# INNOVATIVE PRECAST DECK SYSTEM FOR TAPPAN ZEE HUDSON RIVER CROSSING

**Mike LaViolette,** PE, HDR, Tarrytown, NY
**Mike Lamont**, PE, SE, HDR, Olympia, WA

## ABSTRACT

*The Tappan Zee Hudson River Crossing is the largest and most challenging design/build bridge project currently under construction in the US. The 3.2 mile long crossing includes two adjacent cable-stayed bridges with common foundations, diverging towers, and a 1200 ft main span over the navigation channel of the Hudson River.*

*Each of the ten approach units consists of five nominal 350 ft long, continuous steel girder/substringer spans supported on friction-pendulum seismic isolation bearings. In order to satisfy the project requirement of a 100-year service life prior to major maintenance, the design team developed an innovative precast concrete deck system that eliminates the use of post-tensioning.*

*This paper presents the design and details for the precast deck panel system which includes cast-in-place concrete joints with a permanent, integral bottom form system that eliminates the need for stripping after joint concrete placement. The design incorporates both Eurocode and AASHTO LRFD provisions to satisfy stress and serviceability considerations. A 3D, non-linear finite element model was developed to investigate the applied stresses, tension stiffening of the deck/girder system and estimate the long-term crack width and distribution. With nearly 6000 deck panels required to be installed on the project, constructability was essential; therefore, contractor input into the details was a critical and highly valuable part of the development process.*

**Keywords:** Precast, Concrete, Deck Panels, Composite, Creep, Shrinkage, Design

## INTRODUCTION

The New NY Bridge (Tappan Zee Bridge Replacement) is a Design-Build project with the New York State Thruway Authority. The project consists of the design and construction of new twin, 3 mile structures carrying I-87/287 across the Hudson River between Rockland and Westchester Counties. The twin bridges approach the main-span cable-stayed structure from east and the west with approach lengths of 2 miles from the west and ¾ mile from the east. The new bridges are located on a new alignment that is shifted to the north of the existing bridge. The new alignment parallels the existing alignment through the center of the crossing and then ties into the existing alignment shortly past the shoreline at both ends.

The bridge is owned by the New York State Thruway Authority (NYSTA) and the design and construction of the replacement bridge represents their first design/build project. The project follows a concerted effort to advance the replacement of the existing bridge and eliminate the rapidly increasing maintenance costs that were estimated to approach $2 billion dollars over the next 10 years. In comparison, the replacement project will result in two new bridges which incorporate ped/bike facilities, enhanced aesthetics and a 100 year service life for a relative bargain price of $3.1 billion.

The approaches were divided into design units during the pursuit and the division was carried forward into final design. The west approach begins with Unit 1 and continues to Unit 7 which shares support at the anchor pier with the cable-stayed main-span. The east approach begins at the opposite anchor pier with the start of Unit 8 and touches down on the east shore with the conclusion of Unit 10 for the westbound structure and Unit 9 for the eastbound structure (see Figure 1).

## SUPERSTRUCTURE DESCRIPTION

The design units of the Tappan Zee Hudson River Crossing (TZHRC) include simple-span, three-span continuous and five-span continuous bridges. Typically, the units are 5-span continuous, which provided a repetitive design for the majority of the crossing. The span arrangements were controlled at various locations by interference between the new foundation locations and the existing bridge and marine structures throughout the crossing. In general, the 5-span continuous bridges had 5 equal spans of 350’-0” for a total length of 1750’-0”.

The typical section of the bridges included both conventional girder cross-sections and girder/substringer cross-sections. Units 1 EB & WB, Unit 9 EB Span 6, and Unit 10 WB used girder cross-sections for the structural framing, while Units 2 EB & WB through Units 9 EB & WB used the girder/substringer structural framing system. The steel framing supports a 10¾” thick precast deck panels that are made composite with the girders through the grouting of the haunches and the precast panels are joined together through the grouting of both longitudinal and transverse closure pours. The deck panels are made continuous using conventionally reinforced joints that provide reinforcement continuity between the adjacent panels. The finished deck system includes a 1” polyester concrete overlay that facilitates future replacement at regular intervals and will provide traffic a smooth, durable surface while protecting the structural concrete deck below.

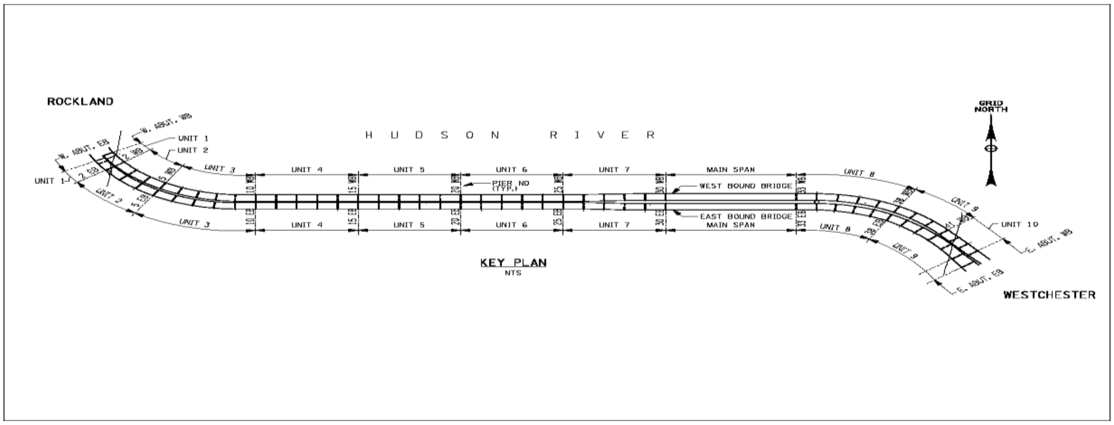


Figure 1. Layout of New NY Bridge

| Design Unit | Span Arrangement | Total Length | Span Lengths | Number of Girders | Notes |
|---|---|---|---|---|---|
| 1 WB | 2-Simple Spans | 388’-10 ¼” | 116’-0 & 272’-9” | 10 Girders | Curved |
| 1 EB | 2-Simple Spans | 332’-7 ¾” | 135’-5” & 199’-2” | 9 Girders | Curved |
| 2 WB | 3-Span Cont. | 970’-10 5/8” | 327’-4”; 309’-5”<br>& 334’-0” | 5 Girders* | Curved |
| 2 EB | 3-Span Cont. | 1024’-7 1/8” | 346’-5”; 328’-2”<br>& 350’-0” | 5 Girders* | Curved |
| 3 WB | 5-Span Cont. | 1673’-0” | 4@330’-9” & 350’-0” | 5 Girders* | Curved |
| 3 EB | 5-Span Cont. | 1750’-0” | 5@350’-0” | 5 Girders* | Curved |
| 4-7 WB | 5-Span Cont. | 1750’-0” | 5@350’-0” | 5 Girders* | Tangent |
| 4-7 EB | 5-Span Cont. | 1750’-0” | 5@350’-0” | 5 Girders* | Tangent |
| 8 WB | 5-Span Cont. | 1750’-0” | 5@350’-0” | 5 Girders* | Curved |
| 8 EB | 5-Span Cont. | 1661’-0” | 350’-0” & 4@327’-9” | 5 Girders* | Curved |
| 9 WB | 3-Span Cont. | 1075’-0” | 2@365’-0” & 345’-0” | 5 Girders* | Curved/Flared |
| 9 EB | 5-Span Cont. & Simple Span | 1890’-0” | 340’-9”; 349’-3”; 321’-0”;<br>301’-3” & 353’-9”<br>224’-0” (Simple) | 5 Girders (5-Span)*<br>9 Girders (Simple) | Curved/Flared |
| 10 WB | 3-Span Cont. | 745’-0” | 235’-0”; 262’-0” & 248’-0” | 9 Girders | Curved/Flared |

* - Denotes Girder/Substringer Framing System

Table 1. Approach Span Arrangement Including Span Lengths and Features

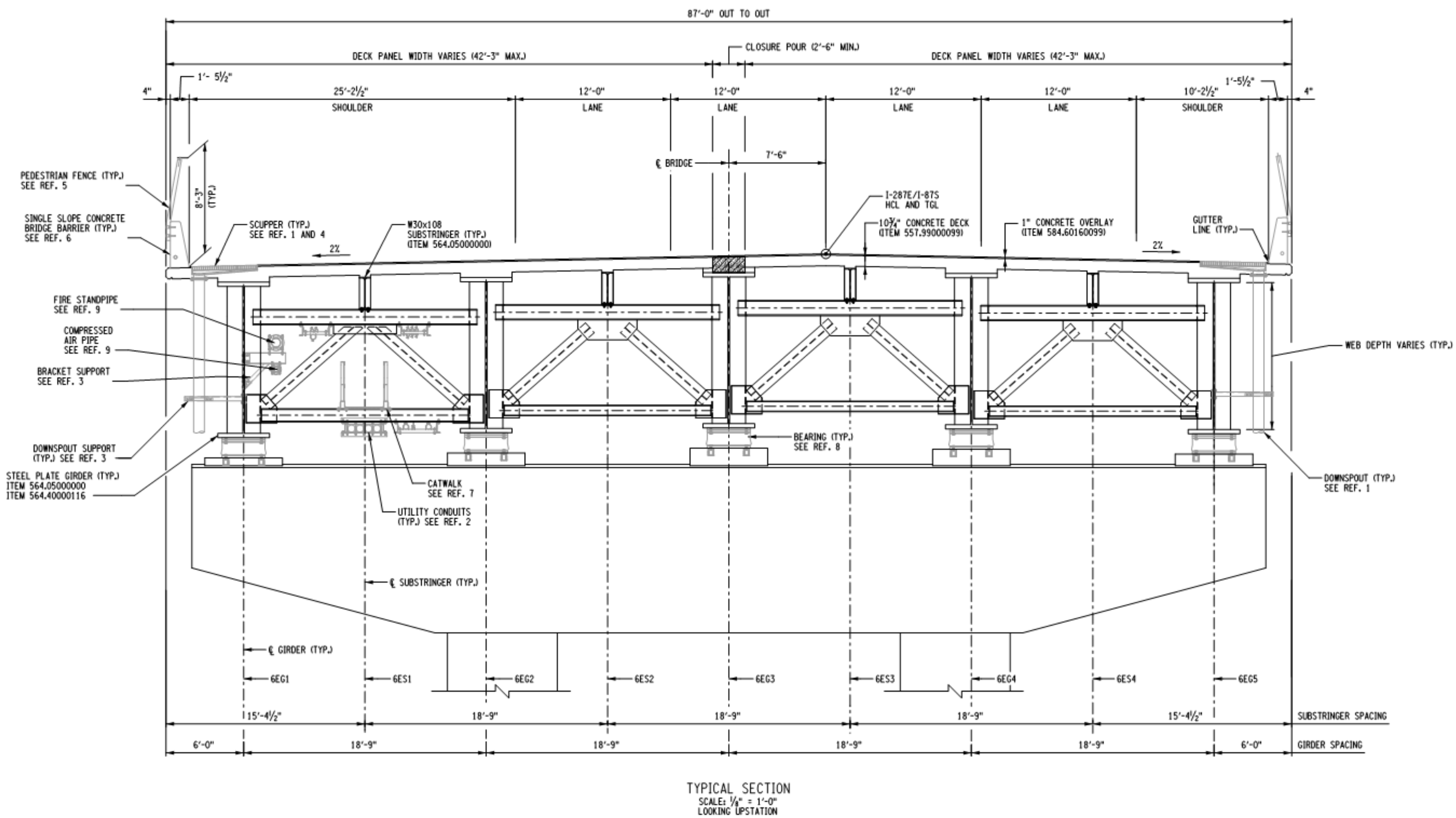


Figure 2. Typical Section – Eastbound Bridge

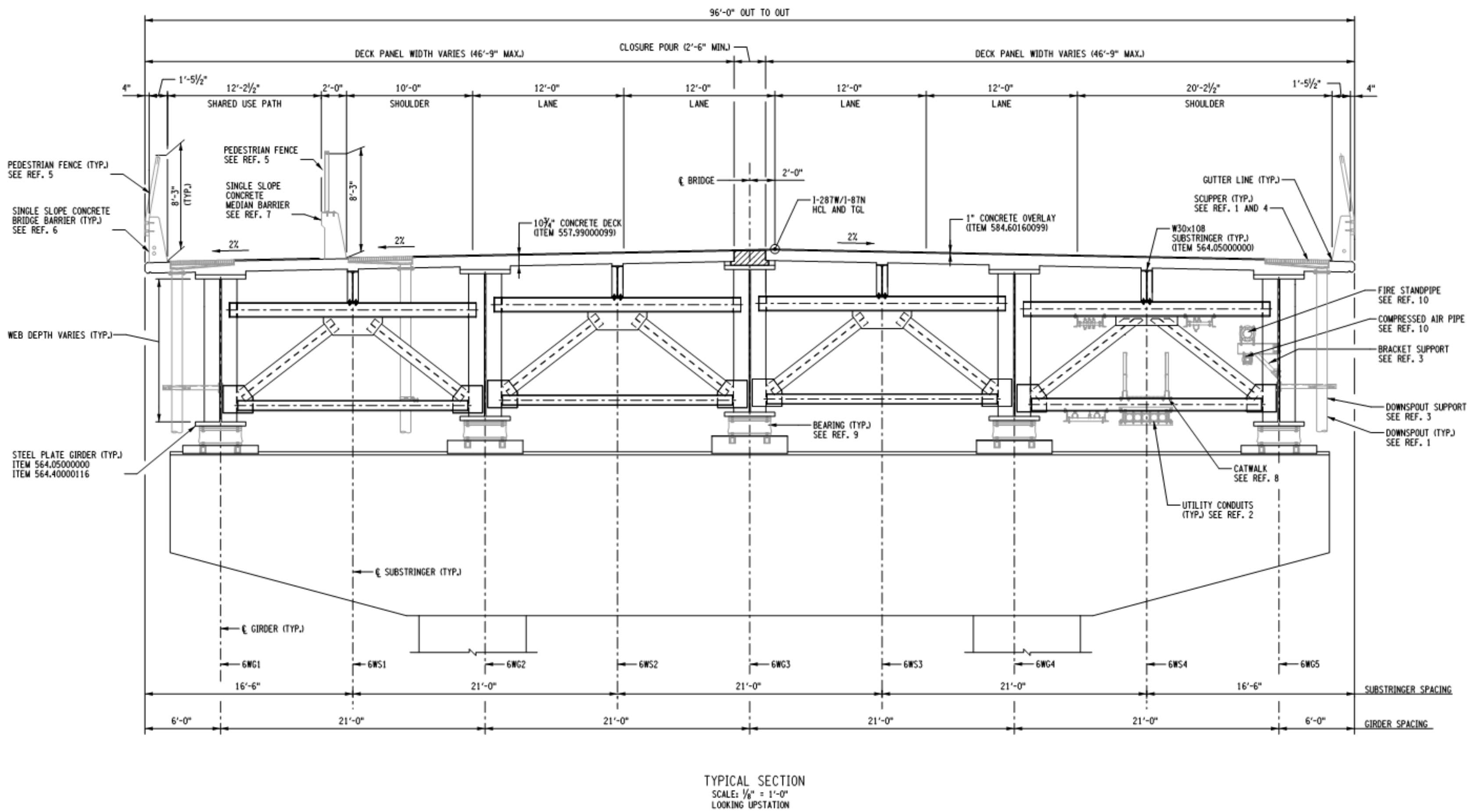


Figure 3. Typical Section – Westbound Bridge

## SERVICE LIFE REQUIREMENTS

The project requirements for the TZHRC defined a primary objective of the project would be to provide a 100-year service life of major structural components prior to major maintenance. A project-specific corrosion protection plan (CPP) was developed for the entire project, including all steel and concrete components. For the deck system, it was acceptable to provide a deck that could last 100 years with multiple overlays that could be milled and replaced over time.

The 100 year design life for the deck required a very specific approach to the design and detailing of the deck panel system. Rather than relying on the typical moment and shear design checks under strength and service limit states, the design was governed by a crack width limitation based on modeling and empirical data which was far more restrictive than the strength requirements.

In order to achieve a 100-year service life for the deck system, a one inch polyester concrete overlay was specified in the project requirements. Polyester concrete has demonstrated the ability to bridge small cracks and provides an elastic moisture barrier with an estimated 30 year replacement interval. In order to ensure that essential reinforcing clearances are not impinged during future milling operations, the structural thickness of the deck panels includes an additional ¾” of top cover. This additional cover is intended to accommodate three milling operations (at 25, 50 and 75 years in the future) with a ¼” overmilled, or sacrificial layer to ensure any deteriorated or chloride-influenced areas.

Deck reinforcing steel was designed to achieve a crack width of less than 0.012” in accordance with the finite element analysis presented later in this paper. In addition, all reinforcing steel in the precast deck are hot-dip galvanized to maximize corrosion protection.

## DECK PANEL DESIGN

During the bid phase of the project, the TZC team determined that precast concrete deck panel system in an excellent solution for all of the approach span units. Precast concrete deck panels offer many advantages for a project of this size including:

- standardization of details and repeatability in the casting yard
- speed of construction
- control of material quality
- ability to cast panels in climate-controlled conditions throughout the year

These advantages are somewhat offset by the sheer volume of precast concrete panels to be handled and placed in the field and the need to incorporate superelevation transitions and fully curved spans at each of the bridge.

Potential challenges to be addressed in precast deck panel design and construction include:

- need to accommodate full composite action with steel girders
- lifting and erection deck panels without inducing tensile stresses that could induce excessive cracking
- accommodation of superelevation transitions in several spans

Three options were carefully considered for the precast deck panel design. These include:

- full longitudinal post-tensioning with non-continuous mild steel reinforcement
- partial longitudinal post-tensioning with continuous mild steel reinforcement through the joints
- continuous, conventional reinforcing through the joints and no post-tensioning

Ultimately, the decision was made to utilize a conventionally reinforcing concrete deck without any post-tensioning. This design eliminates the complication of stressing panel tendons, grouting and the potential for long-term inspection and maintenance challenges that would be faced by NYSTA. In addition, this system allows the contractor to erect deck panels and make all of the necessary elevation adjustments in any weather conditions. Placement of the closure pour concrete can be performed when suitable ambient temperatures are achieved.

The primary girders are designed as fully composite with the precast concrete deck, while the intermediate substringers are designed as non-composite for dead and live loads. In order to eliminate any tendency for rattle, a single row of shear connectors is provided along the longitudinal stringer lines.

Figure 4 presents a section view of the transverse construction joint between adjacent deck panels, while Figure 5 shows an orthogonal view of the transverse joint including projecting hairpin reinforcing and the transverse bars which are threaded through the hairpins to provide a continuous connection between panels. The integral bottom form eliminates the need for the contractor to strip conventional wood forms after deck construction and provides for future inspection of the entire bottom surface of the deck. Concrete strength for the closure pours is specified to meet at 28 day compressive strength of 8 ksi.

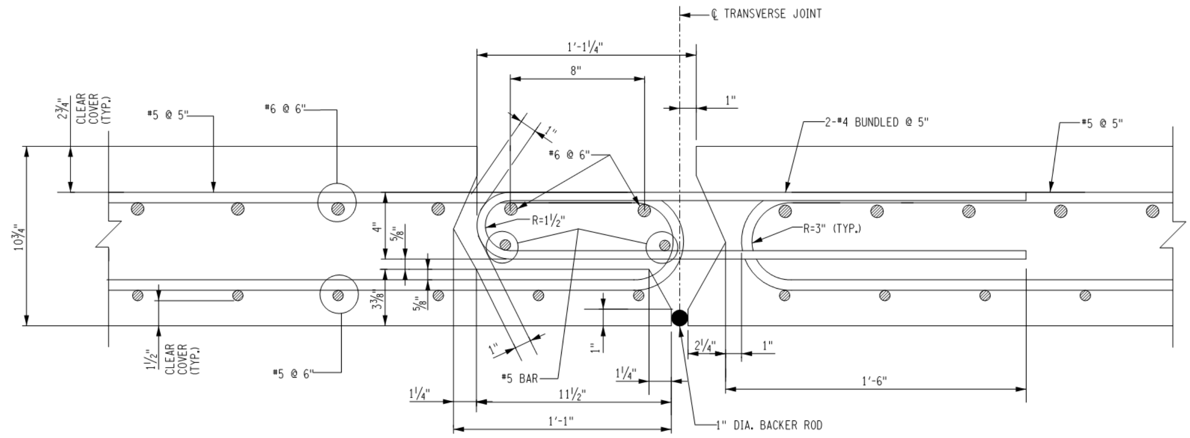


Figure 4.  Section view of transverse construction joint between deck panels

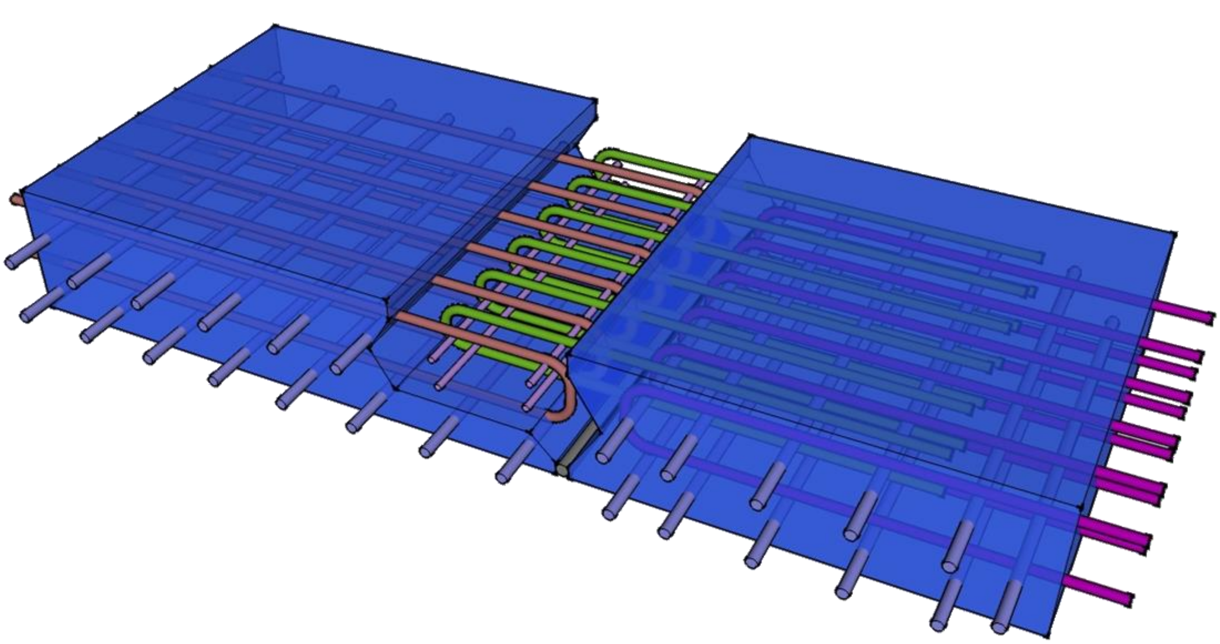

Figure 5.  Transverse Joint Reinforcing Detail - orthogonal view

**DECK PANEL DETAILS**

In order to reduce the size of the panels to be lifted into position, a longitudinal closure point joint over the center girder was developed (See Figure 6). This joint is designed to resist the tranverse flexure of the deck when subjected to truck wheel loads between the girder/stringer lines and this sees a much reduced demand than the transverse closure pour.

In regions of superelevation transition, a longitudinal closure pour joint was added over each interior girder to facilitate additional points of vertical adjustment points for the panels to ensure a smooth ride in the final condition and eliminate abrupt “hard points” in the deck that would be subjected to truck wheel impacts over the 100 year service life.

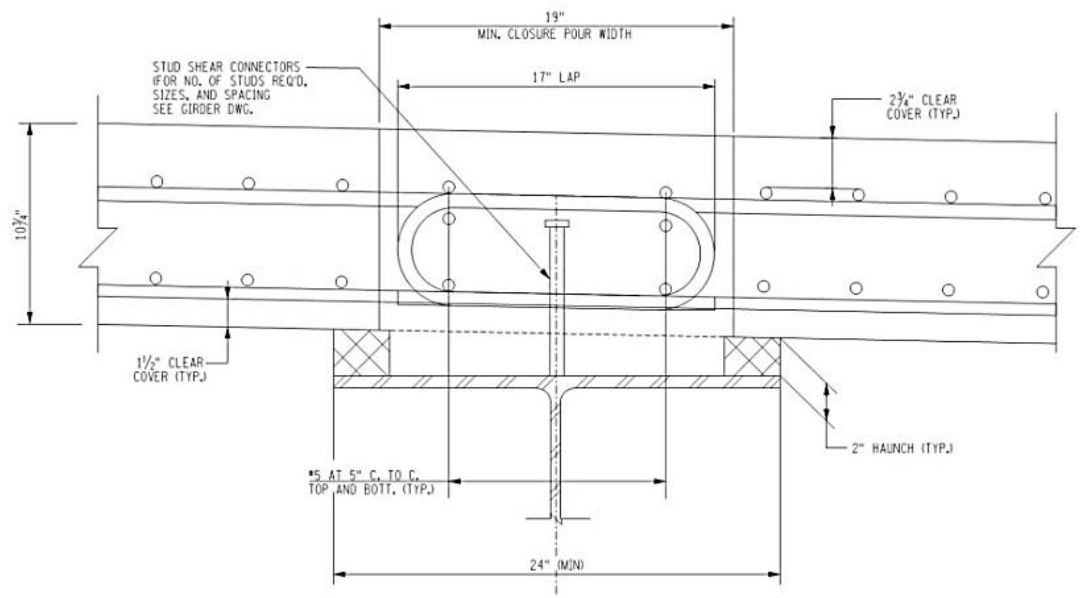


Figure 6. Longitudinal closure point joint over girders

Along the length of the TZHRC, the steel girder top flange plates vary from 36 inches to 60 inches in width. In order to avoid possible problems in placing haunch grout beneath the deck panels supported on the wider girder flanges, the longitudinal closure pour joint varies in width in accordance with changes of the girder top flange.

The combined length of the two parallel bridges exceeds 6.2 miles, which results in approximately 6000 individual deck panels to be cast, handled, shipped and installed. In addition, the number of shear pockets, lights, embedments, belvederes and crossovers could potentially generate over 1150 different deck panel piece marks that would be required. A deck panel casting yard has recently been contracted and will now begin to study the deck panel layout and combine as many of these different pieces marks as possible.

As is typical with all successful design/build projects, many design and detail decisions are driven by the contractor’s means and methods of construction as well as by a thorough understanding of material quantities and supplier pricing. In order to achieve the most economical structure for construction, the design of the TZHRC deck panel system incorporates much wider deck overhangs than are typically used for conventional bridges. The primary advantage of this approach is to allow the deck cross section to achieve the necessary roadway width, while eliminating one girder line from the cross section, thus significantly reducing superstructure material, fabrication and erection costs.

For a conventional bridge, the deck overhang is commonly in the range of 3'-6", while the TZHRC design utilizes an typical deck overhang of 6'-0"; and at locations where lighting fixtures are provided, the deck overhang width extends to over 8 feet. See Figure 7.

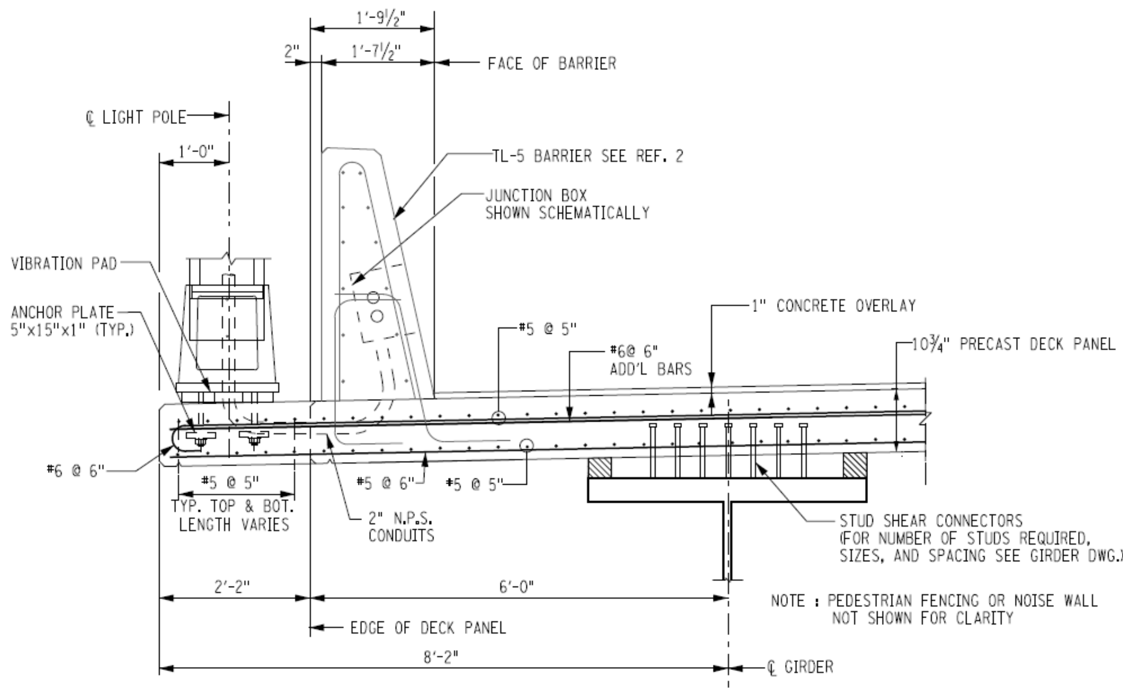


Figure 7. Deck panel overhang at light fixture location

These wider deck overhangs, combined with the use of an FHWA TL-5 traffic barrier generate the need for large, closely spaced reinforcing steel. In addition, stringent crack control requirements along with the physical constraints imposed by working around scuppers, stud pockets, and other deck penetrations lead to local congestion issues. The current code allows for the stiffness of the barrier to help in local loading conditions. The final configuration was a complicated balance of reinforcement located strategically to meet the loading needs.

## SHEAR CONNECTOR DETAILS

The design of a typical deck panel, as shown in Figure 8, utilizes an average shear connector spacing of approximately 22 inches, which satisfies the intent of AASHTO LRFD for maximum shear connector spacing of 24 inches. In the region near the transverse closure pour joints, the details utilize a single, wider stud spacing of nearly 48 inches to avoid conflict with the closely spaced hairpin reinforcing bars which project into the closure pour from each of the adjacent panels.

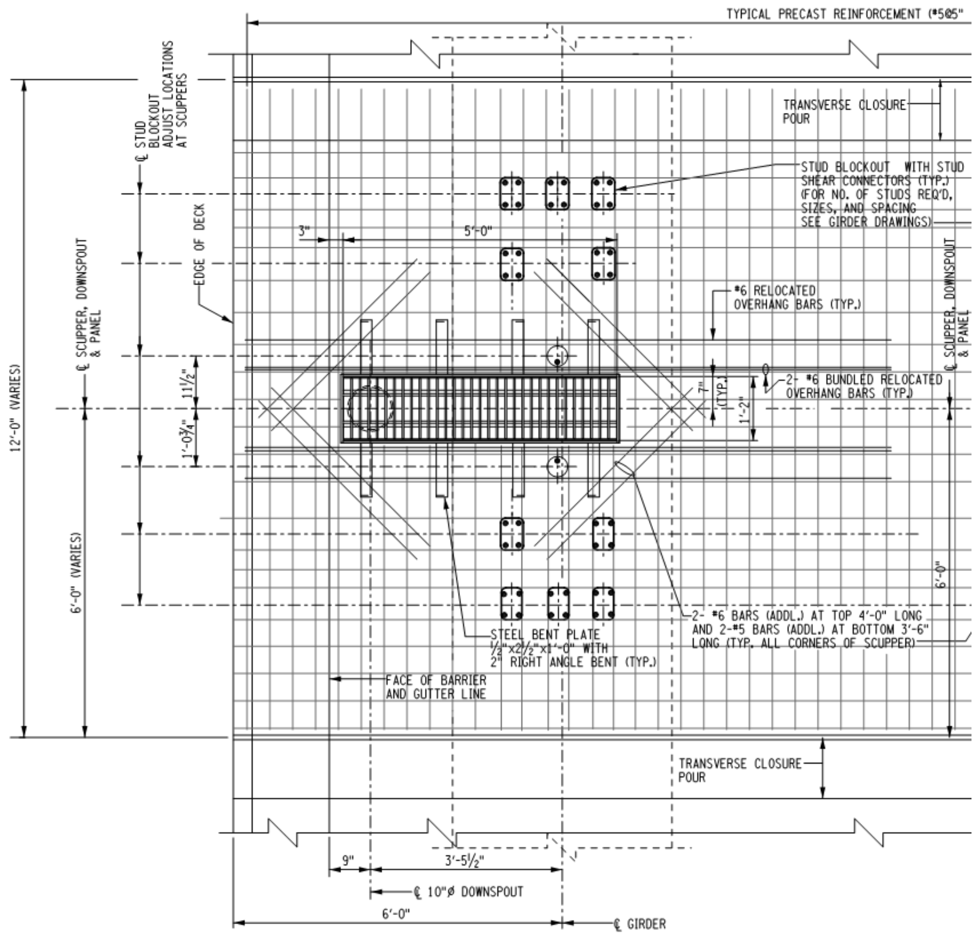


Figure 8. Partial plan view of typical deck panel showing shear connector pockets

The design of the shear connectors for the TZHRC satisfies the required fatigue criteria as specified in AASHTO LRFD. The LRFD code does not specify an explicit limit on shear connector spacing, but rather requires a design which provides a particular number of shear connectors per length of girder to transfer the applied factored shear force. The shear connectors were designed based on the principles of interface shear between the concrete deck and the steel girders to ensure that the section is composite. The intent of the design requirements is met when the structure experiences uniform deflections and stresses due to that deformation.

In addition, the TZHRC design satisfies the required strength criteria as specified in LRFD which specifies a minimum number of shear connectors between the point of maximum positive LL+I moment and adjacent points of maximum negative LL+I moment. It should be noted that the strength requirements of the AASHTO code are based on developing the full plastic capacity of the section based on the strength of the deck or the strength of the steel section, while the design of the approach girders is based on elastic sections and therefore, the strength design of the shear connectors is very conservative.

Although installing a single shear connector that would penetrate the 3 3/8" thick bottom lip of the deck panel would technically satisfy the 24 inch spacing requirements of AASHTO LRFD, the designers felt that such a detail would be extremely detrimental in meeting the 100 year service life requirements for the deck.

In those regions where the girder flange is particularly wide (up to 60 inches), a maximum transverse distance from the edge of the girder flange to the nearest shear studs was investigated. Although AASHTO LRFD remains silent in regard to a maximum edge distance for shear connectors, Eurocode EC4.2 provides some guidance in this area; though it does not specifically address limitations on the transverse spacing of connectors.

Shear lag within the top flange plate, wherein the total width of the top flange is not fully engaged in resisting the horizontal shear force transferred from the deck, is not seen as a significant concern given the relatively low compressive stress (approximately 28 ksi) which exists under strength load conditions in these plates.

The regions with the widest top flanges on the TZHRC are located in negative moment regions of the continuous spans. These flange plates are in tension under strength and service load conditions and thus local buckling of the top flange is not a concern. In positive moment regions where the top flange is in compression, the potential for local buckling of the flange plate was evaluated using AASHTO LRFD and found to be adequate for the strength limit state.

Separation of the deck panels from the girder top flanges was considered in the assessment. In the negative moment regions of the continuous girders, the deck is in tension and will tend to exert a downward force on the top flange and therefore, will not lead to a separation of the components.

In the positive moment regions, the maximum top flange compressive stress is approximately 28 ksi, which is far below the yield strength of the flanges. For the curved spans, the large radii and wide flanges lead to low levels of lateral bending stresses to be superimposed on the normal stresses in the girder flanges and are thus not significant in the assessment. As noted above, the top flanges meet the strength limit state for local buckling, thus it is unlikely that the flange will separate from the deck/haunch, especially considering that the steel flange cannot buckle upward due to the presence of the concrete deck panel.

In the standard precast deck panels designed for the TZHRC, plan details provide a total of 15 shear pockets and up to 60 shear connectors per panel. The number of shear pockets within a given panel was selected to provide sufficient space to install the shear connectors required by design, even in locations where scuppers and bolted girders splices present potential conflicts. In order to minimize the spacing between clustered shear connectors in adjacent panels and promote composite action near the transverse closure pours, plan details specified that pockets nearest the joints are filled first with others distributed uniformly.

In the area near the scuppers above the main girders, additional round stud pockets will be provided adjacent to the scuppers, and a single stud will be placed in each of these pockets to anchor the deck panel to the girder top flange in that area.

## NONLINEAR FINITE ELEMENT ANALYSIS

To validate the reinforcement-only option for the cast-in-place deck joints, a SAP2000 finite element model was used to evaluate the service-level stress condition of the bridge deck for a full continuous superstructure unit (five 350-foot-spans). The SAP2000 analysis utilized staged construction, time-dependent effects, and non-linear post-cracking behavior of the reinforced concrete deck and joints.

The 3D finite element model utilizes plate and shell elements for the girders and the precast deck, respectively; and models all bracing and sub-stringers explicitly using frame elements, as shown in Figures 9 and 10. Partial-depth (7-inch) cast-in-place joints and full-depth (10-inch) precast deck panel elements were each modeled with non-linear material properties, as well as non-linear stiffness to account for pre- and post-cracked section properties. The sectional analysis program Response-2000 was used to develop non-linear stress-stain curves in compression and tension for the deck panel and cast-in-place joint elements, which were used as input into SAP2000. The girders and sub-stringers were discretized every two feet along station at the shear stud pockets, and at each side of the cast-in-place joints. However, half of the 1750-foot-long structure was discretized at four feet to decrease run time of the model (See Figure 11). Results were only taken from the side with the more refined meshing.

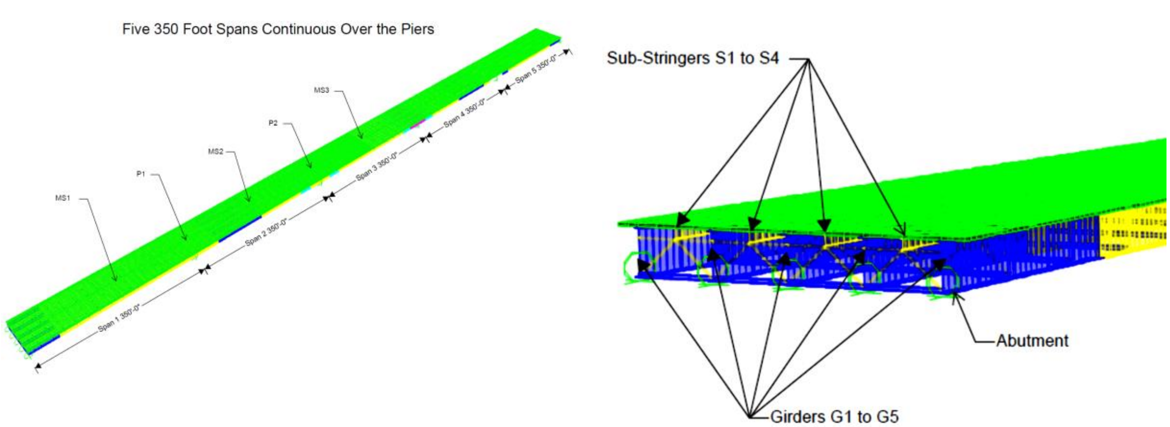


Figure 9. Extruded View of SAP2000 Model

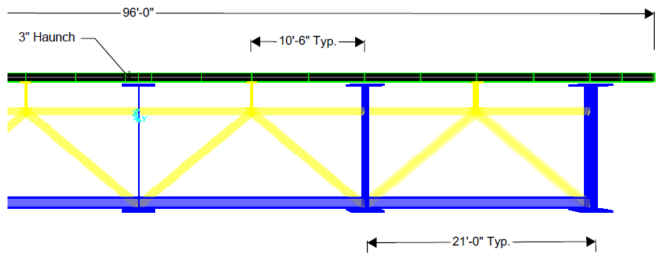


Figure 10. SAP2000 Composite Cross Section

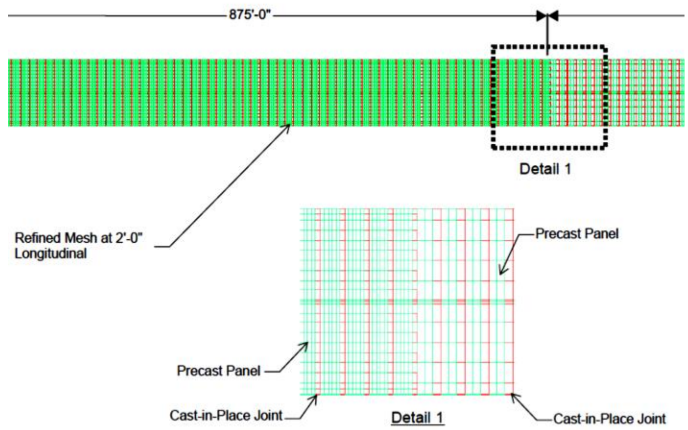


Figure 11. SAP2000 Deck Meshing

The construction staging sequence used in the analysis model was as follows:

1. Erect girders, substringers, and cross frames.
2. Apply catwalk and utility loads.
3. Apply precast deck panels (age = 60 days) and cast infill joints (weight applied to non-composite girders).
4. Apply superimposed dead loads (fire/air pipes, polyester overlay, barriers) to composite deck section.
5. Time-dependent effects (1 year)
6. Time-dependent effects (10 years)
7. Time-dependent effects (30 years)
8. Add future wearing surface (30 years)
9. Time-dependent effects (100 years)

Time-dependent effects considered included concrete material properties, creep, and shrinkage. Creep and shrinkage provisions of AASHTO were used to calculate equivalent strain loading that was applied to the non-linear deck panels in the model, since SAP2000 is not able to apply creep and shrinkage to non-linear elements. While creep was found to have little effect on the deck stresses, the effect of shrinkage on the panels is significant.

To assess the service-level deck cracking, live loads corresponding to worst-case negative bending over the piers were added to the model at the beginning and at the end of service life. The tension in the deck and deck joints resulting from the combined time-dependent shrinkage and negative bending due to live loads were extracted from the model using section cuts, and the average and peak axial loads were evaluated in Response-2000 to come up with an estimated crack width.

The analysis shows that as the cracking tension of the deck is exceeded over the piers due to shrinkage and negative live load moment, the deck becomes effectively softer, and load is diverted into the 3” thick top flange of the girder. The stiffness of the girder flange minimizes the strain in the deck, and thus limits the width of the cracks. Figure 12 shows a service stress plot of the deck, with the cracked areas shown in blue.

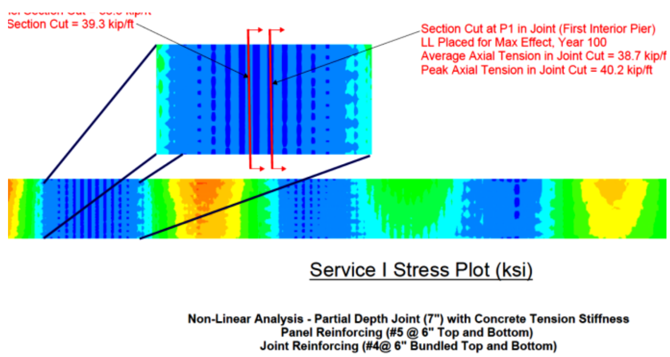


Figure 12. SAP2000 Results

Force results from the SAP-2000 analysis, both peak and average loads in the panels and joints, were input into Response-2000. The sectional analysis includes the effects of tension stiffness of the concrete to capture the additional rebar stress at the cracks. From the analysis, it was found that both the deck panels and the partial depth cast-in-place joints, if reinforced with 1% steel, produce maximum Service I crack widths below the limiting value of 0.012 inches provided by the project Corrosion Protection Plan. See Figure 13.

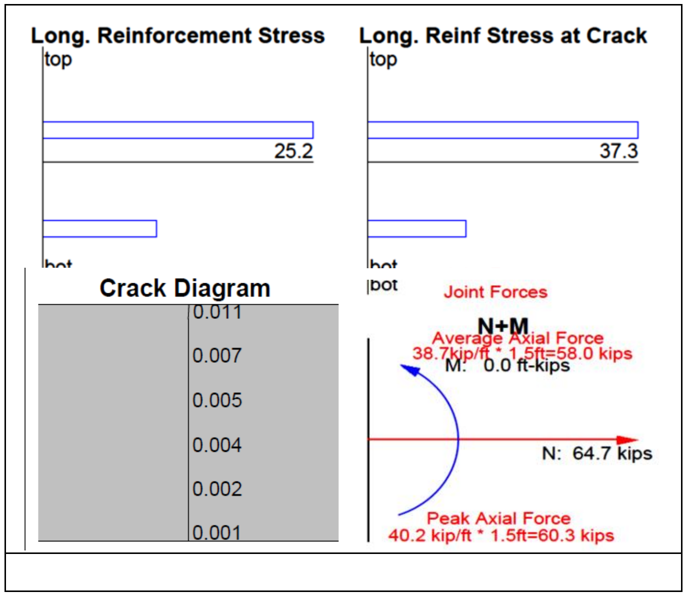


Figure 13. Response-2000 Sectional Analysis

The results of the finite element analysis with non-linear deck elements confirms the feasibility of the conventionally reinforced deck with partial-depth cast-in-place joints, and also validate the assumptions and results of the design spreadsheets, based on AASHTO and Eurocode EN 1994, used for production design of the deck.

## DECK PLACEMENT SEQUENCE

The superstructures were designed to consider a deck placement sequence for the installation of the precast deck panels. The deck placement begins at each end of the approach units with a small cast-in-place closure pour for the embedment of the modular dams and to permit a tolerance in the placement of the precast deck panels. Discussions between the design team and the contractors resulted in a deck placement sequence that begins at one end of the approach unit and continues to the other end. This permits the contractor to minimize the amount of time that is spent mobilizing equipment from location to location and allows the contractor to begin placement of the conventional reinforcement in the longitudinal and transverse joints beginning at one end and moving towards the other. The operations that finish the deck panel placement, including the grouting of the longitudinal joints, grouting of the transverse joints, and grouting of the haunches will not begin until all deck panels have been placed within a given design unit. After the completion of the panels, a 1" polyester concrete overlay will be placed to provide a smooth riding surface for the travelling public.

## CONCLUSION

The New NY Bridge represents perhaps the largest bridge yet constructed that utilizes a precast concrete deck panel system designed to satisfy a 100 year service life requirement. The design and detailing of the concrete panels, along with the reinforced concrete joints and associated shear pockets and attachments is an impressive achievement. The casting and installation of these deck panels will represent an ongoing series of challenges that TZC is ready to address. However, the performance of these deck panels that are designed to satisfy a 100 year service life without major maintenance will set a new standard for long-span bridge decks for generations to come.

## ACKNOWLEDGMENTS

The author is pleased to recognize the New York State Thruway Authority as the owner of the New NY Bridge. Many members of the Authority have been involved during the planning, design and construction of this project and their contributions are greatly appreciated. In addition, we would like to recognize significant contributions from the New York State DOT as well as the Federal Highway Administration.

## REFERENCES

1. Comité Euro-International du Béton, "CEB-FIP Model Code 1990: Design Code," Thomas Telford Ltd., London, UK, 1993, 464 pp.