WASHINGTON STATE PRECAST SINGLE SPAN BRIDGE RECORD – CHIEF JOSEPH DAM BRIDGE REPLACEMENT PROJECT

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ABSTRACT

The existing two-lane 1950's built 94.2 m (309 feet) long bridge, consisting of 39.6-m (130-foot) historic timber truss and five approach spans, was deemed structurally deficient in 2011. In 2014-16 a concrete replacement was designed and constructed. The new record breaking single span bridge is comprised of five 72.6-m (238-foot) long 2.5 m (100 in.) deep post-tensioned deck-composite girders. The substructure is comprised of two concrete H-pile supported abutments. Each girder consists of three precast segments, shipped more than 370 km (230 miles) from the fabricator to site. Stirrups were pre-bent to make clearance. The girders were carried through the existing truss span erected on false work and installed, and then post-tensioned together after closure pours attaining the required strength but before deck slab being formed. High-performance concrete is used for the precast girder segments. Four tendons per girder are comprised of seventy-six 15-mm-diameter (0.6 in.) low relaxation 1862-MPa (270-ksi) strands. Challenges included environmental, geotechnical, life cycle cost, demolition and constructability.

Keywords: Post-tensioning Splice, Precast, Concrete, Single Span, Girder Bridge, Construction

INTRODUCTION

The existing 309-foot long Chief Joseph Dam Bridge carried traffic on Pearl Hill Rd NE and spans Foster Creek. This bridge was originally built in the 1950's and is comprised of a single, 39.6 m (130-foot) long and 6.1 m (20 feet) deep, timber Howe Truss and five timber girder approach spans. It is registered as a National Historic Place because of the rarity of a timber truss of this structural form, size, and age. This aged bridge had structural deficiencies and was replaced with a new precast concrete girder bridge.

Foster Creek is the last creek along Columbia River that reportedly supports wild salmon and steelhead spawning because no fish ladder is provided at the nearby Chief Joseph Dam. Due to the environmentally fragile nature of the Foster Creek, no interruption was allowed between the two existing truss piers during the construction.

The new bridge carries two lanes of traffic and was built on the same alignment as the existing one (Fig. 1). It is a 73.2 m (240 feet) long (back of pavement seat to back of pavement seat) and 9.8 m (32 feet) wide single span post tensioning spliced concrete girder bridge. There is a 7.6 m (25 feet) long approach slab on each end. Figure 2 shows the bridge elevation view. The total deck area is 866 m^2 (9,280 ft^2) including the approach slabs.

The total project cost was \$3,880,000, and the bridge portion cost \$3,440,000, including the removal of existing bridge, resulting in a unit cost of \$3,972/m^2 (\$371/ft^2). The high unit cost is due to the construction challenges. For instance, the banks of the deep ravine are significantly steep (close to 1:1 at some locations). To access the site, a pile supported work bridge was constructed next to the existing bridge. The other challenge was the existence of cobbles and boulders underground, which obstructed pile driving and resulted in over excavation to remove obstructions during pile driving.

Fig. 1 A View of Chief Joseph Dam Bridge, Douglas County, WA

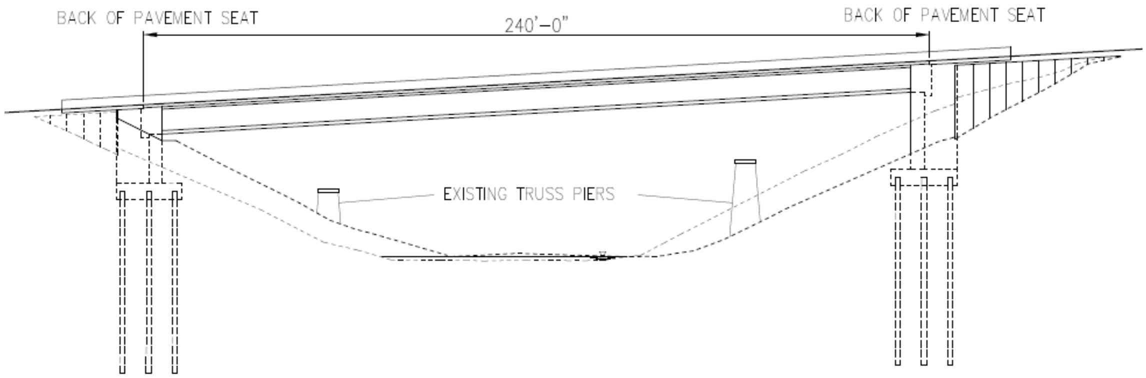

Fig. 2 Chief Joseph Dam Bridge Elevation View (Note: 1 in. = 25.4 mm; 1 ft = 12 in.)

The design was in accordance with the AASHTO LRFD Bridge Design Specifications, 6^{th} Edition, 2012[1] and AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2^{nd} Edition, 2011[2]. The Bridge Design Manual[3] developed by Washington State Department of Transportation (WSDOT) was extensively referenced in the design.

The substructure of the new bridge is composed of two abutments which are supported by H piles. The abutments are deep due to the steep slope of the ravine and the bridge length limitation. The deeper one is around 11.3 m (37 feet) from grade line to bottom of pile cap. A total of 21 HP16 piles are used under a 5-ft thick cap beneath the deeper abutment. A total of 15 piles are used beneath the other abutment. Structural earth walls are used behind the abutment wing wall to retain approach fills.

The bridge is framed by five girder lines. Each girder line consists of three precast pre-tensioned segments, which were erected on the false work, and post-tensioned together after attaining the required strength at the closure pours and before forming the deck slab. All girder segments have a 2.5-m deep (100 in.) WSDOT WF100PTG cross section. The length of each end segment is 15 m (49 feet), and the length of each middle segment is 41.5 m (136 feet). A total of ten end segments and five middle segments were used. The span length is 71 m (233 feet) from center of bearing to center of bearing. The long span minimized the impact to the creek and helped protect the fish habitat. This span length makes this structure the longest precast concrete girder single-span bridge in Washington State up to the date when its construction was completed.

The project design was completed in November of 2014. The contract was awarded in March of 2015, and construction started in April of 2015. The new bridge was open to traffic on June 27^{th} 2016.

In 2016, the Washington State Aggregate and Concrete Association named the bridge 2016 Excellence in Concrete Construction in the Category of Bridges.

TYPE, SIZE & LOCATION (TS&L) STUDY

The restrictive environmental requirement excluded any piers between existing truss piers. The span length of the existing truss was 39.6 m (130 feet). The first type of structural system that was considered was a three-span continuous girder bridge. In this system, the middle span would have been at least 51.8 m (170 ft) long to clear the footing of the existing piers. The side spans would have needed to be around 21.3 m (70 ft). This system needed two more intermediate piers and longer overall superstructures. Given the difficult soil conditions, it was believed that the three-span system would result in a higher cost compared to the selected system. Besides, shipping 170-ft precast concrete girders to this site would be very challenging if not impossible. The second alternative was a single span steel plate girder bridge. It was ruled out because the owner had a big concern about the potential high maintenance effort in the future.

After investigating all alternatives, the designers selected a WSDOT W100PTG post tensioning spliced girder superstructure system. The main reasons are that this system is able to make a long single span bridge, which eliminated extra substructures, and the existing piers could be used as temporary supports at the closure joints. The spacing between the 5 girder lines is 1.9 m (6.25 ft). Girders are topped with a 27.5 MPa (4 ksi) 190-mm-thick (7.5 in.) cast-in-place concrete slab. Clear cover is 63 mm (2.5 in.) thick at top and 25 mm (1 in.) at bottom. Top and bottom reinforcement in longitudinal direction of bridge deck were staggered to allow better flow of concrete between the reinforcing bars. High-performance concrete (HPC) with a 28-day design compressive strength of 74.2 MPa (10.8 ksi) was used for the precast girder segments. The bridge typical section and WF100PTG girder section are shown in Figures 3 and 4 respectively.

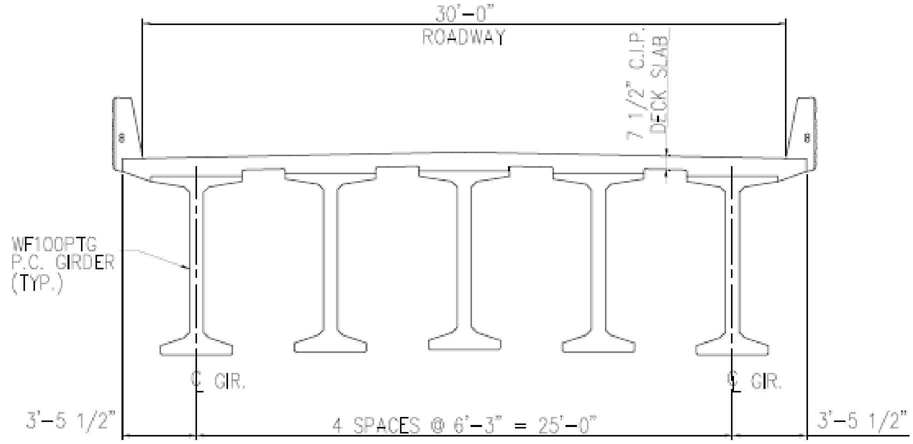

Fig. 3 Chief Joseph Dam Bridge Typical Section (Note: 1 in. = 25.4 mm; 1 ft = 12 in.)

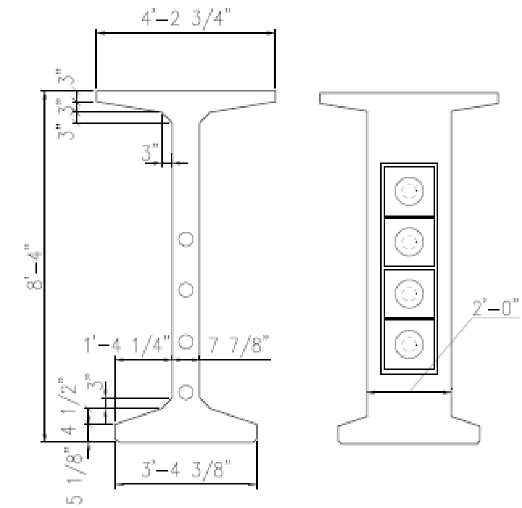

Fig. 4 Girder Section (Note: 1 in. = 25.4 mm; 1 ft = 12 in.)

All post tensioning was applied before the deck was placed so that the deck may be removed or extensively repaired at any time without potentially overstressing girders. The other reason for doing so is the extra weight from concrete deck may overload the existing truss piers which were used as temporary supports at closure joints.

DESIGN AND ANALYSIS

The bridge was designed by KPFF Consulting Engineers for HL-93 live load and a 1000-year return period earthquake, with an acceleration factor of 0.17.

Given the relatively simple bridge layout and construction sequence, instead of using commercial computer programs, designers at KPFF Consulting Engineers developed a spreadsheet to complete the design. This spreadsheet tracked girder section properties and checked all design criteria at 1/10 span points in each construction stage. The design criteria that have been checked included compressive stress and tensile stress in girder concrete, tensile stress in strands, flexural and shear resistance, and girder deflections. Zero tension in concrete at all time was required in this design according to WSDOT BDM[3]. The construction stages that have been evaluated included prestressing release at casting yard, post tensioning assembly, deck concrete placement, traffic barrier placement, and final service stage. Stress losses in strands due to concrete creep and shrinkage are time dependant. The equations in Chapter AASHTO LRFD Bridge Design Specifications[1] Articles 5.4.2.3.2 and 5.4.2.3.3 were used in the spreadsheet to calculate the creep and shrinkage of concrete at each stage, and those in Article 5.9.5.4.2 were used for stress loss due to creep and shrinkage.

The pretensioning force was designed to resist the girder weight and ensure zero tension in concrete at bottom of girders during shipping and installation. Thirty two permanent 15-mm-diameter (0.6 in.) low relaxation Grade 1862 MPa (270 ksi) pretensioning strands were used in each middle segment, and eight pretensioning strands were used in each end segment. Besides the permanent strands, two temporary strands were used in each middle segment to control the tensile stress in concrete at top of girder section especially at the ends of the girder segments. A 5cm x 5cm x 6.3" deep (2" x 2" x 2.5" deep) recess at top of girder was provided for detensioning of each strand. The recesses were patched with grout thereafter.

Four tendons of total seventy-six 15-mm-diameter (0.6 in.) strands were used in each girder line. Because the closure joints are relatively far from abutments, the stress level is high over there due to high flexural moment demand. Therefore, instead of a conventional single parabolic profile, a tendon profile which consists of three different segments was employed. In this profile, two parabolic curves in the end girder segments tangent into one straight in the middle segment. It made the tendons locate at the bottom at closure joints and most effectively reduced compressive stress at top of girder and tensile stress at bottom. The conventional single parabolic profile will end up with a compressive stress of 33.4 Mpa (4.84 ksi) at top of girder for Service-I limit state and a tension stress of 3.04 Mpa (0.44 ksi) at bottom of girder for Service-III limit states. Both of them are beyond the design limitations which are 31.0 Mpa (4.50 ksi) compression and zero tension respectively. By using the three-segment tendon profile, both the compression stress and the tension stress were reduced by around 7.87 Mpa (1.14 Ksi). The post tensioning profile is shown in Figure 5. The analysis showed that the minimum strength of the cast-in-place concrete at closure joints at the time of post tensioning and at 28 days should be 41.4 MPa (6 ksi) and 51.7 MPa (7.5 ksi), respectively.

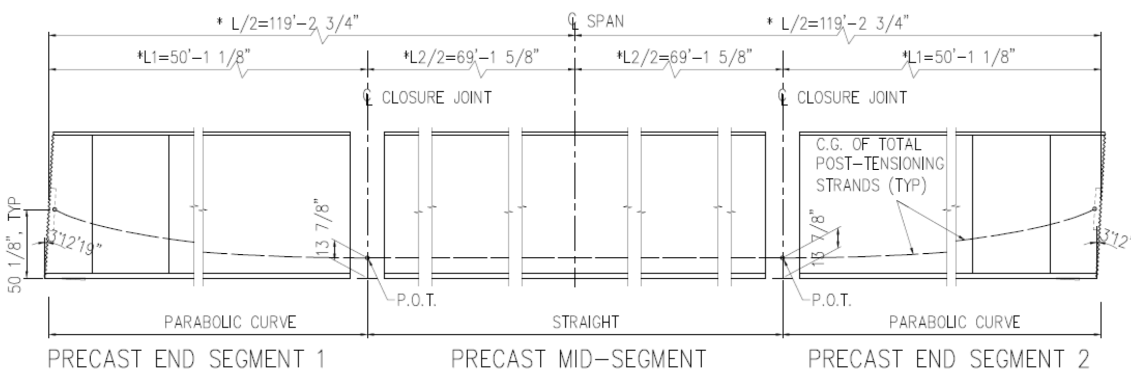

Fig. 5 Post-tensioning Profile (Note: 1 in. = 25.4 mm; 1 ft = 12 in.)

Because of the large span length, girder vertical deflection due to self weight and superimposed dead load is estimated to be around 381 mm (15 in.) which is significantly high. To ensure a smooth top-of-road profile with the least haunch concrete (the concrete directly above girder flanges) and no sag in the girder soffit profile, the elevations of the temporary supports at closure pours are raised by around 127 mm (5 in.) from the intersection points with the straight line connecting abutment supports at two ends. A final camber of 76 mm (3 in.) in the girder soffit profile was achieved. The uplifts at the temporary supports after post tensioning application were checked to ensure the separation of girder from the supports.

GIRDER SHIPPING AND HANDLING

The distance from the precasting yard to the project site was more than 370 km (230 miles). The middle segment weighed more than 86 ton force (190 kips), and the girder section is 3.0 m high (9'-9"), including the length of stirrups extruding from top of the girder. To clear the lowest bridge along the hauling route, the hauler required girder stirrups being pre-bent and only 127-mm (5 in.) extension was allowed above top of flange (Figure 4).

WSDOT requires the extended hook of the stirrups to be within the slab core (the hook shall terminate above bottom mat of deck slab reinforcing). Given a variable haunch dimension, the 127-mm (5 in.) constant extension length was not able to satisfy this requirement, and the bars would need to be straightened and re-bent at most locations. It raised the concern of strength reduction of the rebars. A detail called hat bars was selected to remove this need. It is shown in Figure 6. The hat bar is labeled as "H1". Two additional longitudinal hanger bars, labeled as "H2", were added at the extended hook of pre-bent bars. The test showed that the hat bar design was sufficient to develop the design yield strength of the stirrups [4].

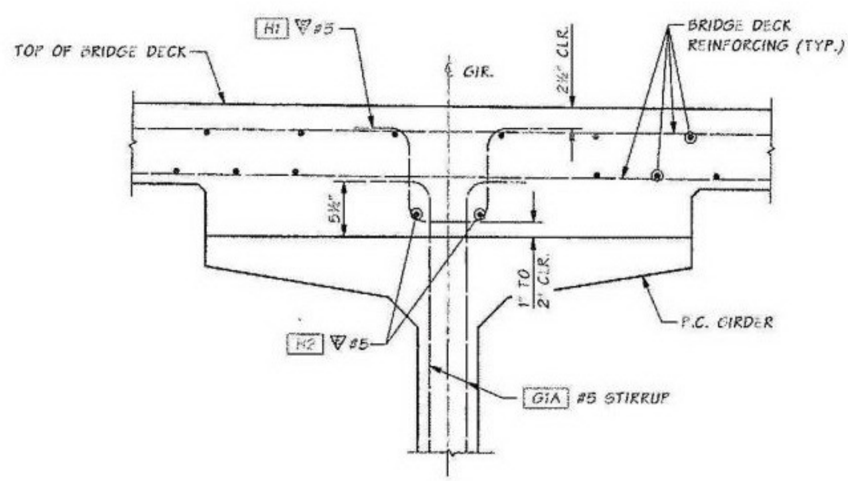

Fig. 6 Hat Bar Detail (Note: 1 in. = 25.4 mm; 1 ft = 12 in.)

The factors of safety against cracking and failure due to shipping and handling were found to be adequate for all girder segments.

BRIDGE CONSTRUCTION

Chief Joseph Dam Bridge Replacement construction started in May of 2015. Because of the low temperature in winter, construction was put on hold after the deck slab was poured and cured in the end of 2015. The contractor remobilized in April of 2016 and finished the remaining construction in June of 2016.

The steep slopes on both sides of the ravine and restrictive environmental requirements brought many challenges to the construction. The biggest challenges include truss removal and girder erection. Because the existing timber truss is registered as a National Historic

Place, the owner of the bridge wanted to salvage the whole truss for exhibition purpose at the early stage of the project. Although it was determined that salvaging the whole structure was not necessary later on, the construction sequence was still designed to address the original design goal of salvaging the truss.

Figure 7 illustrates the whole construction sequence.

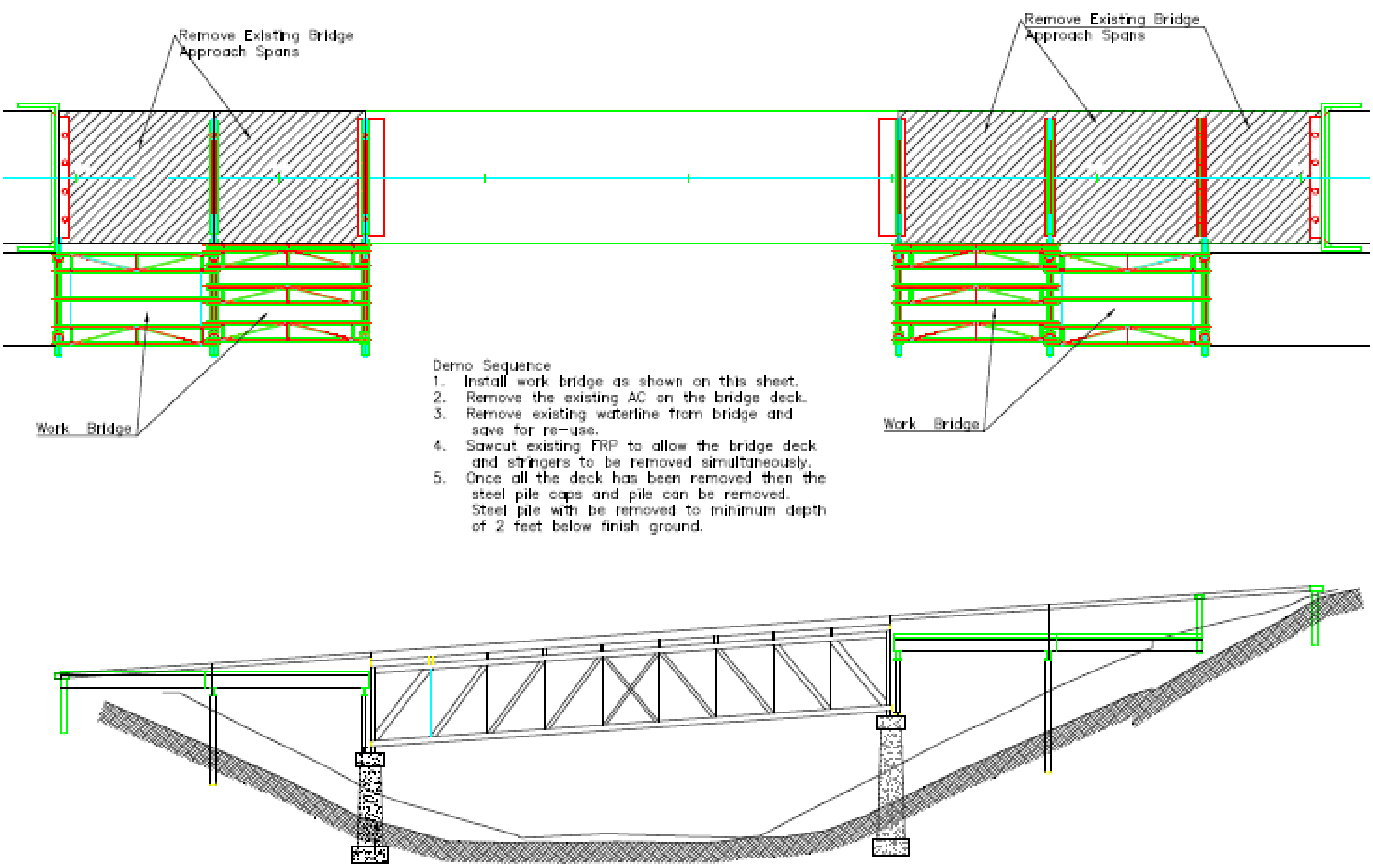

Fig. 7 Bridge Construction Sequence – Stage 1

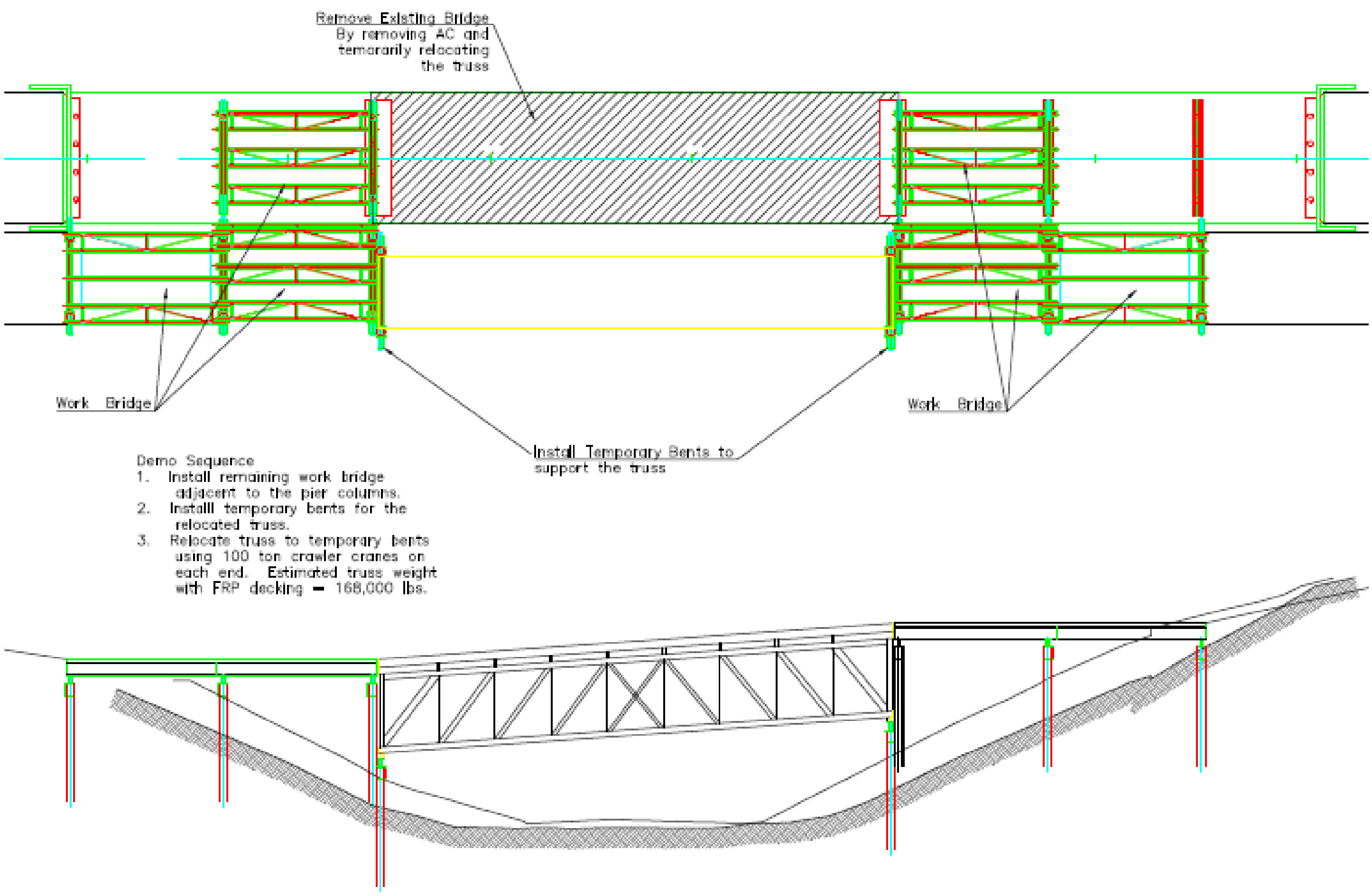

Fig. 7 Bridge Construction Sequence – Stage 2

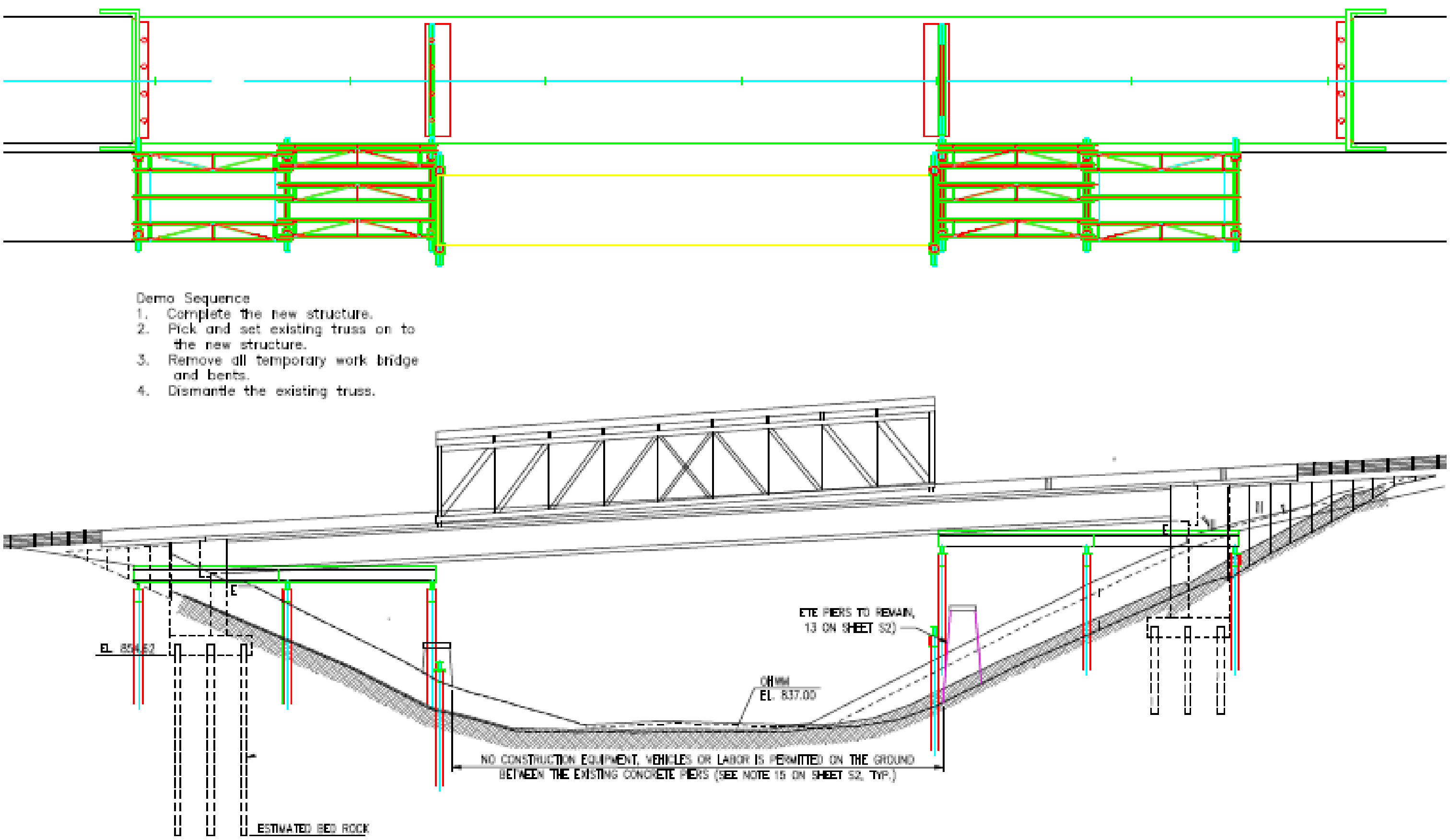

Fig. 7 Bridge Construction Sequence – Stage 3

First, the existing approach spans were removed, excavation for new abutments initiated and shoring was installed. A pile supported work bridge was constructed next to the existing alignment concurrently for construction access (Fig. 8). The work bridge stopped at the ends of existing truss span at both sides, and a finger structure, which was a part of the work bridge and extruded into the existing bridge alignment, was constructed at each end (Fig. 9). The finger structures were partially supported on the existing truss piers. They served not only as the temporary supports to the girder segments during girder erection but also were the construction access to move the existing truss and install new girders. After the abutment concrete was poured and cured, the existing truss was moved to the work bridge alignment by two cranes to make a complete access from both ends of the bridge (Fig. 10). The innovation of this construction sequence was the reusing the existing truss as part of the work bridge. The structural resistance of the truss was checked against the construction loads, which included the weight of girder and hauling truck. The latest inspection report of the existing bridge was referenced to get reasonable strength reduction factor for each structural element. After the existing truss was moved and settled on the temporary piers, the girder segments were installed (Fig. 11). A gap of 0.61 m (2 feet) was left between precast concrete girder segment ends for the closure joint girder splice. Temporary braces and cables were provided during girder erection and before diaphragms were placed (Fig. 12).

Fig. 8 Work Bridge Installation

Fig. 9 Finger Work Bridge Installation

Fig. 10 View from Bottom after Truss Movement

Fig. 11 Girder Installation

Fig. 12 Temporary Braces and Cables for Girder Stabilization

Once the segment closure joints were formed, and concrete was placed and had attained the required strength, the post-tensioning was applied. The post-tensioning force was controlled by gauge readings and verified by strand elongation (Fig. 13). The girders lifted off from the temporary support seats approximately 25 mm (1 in.) after post-tensioning was applied, which matched the theoretical calculations. The tendon ducts received grouting the day after post-tensioning was applied. The grouting was used for corrosion protection and to achieve a reliable bonding between strands and girder.

Intermediate and end diaphragms were placed 2 days after grouting. After temporary supports and bracing were removed, the bridge deck slab was placed. It was mid November when the deck slab concrete was poured, and the local low temperature had dropped below 2°C (35°F). Concrete strengths can be adversely affected by the low temperature. These affects can range from durability issues to low compressive strength neither of which is desirable. Based on the requirement of WSDOT Standard Specification[5], to achieve adequate curing, the temperature of the concrete shall be maintained above 10°C (50°F) during the entire curing period or 7 days, whichever is greater. The concrete temperature shall not be allowed to fall below 2°C (35°F) during this time. The protecting measures included the use of blankets placed over the concrete, enclosure on bottom, and use of space heaters below the deck (Fig. 14). Concrete temperature was monitored during the whole process.

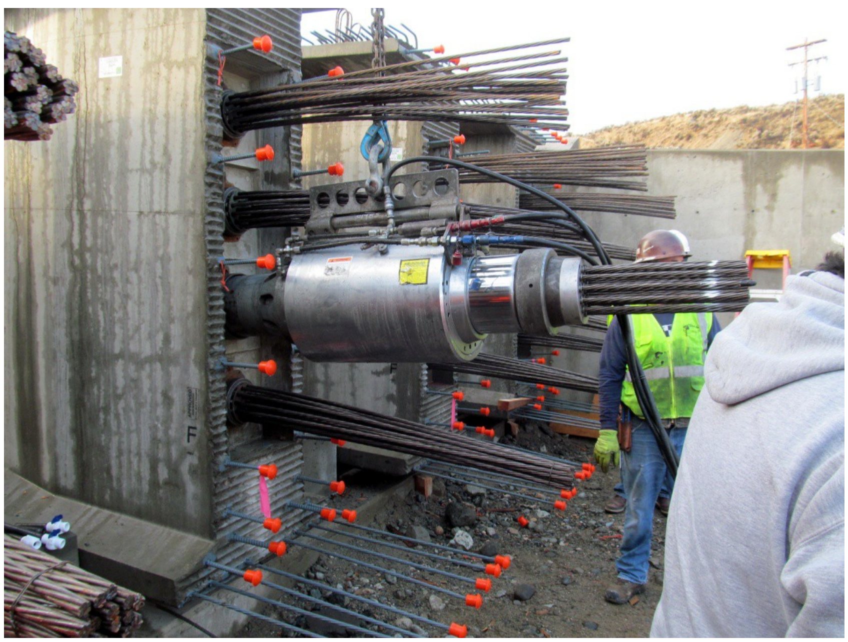

Fig. 13 Post-tensioning Application

Fig. 14 Insulating Blanket on Curing Deck Concrete

After the deck slab was placed, the existing truss was lifted by cranes up to top of the new bridge deck, and the project was put on hold due to the extreme cold weather (Fig. 15). The traffic barriers, approach slabs and retaining walls were installed in 2016. The bridge was opened to traffic in June 2016.

Fig. 15 Project on Hold

This bridge generated a span length record of structures of similar type and has already become a pride of the community.

SUMMARY

Normally, precast girders can be fabricated and transported in lengths up to 52 m (170 ft) and weights of up to 90 ton force (200 kips). The spliced girder method of construction is an efficient way to extend the span range of precast concrete girders. Precast girder segments with manageable weight and length can be transported to the site and then either spliced and post-tensioned on the ground and installed, or erected on temporary supports in their final position and post-tensioned together. This method has extended the practical use of precast girders to span lengths of 71 m (233 ft) or more. In certain length range, longer span length leads to fewer piers that usually result in lower overall cost, especially where soil conditions are problematic. A longer span is also more aesthetically attractive and causes minimal disruption to the natural environment at water crossing. Compared to steel girder structural system, spliced concrete girder bridge system brings higher durability and requires lower maintenance.

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