RICHMOND SAN RAFAEL BRIDGE PROJECT—
SPLICED GIRDER CASE STUDY

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ABSTRACT

The Richmond San Rafael Bridge Western Approach Trestle Replacement, consists of two independent structures, with a combined overall length of 6500 feet of new precast/prestressed concrete spliced girder. The innovative spliced girder structure was developed to allow removal of two 50-foot spans and erection of one 100-foot span during each night-time traffic closure. After four new spans are erected, concrete closure pours are placed and continuity post-tensioning is completed to yield a four-span continuous frame. During construction, closure pour cracking required modifications to the construction procedures and typical closure pour details. The unique structure type and the closure pour details created a challenge and a learning experience for the Contractor, Caltrans, and the Design Team.

Keywords: Trestle, Prestressed, Spliced Girder, Closure Pour, Cracking

INTRODUCTION

The Richmond San Rafael Bridge, designed in the early 1950s, carries four lanes of Interstate 580 over the southern end of San Pablo Bay between the cities of San Rafael in Marin County and Richmond in Contra Costa County. One of five toll bridges scheduled for seismic retrofit in the Bay area, the bridge consists of the following structure types:

- Two single-deck, reinforced concrete approach trestles with 50-foot spans and a combined length of approximately 6,500 feet
- Built-up steel-girder spans at both ends of the bridge
  – Spans convert from single-deck to double –deck structures
  – Combined length is 3,600 feet
  – Typical spans are 100 feet long
- Two variable-depth, double-deck cantilever-truss-type structures spanning the navigational channels, with 537.5-foot anchor spans and 1,070-foot center spans
- Constant-depth, double-deck trusses that span the distances between the two cantilever structures and between the cantilever structures and the approaches
  – Typical spans are approximately 292 feet long
  – Combined length is approximately 10,600 feet

Figure 1 Richmond San Rafael Bridge

At the time of award in 2001, the $485 million Richmond San Rafael Retrofit Project was Caltrans largest project ever awarded. While the construction project was considered to be a complex retrofit of a major San Francisco Bay Toll Bridge, the replacement of the western approach trestle portion as part of the retrofit solution has proven to be one of the most significant challenges of this construction project.

PRECASTING OPERATIONS FOR TRESTLE REPLACEMENT GIRDERS

The existing Richmond/San Rafael bridge trestle was comprised of two parallel structures, one carrying traffic in the westbound direction and the other carrying traffic in the eastbound direction. The replacement of the existing 50’-0” simple span precast/prestressed trestle structure required the offsite precasting of sixty-four nominally 100’ long, by 43’-8” full width, precast/prestressed double-tee girders. The new trestle girders are 5’-6” deep at the ends and incorporate a haunched transition to a reduced depth of 3’-9” for the center 55’-0” portion. This reduced structure depth was necessary since the new girders needed to clear the support bents of the existing trestle structure that were to be removed at a later date.

Figure 2 Two Parallel Trestles

In order to facilitate the offsite precasting and handling of the girders, the girders were broken up into a center 40’-0” long pretensioned middle unit and two 30’-0” end segments that were match-cast simultaneously against the previously cast middle unit. Additionally, the deck portion of the girder was transversely post-tensioned before being removed from the casting forms. The middle unit was cast in a single purpose, self-stressing form that accommodated the twenty 0.6” pretensioned strands. Once the middle segment reached sufficient strength, it was removed from the form and placed into the match-casting position. The end segment reinforcing and post-tensioning ducts were placed in the second casting area and the forms hydraulically closed for casting both ends. Upon completion, the three segments were water cured in storage for seven days. The precasting operation produced a complete 100’ girder on a weekly cycle.

Figure 3 Precasting Double Tee Girder

After curing, the three segments were loaded onto a barge for post-tensioning together to complete the 100’ simple span girder. The match cast faces of the girder segments were coated with segmental bridge adhesive just prior to beginning the simple span post-tensioning operation. The delivery barge was outfitted with four temporary, fixed dunnage points for the middle segment while the end segments were temporarily supported by large rollers near the match cast joints and greased bearing surfaces at each end. The four simple span tendons were sequentially stressed to pull the three segments together.

Figure 4 Loading Girder Segments on Barge

Figure 5 Post Tensioning on Barge

FIELD INSTALLATION PROCEDURE

The trestle replacement superstructure was comprised of 16 post-tensioned frames. Each frame was made up of four simple span girders that were post-tensioned together for continuity after a frame had been completely installed. The Project's specifications required that one lane of traffic, in each direction, had to be maintained during each anticipated night closure. The westbound trestle was replaced first, followed by the replacement of the eastbound alignment. The replacement of each trestle progressed from west to east along each alignment.

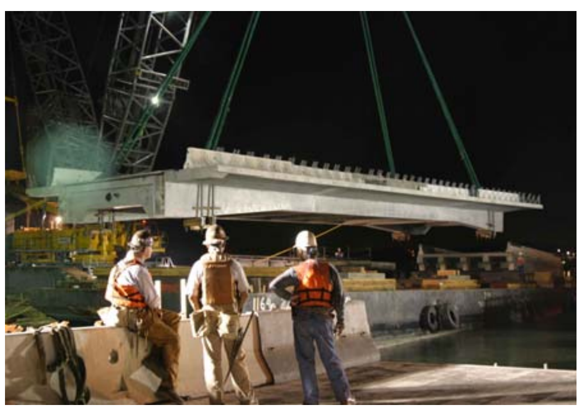


Figure 6 Lifting New 100-Foot Span

Originally, the Contractor had anticipated being able to remove two existing fifty-foot spans and install one new 100' girder during each night's bridge closure. It should be noted that the new girders were immediately opened to two lanes of traffic loading the morning following installation. All of the trestle material handling was done using a nine hundred ton, barge mounted crawler crane that the Contractor had specifically purchased for this project. At erection, the 100’ girders weighed approximately 500 tons. As installation progressed, the Contractor was actually able to routinely replace two hundred feet of trestle during the ten-hour closure window. This outstanding success was due to the coordinated efforts of the entire construction team.

Figure 7 Placement of 100-Foot End Span

CONTINUOUS SUPERSTRUCTURE

To allow the replacement trestle to carry three lanes of future traffic, the superstructure is made into a continuous four span unit by using closure joints and continuity post-tensioning between spans. Each completed frame consists of four spans with three 6-inch wide closure joints located over the interior piers. Two girder tendons and four deck tendons make up the continuity stressing.

The placement of each closure joint is completed during night time traffic closures. Prior to placement of the closure pour concrete, shims are placed in the closure joint and 2 of the 4 deck continuity tendons are stressed to 15% of their final stress. Prior to completing the remainder of the continuity stressing, the closure pour concrete is required to reach an initial strength of 2500 psi. Prior to opening to traffic within an 8 hour period, the closure pour concrete is required to reach a strength of 4000 psi.

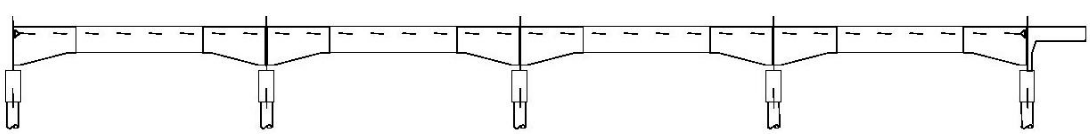


Figure 8 Four-Span Continuous Frame

CLOSURE POUR CRACKING

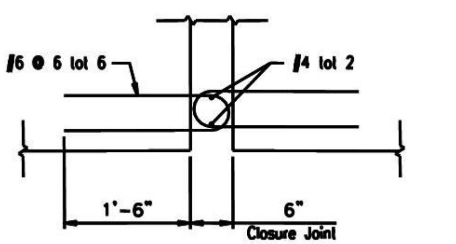
Immediately after the first closure joints were constructed in April 2004, cracking was observed along the bottom of the vertical interface between the closure pours and the girder ends.

Figure 9 Closure Pour Cracking

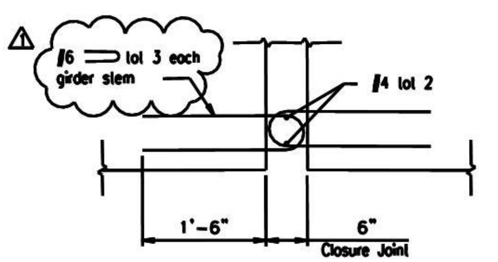
Initial review by the design team determined several construction modifications were not adequately accounted for, the initial design had little conservatism in the design of the joints, and the unique structure type and traffic constraints made the construction of the closure joints a challenge.

Key findings outlined in a Technical Report on the Investigation of Trestle Spans Closure Pour concluded:

- Continuity stressing causes the bottom of the girders to rotate and move apart.
- The original joint design which required auxiliary bonded 6 - #6 U-stirrups and shear keys was reduced.



CLOSURE JOINT DETAIL - ORIGINAL



CLOSURE JOINT DETAIL - REVISED

Figure 12 Girder Joint Reinforcement

Figure13 Closure Joint Ready for Pour

- Use of steel shims as temporary closure joint blocking was not allowing the closure concrete to see full compression.
- Time-dependent effects due to creep and shrinkage significantly impact concrete stresses.

To reduce the joint tension stress and avoid costly construction schedule delays the design team specified the following changes:

- Replace steel pipe shims with concrete shims
- Use a two stage closure joint pour. The upper 2'-6" feet section of the girder poured first. The remaining lower 3 feet section poured after 28 days.

In November of 2004, approximately one month or more after construction of the new two stage closure joints had been completed, cracking of the closure joints was again observed. This time cracking appeared to be significantly lessened when compared to the single stage pour. In fact, some joints had only one of two web areas cracked.

Figure 14 High Side Cracking

Figure 15 Low Side, Little to No Cracking

LONG TERM SOLUTION

Concerned with the need to achieve a full depth closure pour to allow 3 lanes of future traffic plus the effects of cracking and corrosion on the design service life, a long term solution was proposed. This solution included the use of elastomeric concrete in the 2nd stage closure pours, new closure joint bolsters and prestressing rods.

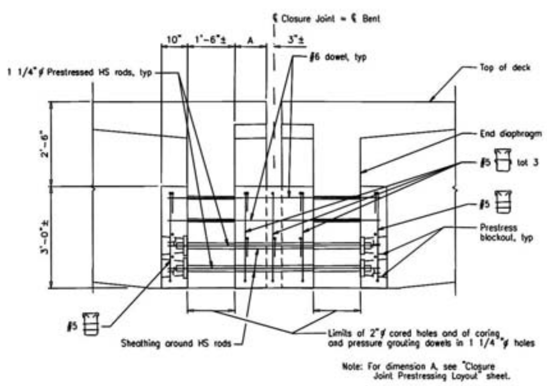


Figure 16 Closure Joint Bolster, Longitudinal Section

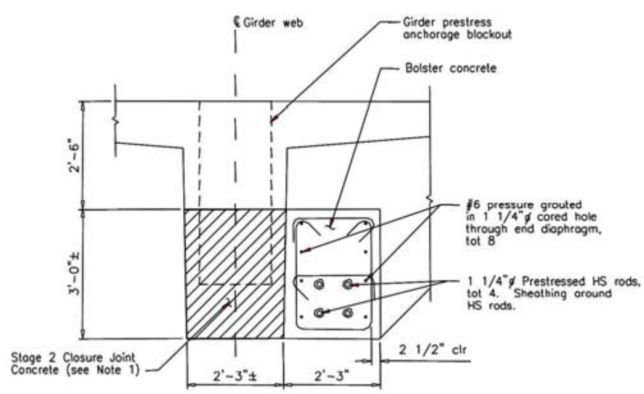


Figure 17 Joint Bolster, Typical Section

Using elastomeric concrete in the area of the new or repaired Stage 2 closure pours provided a more durable concrete for structure traffic vibrations, alleviated the corrosion concern, and minimized the required traffic closures and working days required to complete the additional closure joint work.

The benefits of the additional closure joint stressing allowed the bottom fiber joint stresses to change from mild tension to slight compression without exceeding stress limits at other span locations.

Figure 18 Completed Closure Joint Bolster

Figure 19 Second Stage Closure Pour with Elastomeric Concrete

CONCLUSION

The long term solution using elastomeric concrete in the closure pours, closure joint bolsters, and additional stressing alleviated the closure pour cracking. While closure pours over pier caps in segmental construction are typically deemed to be of little complexity, the lessons learned from the unique structure type and traffic constraints associated with the Richmond San Rafael Bridge Retrofit are expected to impact segmental design and construction considerations on future Caltrans projects.