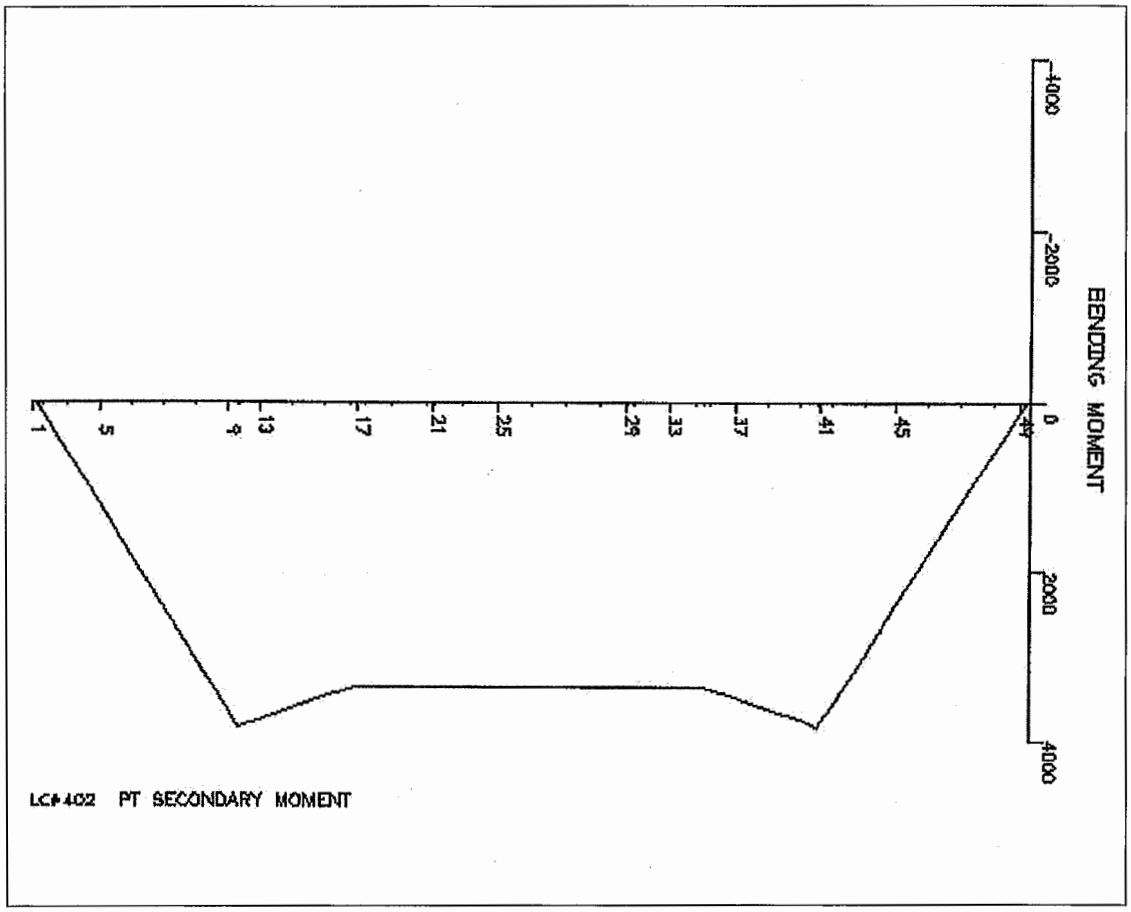

FIGURE 8. Post-tensioning secondary moment.

The splices were located near the points of counterflexure within each span and resulted in girders with reasonable lengths and weights for shipping and lifting (a maximum girder segment length of 104.5 feet and weight of 59 tons). Reinforced concrete diaphragms were provided at the pier and abutment locations within each span, and three galvanized steel intermediate diaphragms were provided in each span. The contractor elected to take advantage of these permanent steel diaphragms during the erection of the girders and used them to stabilize the girders, thus eliminating the need for temporary diaphragms. Steel diaphragms are generally easier and faster to erect than concrete diaphragms and are more advantageous where erection takes place above railroad tracks and active roadways. However, this advantage is somewhat offset by the superior durability of concrete diaphragms.

The splices were made of 1-foot-wide CIP concrete. Shear keys were provided at the ends of girders for splicing, while U- and hook-shaped reinforcing steel bars extended from the bottom bulb and top flange, respectively, into the splice zone. Shear stirrups were also provided in this zone. The post-tensioning ducts, which were each protruding 3 inches beyond the girder ends, were spliced in this zone (see Figure 9).

Erection Means, Methods & Sequences. The erection of the NEBT girders called for careful planning. The challenges included heavy girder weights (1.14 klf), adjacent active roadways and the electrified Amtrak NEC (which limited access to most areas). Several "pick points" were needed for each girder line that required the use of a very large cribbing area for the placement of multiple cribbing mats. Beam loads of 122,000 pounds with a lifting radius of 85 feet and an

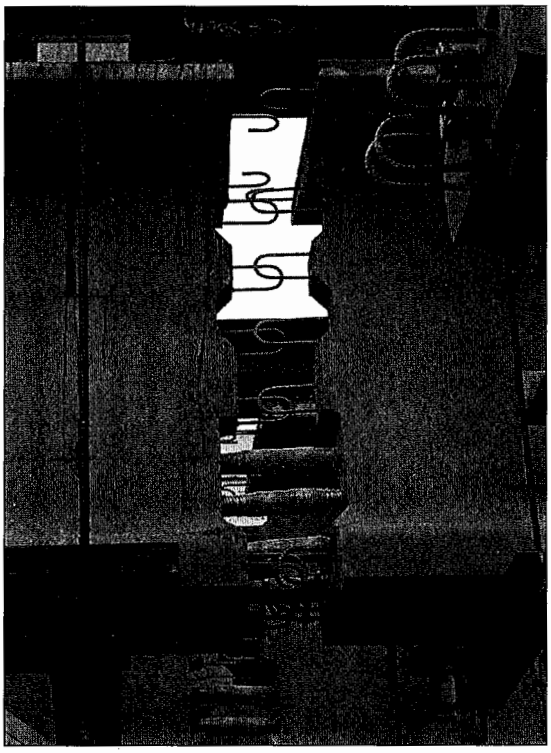

FIGURE 9. A splice detail.

Amtrak requirement that the crane be designed for 150 percent of the loads necessitated the use of a larger crane. For the picks with better access such as directly over Cranston Street or Pier 2, a smaller crane was used.

During the erection of abutment and pier girders, temporary erection towers (TET) were required at the back spans to support these girders. The TETs were centered along the splice locations supporting both the abutment and pier girders (see Figure 10). Erection brackets were required for the placement of the drop-in or center girders. These brackets were attached to the ends of these girders prior to lifting. Upon placement of the girders, the brackets were secured to the ends of the pier girders (see Figure 11). After completion of the SB viaduct, the TETs and the erection brackets were removed and reused for the erection of the NB viaduct.

The erection sequence is shown in Figure 12 and follows this order:

1. Once the abutments and piers were in place, and the TETs were placed, the abutment girders were erected, and permanent intermediate diaphragms were also installed.

2. Pier girders were then erected and the pier and abutment girders were connected at their ends using strong backs. Tie downs were provided to resist the uplift that would occur during the next step. Pier and intermediate diaphragms were also constructed.

3. Erection brackets were attached to the two ends of the drop-in girders and then the girders were hoisted in place and secured to the pier girders. After the post-tensioning ducts were spliced and rebar placed, concrete was placed at the splice locations. Once the concrete achieved a strength of 4,500 psi, two tendons in each girder were stressed and grouted.

4. The end diaphragms and the partial depth precast deck panels were constructed next. High-performance concrete (HPC) for the deck was placed after deck rebar was installed, and the two remaining tendons were jacked and grouted.

5. Finally, abutment backwalls, approach slabs and traffic barriers were constructed and the viaduct was opened to traffic.

Electrified Track & Railroad Coordination. A major concern during the design and construction was working in close proximity to the electrified Amtrak high-speed Acela service. This track alignment traversed the center of the project footprint at a significant skew (53 degrees from perpendicular to the baseline). The presence of the tracks affected many items, including the positioning and erection of the drilled shafts and piers, and the erection of the NEBT girders. The overhead catenary system providing the electricity to power the trains further complicated the erection procedure. For example, erection brackets, which were required to erect the center girders at the span splices, were directly over the Amtrak lines (see Figure 13) and at times as close as 6 inches to the catenary wires themselves.

Installation of any girders near the Amtrak lines had to be performed at night, when Amtrak allowed for two track outage and

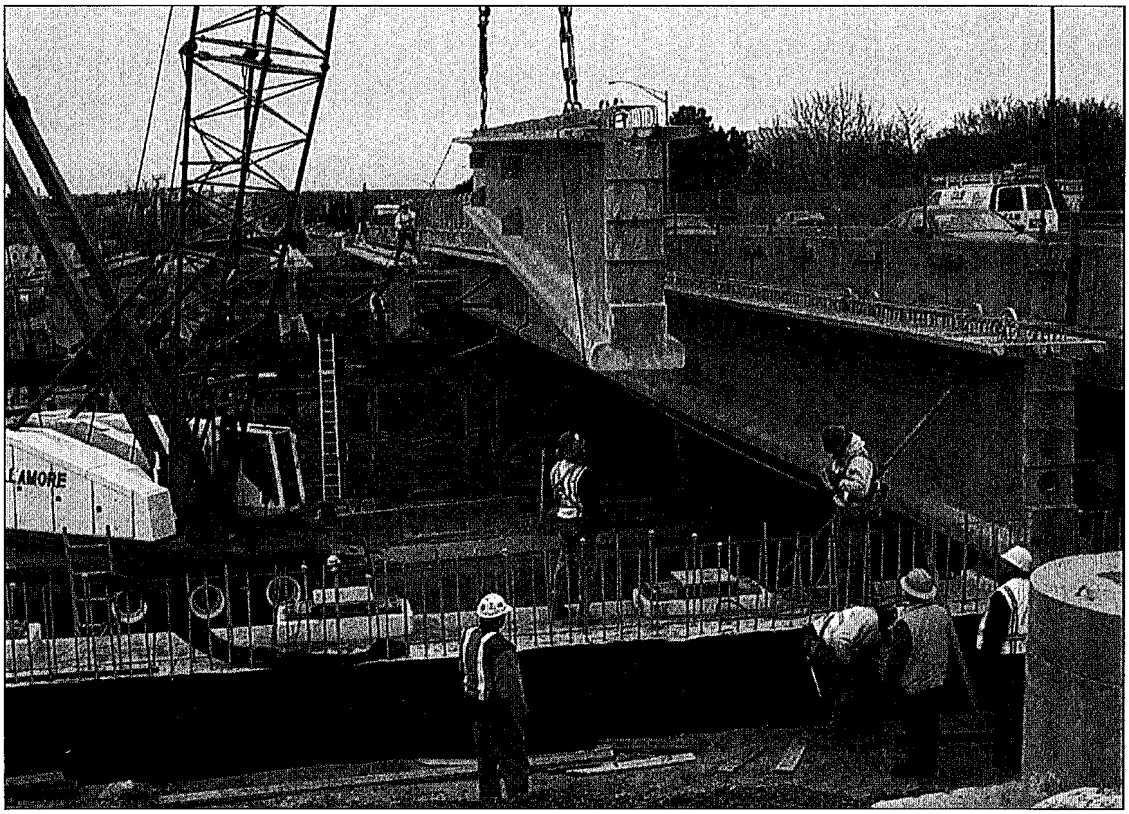

FIGURE 10. Placement of an abutment girder.

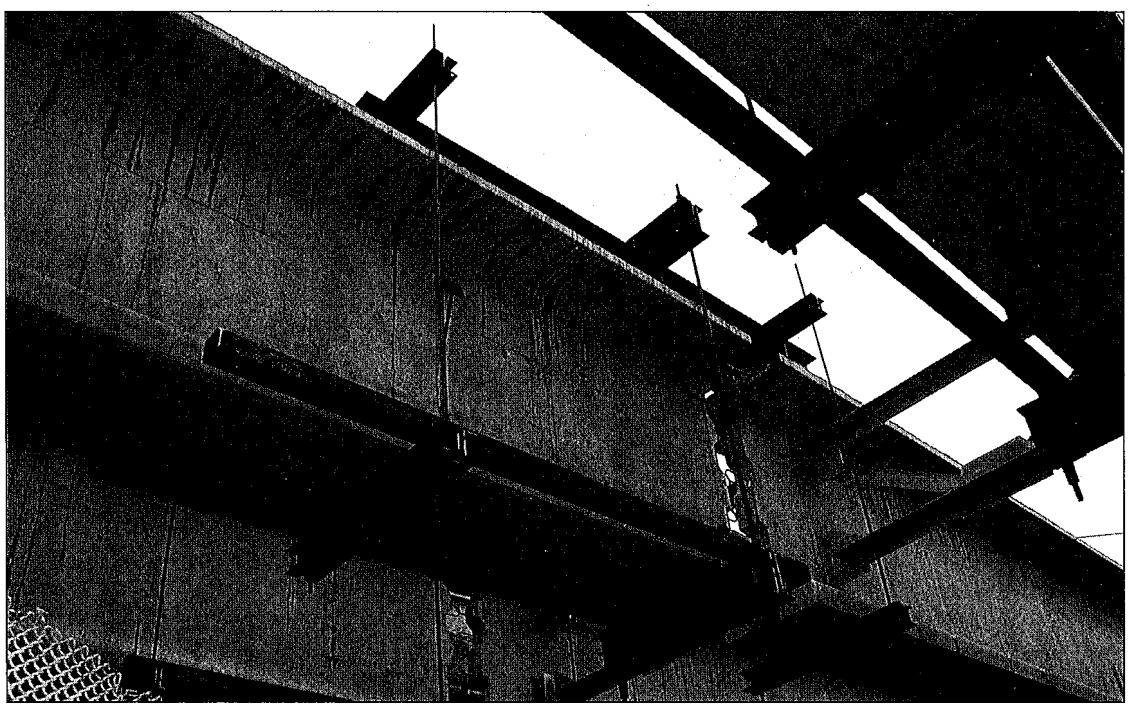

FIGURE 11. An erection bracket.

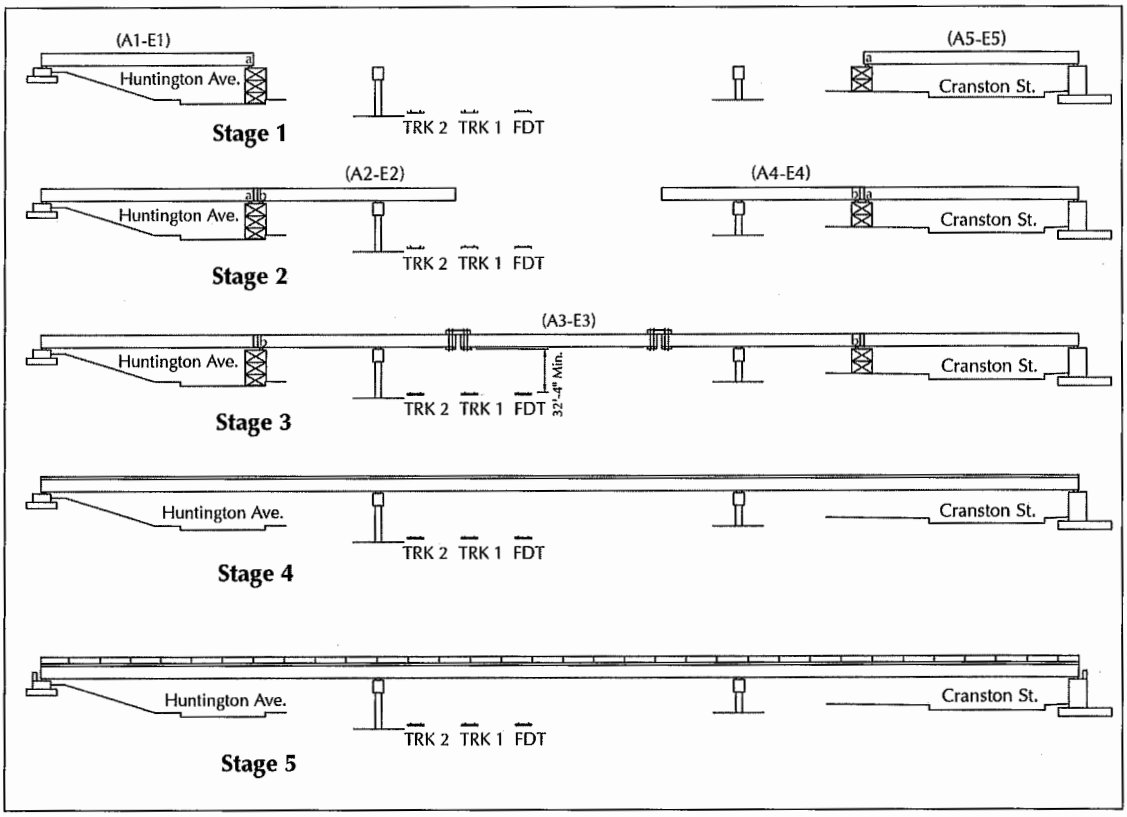

FIGURE 12. The erection sequence.

de-energizing of the lines below. This requirement limited the contractor's production rate. The original contract called for the contractor to have a window of track access between 11:30 P.M. and 4:00 A.M. seven days per week. Following the events of September 11, 2001, Amtrak added several runs between Boston and New York and was forced to perform track maintenance in the very early morning hours. The contractor was sometimes limited to a two-hour window in which to mobilize equipment near the tracks, perform the operation and demobilize away from the tracks.

The Deck System. The deck system consists of partial depth precast concrete stay-in-place (SIP) forms and HPC with a minimum concrete strength of 6,000 psi. The SIP forms were specified because the deck had to be constructed over railroad and vehicular traffic, and also to expedite the construction. During the type study phase, both galvanized steel forms and partial depth precast pretensioned panels were considered and the latter was selected for its superior durability and its compatibility with the NEBT girders. Since reflective cracks have at times been a problem associated with this system, particular attention was paid in developing details and specifications for this item. The designers carefully reviewed the available literature and studied RIDOT's and other state agencies' experience regarding precast SIP forms. A few of the specific measures included:

- Ensuring that the pretensioning strands were well developed beyond their support point.
- Ensuring that the panels were properly grouted at their support point to avoid any settlement after placing the HPC deck.
- Placement of a small amount of reinforcement (#4 bars at 1 foot) in the longitudinal direction directly above the panels.

The precaster of the NEBT girders was also the precaster for the 480 concrete deck panels

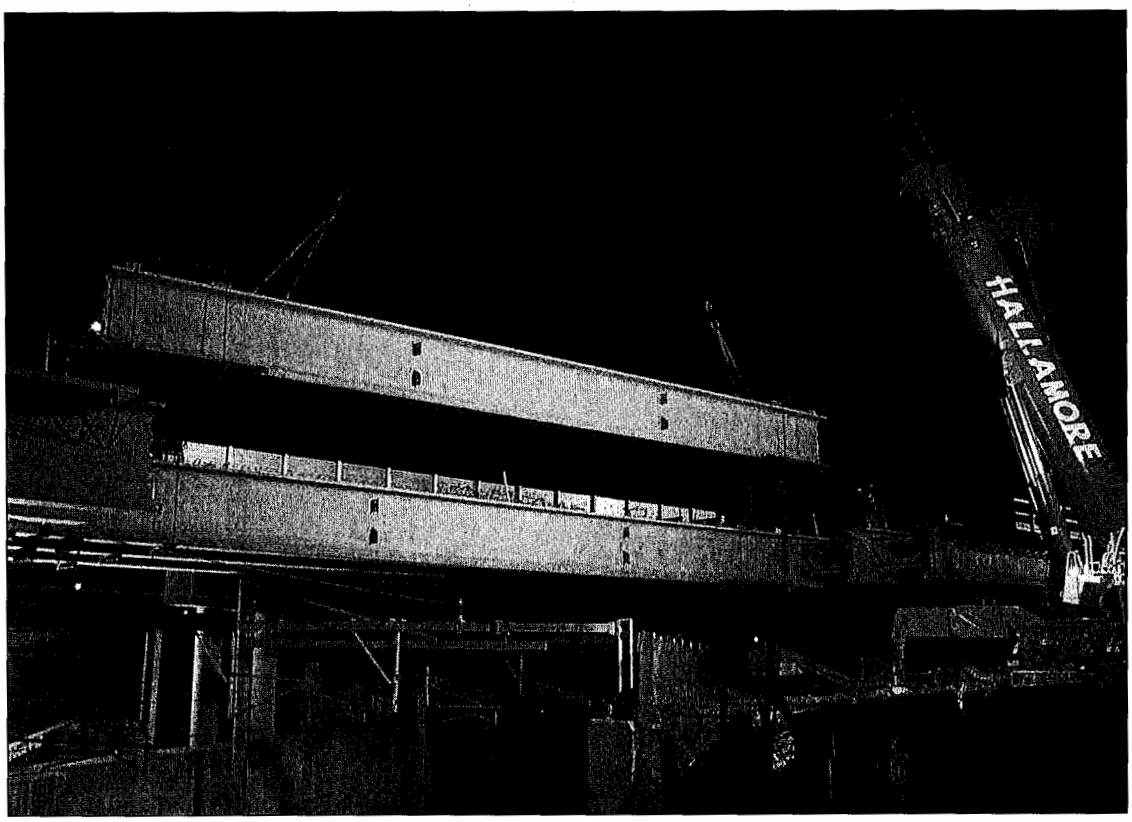

FIGURE 13. Installing girders over the electrified tracks at night.

to be used as SIP forms. These panels varied in plan from roughly 6 to 8 feet in length and from 7 feet (NB) or 8 feet (SB) in width; all were 3.5 inches thick. The panels had pretensioned single strands placed transverse to traffic and mild steel parallel to traffic.

The contractor elected to use plastic jackscrews as a temporary leveling device.[5] Once the panel was placed at the proper elevation, a foam backing previously installed between the panel and the edge of the girder acted as a form that allowed the placement of a grout bed (which would be the permanent load transfer mechanism from the deck to the girder). Once this grout bed achieved adequate strength, the plastic leveling screws were removed and the remaining void filled with grout. This system proved to be very efficient and permitted the contractor to easily adjust the panels' elevation and to place all the deck panels in less than a week per viaduct.

The HPC deck was also placed successfully. RIDOT engineers have substantial experience and expertise in the placing, testing and curing process for HPC and they have developed a thorough standard specification for doing so. To the satisfaction of all project participants, no visible shrinkage or reflective cracks have been observed on either viaduct.

Precasting of the NEBT Girders. The precaster produced 50 NEBT girders for this project (see Figure 14). The precaster was able to simultaneously cast three NEBT girders in the casting bed. On average, the production rate was one girder per day. To produce the modified NEBT girders specified for this project, the precaster simply added a flat spacer piece to the web section of the form to achieve the required added height. Also, the NEBT girders for this project required a 0.5-inch wider web to accommodate twelve-strand tendon ducts. The forms were controlled by hydraulic lines that moved them into place once the reinforcing cage was assembled. To accommodate the additional web/beam thickness, the forms were simply placed at the appropriate dis-

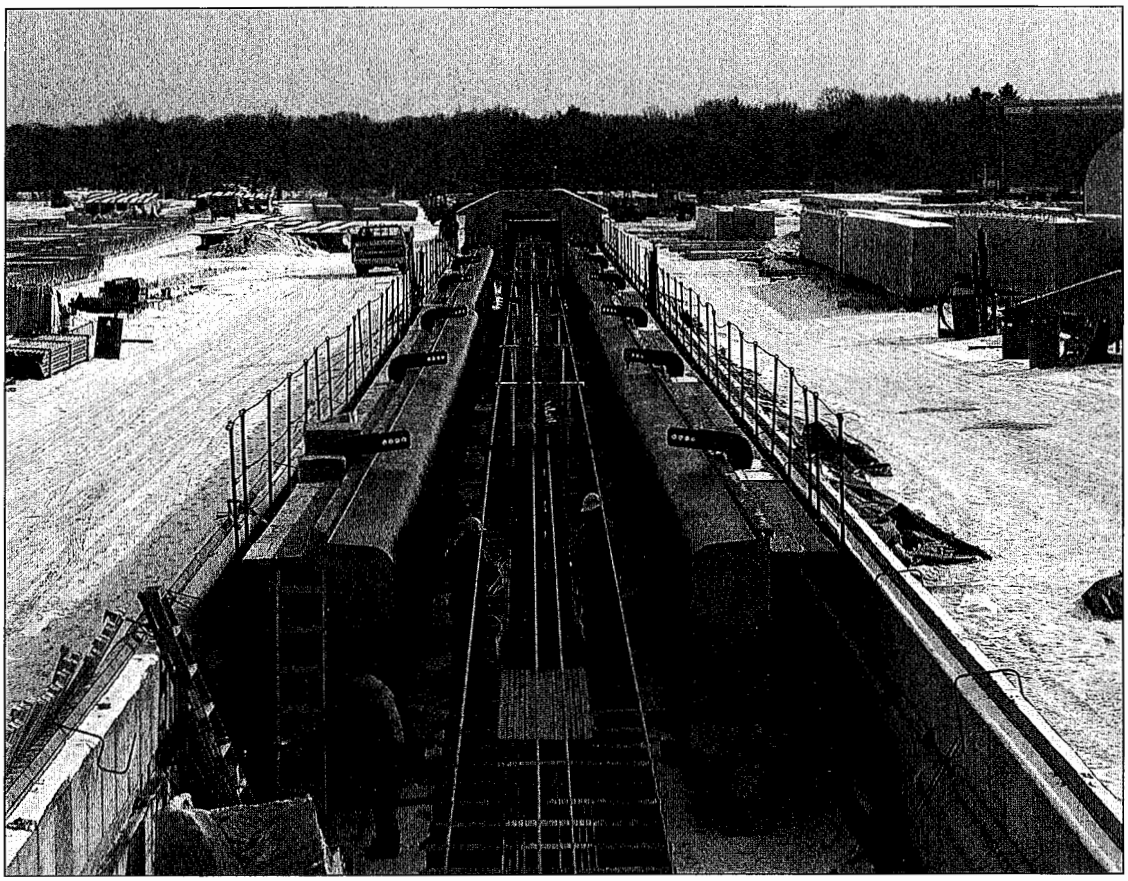

FIGURE 14. The casting bed.

tance apart. The top and bottom flanges were in essence maintained as the standard, though some templates were required at the bulkheads to properly align the pretensioning strands. No harped pretensioning was required for this bridge, but sheathed strands were used.

Post-Tensioning & Grouting Issues. As this project was going out to bid, the standards and specifications for post-tensioning grouts and grouting operations were undergoing major changes nationwide, primarily because of the problems brought to light in Florida in a few post-tensioned structures previously built. These problems were attributed primarily to grout application, such as the bleeding of grout water due to poor mixing controls (which resulted in the corrosion of the strands). Other problems that prompted closer scrutiny of grouting field practices included leakage through end anchorage protection details and an overall deficiency in the implementation and inspection of grouting procedures. According to the National Bridge Institute, the performance of prestressed concrete bridges is significantly better than bridges constructed with other materials. The revised specifications addressing these issues ensure that a higher standard is maintained.[6]

With this background, the designer recommended that the project's specifications be updated to reflect the latest improvements. Therefore, the project's specifications were revised to include:

- prepackaged grout mix for consistent blending of grout and admixtures;
- the inclusion of thixotropic grout properties for even distribution within the duct;
- the addition of downstream grout vents at high points to allow air to escape and

greatly reduce the occurrence of air pockets;

- injection of grout from grout tubes located at low points; and,
- the use of high-speed mixers and specific pumping pressures.

These items, combined with some simple on-site testing procedures, greatly increased the consistency and quality of the mix and application into the ducts. In addition, the post-tensioning subcontractor was required to have experienced technicians who possessed certifications from the American Segmental Bridge Institute.

A Problem Encountered During Grouting & Post-Tensioning

Despite the specification improvements, this installation was not without its unanticipated challenges. A serious problem was encountered during the grouting and post-tensioning operation for the two lower SB tendons (post-tensioning Phase 1) in the exterior girder. This problem happened when the contractor placed the strands and left them in the duct awaiting stressing. The contractor stressed the lower tendon and began to grout it. As grouting progressed, a leak was discovered at several of the grout tubes at the same location. The leaks were discovered to have been caused by a drilled hole through all the four tendon ducts, in effect connecting them. The problem was not identified until the grouting of the first duct took place. Grout infiltrated the second duct and locked the tendon awaiting stressing into a relaxed state. It also placed grout in the third duct, thereby reducing the diameter of the duct and eliminating the required area for installation. The contractor was unable to flush the ducts and the grout hardened in place. This incident highlights the importance of following all procedures, such as performing a pressure check of the ducts prior to grouting and the benefit of having an unlimited supply of water to flush out unwanted grout.

Several forensic processes took place to identify the extent of the problem. The contractor hired another engineering firm to determine how much of each duct was occupied. Both destructive (exploratory openings) and non-destructive (penetrating radar) tests were completed. Only an approximate limit of sound grout and concrete could be determined. Endoscopic photography images were presented to document these findings.

To remove the migrated grout in the third duct, the contractor proposed to cut an access hole into the duct and hydro-demolish the grout (see Figure 15). RIDOT agreed to view a demonstration of this procedure and determine whether the process was appropriate for use on the actual beam. A simulated beam section was created and a grout layer placed and successfully removed with the high-pressure water jets. The appropriate pressure threshold was established through this test to avoid damaging the post-tensioning ducts.

The second tendon was stressed despite having an unknown length of strand grouted in a relaxed state. Careful measurement of the expected and actual strand elongation was recorded. This information, coupled with the information provided by the engineering firm that performed the exploratory program, was useful in determining how much of the strand had been engaged. This length was determined to be 10 to 15 feet within the end span. Vacuum grouting of the un-grouted regions addressed the issue of corrosion protection; however, the loss of post-tensioning force in the unstressed length still remained. The contractor proposed to resolve this problem by external post-tensioning along the affected girder line. RIDOT and the designer allowed the contractor to do so only if the external post-tensioning were placed in between the girders in order to avoid negative visual effect (see Figure 16).

The external post-tensioning design was performed by the contractor's engineer and submitted for approval. Concrete anchorage blocks were constructed at the pier and abutment diaphragms with steel struts to prevent torsion of the eccentric anchorage. An x-ray analysis provided the exact location of post-tensioning, pretensioning and mild steel in the girder and was used to create a bolt template for the strut anchor plates. A single end jacking arrangement was installed at the abutment. Epoxy coated strands, placed within

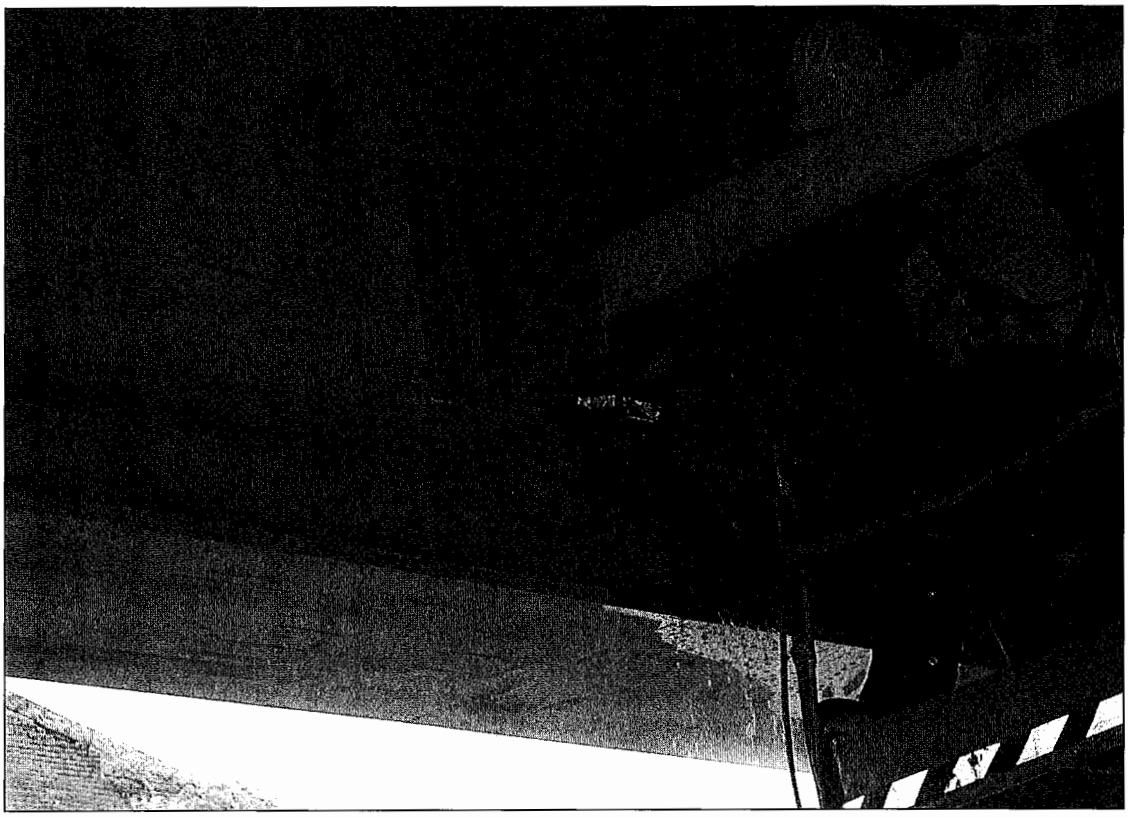

FIGURE 15. Removal of duct grout using hydro-demolition.

high-density polyethylene duct, were then stressed and grouted.

Lessons Learned

The problem encountered above highlights the importance of a well coordinated and executed grouting and post-tensioning operation. For these types of bridges, it is imperative to make sure that the subcontractor performing the post-tensioning and grouting operation is experienced; therefore, inclusion of prequalification requirements in the bid documents is essential. In this project, the NB viaduct post-tensioning and grouting operations proceeded more smoothly than for the SB viaduct because a different crew with greater experience oversaw the NB viaduct operation. The experience gained in the SB viaduct allowed the contractor to build the NB viaduct in nearly half the time that the SB viaduct was constructed.

During the course of constructing the SB viaduct, both the contractor and the designer became aware of several items that could be improved during the construction of the NB viaduct. Some of the improvements were:

- The precaster initiated a decision to pressure test the post-tensioning ducts prior to shipping the NEBT girders. This test was in addition to the pressure test required prior to grout injection.
- Tendon placement hangers were incorporated at the pier locations of the beams. The purpose of these hangers (which were made of #4 bars) was to allow the stressing of a post-tensioning duct while simultaneously protecting an adjacent ungrouted duct from the risk of being crushed. The hangers were arranged similar to a ladder and the ducts were placed at the required elevation. This procedure allowed the contractor to stress the second duct's tendons prior to grouting the first duct (as opposed to the procedure used for the SB viaduct).
- The concrete mix design for the splices

FIGURE 16. External post-tensioning for Girder A.

was revised to include 0.375-inch maximum aggregate size in lieu of the 0.75-inch size stipulated in the contract. This change ensured better concrete consolidation in the CIP splice locations.

The lessons learned through the construction of the SB viaduct proved very beneficial to the project schedule. The NEBT girders of the NB viaduct were erected in a significantly more efficient manner than for the SB viaduct. In the end, the cooperative effort of all project team members yielded a quality product for the owner.

Timeline for the Cranston Viaducts

The feasibility and type studies for the project were completed in November 1998 and the final design in November 2000. The contractor received notice to proceed in spring 2001. The SB and NB viaducts were completed and opened to traffic on December 2003 and 2004, respectively. The project was substantially completed in December 2005.

Project Accomplishments

The completion of this project represents a number of achievements:

- First application of splicing NEBT girders for a continuous bridge in New England.
- Provided the required horizontal and vertical clearances for the third track.
- Replaced aging viaducts that required constant maintenance with one that requires minimal maintenance.
- Improved load carrying capacity from H20 to HS-25 loading.
- Accommodates one additional lane of traffic to each viaduct in the future.
- Use of HPC for the deck and HSC for the NEBT girders.
- Replaced fifteen spans with six spans.
- Replaced 63 columns and six pier walls with eight columns.

- Replaced fifteen joints with four joints.
- Replaced sixteen lines of girders with ten.
- Provided major aesthetic improvements.

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