Spliced Girders in New York State

When concrete girders are too long or too heavy to be shipped in lengths that meet the required spans, in-span splicing and post-tensioning of girders is an attractive solution. Longer spans are often used to reduce the total cost of a bridge, to increase the horizontal clearances under a bridge or to improve the aesthetics of a bridge. Spliced girders also lead to lower long term maintenance cost by eliminating deck joints. Splice girder technology offers a great deal of latitude in selecting the span lengths and splice locations. Splices are usually located at or near the inflection points for multi-span structures. Post-tensioning of splices enhances the durability of the structure by reducing the tendency of the splices itself and decks above to crack.

Many of New York State Department Transportation (NYSDOT) multi-span prestressed concrete bridge superstructures are spliced at intermediate supports with no post-tensioning. The main reason for splicing girders at intermediate supports is to establish live load continuity and to eliminate deck joints. Elimination of joints avoids leakage through them, which has been the main cause for corrosion of substructure elements and ends of prestressed girders. This is extremely important for the durability of bridges in New York where large quantities of de-icing chemicals are applied to the decks during winter. In addition to the above, enhancement in structural performance of these bridges is also derived from the live load continuity. Application of live load continuity splicing is limited to bridges with span lengths that are within the shipping and handling limitation.

In-span splicing with post-tensioning of concrete girders is not a new technology. Some spliced prestressed concrete bridges in New York were constructed back in the fifties. One such application worth mentioning is I-81 (NB & SB) over Oneida Lake inlet, completed in 1960. These three span bridges have two end span 23m (75 ft.) cantilevered over to the mid span by 22m (72 ft.). The 45m (147 ft.) end segments weighed about 250t. A drop in segment of 70.5m (231 ft.) was used to complete the mid span. The drop-in segment weighed about 201t. These bridges have been exposed to a highly corrosive environment due to the use of de-icing chemicals and have been carrying high volume of truck traffic. Both bridges have been performing admirably during the last 45 years.

Another important application of spliced girder technology is Rte.203 over Kinderhook Creek in Valatie, NY. This bridge with two equal spans of 53m (175 ft.) was completed in 1982. Girders were cast in four segments and shipped to an assembly yard near the job site. The segments were spliced with epoxy joints and post-tensioned in the assembly yard before installation. This bridge also is functioning very well without any significant reported problems.

In addition to the above NYSDOT has built a number of prestressed concrete spliced girder bridges during the course of last fifty years and kept up with evolution of the spliced girder technology. With the advent of High Strength High Performance Concrete along with ability of contractors to transport and handle heavier and longer girder segments, the feasibility and economy of bridges using this technology in longer span bridges is improving. Based on the superior durability of the existing spliced girder bridges, NYSDOT is moving forward with new spliced girder bridges.

This paper focuses on two new bridges currently under construction, Route 17 over Wallkill River and Recreational Trail 'A' over Taconic State Parkway. In addition to using the splice girder technology, these two bridges are utilizing the latest developments in concrete technology as well. These two bridges are expected to last more than one hundred years with low maintenance costs. Construction of both of these bridges is expected to be completed by the fall of 2006.

Route 17 over the Wallkill River

1. Spliced prestressed post-tensioned concrete girder
2. Girders made using High Strength High Performance Concrete
3. High Performance Concrete deck
4. Existing piers as temporary supports
5. Staged construction
6. Haunched girders
7. Jointless Abutments

Figure 1-Old steel bridge

A single prestressed concrete bridge with two equal spans of 54.7 m (180 ft.) is replacing two steel through-girder bridges with three spans and a total length of 99 m (325 ft, Figure 2). The old bridges had out to out widths of 44 ft. each and curb to curb widths of 40 ft. each. The superstructure of the new bridge consists of 17 prestressed concrete girders fabricated using High Strength High Performance Concrete (HSHPC) of 70 MPa(10,000 PSI) compressive strength. Girders were cast in 3 segments, installed, field spliced using a cast-in-place closure pour, and post-tensioned. The girder segments at splice locations were temporarily supported on the old bridge piers (Figure 4). These piers were removed after the splicing and post-tensioning of the girders. The concrete girder type used for this bridge is a modified New England Bulb Tee (NEBT) 1800mm (71 inches) deep. The modification was the widening of the webs to accommodate the large post-tensioning ducts. The simplest method of widening the webs was to widen the entire NEBT.

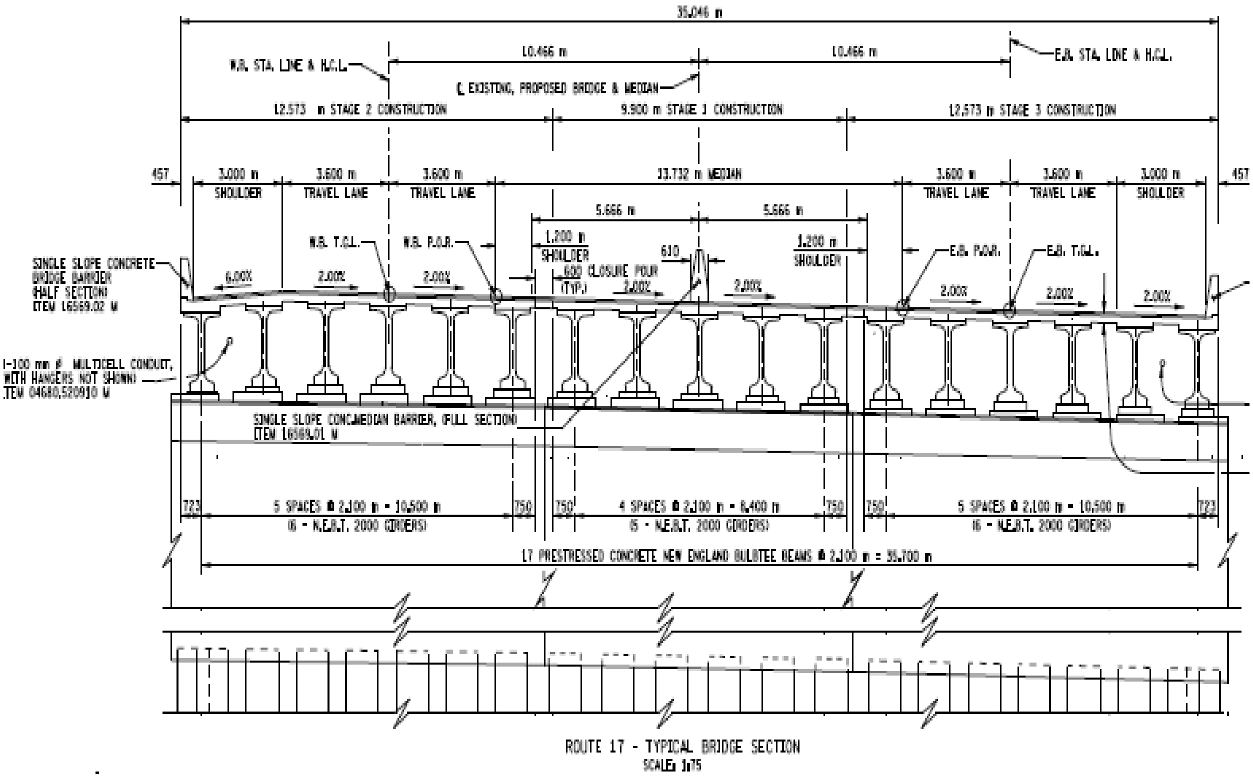
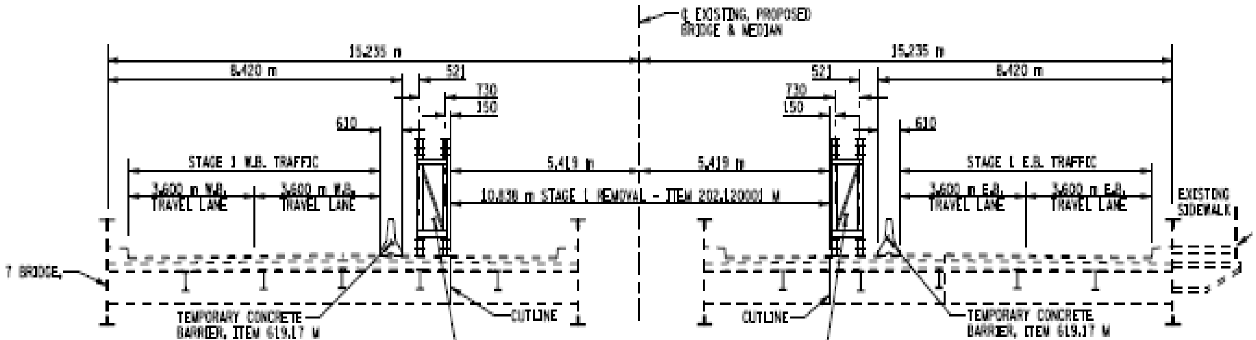


Figure 2 Transverse section of the new bridge

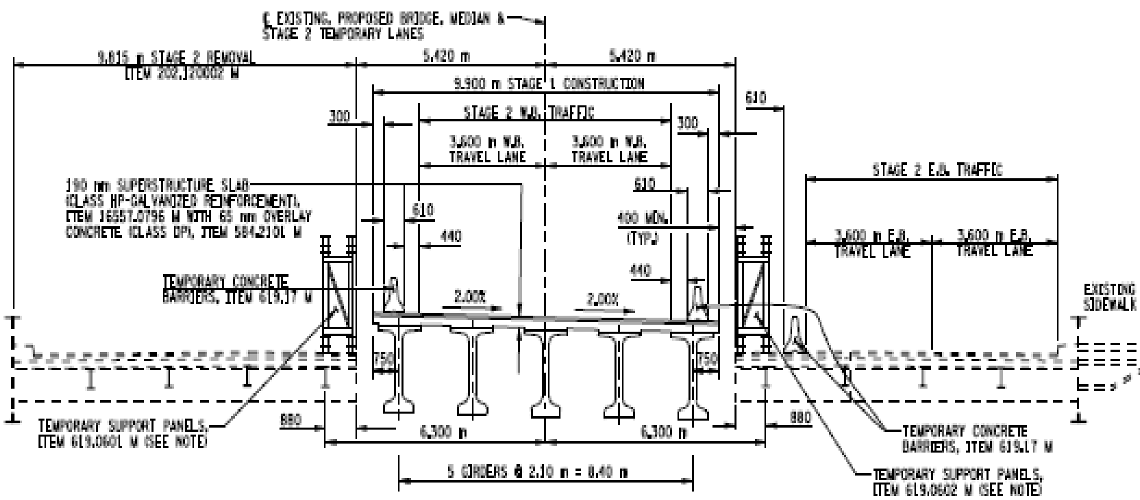
The new bridge was designed to be constructed in three stages. Interior fascia girders along with portions of deck of the existing bridges were removed first to make room for stage 1 of the new bridge. The existing bridges continued to carry traffic during the construction of stage 1. Once stage 1 was completed, traffic carried by the old bridge on the left side was moved over to stage 1 of the new bridge to construct stage 2. After the completion of stage 2 all traffic was moved over to the new bridge while the remaining stage 3 of the new bridge was completed (Figure 3).

Preliminary Design

A two span bridge with spliced girders and a three span bridge with live load continuity were the two main alternates considered for the replacement structure. In span splicing with post-tensioning of concrete girders is more expensive than splicing for live load continuity at intermediate supports. In the case of this bridge, the cost of constructing one less pier in the river more than offset the additional expense for the splicing and post-tensioning of the girders. In addition, one less pier presented less obstruction to the flow of flood water thereby eliminating the need for and cost of raising the highway profile to obtain necessary freeboard.



ROUTE 17
STAGE 1 REMOVAL
AND STAGE 1 TRAFFIC
SCALE: 1:75



ROUTE 17
STAGE 1 CONSTRUCTION AND
STAGE 2 REMOVAL & TRAFFIC
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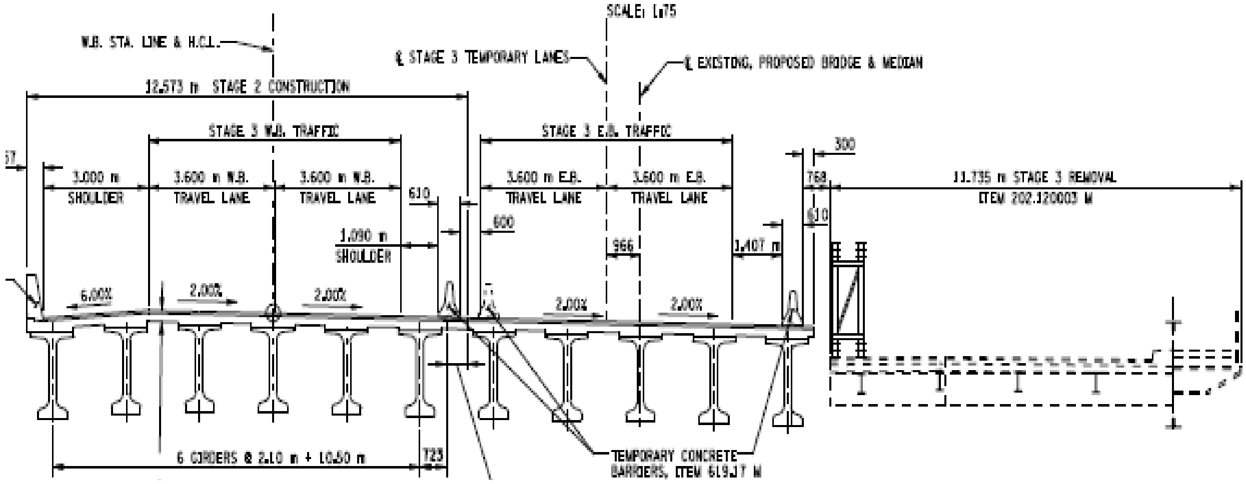

Figure 3 construction sequence in three stages.

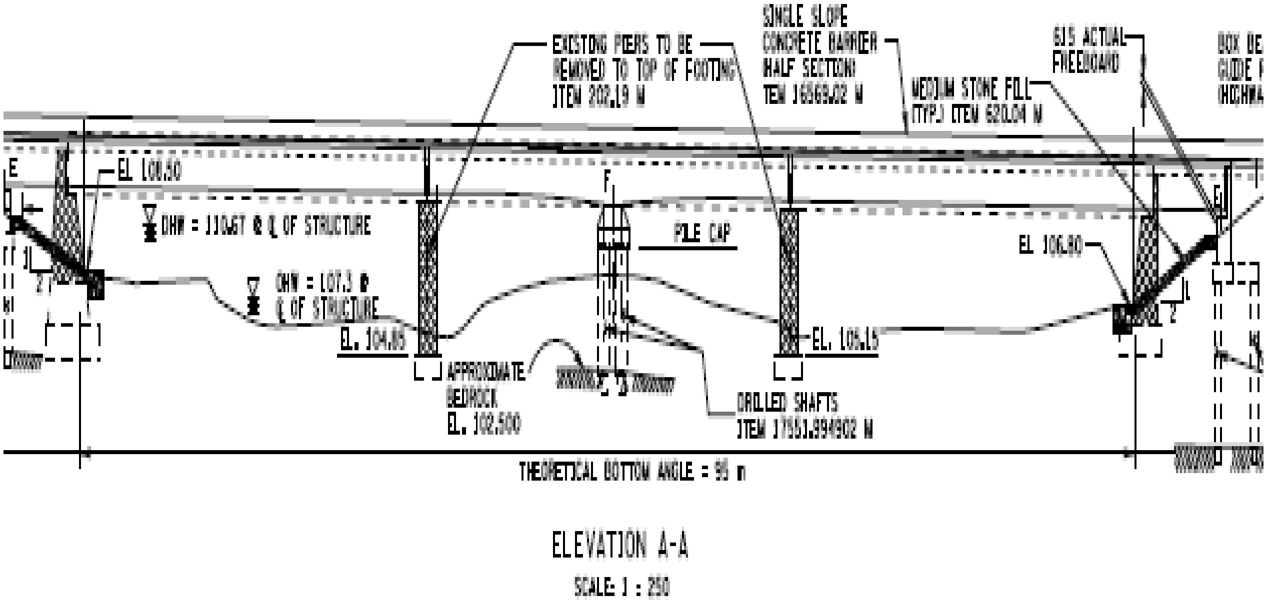

Figure 4-Elevation view of the new bridge

Girder Design

After considering various combinations of girder depth, spacing and segment weights for each design, 17 girders spaced at 2.100m (83 in.) for a total out to out width of 35.046m (115ft.) was adopted. Individual girders are 1.8m deep (71in.) modified New England Bulb Tees (NEBT), haunched at the intermediate support to a depth of 2.6m (8ft. 6in.). The webs were widened to accommodate the large post-tensioning ducts that were required to carry all of the strands. The webs were widened by widening the entire NEBT. A design with 2.0m (78in.) NEBT girder without the haunch was possible, but a deeper girder at the intermediate support was preferred for aesthetic reasons. The girder design is based on AASHTO LFD specifications with NYSDOT modifications. Design service load is HS25. The maximum tensile stress under service load was limited to .25√f’c MPa (3√f’c psi).

The design of the spliced girders used a combination of calculations and analysis by hand, by a spread sheet program and by a proprietary software package. The software has been a valuable tool, especially to compare various alternates, installation and post-tensioning sequences and associated time dependant effects. One significant challenge in designing and constructing spliced girder bridges using staged construction method is the need for accurate prediction of camber. Control of camber growth of girders after post-tensioning due to time dependant effects also is very important. Creep, shrinkage and elastic modulus of the HSHPC used for these girders was known from pre-production lab tests and were considered in determining time dependant effects due to prestress. Time lag between post-tensioning of the girders and deck placement also was considered and was controlled during construction. The haunches over the girders (overbuild above the top flanges) for the second and third stages were increased to adjust for potential variations in cambers of girders between stages.

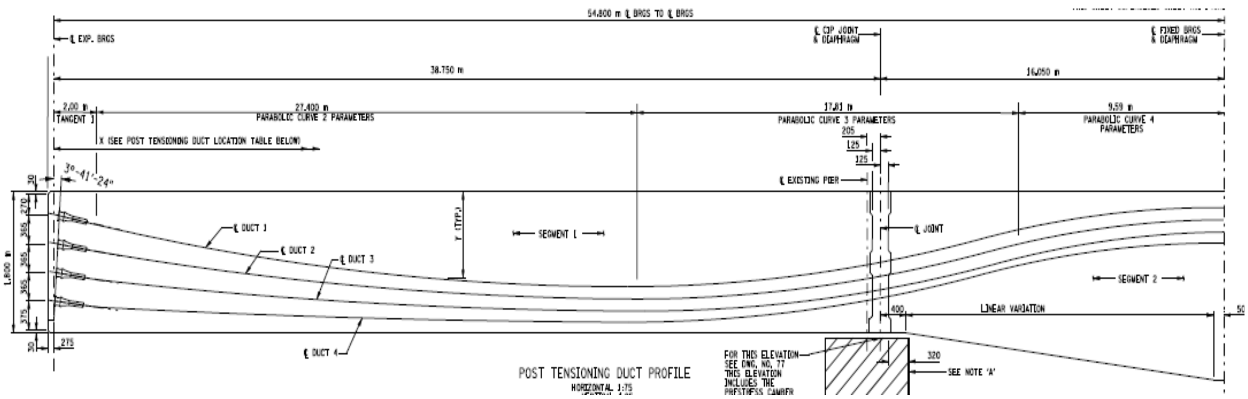

Figure 5 Profile of post-tensioning tendons, half of the girder is shown.

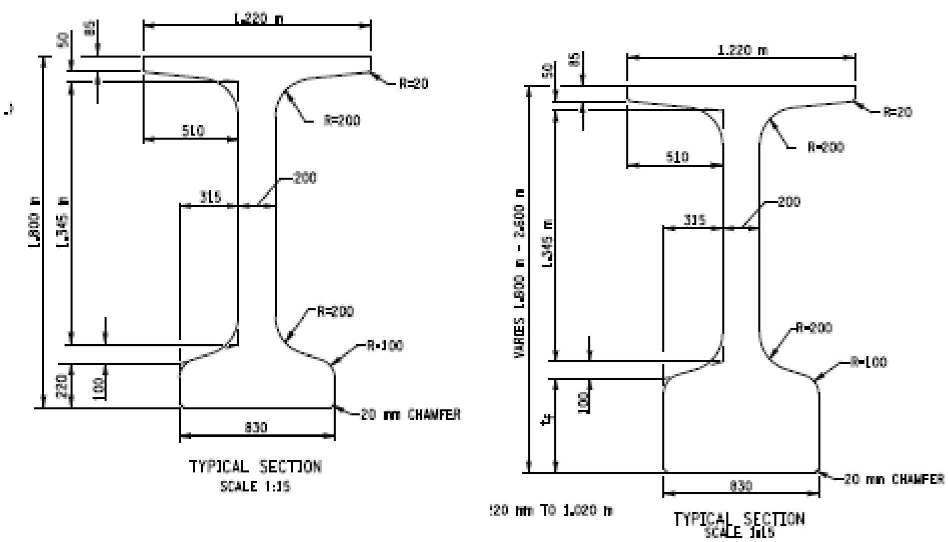

Figure 6 Girder sections, deepened bottom flange at the intermediate support

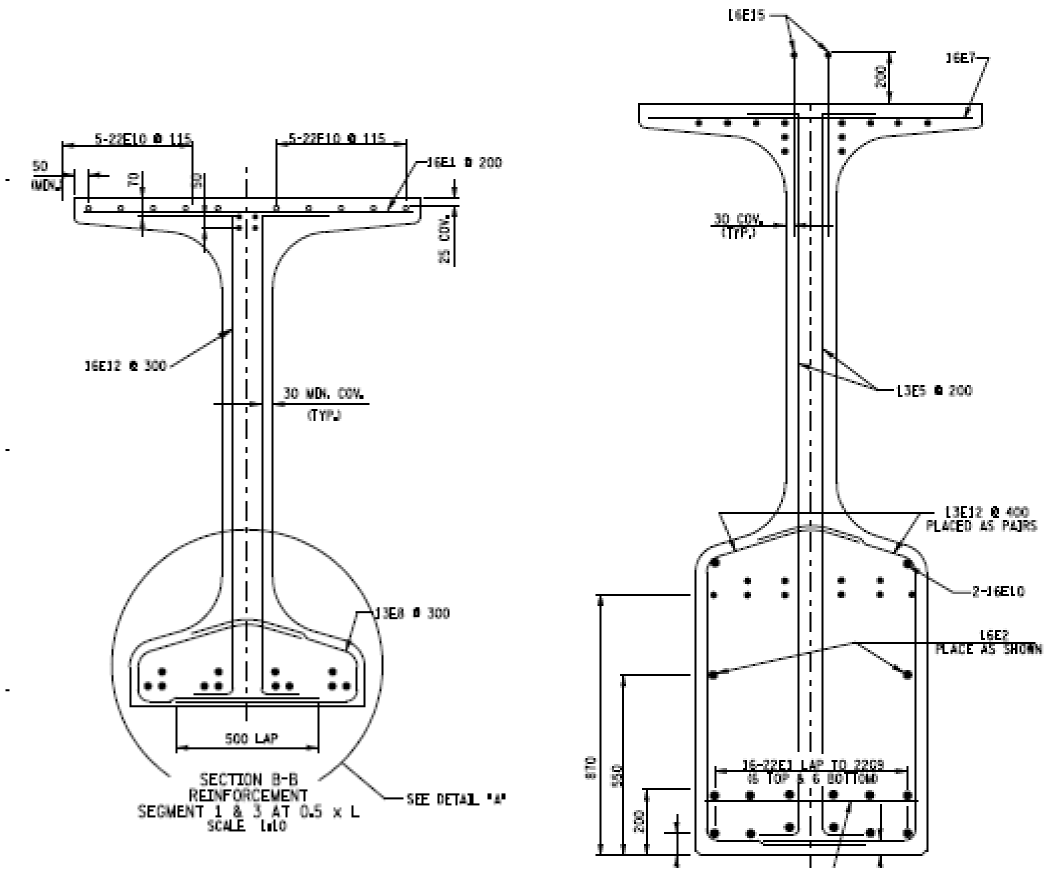

Figure 7 Girder sections showing strand locations and reinforcement detail

These girder segments were pretensioned to keep tension during handling, including 20% dynamic effects, under .25√f'c MPa (3√f'c psi). The end segments were lifted and supported using pick points near the ends during the entire handling and installation process. Twelve strands, (15mm, .6 inch) stressed to 75% ultimate were provided in the bottom flange to limit the handling stresses. The haunched segments had twelve strands (15mm .6 inch) in the top flange and 10 in the bottom flange of to deal with reversing stress conditions due to changing support locations during handling and the installation process. These segments' pick points were located at quarter points for handling. The segments were supported in the middle with the ends touch-shored at the temporary support during installation. The pretensioning provided was adequate to keep tension under the required limits with the changing support conditions.

Each stage of construction was analyzed independently for strength, serviceability and time dependent effects. The girder segments were aged more than 90 days to minimize the time dependent effect due to shrinkage and the deck was poured within 15 days after post-tensioning to limit time dependent effect due to creep. In addition to the above, creep and shrinkage values of HSHPC used are only fractions of those values for conventional concrete. Deck placement for Stage 1 and Stage 2 were successfully completed for this bridge without any complications and the transition of top deck surfaces between stages worked without a glitch.

Figure 8 Typical void in the first few middle segments of girders produced

For the most part fabrication of girder segments went smoothly. Placing and consolidating concrete from the top was difficult since ducts located in the webs of the girders created significant obstruction to the flow of concrete and to the insertion of internal vibrators through the web. Areas of unconsolidated concrete and significant voids were observed around the ducts in the first few middle segments produced. The ducts were draped up and were close to the top flange in the middle part of that segment. External vibrators were attached to the bottom flange and energy available for consolidation of concrete at the top was not sufficient, especially at the deepest part of the girder. These segments were accepted for use by the Department after appropriate repair. Once this problem was identified, the fabricator used a higher slump concrete mixture to allow the concrete to flow more easily around the ducts. Special care was exercised in placing and consolidating of concrete around each duct there by eliminating that problem in all future castings. Use of self consolidating concrete would be ideal for fabricating girder segments of this type for any future project.

Figure 9- Repair of void in the web

NYSDOT will require the contractor to use smaller coarse aggregate and self consolidating concrete for similar future projects. Use of self consolidating concrete will also help in dealing with congestion of reinforcement in the anchorage zone. Further widening of the webs should be avoided since it will result in an increase in the girder weight. Our design called for the closure pour concrete to achieve full strength prior to post-tensioning. In retrospect, the time taken for the closure pour could have been reduced had we designed for a lower strength at the time of post-tensioning. The wide top flange of the NEBT is also negative due to the need for the build up on top of the top flange resulting in increased dead load with no contribution to the girder capacity. One way to deal with this problem is to modify NEBT shapes to have a smaller top flange width. This is fairly easy for the fabricators to accommodate without making any major alterations to the NEBT forms. Cost of fabrication is significantly increased when girders with varying depth are used and should be avoided when possible.

Figure 10 Reinforcing in the anchorage zone

Installation

The installation sequence shown in the following sketch was recommended in the contract plans and accepted and used by the contactor. Use of the piers of the old bridge for the temporary support was beneficial in reducing cost. Control of the support elevation at the center pier is the key for proper installation. The design of the girder assumed that self weight of the middle segment is fully carried by the support in the middle. Careful geometry control is necessary to accomplish that in the field. Camber measured after casting of each middle segment was used to determine the top of bearing elevations so that temporary supports at the ends will be set up at the correct elevation without carrying more than 5 % of the girder dead load. Actual reactions at the ends were checked after the installation of the middle girder segments and were found to be within the required limit. The ends of the end segments at the temporary supports were adjusted to match up with the ends of the middle segments. Provisions for making these adjustments in the temporary support set up proved beneficial. Overall the installation process went well including the splicing of the post-tensioning ducts and the placement of closure concrete. The cast-in-place concrete diaphragms required by the contract plans were replaced by galvanized steel diaphragms based on a substitution request from the contractor. Diaphragms between stages were not connected until the deck placement was completed to allow unhindered deflection of girders under the deck dead load. The use of steel diaphragms made installation faster and easier.

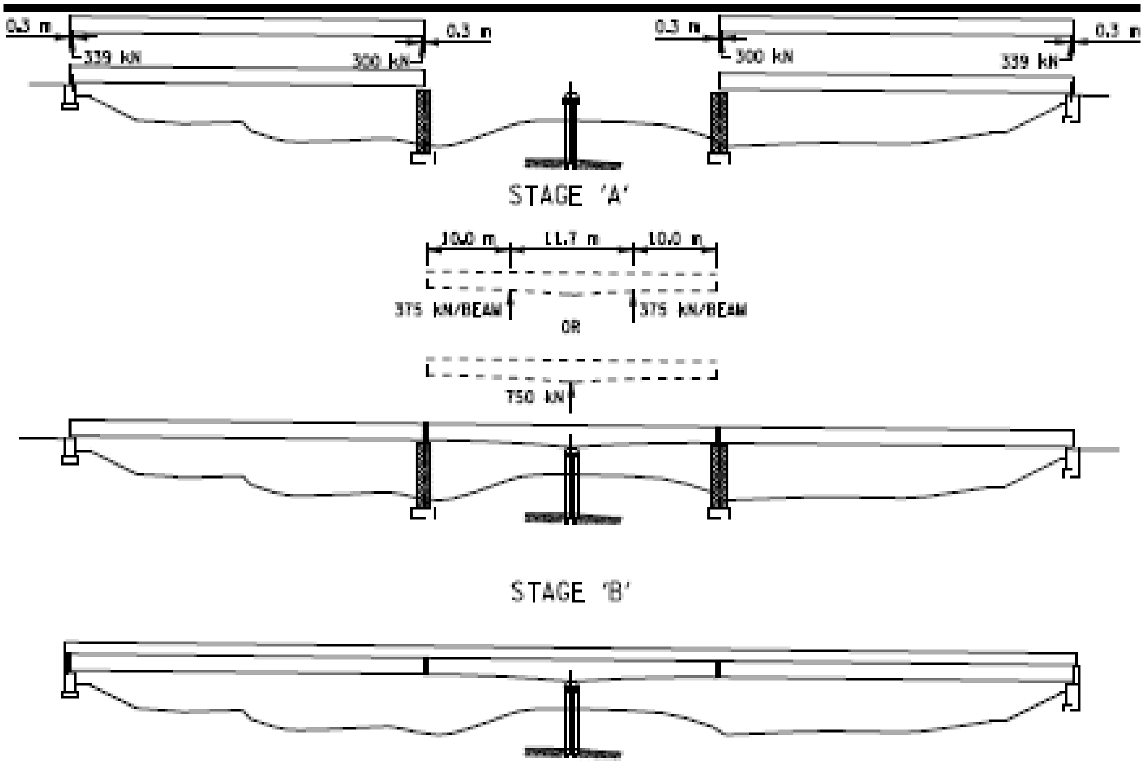

Figure 11 Installation sequence suggested in the contract plans

Figure 12 End segments on the temporary support

Figure 13 Middle segment being picked up at the quarter points

Figure 14 end segments supported at the abutments

Figure 15 view of girders after installation

The girders were fully post-tensioned and grouted before the placement of the concrete deck. This approach has some benefits and some drawbacks. It avoids multi stage post-tensioning and grouting operations and removes any concerns about overstressing girders during future deck replacement operations. Forcing the girder to take the full post-tensioning load before the deck placement produced a less efficient design. Pre-compression of the deck is possible if a portion of the post-tensioning is done after the placement of the deck. In addition, pre-compressing the deck would reduce or eliminate tension in the deck due to shrinkage and temperature gradient during deck hardening which would then reduce deck cracking in newly constructed bridges.

With proper design, overstressing of girders during deck replacement can avoided. Reduced deck cracking reduces the need for deck replacement thereby reducing the maintenance cost of the bridge. NYSDOT will be paying attention to these issues and most probably will post-tensioning girders in two stages. The second stage of post-tensioning will be after the hardening of the deck and will be designed not produce overstressing during deck replacement but will pre-compress the decks sufficiently to reduce tension cracks.

Figure 16 Deck placement operation

Recreational Trail 'A' over Taconic State Parkway

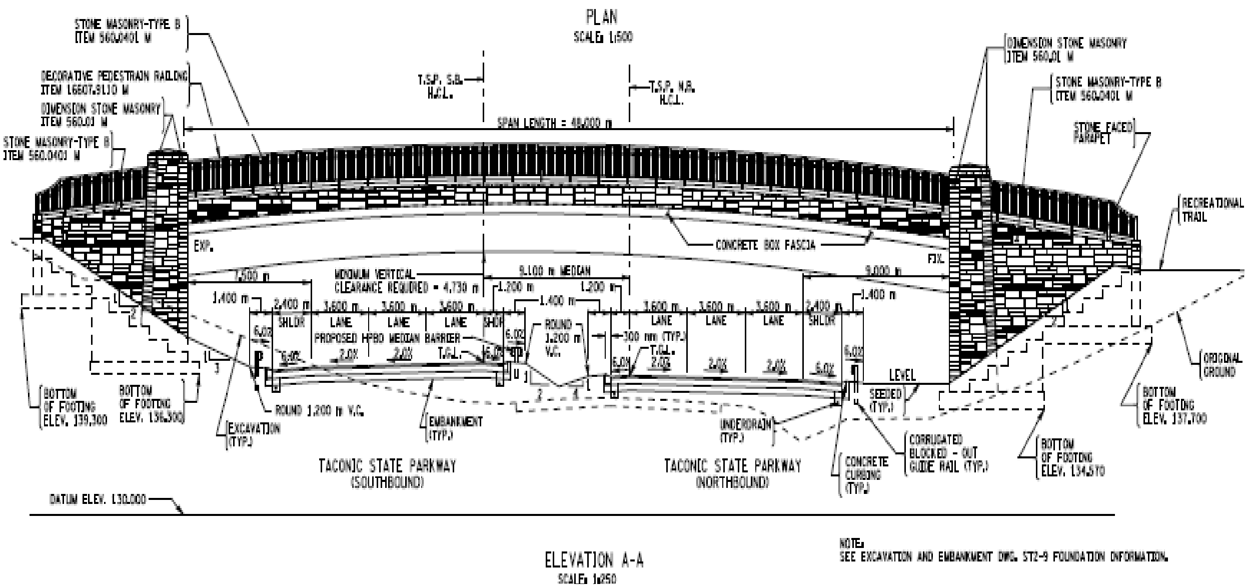

Figure 17 Elevation view of the bridge.

Preliminary Design

This is a single span pedestrian bridge with a span of 48M (160 ft) and width of 4M (13 ft). Unique features of this bridge are:

1. Spliced prestressed concrete girder
2. Trapezoidal box shapes
3. Girders are made using HSHPC
4. Concrete used is self-consolidating
5. Unique placement system for concrete utilizing the advantage of SCC
6. Architectural stone masonry finish
7. Arch shaped girders achieved using splicing of the girder

This is a unique application of splicing of girders to create the arch effect. The girder consists of three precast segments spliced and post-tensioned at the project site. Individual segments are slightly curved by the use of pre-cambered forms and splicing with appropriate adjustment to the geometry produces the arch effect for the girder. Stone masonry on the fascias and the abutments makes this an aesthetically pleasing structure.

Girder Design

This is a unique trapezoidal shape developed for this specific application since no standard shapes were available that fit the need. Individual segments were post-tensioned to resist handling loads with dynamic effects without exceeding a maximum tensile stress of .25√f'c MPa (3√f'c psi). The final post-tensioning in the field develops required strength. The post-tensioning tendons are located inside the webs of the boxes. The girder design is based on AASHTO LFD specifications with NYSDOT modifications. The maximum tensile stress under service load was limited to .25√f'c MPa (3√f'c psi). Time dependant effects were considered in the design even though the secondary effects were less critical since this is a single span bridge.

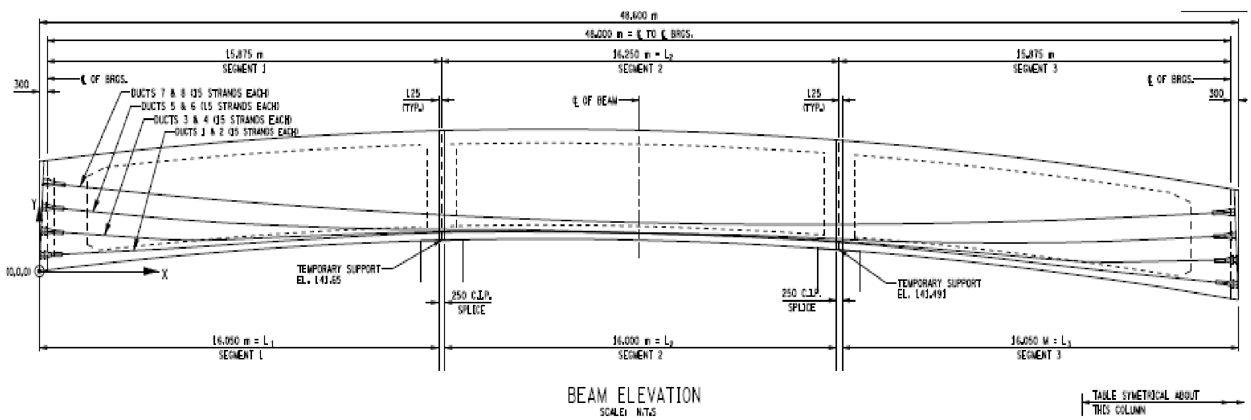

Figure 18 Elevation of the girder showing post-tension tendon profile

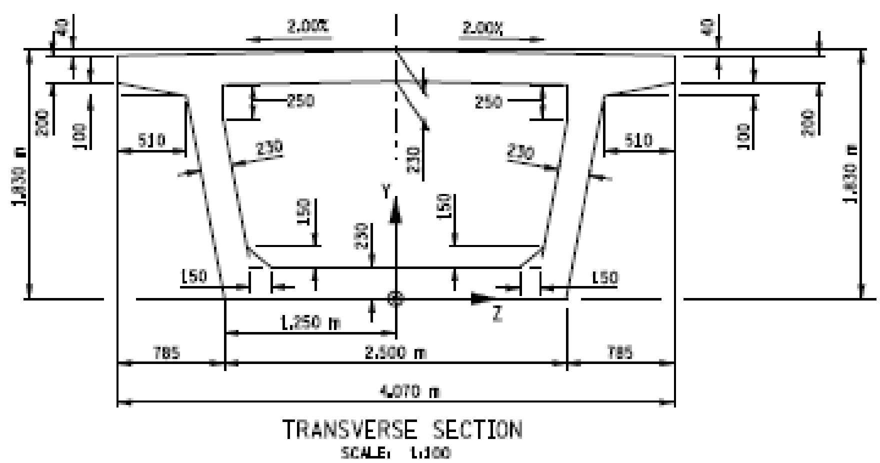

Figure 19 Box girder transverse section

Fabrication

These segments were fabricated using Self Consolidating High Strength High Performance Concrete (SCHSHPC) of 70 MPa (10,000 psi) compressive strength. Concrete was deposited on one side of the form and flowed across the bottom flange by the fluid pressure of the self consolidating concrete. The spread of SCHSHPC used was around 700 mm (28”). Since NYSDOT did not have experience with this method of concrete placement for box girders of this size, a mock up section was poured and tested before this method of placement was approved for use on the actual segment. The placement operation went very well and segments came out near perfect. In fact, this may be the way segments will be fabricated using self consolidating concrete in future. Careful consideration was given to the direction of the concrete flow and the continuity of the flow to avoid chance of entrapment of air. Segments produced for this bridge are proof that if properly executed box segments of any size and shape can be produced using SCC with considerable savings in labor cost. There is a tendency for voids to float up due to the upward pressure from SCC. Use of a robust hold down system (Figure 23) is very important to avoid problems during casting. Figure 20 shows the details of the anchorage zone. Reinforcement shown in these sketches along with the additional anchor reinforcement created a very congested location for placing concrete. Use of self-consolidating concrete proved very helpful. In fact there were no significant problems in placing and consolidating concrete in this area.

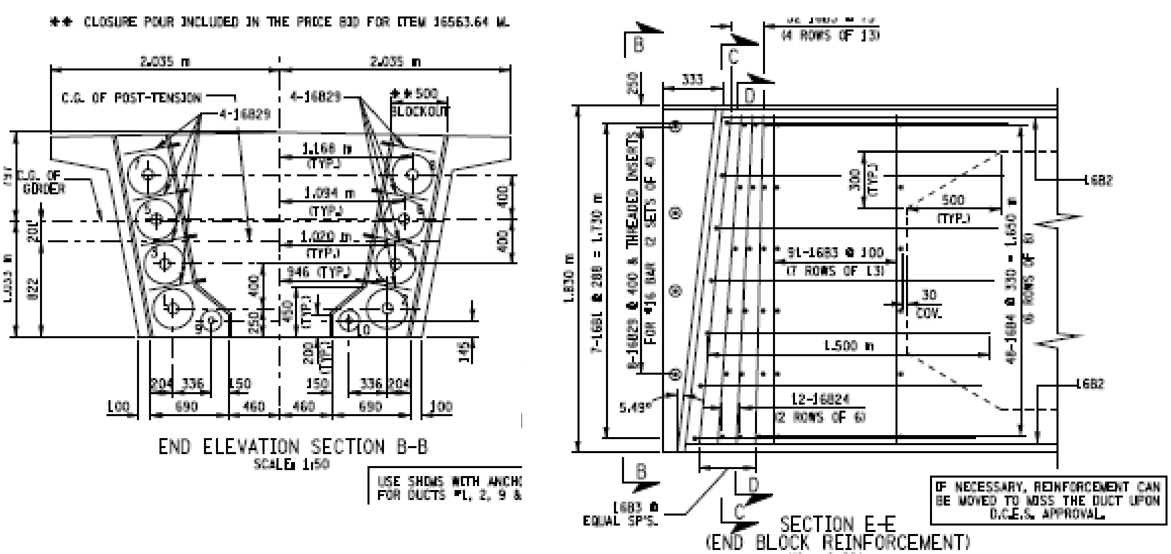

Figure 20 Post-tension anchors in the end and details of anchorage zone reinforcement.

Figure 21 Reinforcement for the bottom and sides of the segment and ready to receive void producing forms

Figure 22 Hold down system for the void producing form

Figure 23 Finished girder segment

Figure 24 Finished girder segment end view of the splice end

The girder shown in Figure 23 consists of three segments. Each segment was designed with prestressing to prevent tension during handling and erection process. Each segment has a small curvature along the bottom. The segments are spliced together in the field and post-tensioned to obtain the desired curved shape for aesthetic reasons. Figure 24 shows the typical end of the girder at splice locations. The internal void allows inspection of the inside of box girders. The internal diaphragms at the splice locations have openings which will allow inspectors to pass through. An access opening from the bottom of the girder is provided at one end of the box girder.

Installation Sequence

Figure 25 shows the sequence of the installation of girder segments. Temporary bents, designed and detailed by the contractor and approved by the Department are used for supporting the girder segments at the splice locations. Cast-In-Place concrete closures (Figure 26) establish the continuity of the girders. Longitudinal post-tensioning is done after the closure concrete reaches required strength to develop full capacity for the girders. The Cast-In-Place closure also uses HSHPC with calcium nitrite corrosion inhibitor. Once the grout in the post-tensioning tendons reaches sufficient strength the intermediate supports are removed.

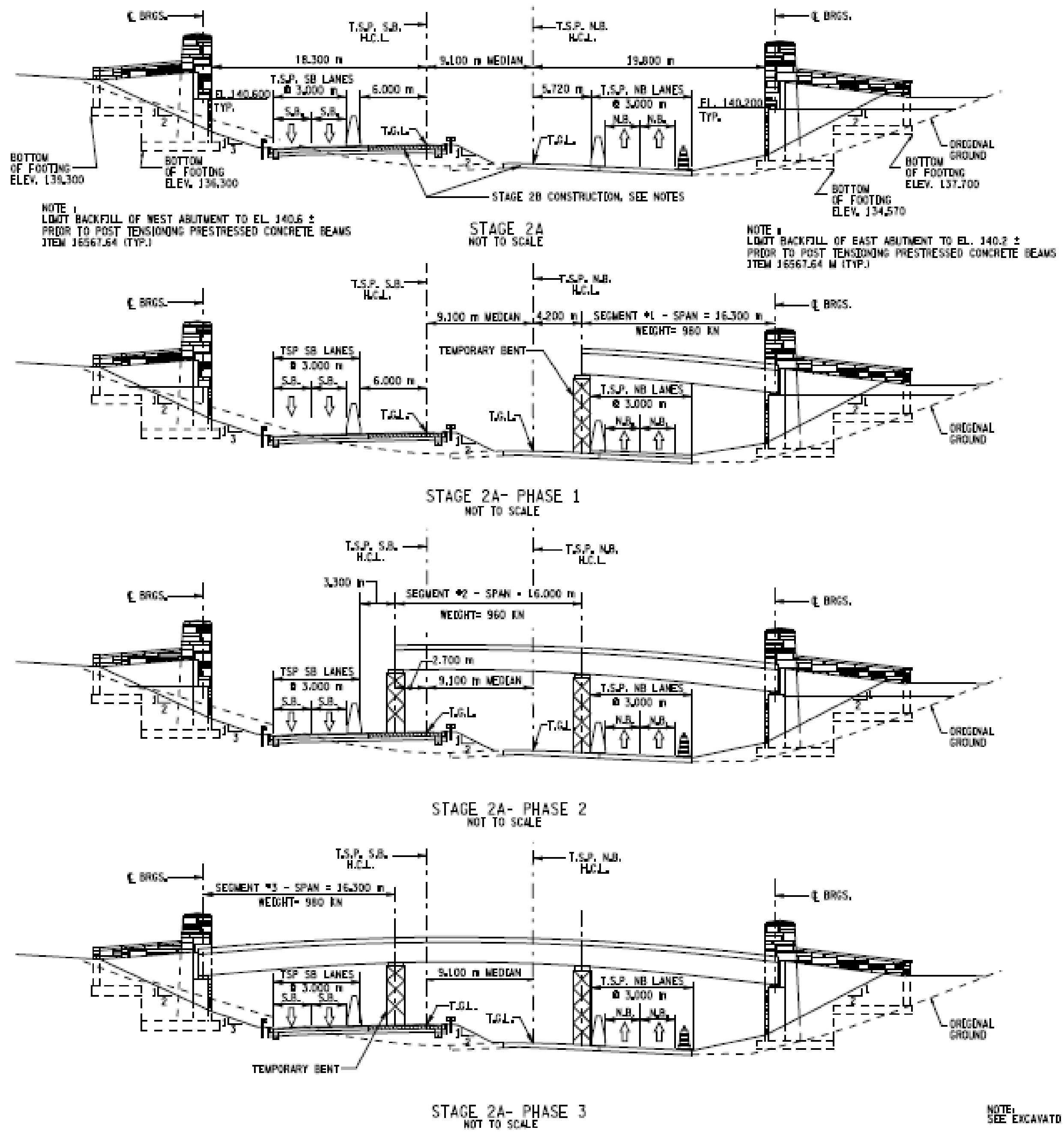

Figure 25 Installation sequence of girder segments

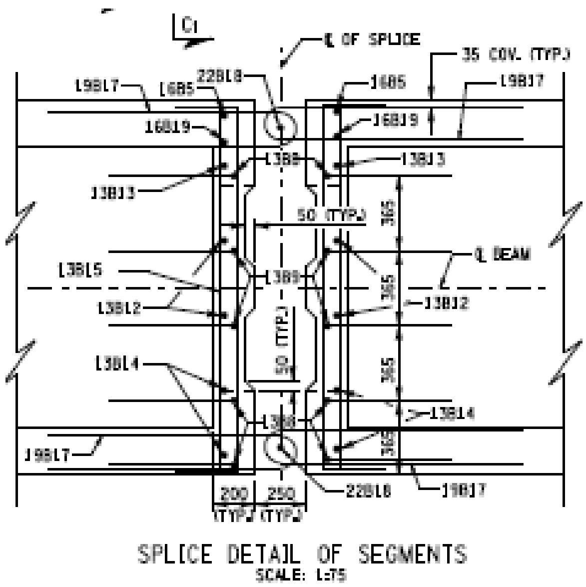

Figure 26 Detail of the splice used, ducts are not shown.

Conclusion

New York State has built a number of post-tensioned prestressed concrete girder bridges over the years. With contractor's increasing ability to handle and transport large, heavy sections coupled with the use of higher strength concrete, this type of structure is becoming more practical and economical. This type of bridge is more desirable because of the greater durability they offer especially those made with High Performance Concrete. Our ability to take concrete structures to new span ranges promises better, more economical bridges for years to come.