Continuity Strengthens South Fork Hoh River Bridge Replacement

Craig R.Owen
Consulting Structural Engineer
Port Angeles, Washington

The South Fork Hoh River Bridge (see Fig. 1) is a 142 ft 3¾ in. (43.4 m) long, three-span continuous prestressed concrete structure made up of pretensioned decked bulb tee girders. The bridge replaces an old deteriorated timber structure using the existing middle piers. To increase the load capacity of the bridge, the girders were post-tensioned for continuity. The bridge is designed for off-highway logging trucks and overload crossings of large tower and yarder units.

This article describes the preliminary studies, design and economic considerations, advantages of continuity, post-tensioning layout and joint details, construction aspects and post-tensioning operations used to construct this unique bridge.

The South Fork of the Hoh River is located on the Olympic Peninsula, southeast of Forks, Washington (see Fig. 2). This region has the highest average annual rainfall in the continental United States, averaging 140 in. (3556 mm) per year. Storms have been known to bring up to 6 in. (152 mm) of rain in a 24-hour period and this, when combined with melting snow, can cause the river to rise quickly, sending logs and debris rushing downstream. In this constantly moist environment, decay is rapid and omnipresent, thus presenting formidable obstacles to bridge construction and longevity.

The South Fork Hoh River Bridge, also known as the Marsh Bridge, was originally built in 1960 for HS20 loading using log stringers and timber decking over two interior reinforced concrete support piers on footings embedded in rock. Twenty-two years later, the log stringers were decaying, threatening structural integrity.

After the original bridge was built, the

Presents the design-construction highlights of the South Fork Hoh River Bridge — a three-span, precast prestressed concrete structure using decked bulb tee girders. A major feature of this bridge was the efficient use of post-tensioning to create continuity in the structure.

Washington State Department of Natural Resources (DNR) attempted to have bridges built in this area with off-highway logging truck capacities (U54) and with the capacity to carry the load of a fully assembled and equipped, large capacity yarder and tower assembly (Skagit BU199 and T110) (see Fig. 3). This heavier equipment allows more economical logging and access to sawmills without using the highway system. In addition, timber sales accessible only by way of this bridge were anticipated in early 1984.

Preliminary studies indicated that the original concrete pier system was efficiently designed for HS20 loading by Harold Sargent, P.E. However, additional axial loads could only be resisted by the concrete columns and footings if bending moments were reduced. Bending moment reductions were made

Fig. 1. South Fork Hoh River Bridge Replacement.

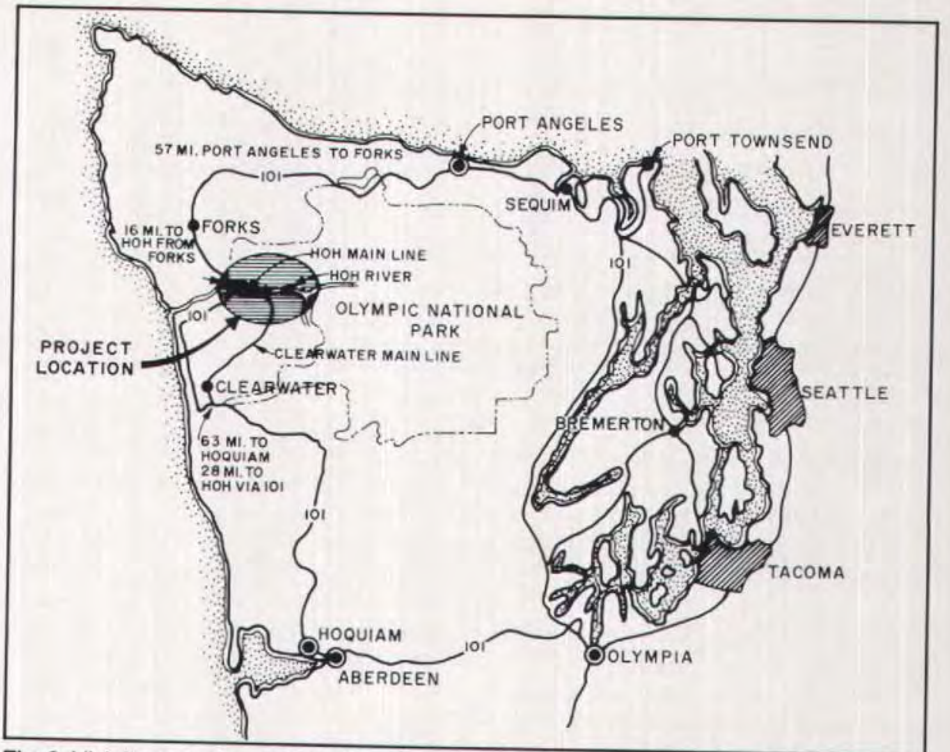


Fig. 2. Vicinity map showing location of bridge project.

possible by eliminating the concrete girder bearing on the pier beam and by locating the exterior girders directly over the center of the columns. A continuous deck system reduced moments from lateral forces on the superstructure while also reducing bending moments on the other axis from the live load effect on one of the simple spans (see Figs. 4 and 5).

Continuity creates additional reaction at the interior supports, so it appeared the desired, heavier loadings were feasible with this system only if continuous steel girders were used. The DNR preferred the use of precast prestressed decked bulb tee girders and wanted to avoid substantial pier modification, even if it meant limiting the new bridge to HS20 requirements. This preference was due to the Department's favorable experience with precast concrete and the relatively low cost and maintenance of the simple span, precast decked bulb tee girders used on other bridges.

Design for Continuity

Since precast prestressed decked bulb tees were desired without pier modification, the bridge appeared to be limited to HS20 truck loadings and a series of three simple spans. The design phase began with this assumption, but proceeded with the idea that U54 load capabilities would be preferred.

When designing the diaphragms for the simple span ends over the existing piers, it was decided that the diaphragm capacity should be increased to carry the total load of the interior decked bulb tee as a raised cross beam. This would eliminate the need for the existing pier beam to carry the load, considerably reducing frame action bending in the columns and pier beam. Continuity would still be required to reduce column bending moments in the other axis, by keeping the loads centered over the columns for the heavier loadings (see Fig. 5). Although the decked bulb tee system selected was heavier than a steel girder bridge, the increase of load to the interior, existing pier supports was substantially reduced, since the girders acted as simple spans carrying their own dead load and were continuous only for live loads.

The interior diaphragms were modified to act as raised cross beams (see Fig. 8) and to carry loads from the center girder. They also act as a connection for continuity of live loads, causing such loads to be centered over the existing columns. This allows a much greater load carrying capacity.

This is due primarily to the connection detail between the diaphragm cross beam and the existing pier which minimizes lateral load transfer from the superstructure to the pier. However, the decked bulb tee girders provide some restraint to the existing piers with regard to severe lateral overloads from potential debris flow.

Post-tensioning of the pretensioned decked bulb tee girders allowed for continuity and enabled the existing interior piers (originally designed for significantly lighter loads) to be used without modification. In addition, the proper combination of pretensioning and post-tensioning in various strand and tendon layout patterns sufficiently controlled the reaction to the interior piers and eliminated the need for widened end blocks. Further, it reduced long term secondary restraint moments and eliminated the need to field weld heavy positive mild steel reinforcement at the interior supports with their relatively narrow, raised diaphragm cross beams.

Achieving continuity with post-tensioning, rather than mild steel reinforcement, also eliminated the need for a cast-in-place concrete top slab for

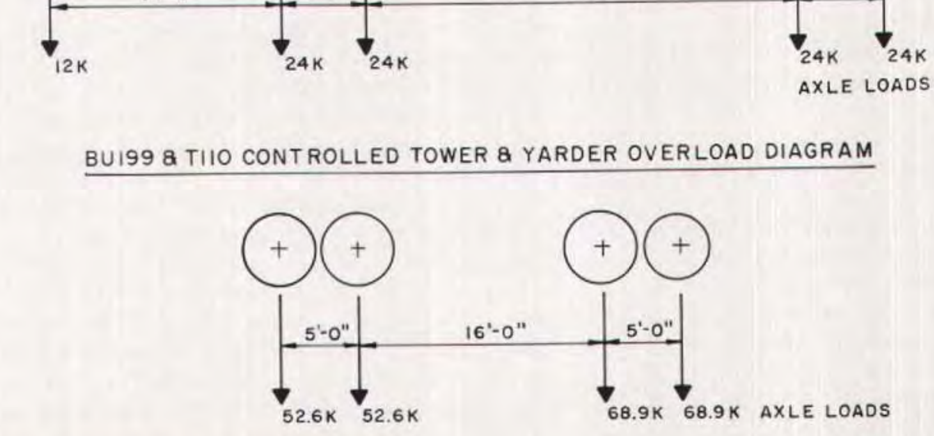
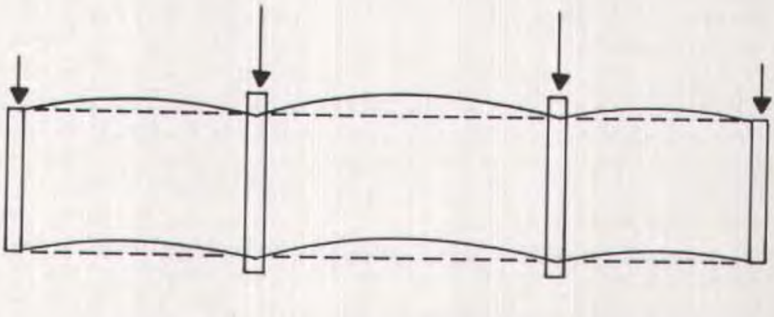


Fig. 3. Bridge design loadings.



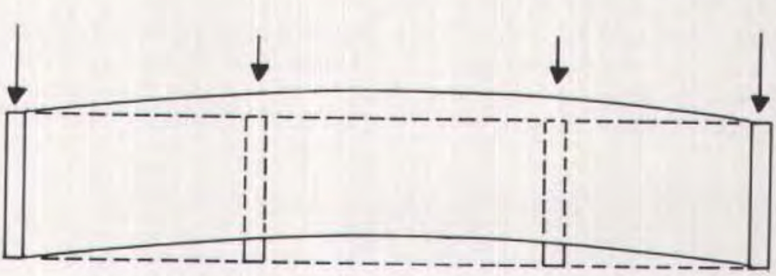


Fig. 4. Lateral force comparison between simple span and continuous bridge systems.

crack protection.

Fig. 6 shows the plan and elevation of the bridge that was finally selected.

Advantages of Continuity

Continuity considerably reduced the lateral loads required to be resisted by the interior piers. By using flexible bearing pads at the interior supports, lateral loads bypass the interior piers and are transferred to the new end abutments where stiffer lateral connections are provided (see Figs. 4 and 8). A simple span system would have produced lateral loads to the piers tributary to half the span length of the girder which they support.

Continuity also provides a better riding surface due to less camber and the elimination of joints at the interior supports. Eliminating these joints can also reduce long term maintenance costs. A continuous system provides for more ultimate strength due to redundant capacities which would be used prior to failure. Economies are also gained by reducing the maximum positive moments at midspan with the formation of negative moments at the supports.

Commonly used methods of creating

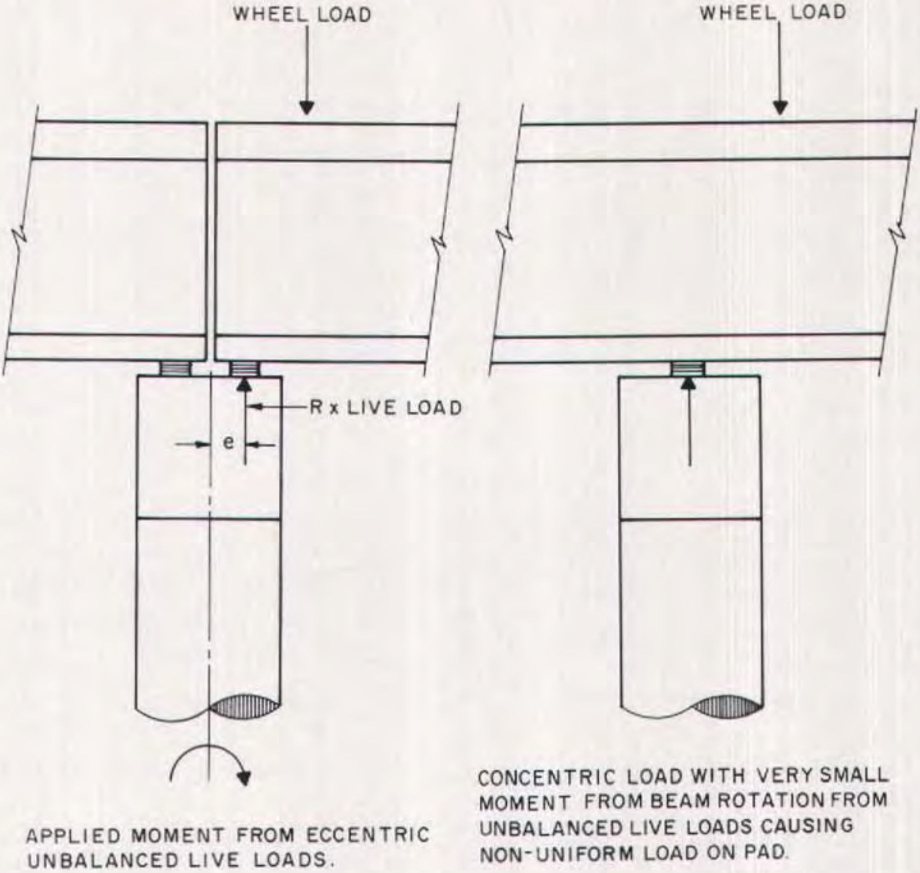


Fig. 5. Moment to interior support from unbalanced loads. Comparison between simple and continuous bridge systems.

continuity for live loads with precast prestressed concrete members include incorporating mild steel reinforcement in the deck slab capable of developing negative moments over the supports. Since this total negative moment is cyclic and is created by live load, fatigue considerations under working stresses often control the design. One possibility is to field weld the negative reinforcement, which would be extended from the decked bulb tees and joined in the cross beam.

This system has been used for making short, prestressed pier sections continuous in dock and wharf construction. Many highway bridges are built using AASHTO or Washington State Department of Transportation (WSDOT) standard I-beam prestressed girders with a cast-in-place concrete deck slab and negative reinforcement placed over the interior supports. This is appropriate only if the entire deck is intended to be cast in place. Due to cracking potential at the supports with prestressed members made continuous by use of mild steel reinforcement, these structures are generally covered with a protective wearing surface of asphalt.

The design of the South Fork Hoh River Bridge as a continuous system for

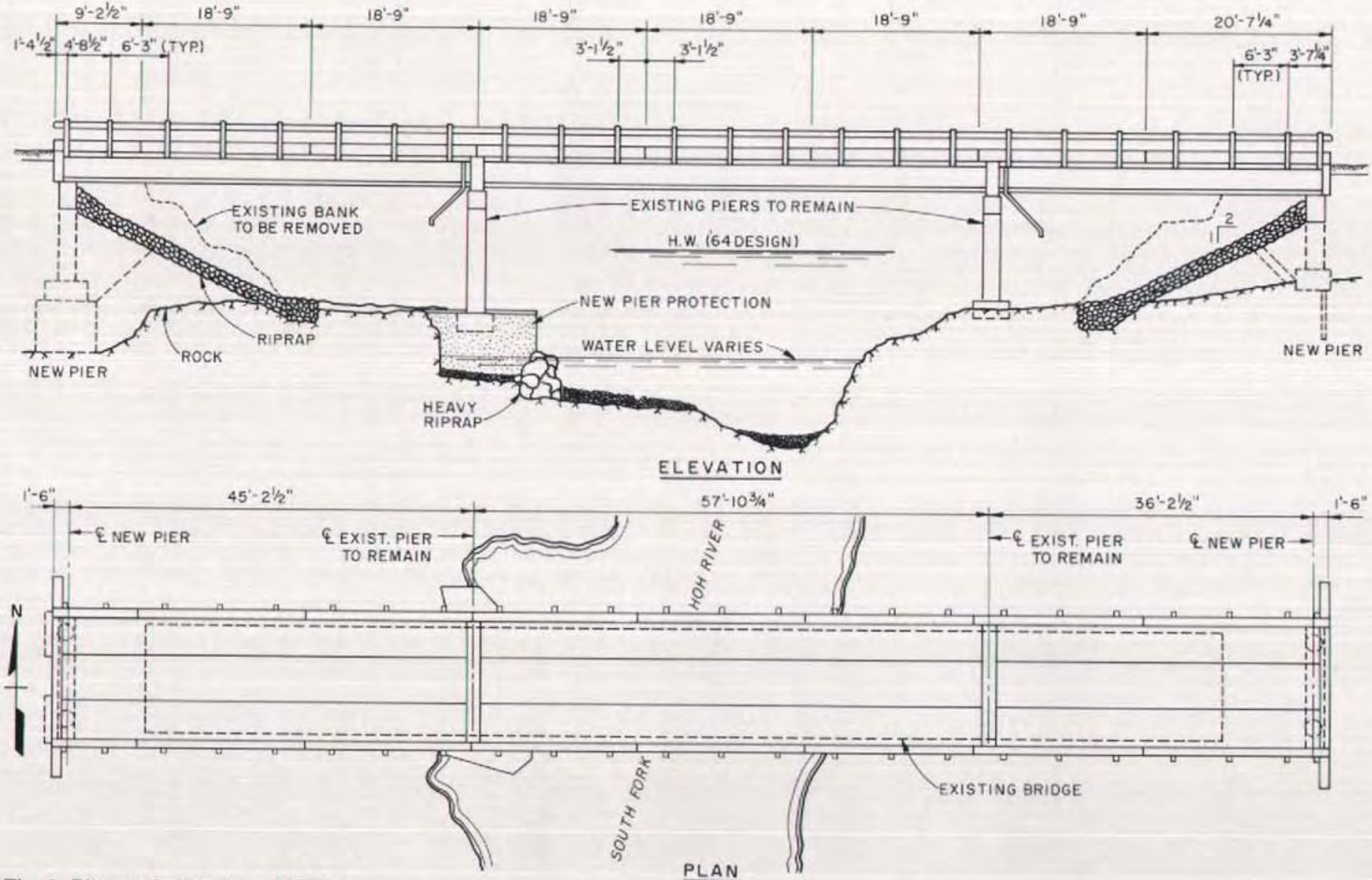


Fig. 6. Plan and elevation of bridge.

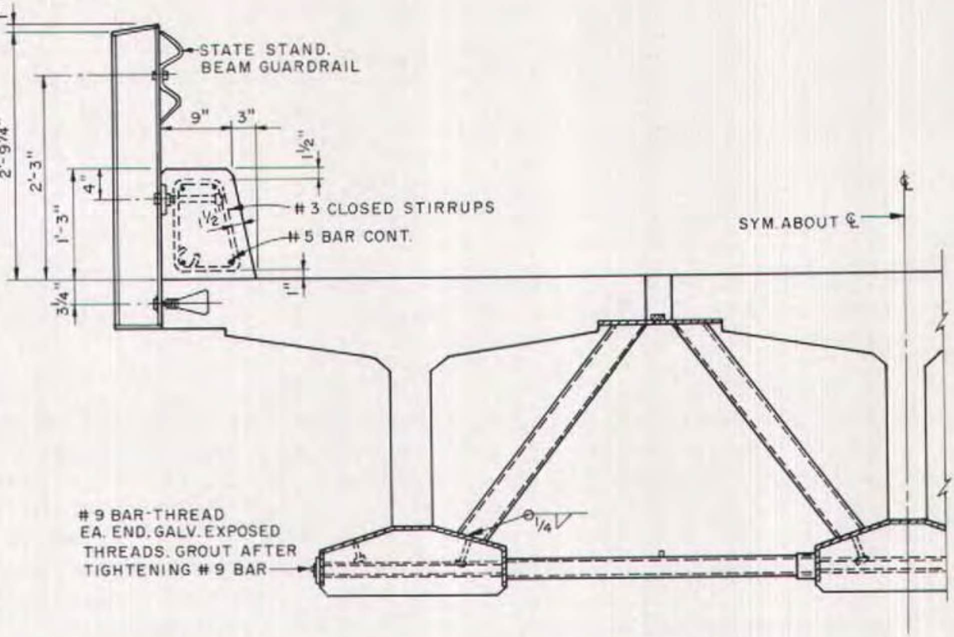


Fig. 7. Section showing diaphragm assembly.

Fig. 8. Raised interior cross beams allowed load to be centered over columns, eliminating moment transfer to the columns from frame action with vertical loads.

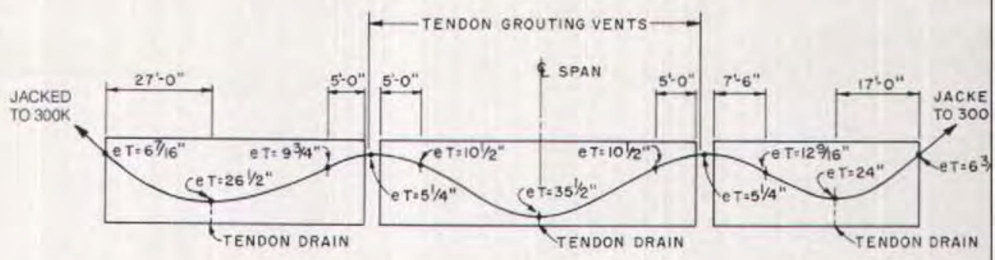


Fig. 9. The selected tendon pattern maintained compression at the top of the exposed slab and cast-in-place cross beams while reducing the load carried by the existing interior piers.

live loads increased the complexity of the analysis over that required for a series of three simple spans. The varying span lengths were such that no specific loading controlled the design. Therefore, U54 lane loading and axle loadings with varying trailer lengths and yarder and tower assembly overload had to be checked for all possible positions on the bridge. A variety of design vehicle configurations and directions controlled the working stress and ultimate strength stresses at different locations along the bridge.

Post-Tensioning for Continuity

Post-tensioning was chosen as the most cost effective method for obtaining continuity with decked bulb tees. Post-tensioning eliminated the need for a cast-in-place deck or a topping slab. With the selected draped tendon pattern (see Fig. 9), post-tensioning reduced the load carried by the interior supports at the existing piers. The cast-in-place slab area at the top of the cross beams over the interior supports is always in compression, even with long term creep, live loads and the controlled overload.

A challenging aspect of the design was to obtain the optimum profiles and strand distribution for the post-tensioning and prestressing layout. The use of #5 40 grade mild steel reinforcement was chosen to resist positive moments at the interior supports since it could be lap spliced in the relatively narrow cross beam, avoiding the expense of field welding. To achieve this, the tendon pattern was adjusted and a high proportion of harped prestressing strand was used to reduce long term positive moment effects from creep (see Fig. 10).

Tension was limited to 3 $\sqrt{f'_c}$ for the lower precompressed tensile zone of the decked bulb tee girders, and no tension was allowed in any portion of the top deck slab under design loads. The tensile stresses from controlled overloads were limited to 6 $\sqrt{f'_c}$ at the lower precompressed tensile zone and $\sqrt{f'_c}$ at the top deck. Tensile stresses were limited to 7.5 $\sqrt{f'_c}$, where the normal steel reinforcement carried the total tension force which occurred at the lower girder flanges with positive moments adjacent to the interior supports.

The practice of using partially unbonded prestressed strands raised questions regarding development lengths, concrete strains and ultimate moment capacities in the relatively short, heavily loaded girders. Since the post-tensioning tendon was placed to produce compression in the deck slab under design loads, the positive ultimate moment strength was reduced in some portions of the girders. The raising of the tendon required high strains to be developed in the bottom fibers of the girders prior to developing the ultimate

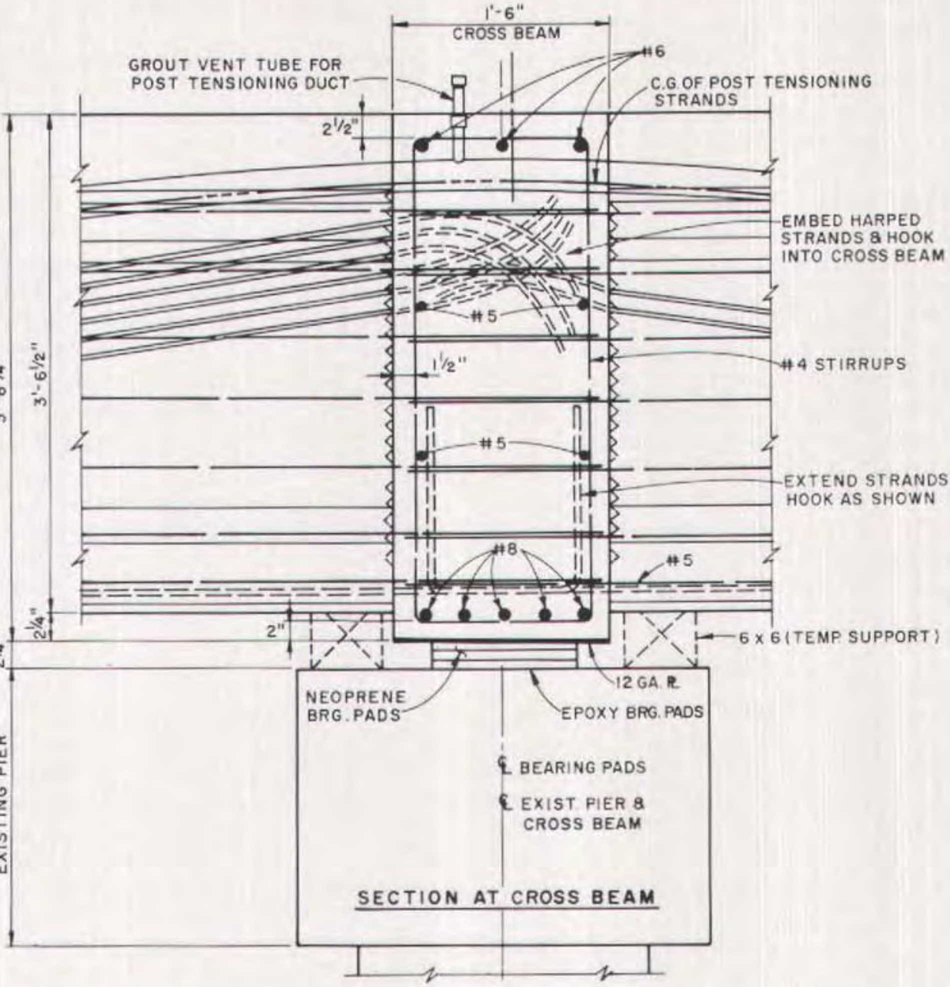


Fig. 10. Positive moment reinforcement at the interior piers consisted of #5 grade 40 bars to allow lap splicing within the narrow cross beam. The high percentage of harped strands reduced long-term positive moments from creep while avoiding debonding of straight strands. More reinforcement at the sides of the diaphragm beam would assist in resisting post-tensioning stresses during erection.

strength of the post-tensioning strands (see Fig. 11).

These high strains also created stresses in the ultimate strength range of the prestressing strands near the bottom of the girders. And, since these strains were within the ultimate strength development length of the debonded prestressing strands [about 14 ft (4.3 m)], there seemed to be a possibility of the strand suddenly losing bond. This would be contrary to the current design practice of assuming strand development effectiveness in proportion to the amount of bonded length.

The high percentage of unbonded to continuous strands (due to the relatively small amount of total prestressed strands) could make such an occurrence significant. Partial debonding of some strands would normally be required to reduce the top tensile stresses under working loads and long term positive moments from creep effects.

The harped strands produced better results and also eliminated the longer development length requirements of partially unbonded strands in locations of high bottom fiber strains under ultimate load conditions. Thus, the need for mild steel reinforcement at interior portions of the spans for assurance of ultimate moment development was eliminated. However, harping the strands did require reduction of the stirrups to #3 to avoid clearance problems with the shear reinforcement and to allow the post-tensioning tendons to fit in a 5 in. (127 mm) web (see Fig. 12).

Elimination of End Blocks

Attachment of the decked bulb tees to the interior cross beam supports was accomplished by shear friction reinforcement. However, the surface area requirements necessitated using a portion of the flange area extending from a 45-deg angle at the web intersection. This is a liberal approach by code requirements, but one that is commonly used in the prestressed concrete industry for hollow-core slab shear capacity analysis. WSDOT standard shear keys were used in these flange areas to ensure adequate surface roughness (see Fig. 13).

This analysis also supported the elimination of end blocks at the interior supports. In addition, by raising the post-tensioning termination anchorage

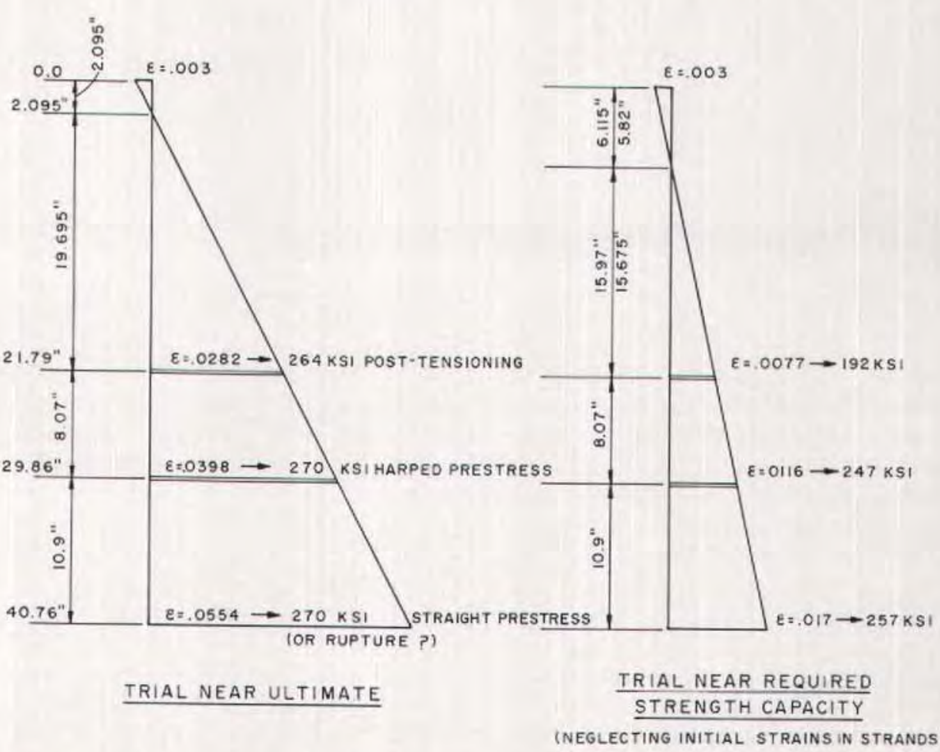


Fig. 11. High strain in the few lower prestressing strands at ultimate moment caused concern for maintaining development length and avoiding debonding.

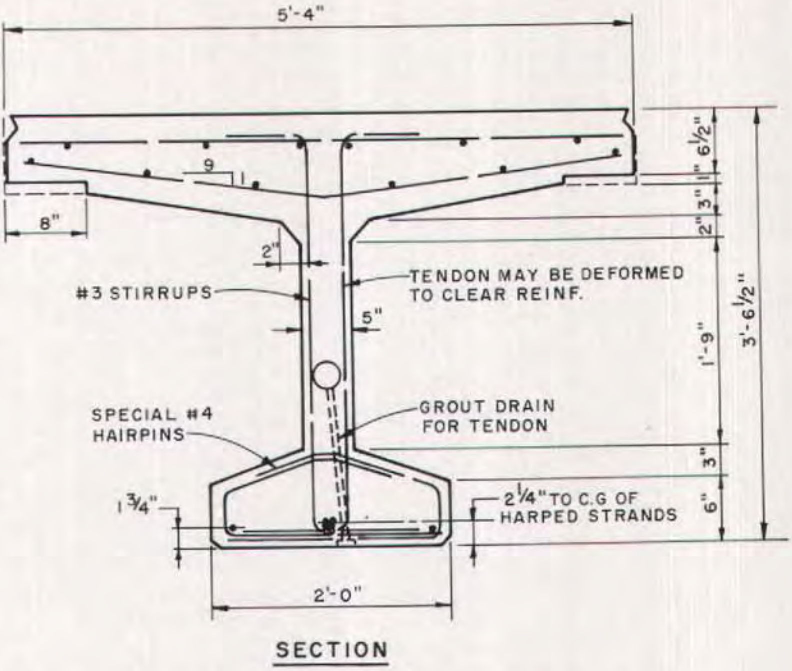


Fig. 12. The 5 in. (127 mm) web required the use of smaller #3 stirrups to provide clearances for harped strands and post-tensioning.

into the top flange above the optimum flexural location, use of end blocks at the exterior ends was also avoided (see Figs. 14 and 15). This resulted in substantial cost savings.

Construction

Construction of the bridge went very smoothly. As low bidders on the project, Rognlins Inc. incorporated some innovative ideas to reduce construction costs, including dismantling and removing the existing bridge so that the new bridge could be put in its place. Also, since this bridge served as the only access to the opposite side of the river, the contractor moved the old center span to some rock outcroppings downstream for use as a temporary bridge during construction. This enabled more efficient phasing of construction and installation of the new girders than if a portion of the existing bridge had remained (see Fig. 15).

The new prestressed girders did not have much capacity for construction loads since most of their live load flexural strength would be provided by post-tensioning. Therefore, river transit was used during all phases of construction while the old bridge served as a work platform to reduce the equipment required at this remote worksite. And, even though late spring rains, coupled with snow runoff, caused the river level to reach the improvised bridge, it withstood the high water.

Backhoe investigation and initial excavation uncovered rock which was designed to support the south footing. However, an engineering geologist for the State Department of Transportation determined that this rock was not the bedrock for that side of the river and

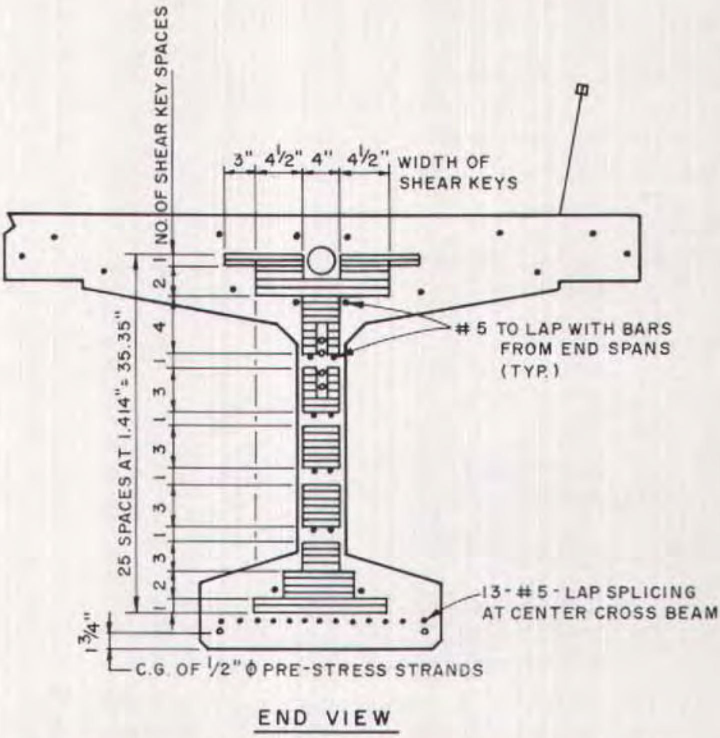


Fig. 13. End blocks were avoided at the interior supports by using a 45-deg angle into the heavier flange areas to keep shear friction stresses below 800 psi (5.5 MPa). Shear keys are incorporated in the area to maintain surface roughness.

would not allow the footing to be placed. Further excavation showed that his analysis was correct, and a large boulder was eventually uncovered and removed. Lean concrete was then used for fill and the fabricated pier reinforcement was placed. This procedure added minimal additional expenses since coring for rock anchors at that end of the bridge was eliminated.

Post-Tensioning Operation

One easily overlooked variable in the design phase is consideration of the stresses generated by typical construction sequences. A case in point is the effect of jacking stresses generated during the post-tensioning sequence. Jacking was required at both ends of the tendons, but full jacking at one end is done prior to moving to the next girder and, eventually, to the opposite tendon end (see Fig. 16).

Potential overstress and cracking of the interior cross beams acting as continuity connections would occur due to axial shortening of the center midspan girder under post-tensioning (see Fig. 17). Therefore, the diaphragm connectors at the flange edges of adjacent girders were welded prior to post-tensioning, contrary to common practice, to avoid overstress of the cross beams by shoring the post-tensioning forces with adjacent girders.

Analysis of initial and residual stres-

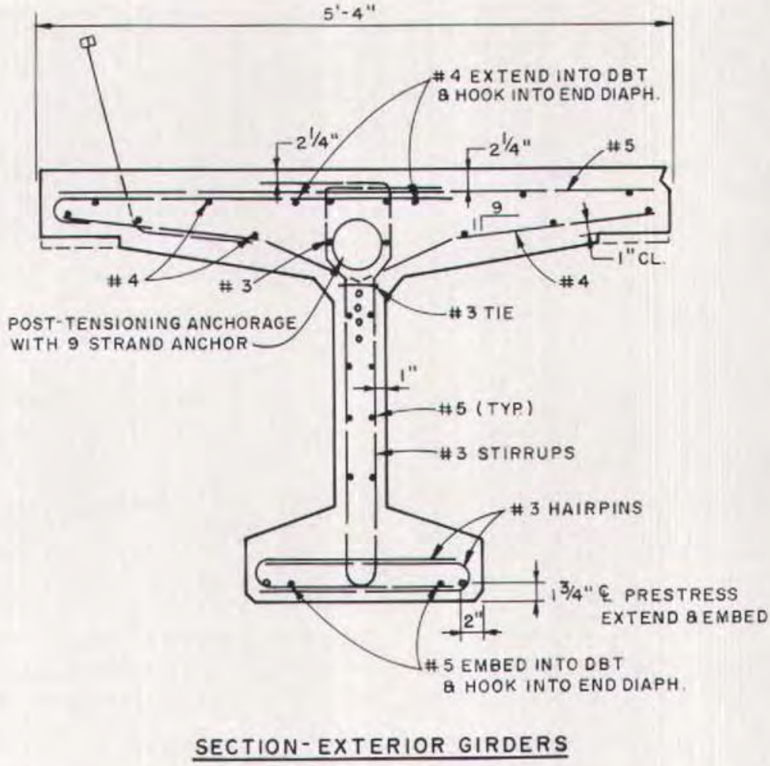


Fig. 14. The termination anchorage at the post-tensioning was high, eliminating the need for end blocks at the exterior ends and to assist in reducing load transfer to the existing interior piers.

ses in the diaphragm clips, as well as transfer of post-tensioning forces to the adjacent girders during the jacking operation indicated this to be the best solution. Additional reinforcement to resist the vertical axis bending at the cross beams may have been a better alternative.

The post-tensioning operation was performed by Concrete Technology Corporation, the supplier of the precast girders. The company requested the use of ten strands, in place of the nine specified per girder line, in the post-tensioning operation to reduce strand stress. This resulted in a slight increase in the over-all jacking force to obtain the design stresses after losses.

Each girder line is jacked to 300 kips (1334 kn) from each end and seating losses were a maximum of ½ in. (13 mm). The post-tensioning sequence was observed by engineers from P.C. Bridges, Ltd. of Tokyo, Japan, guests of Concrete Technology (see Fig. 18). Next, the post-tensioning ducts and shear keys were grouted and the precast curb set prior to guardrail installation. Backfilling of the abutments completed the project.

Aesthetics

In situations where cost control and standard practice dictate bridge geometry and form, it is still possible for the engineer to use design details to enhance appearance. On this project, for

Fig. 15. The center span of the old timber bridge was used as a temporary construction bridge. The high termination anchorage of the post-tensioning is shown. Duwayne Hankins, project engineer from Concrete Technology, directs the post-tensioning operation.

Fig. 16. Jacking was performed on one end of each girder individually.

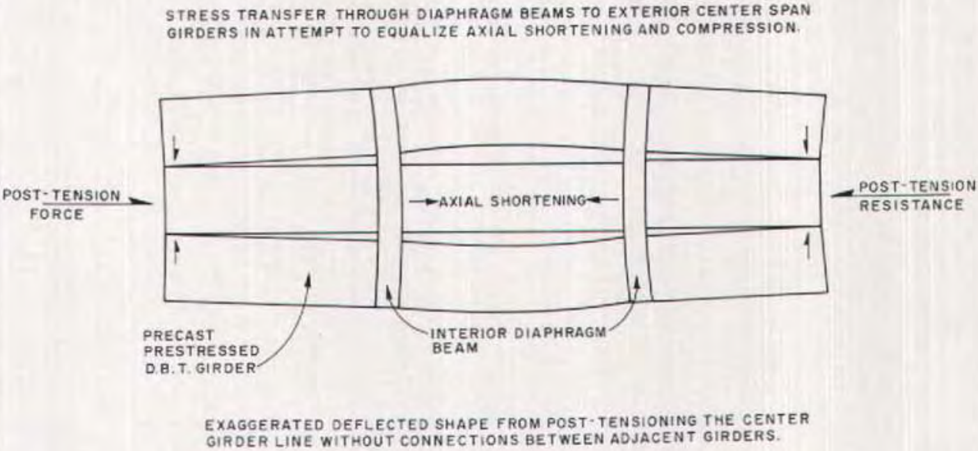


Fig. 17. Flexural and shear stresses are created at the interior diaphragm beams as post-tensioning of the center girder tends to distribute to all three girders of the interior bay.

Fig. 18. Engineers from P. C. Bridge, Ltd. of Japan observe the post-tensioning operations.

Fig. 19. Galvanized steel pipe diaphragms were used at maximum positive moment locations.

Fig. 20. Tapering the ends of the raised cross beams and providing sloped stiffeners at the top of the guardrail posts do their part in enhancing the aesthetics of the bridge.

example, center diaphragms of galvanized steel pipe were specified rather than the more typical angle braces (Figs. 7 and 19) and the raised cross beams were tapered at the ends for a better transition to the narrower interior piers (Fig. 20).

In addition, stiffener plates were installed diagonally on top of the DNR standard guardrail posts for appearance, as well as to utilize the total vertical axis to resist AASHTO longitudinal rail forces (see Fig. 20). The cost of these minor modifications was insignificant and the appearance was much more aesthetically pleasing, which was important due to the proximity of a public campground.

Costs

DNR project cost estimate prior to design, based on lighter HS-20 capacity $200,000

Engineer's estimate for the bridge, as designed for U54 off-highway logging trucks and the large tower and yarder units, including reinforcement and scour protection at the south pier requiring over 20 cubic yards (15.3 m^3) of reinforced concrete placed below water level $180,000

Bid price $157,000

Final contract price including modifications for foundation change orders $160,259

Total cost including tax, advertising, and testing $173,700

Concluding Remarks

Since completion, the bridge has performed very well with off-highway log truck traffic and overload crossings of large tower and yarder units without signs of any distress. There are definite advantages in using post-tensioned precast members for continuity, even on short multispan bridges. Indeed, this method could be used effectively to extend span capacities of standard precast members for longer multispan bridges.

Credits

Design Engineer: Craig R. Owen, Consulting Structural Engineer, Port Angeles, Washington.

Draftsman: William Dawson.

Geotechnical Agency: State of Washington Department of Transportation, Materials Laboratory, Olympia, Washington.

Geotechnical Exploration: Department of Natural Resources.

Engineering Geologist: Tom Zimmerman, Washington State Department of Transportation.

Precast Concrete Manufacturer and Post-Tensioning: Concrete Technology, Tacoma, Washington.

Owner and Job Coordinator: Washington State Department of Natural Resources, Olympia and Forks, Washington.

Contractor: Rognlin's, Inc., Aberdeen, Washington.

Photographer: Bill Rinehart.

* * *

NOTE: Discussion of this paper is invited. Please submit your comments to PCI Headquarters by September 1, 1987.