Russian River Bridge Replacement

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ABSTRACT

Emergency replacement of the Russian River Bridge in Northern California presented a unique design challenge and an opportunity for creative solutions. With sensitive environment, tight construction window and local community demands to complete the replacement in about 8 months, the design reflected innovative ideas that are not typical to the State bridge industry. The bridge is the first built in the State using a specially dimensioned precast prestressed Double Tee girder with a two stage post-tensioning. The bridge superstructure has a depth-to-span ratio of 0.037, much smaller than 0.045, the typical for similar precast superstructures. This paper presents the successful delivery of such an accelerated project. Creative solutions provided by the designers and the Contractor's team to expedite construction are discussed. Lessons learned during design and construction phases are concluded.

Keywords: Precast, Double Tee, Post-Tensioning.

INTRODUCTION

The existing steel pony truss bridge over the Russian River in Geyserville, Sonoma County, California was severely damaged during the series of storms in the last two weeks of December 2005. The bridge was closed to traffic causing a hardship to the local community. It is the shortest route to the high school on the other side of town and closure of the bridge resulted in a 40 minutes detour every school day.

The existing bridge, built in 1932, consisted of six 30.5 m (100 ft) long riveted pony truss bridge spans over the main channel and fifteen 7.3 m (24 ft) reinforced concrete T-beam approach spans on the east and west sides. The substructure consisted of reinforced concrete piers parallel to the original river flow and supported by 7.6 m (25 ft) long timber piles (See Fig. 1)

California Department of Transportation's (Caltrans) management decided to replace the bridge and open to traffic in about 8 months before the next scholastic year begins. Caltrans engineers started the design of the replacement bridge first week of February 2006. Sensitive environmental issues and the necessity for a fast construction schedule led to a precast superstructure type as the most feasible design. Standard AASHTO Adjacent Box Girders[1] transversely post-tensioned were selected for the design of the 10 spans, 298.7 m (980 ft) long replacement bridge with a depth to span ratio of 0.037. Total of 120 box girders were used in the original design. Substructure was made of a drop bent cap with two 1.22 m (4 ft) diameter pile shafts, and a stand-alone seat type abutment. By mid-March, the biddable design package was ready to list, and the general contactor was awarded the project in early April, 2006.

Contractor's team proposed an alternative superstructure design. A non standard and a wider than typical Double-Tee girders with two stages of post tensioning was proposed instead of the AASHTO Boxes to reduce number of precast girders (60 girders) and expedite the construction in 80 days. Caltrans agreed to the stage construction concept and performed an independent check and evaluation of the alternative superstructure design. Check of the alternative design resulted in some modifications that were incorporated in the final design.

Construction of the bridge proceeded in mid-May, 2006 with successful coordination between Caltrans construction and the Contractor. The replacement bridge was opened to traffic on August 17th, 2006, just one week before school year and to the delight of the local community.

This paper presents the successful delivery of such an accelerated project. Creative solutions provided by the designers and the Contractor's team to expedite construction are discussed. Challenges faced during bridge design, alternative design and construction phases are concluded.

PROJECT HISTORY

Over the past decade, the Russian River in Sonoma County, Northern California has meandered, subtly changing direction from season to season. Near Geyserville, the river flow eventually began impacting the existing bridge support piers at 30º skew angle relative to the original river-to-bridge pier orientation. While there was evidence of low-level, long term scour occurring around the timber piles supporting the concrete piers and the bridge rating was subsequently reduced, there ware no other obvious structural issues.

During the last two weeks of December 2005, high seasonal rainfall resulted in exceptionally high river flow and flooding (Fig. 2). In addition, debris from the storm that accumulated at the leading edge of mid-channel bents exacerbated the scour conditions at one critical pier. Local authorities and Caltrans maintenance engineers observed rotation of Pier 6 in the transverse direction and approximately 8 in. of differential settlement between upstream and downstream side. The bridge was closed to traffic on January 1, 2006 cutting Geyserville in two.

Caltrans maintenance engineers studied the parallel options of repair or replace. After site geology and scour mitigation studies completed, Caltrans decided to replace the bridge and open to traffic in about 8 months. The replacement bridge layout was decided to have the same overall length, profile and vertical clearance over the channel. Matching the existing layout was made mainly to minimize the time in acquiring right of way and to keep the permit process to a minimum. Raising bridge profile and consequently extending bridge length would have led to legal issues with the locals in the area.

CALTRANS' PLANS SPECIFICATIONS AND ESTIMATES

Caltrans structural engineers started the design towards the first week of February 2006. During the initial design phase, designers took into consideration availability of materials and accelerated delivery methods to expedite project construction. Several meetings were held with contractors, suppliers, and manufactures to exchange ideas and ensure the design and detailing met industry expectations with no prospective delay in construction.

The replacement bridge was designed to carry two 3.6 m (12.0 ft) traffic lanes; 2.4 m (8.0 ft) shoulder at each side; a 1.6 m (5.3 ft) sidewalk with an overall width of 14.98 m (49.15 ft). Overall length of the replacement bridge was 298.7 m (980.0 ft). The bridge alignment is straight horizontally, with a vertical parabolic curve of +0.7% and –0.6% upward and downward grades respectively.

Hydraulic concerns of the river migration required the use of fewer numbers of spans for the new replacement bridge. Eight 31.2 m (102.5 ft) long spans and two 24.4 m (80.0 ft) long spans at the ends were used. To provide free board clearance for the 50–year design flood, the superstructure depth was limited to 1143 mm (45 in.) with a depth-to-span ratio of 0.037 for the longer spans.

Fig. 1 Existing Bridge over the Russian River in Geyserville, CA.

Fig. 2 Series of Storms Damaged the Bridge in late December 2005.

During type selection process the designers dealt with the following restrictions:

1- Environmental – no falsework in the channel;
2- Tight schedule – 8 months from closure of the bridge to open to traffic;
3- Better quality control – less demand on the field staff;
4- Construction window – from May till August.

These restrictions led to precast prestressed bridge type as the most suitable alternative. Four standard sections were examined during type selection: I–Girder; Bulb–T Girder; Spread Box Girder; and Adjacent Box Girder. The dictated depth to span ratio is much less than 0.045, a typical value for such a precast system.

All of the considered four alternatives were simple span girders made continuous with cast-in-place composite deck. Every effort was made to produce a biddable design that uses State standard sections and thus a fair competition to all precast manufacturers. Non-standard sections would have required additional time to manufacture new forms or modify existing ones.

Standard I, and Bulb-T sections did not work due to limited superstructure depth and required the use of non-standard sections. Spread Box sections required less than 0.61 m (2 ft) distance between the girders, and the use of forms for deck placement in such short distance and thus deemed unpractical. Adjacent Box girder was the only standard section that met the design demand with tight superstructure depth-to-span ratio. No forms were needed for deck placement and hence a faster field operations. Standard precast prestressed AASHTO box girder 1220 mm (48 in.) wide and 990 mm (39 in.) deep, with a 152 mm (6 in.) cast-in-place reinforced concrete deck was selected during the type selection process (Fig. 3). The adjacent girders were transversely post-tensioned at ¼ span distance at locations of 200 mm (8 in.) thick intermediate diaphragms. A total of 120 girders were used in the original design. Superstructure box girders were made continuous for live loads using cast-in-place diaphragms in-between girders.

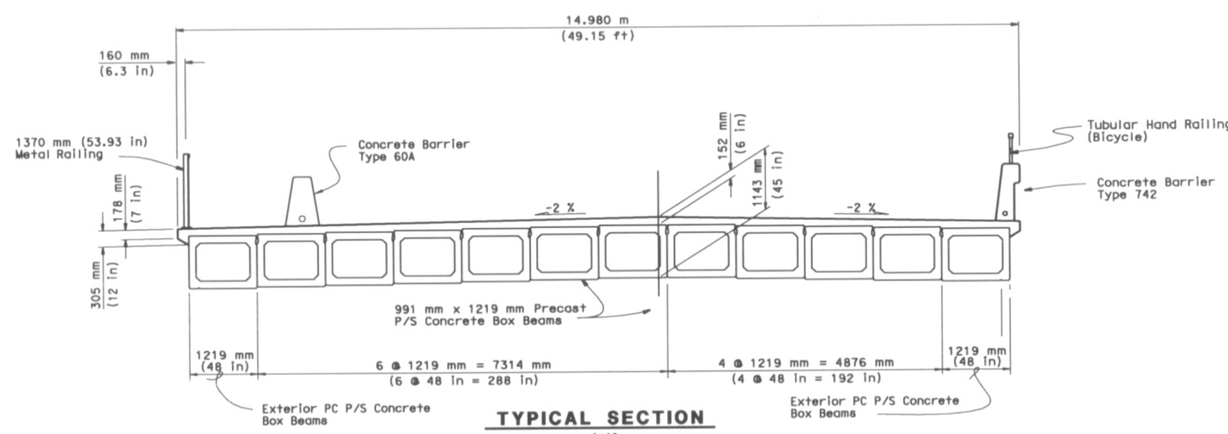

Fig. 3 Typical Cross Section – Original Design.

Superstructure precast box girders were supported on cast-in-place drop bent caps via 300x200x75 mm (12x8x3 in.) elastomeric bearing pads (2 per each box). The drop bent cap has a constant width of 1.8 m (6 ft) and a variable span depth with minimum dimension of 1.8 m (6 ft).

Each drop bent cap was supported by two Cast In Steel Shell (CISS) pile shafts. Cast In Steel Shell pile shafts were chosen based on their high load bearing capacity, site conditions and hydraulic suitability and were preferred over Cast In Drilled Hole (CIDH) piles because of their potential severe caving during drilling.

Caltrans structural engineers considered State furnished materials option for the steel shell pipes to ensure no delay in construction. The size and thickness of the readily available steel shell pipes in the market were used in design and procured for project construction prior to Contract award. Each pile shaft has a 1220 mm (48 in.) diameter with shell thickness of 25 mm (1 in.).

Seat type abutments were designed to stand alone during the 100–year design flood. Potential scour of 11 m (33 ft) at the abutments necessitate the use of three 1220 mm (48 in.) diameter Cast In Steel Shell (CISS) pile shafts.

Bridge site geology consisted of generally interbeded layers of dense to very dense sand, gravelly sand and sandy gravels. Specified pile tip elevations designed for compression and critical scour indicated a driving length of 35 to 44 m (115 to 144 ft) below mudline. In order to determine the final pile geotechnical capacity, a pile load test was designed and recommended at two alternate locations (Abutment 11 and Bent 8) during construction. Possible higher pile geotechnical capacity determined in the field test can lead to a lower pile tip elevation and a reduced driving length, thus a faster construction schedule.

The controlling fault for the bridge site has a maximum credible earthquake moment magnitude of 7.25. This fault is located less than 3 km (1.86 Mile) northeast of the bridge site. Geotechnical study shows that the Peak Bedrock Acceleration (PBA) at the site is 0.6g. The structure spectral acceleration curve shows a maximum of 1.58g for the bridge.

In order to accommodate such large seismic displacement, and provide integral seismic resistance of the bridge structure, the superstructure box girders were pinned to the drop bent caps using reinforcing rebar #29 M (# 9) every 610 mm (2 ft) along the length of the bent cap. Shorter column shafts at Bents 2 and 10 were isolated from the surrounding soil for about 6.5 m (21 ft) below mudline to increase their free length and hence increase their ductility capacity (Fig. 4). Finally, additional mild steel was added to the superstructure girders to resist one quarter of the dead load weight acting as vertical seismic acceleration[2].

Despite the expedited schedule for this replacement project, a few aesthetic measures were considered for bridge bent caps, girders and barriers. The bent caps were designed with simulated capitals, rounded noses and arched soffits to visually reduce their otherwise massive appearance. This effort aided in bringing the bent caps and column shafts into a closer proportional relationship to each other. The smooth vertical face of precast box girders contributed to the tidy effect of the superstructures exterior, thus complimenting the nautical theme of the barriers surface treatment and context sensitive handrails (See Fig. 4 and 5).

By mid March, the biddable design package (Plans, Specifications and Estimates) was ready to list. Bids were based on the sum of the item totals for the work to be done, plus the product of the number of bid working days to complete the work and the cost per day shown on the engineer's estimate. This form of bid contract is designed to reward a general contractor with the least working days to complete the construction. Pile shaft steel shells were purchased by the State on a separate contract and delivered to the job site prior to awarding the general contractor (State furnished materials).

CONTRACTOR'S COST REDUCTION INCENTIVE PROPOSAL

CC Meyers Inc general contractor was awarded the contract on April 11, 2006 with the least bid of 14,383,026 dollars to build the bridge in 80 days. The very next day, the general contractor, CC Meyers Inc, along with his consultant designer R.N. Valentine Inc, and precast manufacturer, CONFAB submitted a Cost Reduction Incentive Proposal (CRIP) to use a non-standard Double Tee precast prestressed concrete girder with multiple stages of post-tensioning in the field. The proposed non-standard Double Tee girder was twice as wide as original design [2.44 m (8 ft) versus 1.22 m (4 ft)] which resulted in half as many girders per span (total of 60 girders versus 120 girders) in original design (Fig. 6). Standard Double Tee section is typically suitable for 12 to 20 m (40 to 65 ft) span length and was not an option in the State approved girder sections for such long spans used for the replacement bridge.

The alternative girder design used two stage post-tensioning to maintain continuity of the superstructure under applied loads. Cast-in-place diaphragm between girders and first stage post-tensioning were used to create continuity under the weight of the 152 mm (6 in.) deck slab. A second stage post-tensioning was applied to carry the bridge superimposed dead and live loads. Figure 7 depicts the sequence of construction stages proposed in the alternative design. Superstructure depth was maintained in the proposed design. No changes were made to the substructure design as a consequence.

Not only was the Double Tee section non-standard, but also the two stage post-tensioning was not a standard practice for precast girder design in California.

Caltrans immediately evaluated the proposal and approved the concept primarily to reduce construction time and possibly cost savings. Closure of the nearest precaster yard and the difficulty of transporting long girders in the local roads made this CRIP necessary. The Contractor consultant designer prepared his design plans and submitted to Caltrans by end of April for review and approval.

Caltrans structural engineers cooperated with Contractor's consultant designer to review and approve the design and detailing of the proposed alternative Double Tee girder in two weeks. Caltrans structural engineers performed a time dependant analysis to check stresses in the girder during pre-tensioning, erection, first post-tensioning, deck pour, and second post-tensioning stages. The State independent check also reviewed deflection assessment at various stages of pre-tensioning and post-tensioning; long term camber; details to account for shortening in longitudinal direction during post-tensioning; intermediate diaphragms; and movement rating for the joint assemblies at the abutments.

Fig. 4 Isolation of Shorter Columns at Bents 2 and 10.

Fig. 5 Nautical theme of the Barriers Surface Treatment.

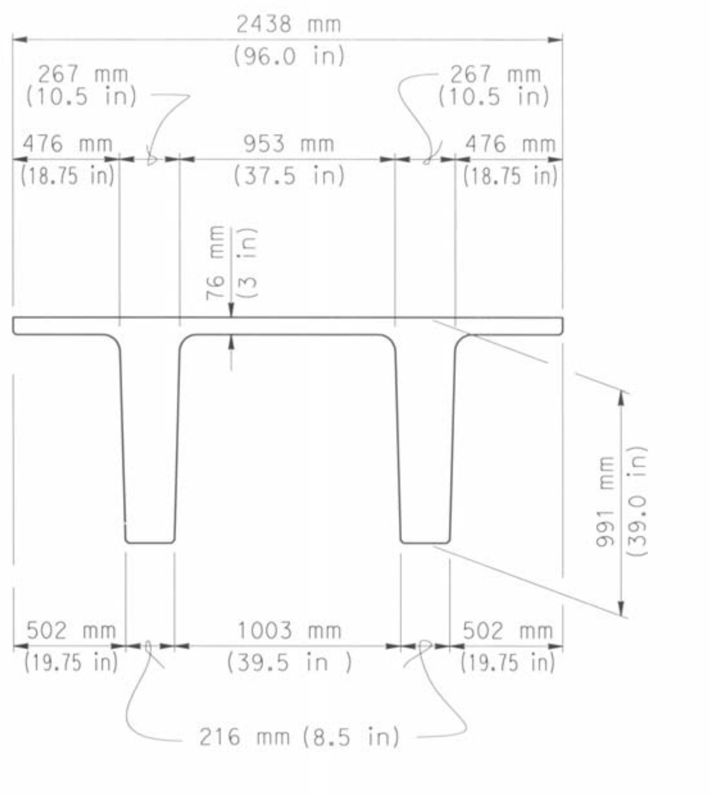

Fig. 6 Proposed Dimensions of Non-Standard Double Tee Girder.

The review process resulted in the following modifications to the proposed design that were later incorporated in the final approved design.

- Time dependant analysis for prestress losses:

Caltrans requested the contractor consultant to perform and submit a staged construction analysis considering time–dependant concrete properties as per NCHRP 517 guidelines[3]. Contractor's consultant used SFrame and Caltrans engineers used RM2000 and CONSPLICE. The independent state review resulted in an increase of number of post tensioning tendons; a minor change in post tensioning profile; and an increase in concrete deck strength from 28 to 34.5 MPa (4000 to 5000 psi).

- Post-tensioning effects during the two stages of stressing:

The proposed design used two separate ducts to perform the two stages of post tensioning in the field. Small clearance distance between the two ducts at the high and low points along the profile and the unknown effects of possible concrete crushing in-between ducts made this proposal risky. Caltrans requested the use of one duct for post tensioning with partial prestressing (i.e., one duct with two groups of strands for each stage of post tensioning). This was later changed during construction to allow for all the tendons in the duct to be stressed partially up to certain percentage of ultimate stress in stage 1, followed by full prestressing in stage 2 (up to 44% of ultimate stress in stage 1 and 75% of ultimate stress in stage 2).

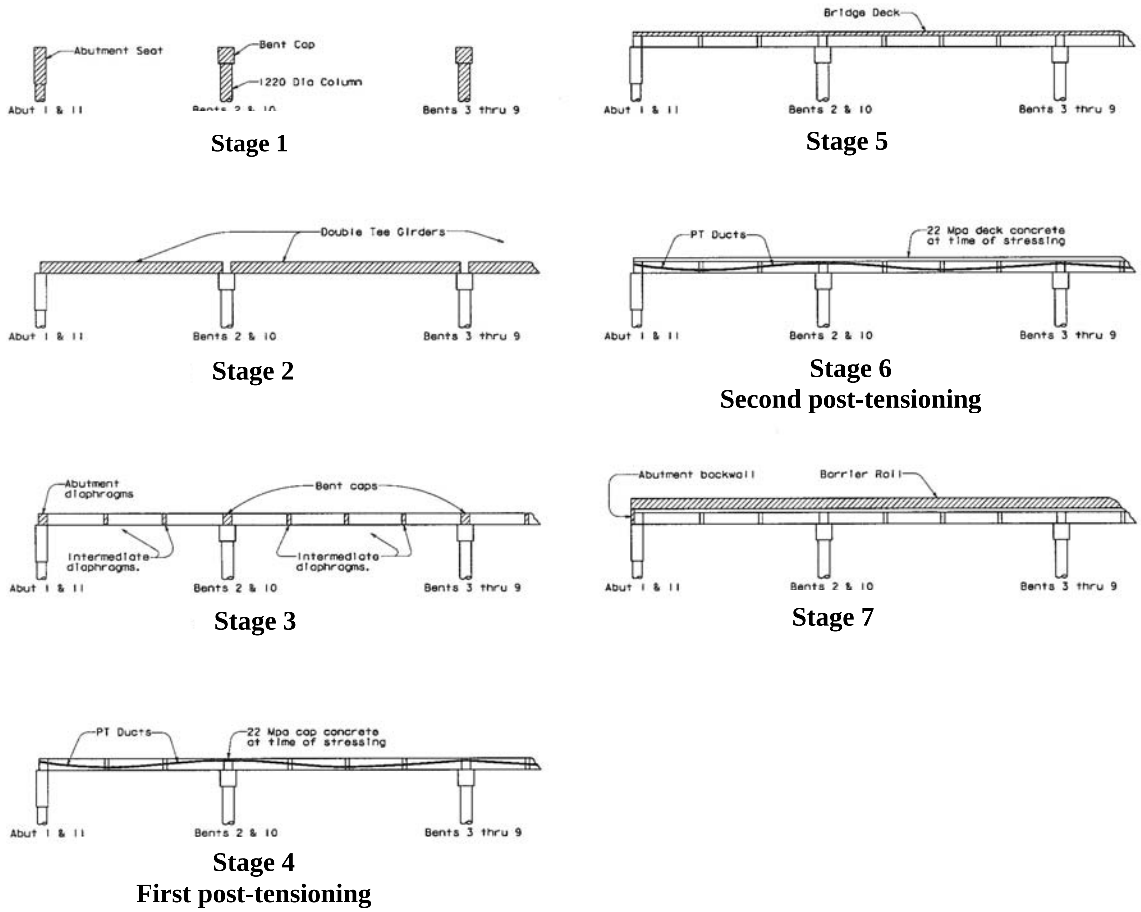

Fig. 7 Sequence of Construction Stages

- Excessive shortening in longitudinal direction during post-tensioning:

Introducing post tensioning of a continuous 298.7 m (980 ft) long frame exerted high longitudinal forces to the substructure pile shafts under service loads. These high forces were not encountered in the original design, and it is particularly important for the outer bents away from the point of no movement of prestressing strands. Caltrans requested the superstructure to bent cap connection to be modified to allow for longitudinal movement during post tensioning at the two outer bents (Bent 2 and Bent 10) without transferring any displacement to the pile shafts. Metal plates with greasy surface were used to allow for superstructure sliding with minimal force transfer to the supporting bent caps (See Fig. 8). After initial shortening of the superstructure and grouting of the prestressing duct, the connection was locked in place.

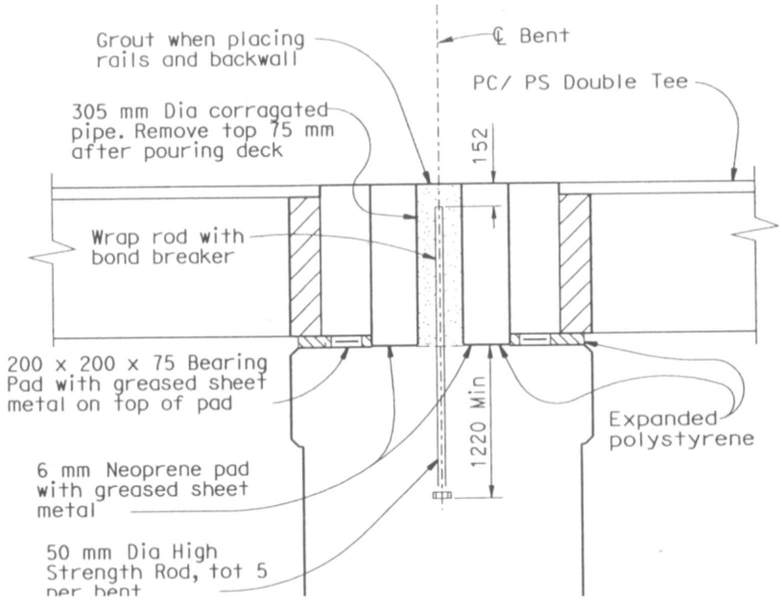

Fig. 8 Detail to Account for Sliding Movement at Bents 2 and 10.

- Movement rating for the joint assemblies at the abutments:

Post-tensioning of the continuous frame also increases the movement rating at the joint seal assemblies at the two abutments. Caltrans requested the movement rating to be increased to 175 mm (7 in.).

- Intermediate diaphragms

Proposed superstructure design eliminated the intermediate diaphragms used in the original design. Caltrans requested the use of intermediate diaphragms at fourth point of longer spans and at third point in the two smaller spans as per Caltrans Bridge Design Specifications[4]. Not only it allows for better load and deflection distribution among girders but also it stiffens the very flexible Double Tee girders during construction and before final post tensioning. Four inch holes were made in the deck of double tee girders during manufacturing to allow for the cast in place pour of the diaphragms in the field (Fig. 9).

ACCELRATED CONSTRUCTION

The general contractor mobilized his equipment, and demolished the existing pony truss bridge while the proposed alternate superstructure design was prepared by his consultant and reviewed and approved by Caltrans. A temporary trestle was built on the upstream side of the bridge to provide access to the work site for demolition of existing bridge and the construction of new bridge. The temporary trestle was about 183 m (600 ft) long, 12.2 m (40 ft) wide and supported by 300 mm (12 in.) steel pipe piles spanning 9 m (30 ft) apart. The trestle was designed to carry the drilling equipment, cranes and served as a platform for double tee girder storage before placing on bent caps.

Construction proceeded in early May with driving the new substructure cast-in-steel shell piles and building the drop bent caps. Pile load testing was conducted at Bent 8 and Abutment 11 to determine actual in-situ soil resistance. No reduction in driving length was gained however, as better than estimated soil skin friction and end bearing were not warranted.

In the meantime, Double Tee girders were fabricated in May/June, and erection was completed in July. Cast-in-place diaphragms and intermediate diaphragms were cast in early August, followed by the first stage post tensioning operation. Deck was poured and the second stage post tensioning took place three days later. Work around the clock resulted in the bridge opening to traffic on August 17, 2006 to the delight of the local community. See Fig. 10.

Fig. 9 Cast-in-place Intermediate Diaphragms Connecting Precast Double Tee Girders.

Fig. 10: Construction of the Russian River Bridge: a) Demolition of Existing Truss Bridge, b) Driving of Cast In Steel Shells, c) Forming Drop Bent Cap with Architectural Features, d) Fabrication of Double Tee Girders, e) Erection of Girders and Deck Slab Reinforcement, f) Post-tensioning – First Stage.

CONCLUSIONS

The emergency replacement of the Russian River bridge presented many design challenges and the opportunity for creative solutions. Successful completion of this environmentally sensitive and accelerated construction drew the following lessons:

- Precast composite deck bridge type is a viable solution for accelerated projects in California. Cast-in-place box girder construction, while favored by most contractors in the State, has its limitations.
- Wider precast sections eliminated the deck falsework, and reduced number of precast girders. Use of such wider precast sections reduced time and cost for fabrication, delivery and erection and was a critical factor for the fast construction of this project.
- Multi-stage post-tensioning with precast sections may be an effective solution for bridges with tight span to depth ratio.
- Concern for the project potential delay because of materials shortage (steel shells) led to State furnished materials option in project contract. This option deemed to be unwise decision as most contractors are adapt in securing materials as fast and cheap as possible avoiding any routine and extra cost.
- Specifications on accelerated projects should allow for greater alternatives such as high early strength concrete and rapid set concrete.
- Better assessment of elongations during jacking operation due to partial prestressing needs to be addressed.
- Finally, exchange of ideas between design, construction and fabricator is essential for out of the box creative solutions. Effective communications and partnering are a key element for the success in such accelerated project.

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