First Two-Stage PT Wide-Flange Girder Bridge in California

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Abstract

The first post-tensioned spliced wide-flange girder bridge, which is Riego Road Overcrossing, has been designed by California Department of Transportation (Caltrans) two years ago. Now it has been constructed and the new bridge is expected to be open at the end of this year. The project is a new interchange at Riego Road and State Highway 99 in southern Sutter County. The bridge has two spans, 154' and 141', over Hwy 99. Cost of the project is $31 millions. Caltrans' new standard PT wide-flange girders have been used for this project. Since this is the first California post-tensioned spliced wide-flange girder bridge designed by Caltrans, it faced many challenges from the different phrases. Feasibility, design ability, constructability and cost are being challenged along the whole project process. During the design, manufacture and construction, Caltrans worked with the Contractor and the precast sub-contractor to have solved various problems. There are several lessons learned from this project. Caltrans has developed two new and improved standard precast wide-flange girder shapes in recent years. One of the new girder types, post-tensioned WF girder type, has the span capability of exceeding 200 ft, which is very comparable to those of cast-in-place post-tensioned box girders. Combined with the use of high-strength concrete, the spans and girder spacing can be maximized. The standard 8-inch web is for post-tensioned girders where splicing for continuity is applied. The new girders serve to promote the Accelerated Bridge Construction (ABC), as well as economic design. This paper illustrates Riego Road Project as a case study to present the design, shop drawing review, manufacture and construction for this new post-tensioned spliced WF girders project. The author wants to share the valuable experience and important information with other State Agencies and PCI members about using precast girders for long span bridges.

Keywords: Wide-Flange Girders; Precast Bridge; Post-Tensioned; Accelerated Bridge Construction (ABC); Long Span

INTRODUCTION

Prestressed concrete superstructure can be categorized as "pre-tensioned" or "post-tensioned". "Pre-tensioned" means the strands are tensioned prior to the placement of concrete, and "post-tensioned" means the strands are tensioned after the concrete has attained a minimum level of compressive strength. Stress transfer to the concrete for both applications occurs once the concrete has reached a design-specified compressive strength; in pre-tensioned concrete, the strands are typically in direct contact with the concrete, while in post-tensioned concrete, the strands are isolated in ducts that are usually grouted upon completion of the stressing operation. In the past ten years, with the increasing demand of longer span lengths, tighter vertical clearances, and rapid construction, a new type of precast/pre-tensioned concrete girder system, named spliced girders, has been type selected and used. A spliced girder is a precast prestressed concrete member that is fabricated in several relatively long pieces (i.e. girder segments). These girders are assembled into a single girder or a continuous girder onsite for the bridge superstructure. Post-tensioning is generally used to reinforce the connection between girder segments longitudinally. The bridge cross section is typically a conventional structure in which multiple precast girders support a cast-in-place concrete composite deck. The spliced girder, with longitudinal post-tensioning, could provide not only superstructure continuity, but also an integral cap-to-column connection if designed properly.

The main advantages of the precast/prestressed concrete spliced girder bridge project can be summarized as follows:

(1) Rapid construction with the use of precast elements to reduce congestion, traffic delays, and the total project cost.
(2) Longer span lengths to reduce the number of piers, avoid obstacles on the ground such as roadways and utilities, improve safety if traffic is underneath, and avoid the placement of piers in waterway to reduce environmental impact.
(3) Eliminating superstructure joints to improve structural performance (including seismic performance), reduce long-term maintenance cost, and increase service life.
(4) Minimizing bridge superstructure depth to obtain required vertical clearance for traffic or railway, through the use of post-tensioning continuous members.
(5) Minimizing falsework to improve the flow of traffic and improve safety for traffic and construction workers.
(6) Increasing the girder spacing to reduce the numbers of girder lines and the total project cost.

Currently, California developed a new wide flange girder shape, which has the span capability of exceeding 200 feet for post-tensioned splicing specifically. The WF girder section has a large bottom flange that can accommodate a large number of strands and post-tensioned ducts which increases flexure capacity for simple span and provides larger compression area for continuous spans. Combined with the use of high-strength concrete, debonding features and post-tensioning, the spans and girder spacing can be maximized.

When considering using post-tensioned spliced girder for longer span bridges, the first thing is to select girder web to be wide enough, such as 8”, to accommodate the post-tensioning ducts. The depth-to-span ratio for post-tensioned spliced girders is approximately 0.040-0.045. The new WF girder has very efficient girder sections and could normally be spaced at the distance of two-times the bridge depth. If girder spacing has been designed closely, the depth-to-span ratio for post-tensioned spliced WF girders even could be in the range of 0.035 to 0.040. That will make the superstructure very shallow and allow more vertical clearance. Or it could decrease the bridge profile to reduce the bridge and roadway embankment backfills and reduce the project cost.

In California, several Amendments to AASHTO LRFD Bridge Design Specifications have been made to make the spliced girder system more feasible. To avoid any congestion of pre-tensioning strands and post-tensioning tendons in the spliced girders, using straight strands is more desirable alternative than harping. In order to control tensile stresses at the top fiber of the girder ends, California Amendments allows maximum up to 33 percent debonded strands, For maximum post-tensioning forces and duct sizes, California Amendments also increases the limit of duct size to the web thickness ratio from 40% to 50%. Therefore, the 8-inch web can accommodate a maximum 4-inch outside diameter ducts, which can fit a large 19-0.6” strands PT system.

CASE STUDY: RIEGO ROAD OVERCROSSING PROJECT

This project is located approximately 10 miles north of downtown Sacramento. The bridge portion is to construct the Riego Road Overcrossing with a 2-span structure that will consist of four-12 feet lanes each direction with shoulder on each side and a median.

The bridge was designed by Caltrans and the contractor is MCM Construction, Inc. The precast girders were provided by Kie-Con Inc.

The Riego Road Overcrossing is 156’ wide and consists of two long spans, 154’ and 141’. The superstructure depth is 6’-5”, amounting to a depth-to-span ratio of 0.041 and 0.045. The superstructure type is precast prestressed wide flange girder with cast-in-place bent cap, deck, and end diaphragms, and then longitudinally post-tensioned for continuity and made integral with a cap-to-column connection. At Bent 2, there are five 6.00 ft diameter columns pinned at the bottom to new piled footings. The structure was designed under the LRFD design criteria with California Amendments. Figure 1 shows the bridge elevation and its typical section.

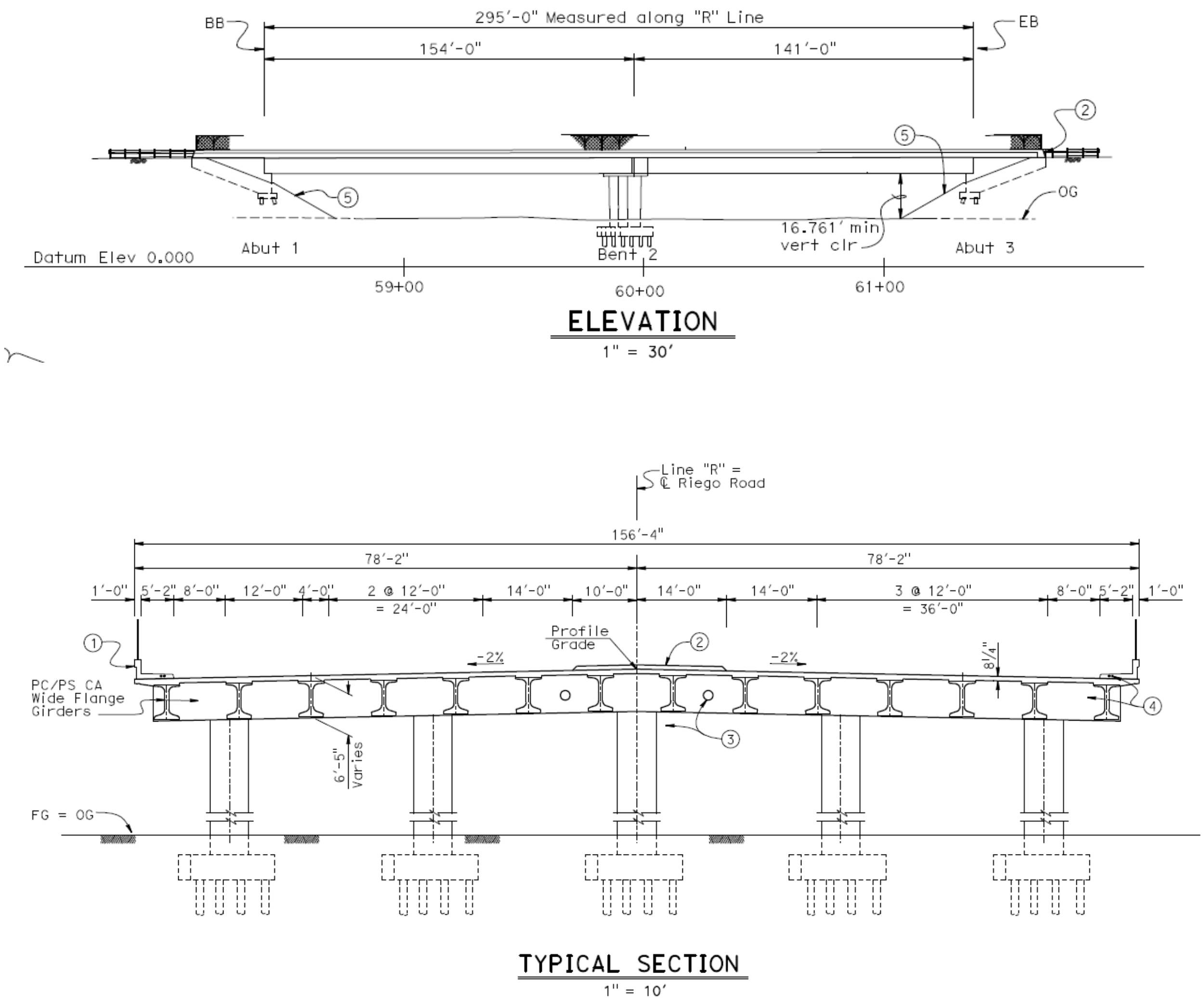


Figure 1: Elevation and Typical Section

Type Selection

Three bridge alternatives were considered in the initial feasibility study. Two staged post-tensioning WF Girder is selected for final design and construction due to its most economical cost and no falsework. Three alternative details are shown below.

Alternative	1	2	3
Structure Type	Post-Tensioned Cast-in-Place PS Box Girder	Two Staged Post-Tensioned PC/PS Bulb-Tee Girder	Two Staged Post-Tensioned PC/PS WF Girder
Total Length	295’	295’	295’
No. of Spans	2	2	2
Span Length	154.0’and 141.0’	154.0’and 141.0’	154.0’and 141.0’
Width	156’-4”	156’-4”	156’-4”

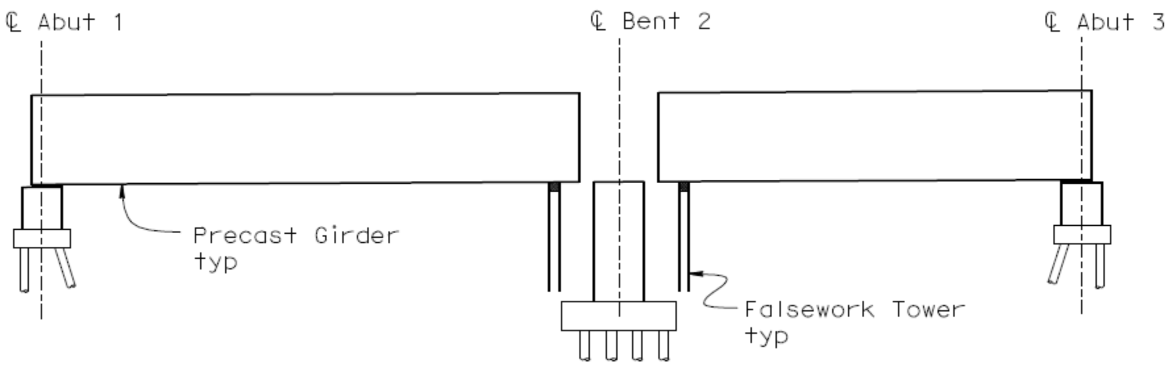
Abutments	Short seat on pile footing supported by concrete driven piles (Class 140). Fill slope 1.5:1 longitudinally and 4:1 transversely. Slope paving. Abutments have skew angle of 12 degree to align with the Route 99.		
Bent	Integral bent cap supported by 5 columns with pile footing. Column diameter is 6.0'. Concrete driven piles (Class 140) support the footings.	Integral bent cap supported by 5 columns with pile footing. Column diameter is 6.0'. Concrete driven piles (Class 140) support the footings.	Integral bent cap supported by 5 columns with pile footing. Column diameter is 6.0'. Concrete driven piles (Class 140) support the footings.
Construction Stages	New construction, only one stage is needed. Traffic on Riego Rd will be detoured.		
Falsework	**YES**	**NO**	**NO**
Approaches	Structure Approach Type N(30S)		
Deck Protection	This bridge is located in Area II. No special deck protection is required.		
Utilities	Future utility opening on north side of bridge is required.		
Future Widening	No future widening is planned		
Cost	**$5,556,000**	**$5,290,000**	**$5,055,000**
No. of Working Days	139	124	124

Two staged post-tensioning WF Girder is selected because of its least cost and no falsework. Two staged post-tensioning Bulb-Tee girder alternative was considered as well. However, since its shape is not as efficient as WF girders, the girder depth needs to be 6" more than WF girders or it needs to add another girder line. Therefore, the estimated cost is slightly higher than that of WF girders. Post-tensioned prestressed cast-in-place concrete box girder alternative initially was studied. Due to its required falsework for the superstructure, the bridge needs to be about two feet higher than WF girder superstructure. In that case, the cost of the structural backfill increases.

Two staged post-tensioning WF Girder was selected because of its other numerous advantages. The WF girders are very stable for transporting and erection. Eight-inch webs can be employed to accommodate post-tensioning ducts. More importantly, after casting the deck, bent, and diaphragms in place then post-tensioning, the integrated superstructure allows for depth-to-span ratios of less than 0.045. This results in the satisfaction of minimized bridge height, which reduced the cost of roadway backfills and saved roadway construction about one million dollars. Finally, the columns were pinned at the bottom to take advantage of smaller footing designs because of the integral girder-cap-column design. The precast prestressed WF girder with longitudinal post-tensioning alternative was ultimately chosen since it posed above mentioned advantages and less falsework congestion and cost less overall.

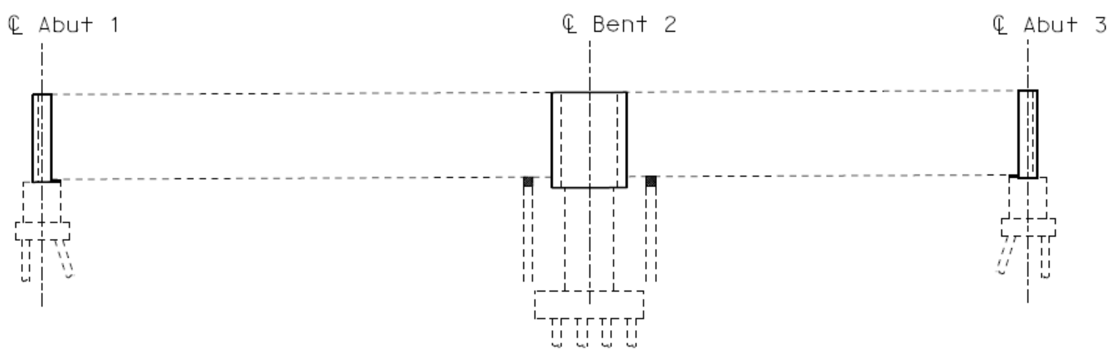
Description of Construction Sequence

The following figure 2 illustrates the bridge construction sequence steps.



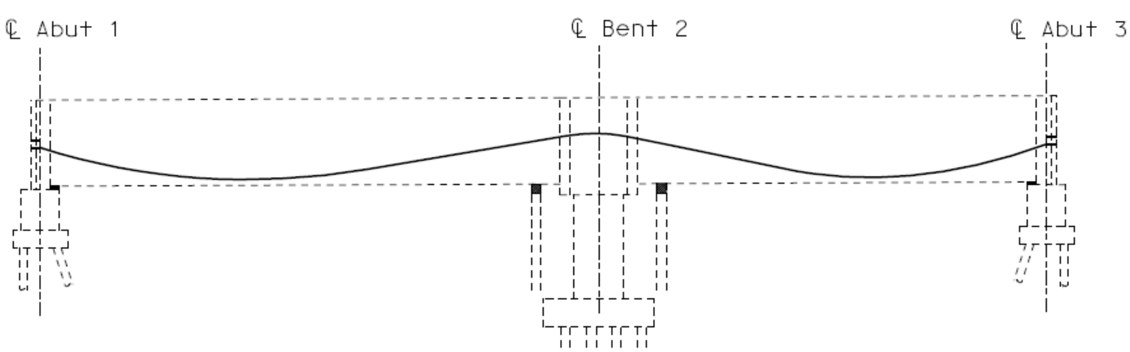
STEP 1, 2, AND 3

STEP 1, 2 and 3: Girders are cast at precasting plant while the substructures are constructed. Erect falsework tower. Erect precast girders.



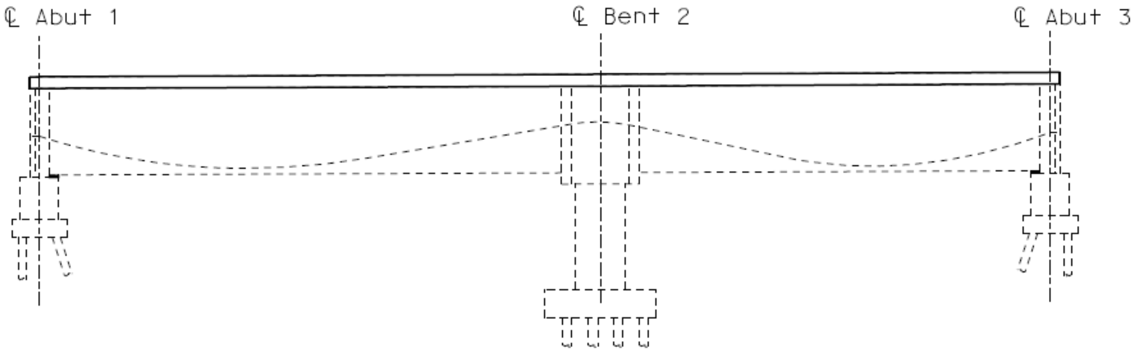
STEP 4

STEP 4: Construct cast-in-place end diaphragms, bent cap, and intermediate diaphragms



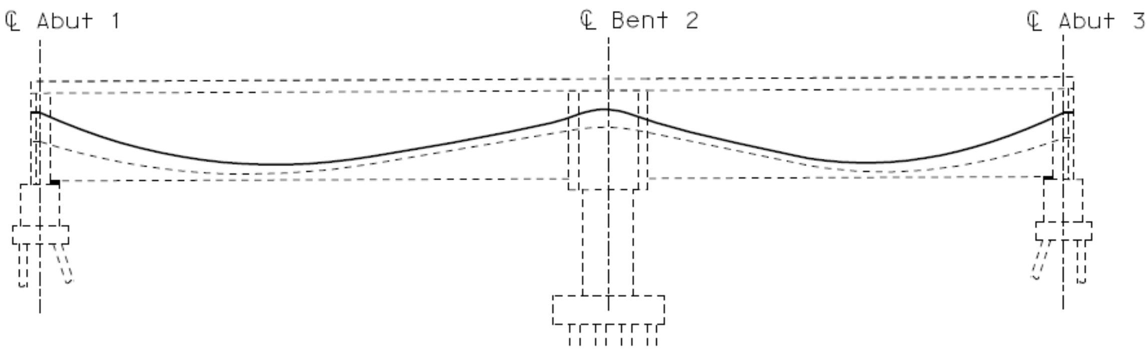
STEP 5

STEP 5: Complete Stage 1 Post-tensioning



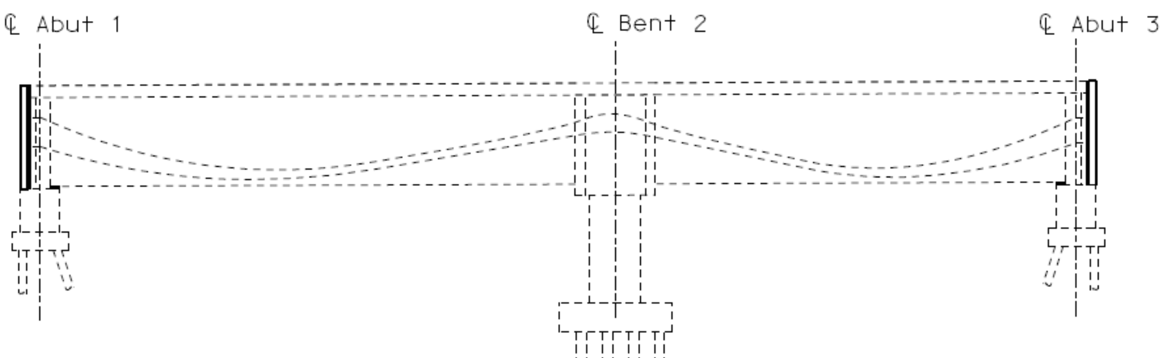
STEP 6 AND STEP 7

STEP 6 and 7: Remove falsework towers, Form and pour deck concrete



STEP 8

STEP 8: Complete State 2 Post-tensioning



STEP 9

STEP 8: Complete prestress blockouts, construct backwalls of abutments and barrier rails.

Figure 2: Bridge Construction Sequence

Design/Detailing/Fabrication Considerations

Since this is Caltrans's first time to design two-stage post-tensioning precast wide flange girder bridge, designing posed some challenges comparable to designing any bread-and-butter type bridges. Firstly, a study was conducted to compare one-stage post-tensioning with two-stage post-tensioning. In general, one-stage post-tensioning is relatively simple in design and construction, and is mostly used with bridge span lengths less than approximately 140 feet. Normally, it is desirable to apply all of the post-tensioning after the deck becomes part of the composite deck-girder section. When the bridge span length exceeds approximately 140 feet and/or the precast girders are in segments, two-stage post-tensioning typically results in a more efficient bridge system. The first-stage post-tensioning force is designed to control concrete stresses throughout the continuous span for loads applied before the second stage of post-tensioning. The second stage post-tensioning force is usually designed for superimposed dead loads and live loads. For this project, the benefits of the two-stage post-tensioning method include: lower required pretensioning force, more efficient total prestressing force for the structure, lower required f'_{ci} and f'_c for the precast girder, and better deflection control.

Secondly, because of lack of available commercial software that provides structural analysis as well as bookkeeping, a spreadsheet has been used for recording and manipulating data. This project required good spreadsheet skills combined with hand calculations for the simply supported wide flange girder analysis and a basic structure analysis tool for evaluating prestressing and live loads forces for indeterminate structures. It is a matter of bookkeeping loads and stresses. In this case, Caltrans Bridge Design System Software (CT Bridge) was used to evaluate superstructure moment and stresses when the structure is continuous during the non-composite stage (before and immediately after deck placement) and when composite (post-tensioned plus live loads). When calculating the effects of pretensioning and two staged post-tensioning, the short-term and long term prestress losses must be considered. Time dependent calculations are used to properly account for prestress losses associated with multiple stages. A detailed spreadsheet and Mathcad were used for bookkeeping and verification of loads, prestress losses and stresses between each stage of construction.

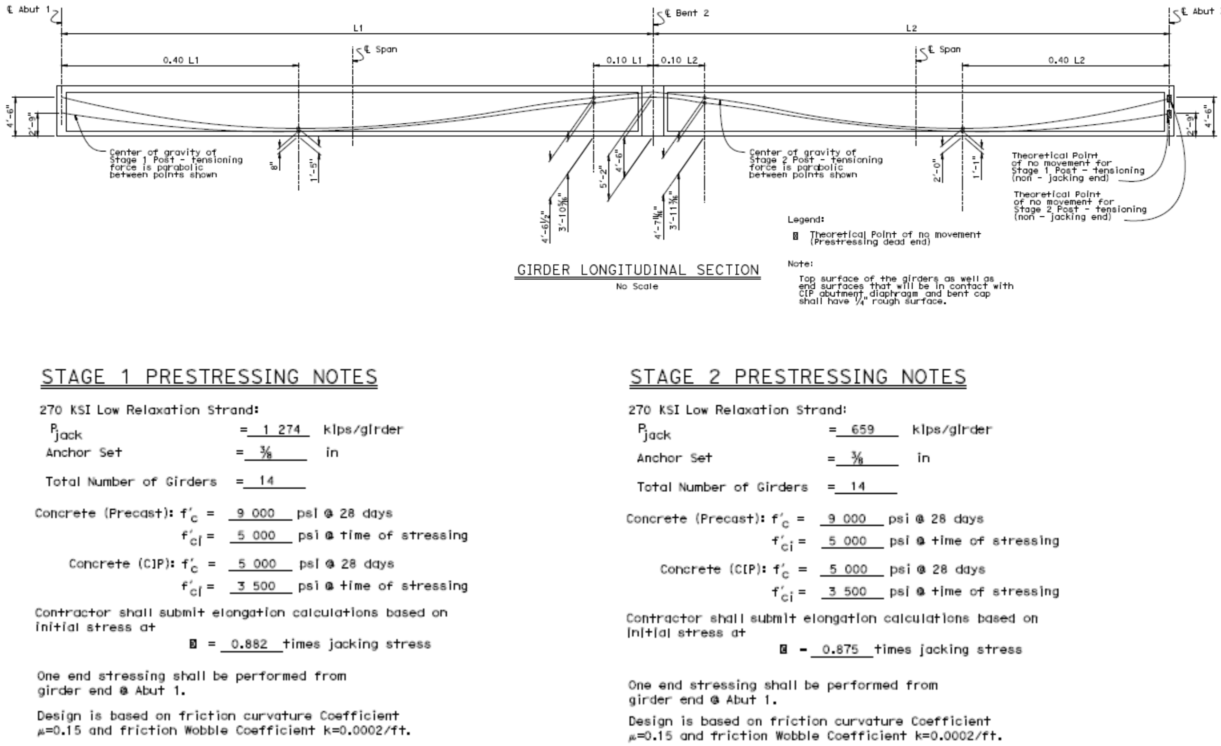


Figure 3: Two Stage Longitudinal Post-tensioning

It is often the case that detailing and fabricating a component for its intended design is even more challenging than the design process. When designing staged post-tensioning, both prestressing forces and the post-tensioning tendon profile must be shown on the design plans as illustrated in Figure 3. Although a specific tendon placement pattern need not be provided in the bridge plans, the designer shall develop at least one workable tendon placement solution at all locations along the span, including at anchorages. Tendon duct size must be based on a duct area at least 2.5 times the total area of prestressing strands. The outside duct diameter shall be less than one half the girder web width per AASHTO LRFD Article 5.4.6.2 and California Amendment 5.4.6.2 and must be shown on the shop drawing plans. The maximum number of strands per tendon shall not exceed 19 0.6”-diameter strands. During the construction, shop drawings for the post-tensioning system and tendon placement have been reviewed to ensure it meets the requirements of its design. Instantaneous deflections due to post-tensioning at different stages were shown on the contract plans. These deflection values are used to set screed grades in the field. In addition, top surface and end faces of the precast girder at joints must be intentionally roughened. For this project, welded wire reinforcement has been used for shear stirrups and their supporting reinforcement and details are provided on the design plans.

Another challenge associated with designing this unique structure is dealing with the positive moment over the bent cap before the cast-in-place deck pour. Bear in mind, this construction stage occurs after the end diaphragms, intermediate diaphragms and bent cap are cast and the first stage post-tensioning was applied. Because this stage occurs when the superstructure is continuous, positive moment originating from the first stage post-tensioning must be accounted for over the bent cap. This challenge was resolved by extending the pre-tensioning strands into the bent cap as shown in Figure 4a&b.

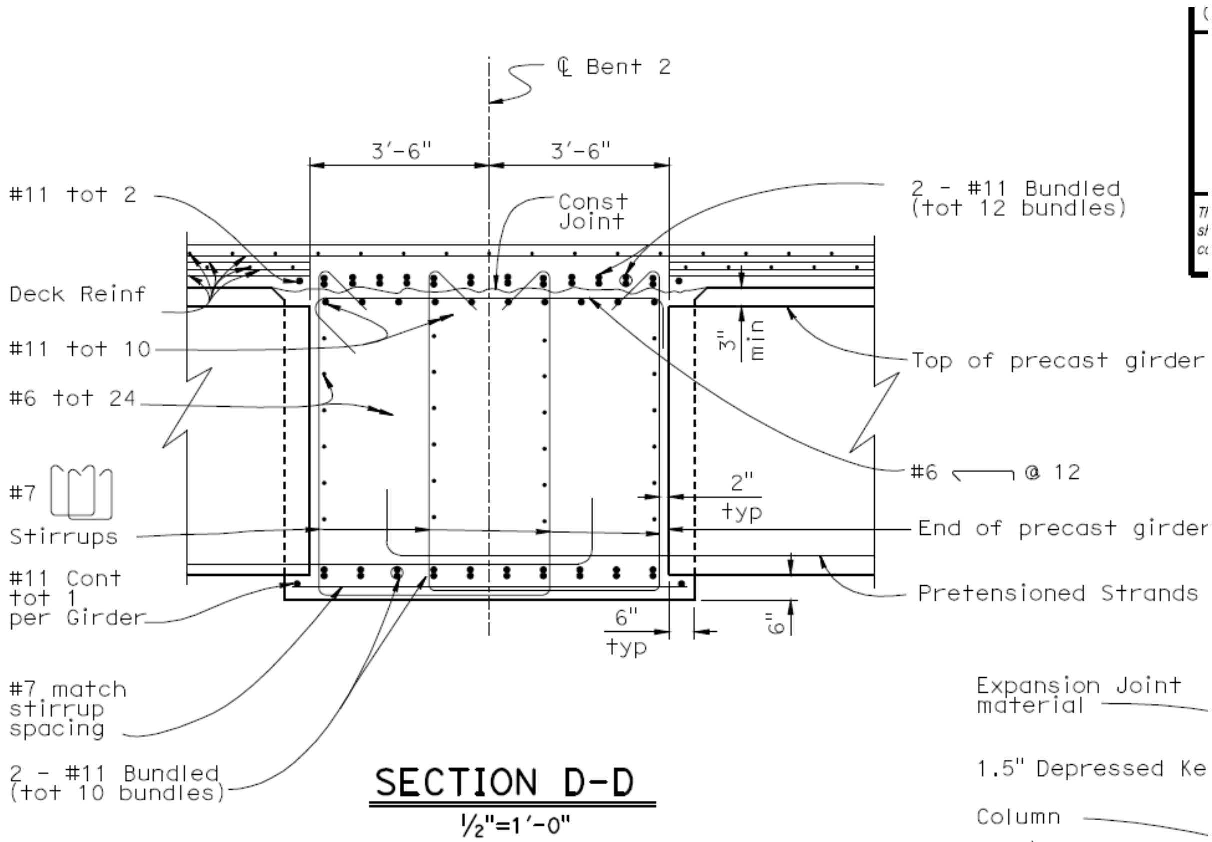


Figure 4a: Bent Cap Connection

Figure 4b: Extended Starnds at Bent Cap Connection

In California, moment reversal at the bent cap due to the cyclic nature of seismic loads has to be considered. It was found that the positive moment associated with seismic loads exceeded the superstructure's dead load moment. This condition was resolved by extending some of the bottom pre-tensioning strands into the bent cap as well in Figure 4a&b.

The designer has to consider that precast prestressed girders usually do not experience negative moment under service load. However, when the girders are made continuous over the bent, then loaded with additional dead load and live load, the bottom bulb of the superstructure is under tremendous compressive stress near the bent caps. Even before the continuous structure is loaded, the bottom bulb is under some compressive stresses induced by pre-tensioning. De-bonding some of the bottom strands and added four strands at the top flange alleviated this problem as shown in Figure 5.

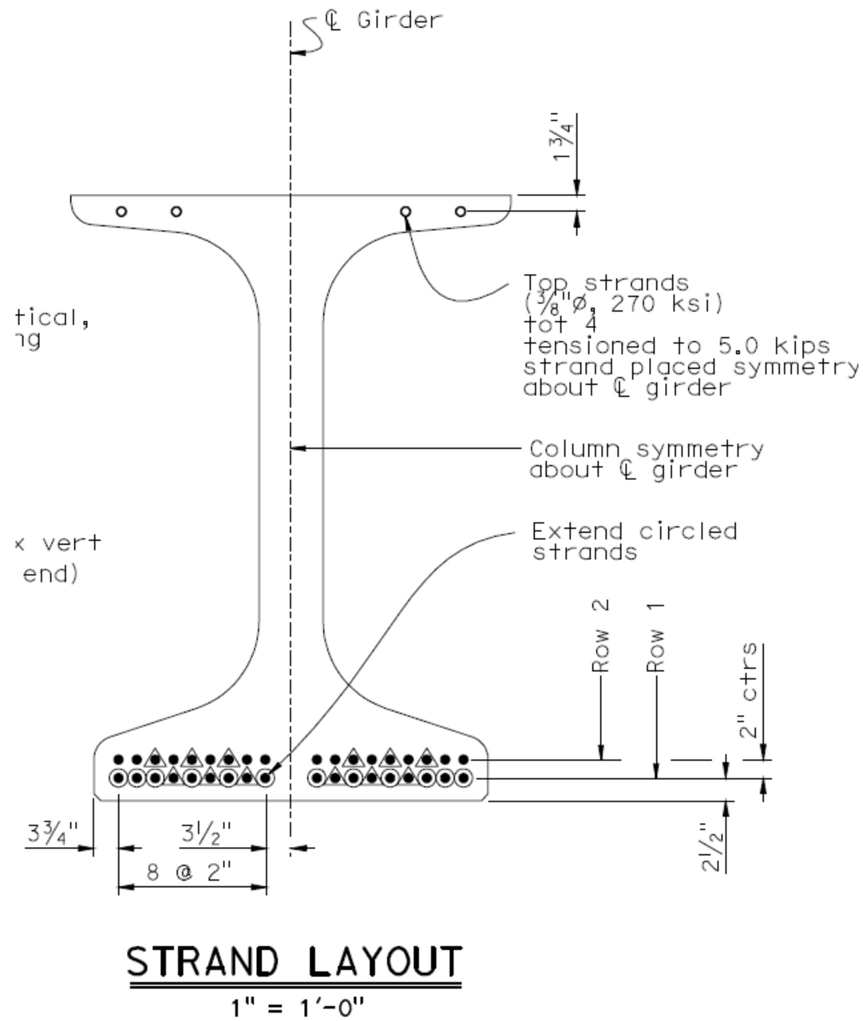


Figure 5: WF Girder Strand Layout

For the shear design, since the girder shear demand is very high due to the California P-15 permit loading. The required transverse shear reinforcement are substantial. Per AASHTO 5.8.3.3-2, the maximum shear resistance $V_n(V_c+V_s)$ shall be less than $0.25f'_c b_v d_v$. That means the maximum shear capacity depends only on f'_c, b_v, and d_v. To certain level, adding reinforcement stirrups will not increase the shear capacity. Because d_v is constant and b_v is constant also. The only factor that could increase the shear capacity is f'_c . Therefore, to meet the applied loading requirement, f'_c had to be raised from 7 ksi to 8 ksi to increase the shear capacity so that shear reinforcement could be utilized. In addition, per AASHTO 5.8.2.9, in determining the web width at a particular level, one-quarter the diameter of grouted ducts at that level shall be subtracted from the web width. For this project, the web width reduced from 8" to 7" due to the post-tensioning ducts. Therefore, the concrete strength f'_c needs to be increased from 8 ksi to 9 ksi to satisfy this requirement. Overall, the concrete strength f'_c is controlled by shear design, not flexure design, in this project.

Because of the post-tensioning anchorage systems, the end block is needed at one end of the girder. The new WF girder has the bottom bulb width of 3'-10 ½". To reduce the weight and satisfying the aesthetic appearance for the exterior girders, two types of end blocks are detailed. Figure 6 shows the details of these two end blocks.

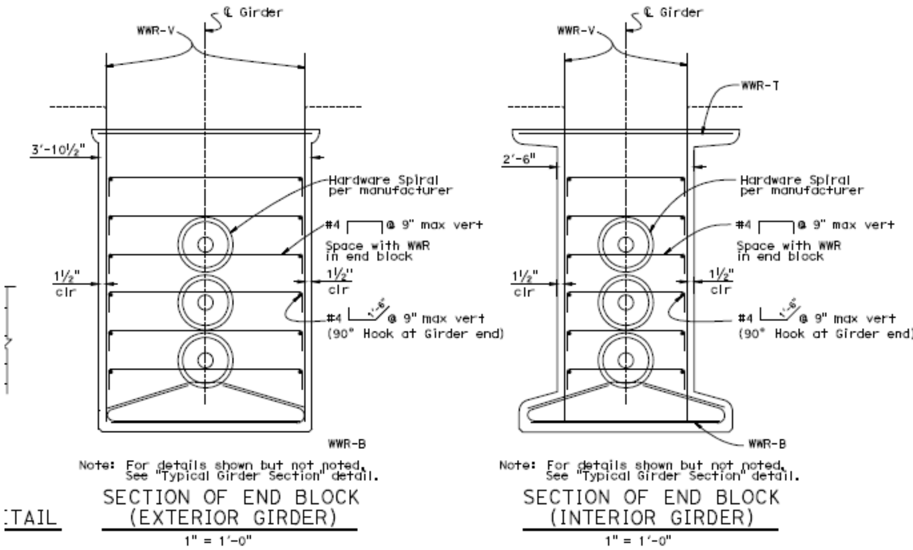


Figure 6: End Block Details

LESSONS LEARNED

There are several lessons learned from this project. One interesting finding is the girder unit weight. During the precast girder transportation and erection in December 2013, the precast manufacture found out the girder weight was more than they calculated for trucking limits. Even though the precaster had used higher unit weight for the girders, the 154' girders weighed more than that was calculated. For that reason, the girders required an 11-axle truck for transportation, instead of a 9-axle truck as planned. Also, a new permit was needed for the 11-axle truck. Therefore, there was a six-week delay in the bridge construction schedule due to postponed delivery of the girders. The first fourteen girders (141') were delivered and erected in early December of 2013 (Figure 7). The remaining fourteen girders (154') were transported and erected in late of the January of 2014. Fortunately, the erection of the bridge girders was not in the critical path of the overall project schedule.

During the bridge design and construction, it is generally assumed that the unit weight of reinforced concrete is 150 pcf. In Bridge Design Specifications 3.3.6, it states "the following weights are to be used in computing the dead load: Concrete, plain or reinforce…….150 pcf". But under AASHTO LRFD Article 3.5.1, it shows "In the absence of more precise information, the unit weights, specified in Table 3.5.1-1, may be used for dead loads". The Table 3.5.1-1-Unit Weights lists "Concrete-Normal Weight with f'c<5.0 ksi, use 0.145kcf; Concrete-Normal Weight with 5.0<f'c<15.0ksi, use 0.140+0.001f'c (kcf)". Under C3.5.1, it states "……. The unit weight of reinforced concrete is generally taken as 0.005 kcf greater than the unit weight of plain concrete…..". Riego Road WF girders have f'c=9ksi. If we use AASHTO LRFD unit weight formula, plain concrete unit weight is 0.140+0.001x9=0.149kcf, add 0.005kcf for reinforcement, the reinforced concrete unit weight is 0.154 kcf. Therefore, based on current code, the unit weight for reinforced girders is more than 0.150kcf we normally use. For high strength concrete, the unit weight of concrete mixture may be higher than the normal concrete mixture. So it is possible that the unit weight of high strength concrete with prestressed strands and reinforcement reaches 160pcf or even higher.

With AASHTO LRFD HL93 loading and CA Amendments P-15 loading, higher concrete strength and more prestressed tendons/strands and more reinforcement may be required for superstructure design, especially for longer span bridges. Therefore, high density aggregate and cement may need to be used for increased concrete compressive strength. Also, larger Pjacking force and increased mild reinforcement will add unit weight of normal girders. The following uses Riego Road WF girder calculation to illustrate the increased unit weight of the girders.

Girder Data: 5'-6" WF Girder, L=154', Girder Section Area=1140 in^2
Assume that unit weight of the concrete for girders with all reinforcement, but without Pjacking force, is 150 pcf.
For each girder, total Pjacking=3514 kips, prestressed area Aps = 3514/202.5 = 17.35 in^2
P/S Steel unit weight = 490 pcf
Girder composite area = (1140-17.35)+17.35x(490/150) = 1178 in^2
Therefore, unit weight for composite girder section = 150x(1178/1140) = 155 pcf

Therefore, the girder unit weight for the long span high strength prestressed concrete bridges needs to be modified from normal 150pcf. AASHTO LRFD Table 3.5.1-1 shall be used. This lesson learned has been implemented to Caltrans Bridge Design Aids.

Figure 7: One-Span Girders Have Been Erected

SUMMARY

The advantages of utilizing the new wide flange spliced girder system is quite evident when one considers the mitigation of impacts to other important aspects of a project such as longer span lengths, minimizing bridge superstructure depths, and reduction in falsework. The Riego Road project case study indicates the feasibility of construction at a reasonable economic outlay. When engineers and planners are challenged with similar constraints as those described in the case study such as lower bridge profile, no falsework, and cost effective, the precast wide flange post-tensioned spliced girder is a viable bridge alternative. In addition, the new wide flange girder type has the span capability of exceeding 200 feet with splicing and its depth-to-span ratios are comparable to those of other types like cast-in-place post-tensioned box girders. Through the Riego Road project, Caltrans has gained various valuable design and construction experiences such as staged construction sequence, two staged post-tensioning, extended strands at bent cap connection, debonding and adding top strands for the girders, time dependent prestress losses calculations, special shear design, and adequate girder unit weight for design and transportation. This case study has approved that Caltrans has taken all the advantages and benefits of mature practice of low cost post-tensioning construction techniques in California and applied it to the precast girder bridge systems and utilized this construction method to meet the California specific challenges. It can be foreseen that more and more WF spliced girder bridges will be built in the coming decades. As California moves forward to improve mobility on an unprecedented scale, the new and efficient wide-flange girder type, combined with post-tension splicing, can provide a cost-effective way to accelerate delivery of bridge improvement projects in California.

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