INNOVATIONS IN PRECAST/POST-TENSIONED CONCRETE CABLE STAYED BRIDGE TECHNOLOGY

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ABSTRACT

Bridges are an integral part of our transportation system. The average age of a bridge in the United States is 42 years old although some are much older. The functionally and structurally deficient Puyallup River Bridge in Tacoma, Washington is more than 90 years old. Due to the condition, rating and serviceability of the structure, the City of Tacoma began a major multi-phase bridge replacement program with an anticipated cost of $100 million. The first phase of the project will replace 950 feet of an aging concrete viaduct and a steel truss span with a signature cable-stayed bridge that features an innovative precast/post-tensioned concrete cable-stayed main span superstructure.

Profile and alignment constraints, the crossing of six active rail lines and construction speed were all factors in the selection of a unique 4.5–foot-deep structure depth for the two 400-foot-long main spans. This paper details the precast/post-tensioned concrete cable-stayed superstructure used for this project. Also discussed is how critical details were integrated into the construction specifications in order to minimize project risk, while still allowing cost saving and construction innovations for the builder. The paper concludes with a summary of how the team succeeded and lessons learned.

Keywords: Cable Stay, Bridge, Precast, Post-Tensioned, Seismic, Bridge Replacement

INTRODUCTION

PROJECT HISTORY

The original Puyallup River Bridge was constructed in 1925, and at the time connected the Cities of Fife and Tacoma in Washington State across the Puyallup River and nine railroad tracks. In 1972 the west approach bridge was partially reconstructed to accommodate an approach roadway re-alignment. The current 32 span - 2,456 foot long bridge is made up of a combination of roughly three different bridge types. Starting at the west end of the bridge there is a 3 span – 269 foot prestressed concrete girder structure, followed by a 197 foot long steel thru truss structure that crosses several railroad tracks, then by a 13 span – 595 foot long cast-in-place (CIP) girder structure that connects to a 3 span -764 foot long steel thru truss structure that crosses over the Puyallup River. The river crossing structure then connects to a 3 span – 95 foot long CIP girder structure, that then connects to a 116 foot long steel through truss structure that crosses two railroad tracks and finally ends with an 8 span – 416 foot long CIP girder structure. Refer to Figure 1 for an overview of the current bridge.

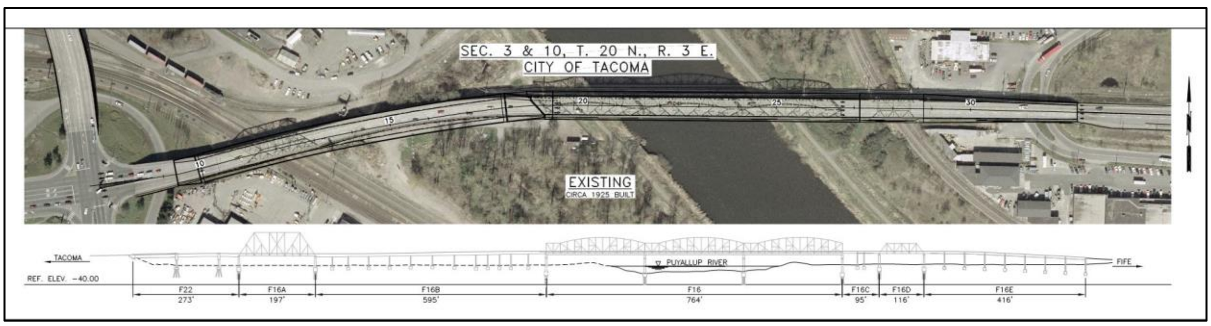

Fig. 1 Overview of Project Site

The 50 foot wide bridge currently carries three lanes of traffic (one westbound lane and two eastbound lanes), has two pedestrian sidewalks and carries roughly 20,000 vehicles per day. It serves a large volume of local commuter vehicles and is also a main route for trucks traveling between the Port of Tacoma and Interstate 5. It is one of the oldest and most heavily used bridges owned by the City of Tacoma (City).

Unfortunately, after more than 90 years of service in a marine and industrial environment, the bridge is showing significant signs of deterioration and is now both functionally obsolete and structurally deficient. Minimal city bridge maintenance budgets over the previous decades have only allowed the repairs and maintenance necessary to keep the bridge open to traffic. The bridge currently has a NBI structural evaluation rating of 3, with many critical WSDOT Bridge Management System (BMS) Elements (such as gusset plates and girders) in Condition State 4. It also has a Bridge Sufficiency Rating of 17, well below that which is considered acceptable for a heavily used bridge. Refer to Figure 2 for a view of the bridges deterioration.

Fig. 2 Typical Deteriorated Steel Gusset Plate Connection

Additionally after the most recent fracture critical inspection conducted last year, the load rating was revised and the bridge was reposted to only be able to carry car traffic. The significant and wide spread deteriorations of the bridge has now severed this important local transit and trucking route.

PROJECT CONSTRAINTS

There are a number of factors that make this a particularly challenging project. The first major challenge is funding. A replacement structure is anticipated to cost $100 million. This is far more than the City is able to afford on its own, and still far exceeds the City's match-funding capabilities if it were to utilize federal funding to replace the bridge.

The project site is also very constrained with a number of restrictions and stakeholders along the length of the structure. The bridge crosses a total of eight railroad tracks that are owned and operated by BNSF and Union Pacific, and crosses the Puyallup River and its levees which are governed by a number of agencies that include the US Army Corp of Engineers, the Department of Fish and Wildlife and other state and environmental agencies. Additionally, the existing right-of-way is only wide enough to accommodate one bridge. A majority of the land adjacent to the bridge has been developed and is owned by various government agencies and private businesses. The Puyallup Tribe also owns land, of cultural significance, directly adjacent to the existing bridge and is also a significant stakeholder in the maintenance of the Puyallup River. Finally, the soils beneath the existing bridge have been rendered hazardous after years of industrial site use and will require special treatment and disposal during construction if disturbed.

With all of these project challenges there is at least one aspect that is not challenging. The river crossing is one of four river crossings within a one mile long stretch along the Puyallup River. This means that it is possible to temporary take the entire bridge out of service since there are multiple nearby detour options.

While not a physical constraint, limited access in areas over the existing active rail lines and a WSDOT access road will restrict time and type of construction to techniques that can work overhead with limited closures and falsework.

BNSF has future expansion plans in the vicinity of the structure which involve a set of spur tracks that would be further east of the existing rail lines. Commitments have been made to coordinate the future bridge structure with the horizontal and vertical clearance envelopes for the new spur. Providing the additional vertical clearance and horizontal clear distances will require a shallow superstructure in order to tie into existing alignments.

The wide river crossing is bordered by dikes operated by the Army Corp of Engineers. This will require a long central or side span to bridge without additional in water piers for environmental, cultural reasons and to protect the integrity of the dikes.

In order to complete a phased replacement of the structure an added geometric constraints are added to the project in order to connect to the intersection on the west end of the project without significant modifications and to continue providing future compatibility for successive portions of the project.

PHASED REPLACEMENT

With the challenging project constraints in mind, a unique solution was needed to replace the aging bridge on this local important route. The existing bridge was further evaluated, and subsequently subdivided and re-inventoried into seven separate structures. The seven partition of the entire Puyallup River Bridge were evaluated and rated separately. It was found that some portions of the bridge were in much better condition than others. With this information and project and funding challenges in mind, the concept of phased replacement of the Puyallup River Bridge was conceived and ultimately adopted. Refer to Figure 3 for the general two phase bridge replacement program.

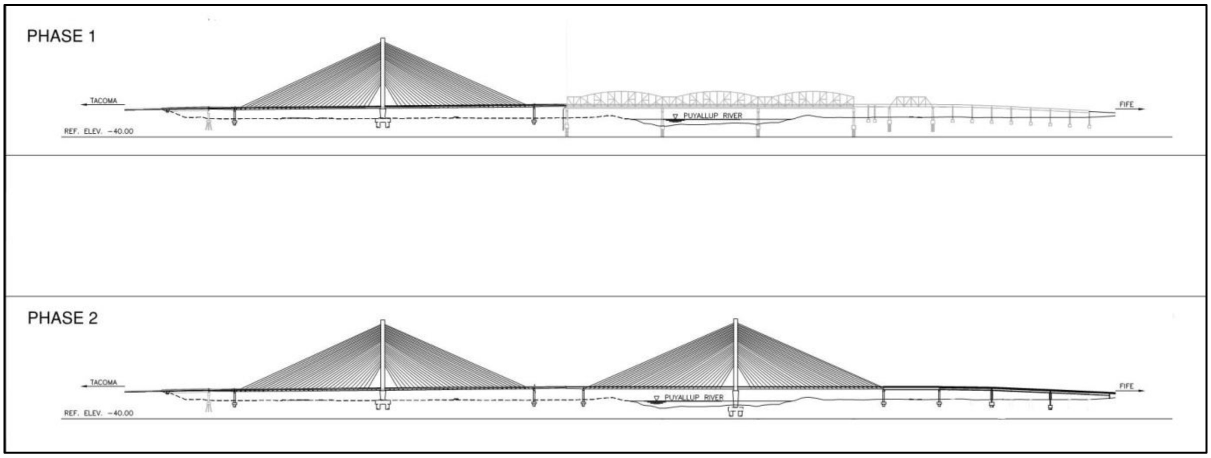

Fig. 3 Two Phase Replacement of the Puyallup River Bridge

The first phase of the project consists of replacing part of the western 3 span – 269 foot prestressed concrete girder structure, the entire 197 foot long steel thru truss, crossing six active railroad tracks, and the entire 13 span – 595 foot long cast-in-place (CIP) girder structure. This is the portion of the bridge found to contain the most deteriorated elements and the lowest remaining service life. The second phase of the project consists of replacing the remaining river crossing structure and east approach structures.

The phased replacement program creates of number of solutions to deal with the projects specific challenges. The first solution was the ability to make a replacement bridge affordable within the current funding packages. The first phase of the project is anticipated to cost about a third of the total anticipated project costs. Although this is still more than the City can afford on its own, it is within the City's match-funding capabilities which allows them to utilize federal funding to replace the bridge. This also allows the City more time to find and accumulate additional funding by utilizing the remaining service life of less deteriorated eastern portions of the bridge.

Another solution was to limit the number of stakeholders and areas needing permit per phase by geographical regions of the project. Fewer permits and involved parties have accelerated the permitting process and negotiations for the project's first phase. Although the phased replacement does increase overall construction duration, multiple available detour routes help minimize impacts to traffic. Other benefits gained with the selected bridge configuration will be discussed later.

REPLACEMENT STRUCTURE

With the constrained urban environment, hazardous soils, narrow permanent right-of-way, limited construction easements, as well as a desire to add additional railroad under crossings, it was determined early on in the project that a long span bridge solution would best fit all of the project's needs. The preliminary alternatives analysis for the first phase of the project determined that a signature cable stayed bridge unit with two roughly 400 foot long precast/post-tensioned concrete main spans would best fit the needs of the project constraints. The superstructure would be supported by a 224 foot tall diamond shaped tower and spliced to the remaining existing structure with short transition spans. This optimized span arrangement minimized excavations in the hazardous soils and provided the best solution for future railroad undercrossing expansion to support the growth of the regional economy.

The two phase replacement also provides additional cost saving benefits to the project. The second phase, currently planned as a cable stayed bridge unit over the Puyallup River is expected to be almost identical to the first phase cable stayed bridge unit. This not only allows for the reuse of a number of engineering calculations, design details and project specifications, it also allows the City to carry forward lessons learned during the construction of the first phase of the project that will reduce project risk and consequently project cost within the second phase of the project.

DESIGN CRITERIA

BASIC DESIGN REQUIREMENTS

Complex structures commonly require project specific design criteria, and this project is no different. The project contains of mix of long cable supported spans and short typical concrete spans, large concrete foundations elements and unique concrete retrofits to be able to tie the new phase one structure to the existing remaining Puyallup River Bridge. The WSDOT Bridge Design Manual, along with the AASHTO Bridge and Seismic Design Specifications, set the basis for the project design requirements. However, unique aspects of the projects required the adoption of other design guides and specifications in order to adequacy design the phase one replacement structure. The PTI Recommendations for Stay Cable Design, Testing, and Installation was required for the project, along with the CEB-FIP 90 Model Code for Concrete Structures and PTI Post-Tensioning Manual.

WIND DESIGN REQUIREMENTS

Even though significant support was provided by the pre-mentioned design manuals, specifications and guides, the project still needed additional specific criteria to be developed by the design team. The wind loads on the structure were developed from a site specific wind load analysis that resulted in the following basic following design parameters;

Design Condition	Wind Speed	Return Period (Years)
Final Design	49 mph	100
Final Stability	74 mph	10,000
Construction Design	43 mph	25
Construction Stability	63 mph	1,000

The site specific design wind speeds were to be utilized in a wind dynamic analysis of the cable stayed precast/post-tensioned concrete main span unit structure. Scale model wind tunnel testing was required to determine the dynamic wind response coefficients. Refer to Figure 4 for a view of the scale superstructure model using for testing.

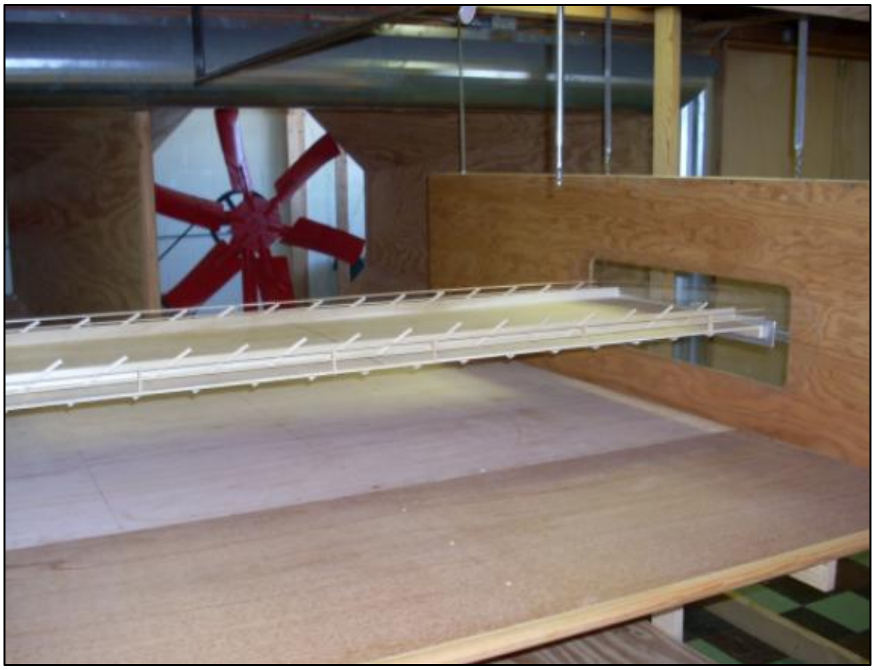

Fig. 4 Scale Cable Stayed Main Span Unit used in Wind Tunnel Testing

The main span unit required numerical dynamic analysis of four different stages during cantilever construction in addition to the final bridge configuration. Stability and buffeting analyses were required for all numerical dynamic analyses. Vortex and buffeting induced analyses utilized to predict pedestrian discomforting motions.

The individual stay cables required localized wind dynamic analyses using the site specific design wind speeds. Cable-stay vibration, rivulet effects, turbulence and wake effects all were included in the calculations. Rain Wind Induced Vibration (RWIV) and dry-cable vibration parameters were utilized to determine the recommended supplemental damping requirements for the individual stay cables.

THERMAL DESIGN REQUIREMENTS

Thermal loadings also needed specific criteria to be developed for the project. Uniform thermal effects and superstructure temperature gradients for concrete structures are established in the pre-mentioned design resources although the specific effects related to stay cables, long span superstructures, tower legs and the integration of the loading conditions needed to be developed for this project. After a qualitative and quantitative evaluation of similar cable stayed structures and the regional thermal variations and demands, the following additional thermal design parameters were incorporated into the project design criteria;

Temperature Gradient Condition	Thermal Variation
Between the Cables and Bridge (Deck, Tower & End Piers)	+/- 25^{o}F (Uniform)
Between the Opposite Tower Leg Faces	+/- 18^{o}F (Linear)

These project specific thermal design parameters were applied to the global analysis model. The number of loading combinations was determined so as to envelope the worst loading conditions on the individual elements of the cable stayed precast/post-tensioned concrete main span unit.

SEISMIC DESIGN REQUIREMENTS

The size and complexity of the cable stayed main span unit in a high seismic zone merited specific design criteria to be developed by the design team for the project. The cable stayed main span unit required three response spectrum-compatible time histories to be used for each component of motion to represent the design earthquake during the analyses. The three orthogonal components (longitudinal, transverse and vertical) of design motion were required to be input into the structural model when conducting the seismic time history analysis. Since the site was relatively short in length and the subsurface conditions varied little from one end of the project to the other, the time histories did not need to include modeling of spatial variations in terms of the differences between seismic wave arrival times at the different bridge piers.

A total of nine separate time histories were used to meet the seismic analyses requirements. See Figure 5 for representative time history plots used on the project.

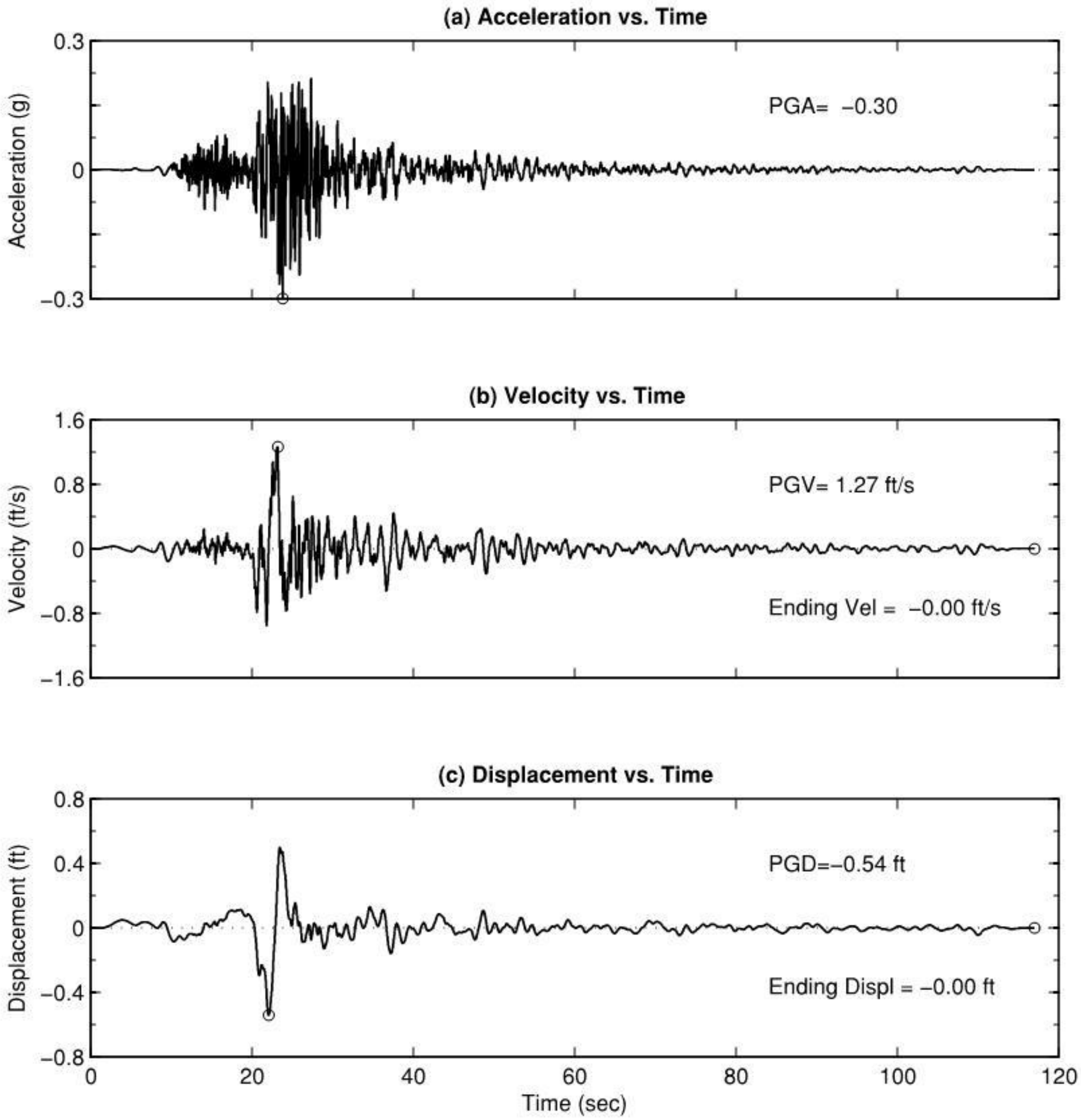

Fig. 5 Soil Matched Time History Plots for the 2001 Nisqually Earthquake

Response modification factors were established and documented for elements not covered in the pre-mentioned design manuals for project consistency. The cable stayed main span units basic Earthquake Resisting System (ERS) was defined to be a ductile substructure with near elastic superstructure. Supplemental seismic detailing requirements were also further defined in the project design criteria.

HIGH PERFORMANCE CONCRETE

The project team limited the height of the cable stayed main span depth to 4.5 feet in order to meet the vertical clearance and tie in criteria. In order to meet project strength and service requirements with a shallow superstructure the project design team evaluated the ability to obtain high performance concrete (HPC) in the northwest region of the United States. It was determined that high performance concrete with a 28 day compressive stress of 10 ksi was readily available in the region however the premium costs associated with high strength mix the design team adopted a single 56 day, 9 ksi concrete to be used on the entire project. This mix was utilized for the precast concrete cable stayed main spans segments, for the precast girders for the transition spans and isolated elements of the substructure.

LONGITUDINAL TENSILE STRESS LIMITS IN THE EDGE BEAMS

Because the precast concrete superstructure is made up of match cast segments, with discontinuous longitudinal reinforcement at joints, the project team limited the longitudinal stresses in the edge beams to 0 ksi. In recent concrete edge beam cable stayed bridge projects it has been common to evaluate service longitudinal stresses in the edge beams with some level of allowable tension (typically between 0.0948*sqrt(f'c) and 0.19*sqrt(f'c)). These structures were typically built with cast-in-place techniques, either utilizing form travelers or falsework where mild reinforcement crossed segment joints.

The zero tensile stress criterion presented challenges during the design process that required unique solutions to be developed for the staged construction of the precast segmental main spans. This requirement in addition to the epoxied joints between segments will result in a very durable superstructure design that has the ability to exceed the City's service life expectations.

INNOVATIVE DESIGN CONCEPTS

SHALLOW SUPERSTRUCTURE CROSS SECTION

The existing 197 foot long steel thru truss structure clearance over the railroad tracks is 23.6 feet. Its structural depth, from bottom of chord to top of deck is approximately 4.0 feet. This provides very little variance when meeting the project vertical clearance requirement of 23.3 feet over the railroad tracks. Additionally, the design speed of the corridor is being increased for the new structure to 35 mph. This requires longer flatter vertical curves to meet both modern roadway geometry and drainage requirements. This was a significant project challenge as there was little ability to rise and fall quickly between the ends of the bridge and the locations of required minimum vertical clearance over the railroad tracks. This created very narrow limits that the new main span superstructure needed to thread through that was roughly 6.5 feet high from the top of deck to the bottom of structure.

The resulting span/depth ratio for the main span superstructure at 4.5 feet deep is roughly 88 which is not excessive when compared to cable stayed bridges utilizing concrete edge beams. The total depth, from top of deck to bottom of anchorage, was a significant problem that needed to be addressed. Traditional cable stayed bridges with concrete edge beams typically have the stay cable run directly through the edge beam and anchor in the underside of the edge beam. See Figure 6 for a typical cable stayed concrete edge beam and anchorage.

Fig. 6 Typical Cable Stayed Concrete Edge Beam and Stay Cable Anchorage Assembly

Although it was conceptually possible to thread a traditional cable stayed concrete edge beam through the narrow limits set above, the window for construction was even more difficult to meet. The staged construction temporary deflections additionally limited new main span superstructure within a similar vertical envelope from profile grade line to top of temporary construction envelopes to 9.0 feet, primarily above the active rail lines. Traditional construction platforms used to install edge beam anchorages elements typically extended another 4 feet to 6 feet below the bottom of the edge beam. Strand tails can also extend another 3 feet to 5 feet below the bottom of the edge beam and are typically not cut to their final lengths until main span construction is complete. The flexible balanced cantilever spans, which have arms deflecting 1.5 to 2.5 feet per construction activity, meant that a unique concrete edge beam cross section was needed to meet the project's design criteria.

The chosen solution was to offset the stay cable to the outside face of the concrete edge beam and connect the load path between the edge beam and stay cable anchorage through an roughly 2 foot thick stay cable shelf/ledge. See Figure 7 for the typical bridge edge beam cross section.

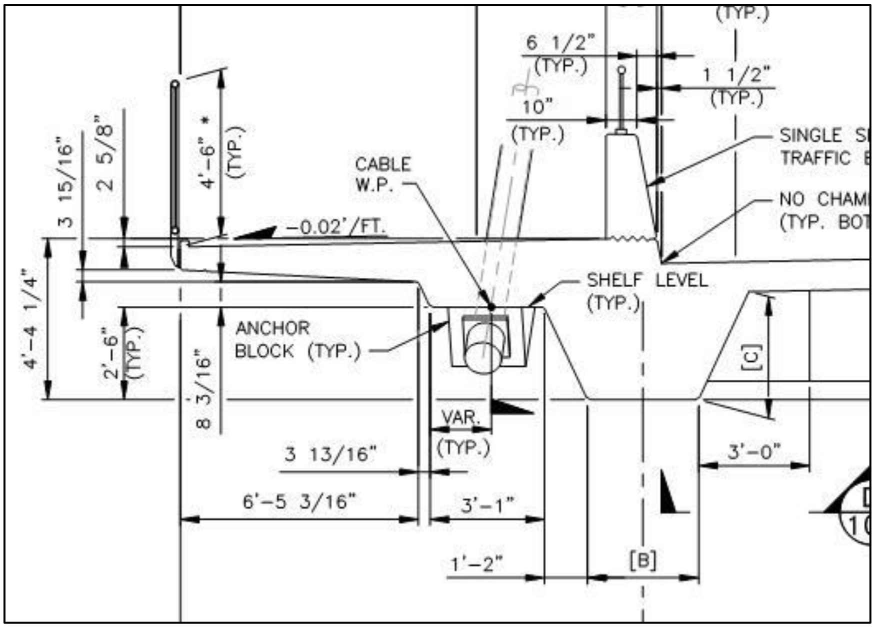

Fig. 7 Typical Bridge Concrete Edge Beam Cross Section

This solution allows the new precast/post-tensioned concrete cable stayed main span superstructure to meet the permanent vertical window of 6.5 feet with room to spare by providing 25.6 feet of clearance over the railroad tracks. Additionally, the solution meets the construction vertical window of 9.0 feet when combined with a balanced cantilever construction method that maintains the out of balance cantilever away from the railroad tracks. The additional permanent space helps prevent the balance cantilever tip over the railroad tracks from dipping downward into the temporary clearance envelope during balanced cantilever construction.

FULL WIDTH PRECAST SEGMENTS

The 4.5 foot tall, 73 foot wide segment required for the new precast/post-tensioned concrete cable stayed superstructure required a unique segment cross section to meet the bridge geometry constraints. CIP cantilever construction was considered; however, it showed to have a number of challenges related to the available room for cantilever traveler formwork. The significant additional weight at the cantilever tip also causes larger downward cantilever tip deflections on top.

Construction experience has shown that it is preferred to not have transverse joints (either match cast or CIP) located within stay cable anchorage assemblies. This type of detail can reduce the constructability of the stay cable anchorage assemblies and edge beams. These joints can also have an impact on the service life of the stay cable anchorage assembly. Consequently, the segment length needed to contain all stay cable anchorage assemblies in individual segments was 12.0 feet.

Superstructure erection speed was also an important consideration. Even though the corridor would be closed again for the phase two construction, the City wanted to minimize the duration of the phase one closure as much as possible. These constraints determined that full width precast concrete segments were the best solution for the project. See Figure 8 for the typical precast/post-tensioned concrete cable stayed main span superstructure cross section.

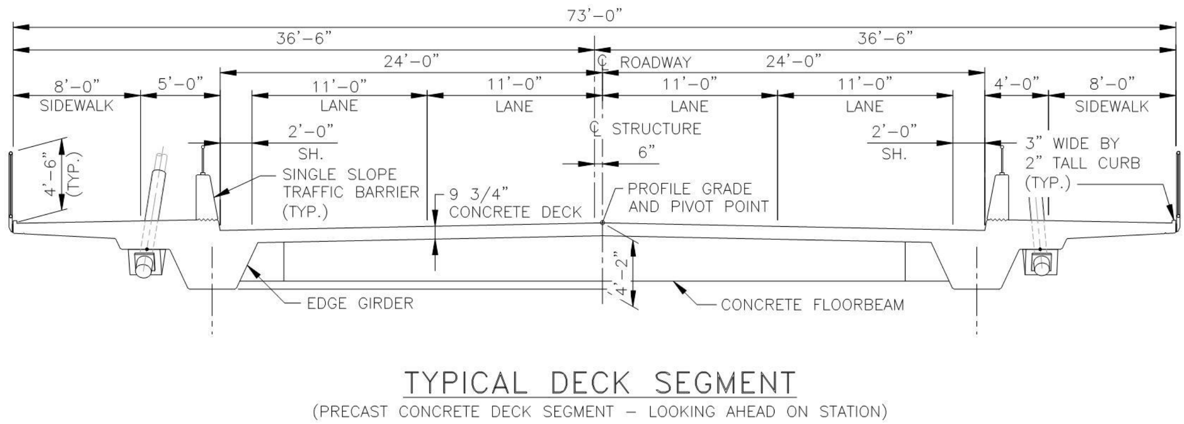

Fig. 8 Typical Cable Stayed Main Span Superstructure Cross Section

The 73 feet wide by 12 feet long by 4.5 feet tall segments weigh between 96 and 116 tons. Although this is relatively heavy for precast segmental concrete construction, the construction site permits erection cranes to get very close to the tips of the cantilevers. This means that readily available cranes can be used to erect the precast concrete superstructure segments.

SUPERSTRUCTURE ERECTION SEQUENCE

The wide, shallow and relatively heavy precast/post-tensioned concrete cable stayed main span superstructure segments presented unique challenges to the balanced cantilever construction method. The primarily challenge presented was how could a builder erect a 96 to 116 ton segment on the tip of a cantilever quickly while keeping the already erected cantilever segments and stay cables within allowable construction stress and force limits. In addition, at the end of construction and long term time dependent effects, how does the superstructure meet the zero tensile stress requirements in the longitudinal direction of the edge beams?

Temporary leading edge stay cable were considered; however, they showed to have a number of challenges related to available room on the cantilever tips for temporary stay cable anchorage assemblies, additional constraints imposed on permanent stay cable erection, constructability issues related to relocating the temporary strand anchorages and construction maintenance requirements for the temporary strands. The balanced cantilever solution recommended involves a multi-stay cable stressing sequence in conjunction with cantilever ballast that is positioned in different locations on the cantilevers during balanced cantilever construction. This solution allows the cantilevers to take the relatively large out-of-balance forces induced by the erection of a precast segment, while still keeping the erected precast segments and stay cables within allowable construction stress and force limits. Additionally, the final stay stressing sequence induces sufficient compression into the precast/post-tensioned concrete cable stayed main span superstructure so that it is able to meet the zero tensile stress requirements at both end of construction and end of all time dependent effects. See Figure 9 for a typical segment erection sequence schematic.

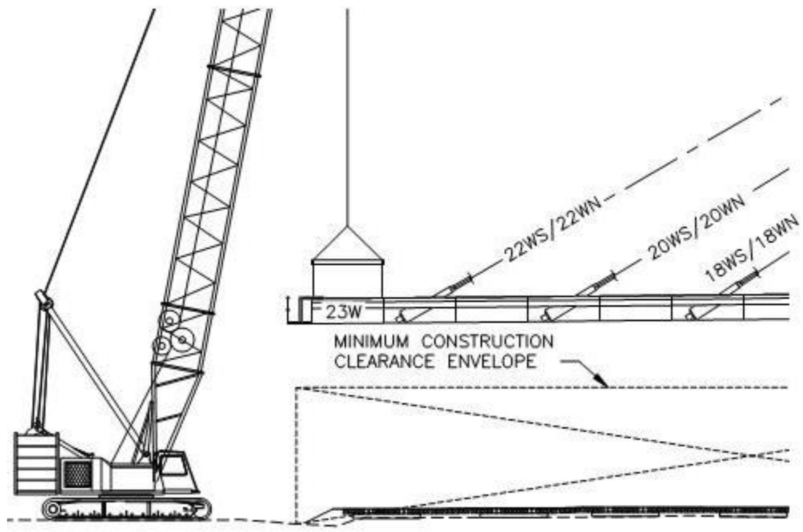

Fig. 9 Typical Balanced Cantilever Segment Erection Sequence Schematic

Superstructure construction over the railroad presented its own unique challenges. Construction over six active rail lines, owned by BNSF and UPRR, that provide access to the port. Both agencies expressed strong messages limiting track closures. In order to ease railroad agency concerns the design team presented before a joint WSDOT/AGC Constructability Team to help augment project knowledge with contractor experience. The panel agreed that the segment hanging operations were possible within the limited work windows allowed by BNSF and UPRR. Additional efficiencies offered by the panel which were incorporated into the proposed construction method.

SUPERSTRUCTURE DESIGN

EDGE BEAMS DESIGN

The design of the edge beams for the new precast/post-tensioned concrete cable stayed main span superstructure had a number of unique challenges that needed to be overcome. The first challenge related to construction stresses and final services stresses in the edge beams. Construction stresses were primarily limited by a tension limit of 0.22*sqrt(f'c) and a compression limit of 0.5*f'c and final service stress were limited by a zero tension limit and a compression limit of up to 0.6*f'c.

To meet the construction stress limits during construction, 42 – 1-3/8 inch diameter ASTM A722, Grade 150, Type 2 post-tensioning bars are required to supply sufficient axial compression to the bridge cross section to minimize tensile stresses. Additionally the post-tensioned bars needed to provide sufficient capacity to the cantilevers when analyzing the segment drop/loss of segment load cases. To help meet final service stress limits, 8 full length 27 strand post-tensioned tendons utilizing 0.6 inch diameter ASTM A416, Grade 270, low relaxation prestressing strands are installed at the completion of cantilever construction. Three partial length tendons are installed in each edge girder during progressive segment placement which balance stresses over intermediate transient load posts. See Figure 10 for a half cross section of the typical longitudinal post tensioning provided in the precast/post-tensioned concrete main span superstructure.

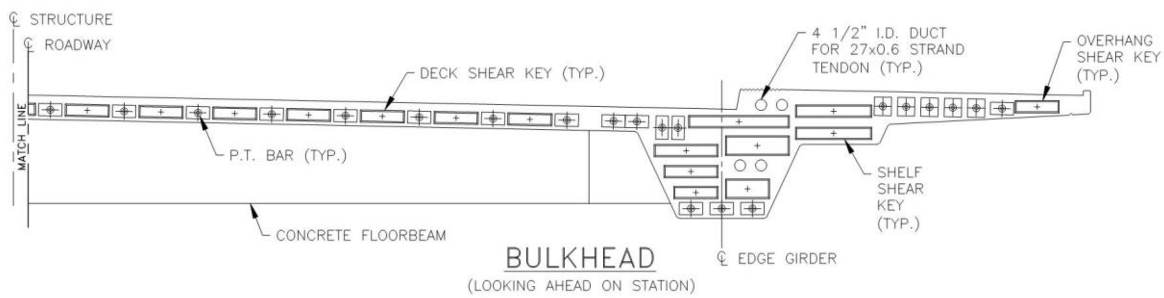

Fig. 10 Half Cross Section of Internal Longitudinal Post-Tensioning Bars and Tendons

Although there is a significant amount of longitudinal post-tensioning in the main span superstructure, the primary means used to manage the longitudinal stresses was to adjust the stay cable forces. This is because in a cable stayed concrete edge beam superstructure, significant bending stresses can be induced into the concrete edge beams by the stay cables. Since the section is relatively shallow in depth, slight adjustments to the stay cable forces can induce significant bending stresses into the edge beams. The final stay stressing sequence is used to adjust the bending stresses in the edge beams so that they can transition from meeting the construction stress limits to the final service stress limits. See Figure 11 for a plot of the final end of construction service stresses after the final stay stressing sequence.

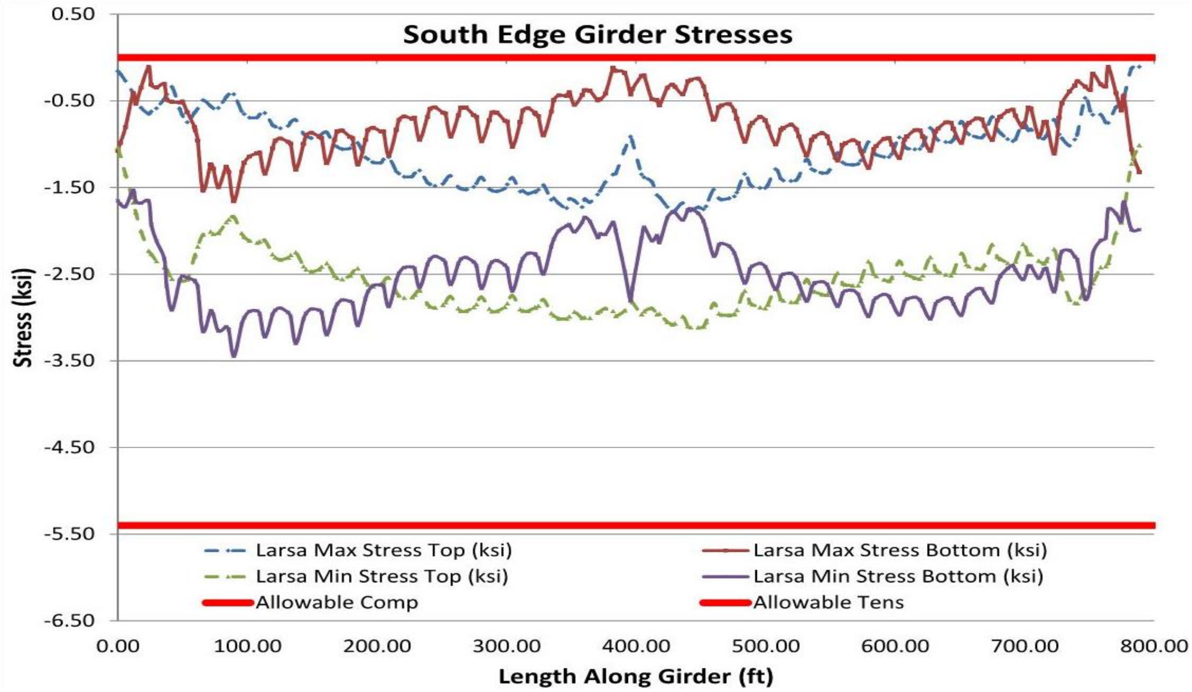

Fig. 11 South Edge Girder EOC SERVICE III Stress Plot

Once the stresses in the edge beams were resolved, nominal moment capacity presented a unique challenge. The primary limitation provided by any superstructure constructed using precast concrete segments is that the only tensile reinforcement, and subsequent moment resistance, is only provided by the post-tensioning steel that actually crosses the precast joints of the segmental superstructure. In order to most accurately calculate the capacity of the edge beam, the method of strain compatibility in accordance with AASHTO LRFD, Section 5.7.2.1 was used to determine the nominal moment capacity of the main span superstructure edge beams. Once the nominal capacity of the edge beams was determined an iterative process was used to modify the forces in the stay cables to change the demand bending forces transferred to the edge beams. Where necessary, the longitudinal post-tensioning tendons alignments were adjusted locally to manage bending forces that could not be manage with stay cable force adjustments alone.

Shear and torsion forces in the edge beams presented another unique challenge to the design of the concrete cable stayed superstructure. Although offsetting the stay cable anchorages to the outside faces of the edge beams solved the geometric clearance issues mentioned earlier, this change did result in greater torsional forces in the edge beams when compared to typical concrete edge beam cable stayed superstructures. An increase in the torsional perimeter was required to help manage the torsional forces in the edge beams, and this resulted in the trapezoidal shape used for the main span superstructure edge beams. A unique aspect of this particular structure is that in many locations along the edge beams the precast joint shear and torsion capacity actually governed the shear and torsion design over the shear and torsion capacity of the monolithic edge beams between the precast joints. Sufficient capacity was ultimately achieved; however, this required refinement to the shear key geometry and shear interface capacity calculations, by utilizing additional code provisions found in ACI and CEB-FIP, to confirm all appropriate variables were included in the shear key capacity calculations.

Finally once service stresses, nominal moment capacities and shear and torsional capacities were checked, principal stresses in the edge beams were checked. During construction principal stresses were limited to 0.126*sqrt(f'c) and 0.11*sqrt(f'c) for final service stress checks. Meeting principal stress requirements was initially achieved after meeting the other edge beam design requirements for both construction cases and final configuration cases.

FLOORBEAM DESIGN

The design of the floorbeams for the precast/post-tensioned concrete segments presented their own set of challenges that primarily related to service stresses and integrating the required transverse post-tensioning and mild reinforcing steel for the floorbeams with the longitudinal post-tensioning, stay cable hardware and mild reinforcing steel required for the edge beams. For most segments, 3 – 7 strand post-tensioning tendons utilizing 0.6 inch diameter ASTM A416, Grade 270, low relaxation prestressing strands were sufficient to meet service stress requirements in the floorbeams. See Figure 12 for a partial cross section of the transverse post-tensioning layout used in the typical floorbeams of the main span superstructure.

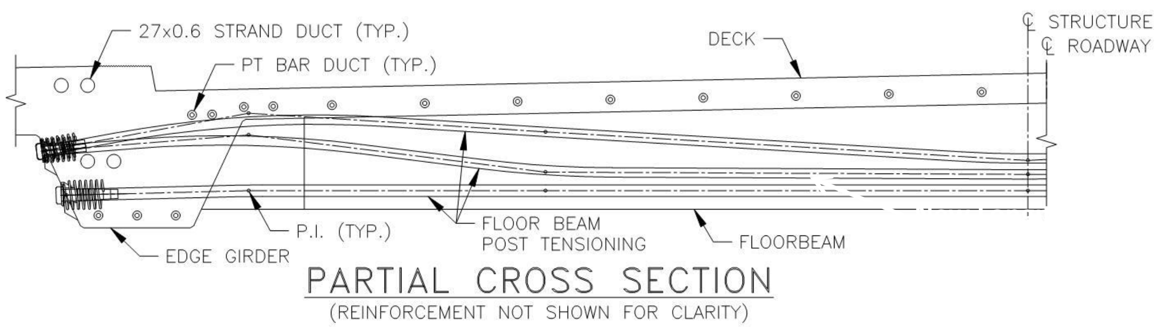

Fig. 12 Partial Cross Section of Typical Floorbeam Post-Tensioning

Since the floorbeams were monolithically cast with the precast concrete segments, final service tensile stress limits of 0.0948*sqrt(f'c) could be used in conjunction with the 0.22*sqrt(f'c) construction stress limit. Final service stresses generally controlled over construction stresses in the floorbeam elements.

Nominal moment capacity was calculated using the method of strain compatibility in accordance with AASHTO LRFD, Section 5.7.2.1. Fortunately with a monolithically cast beam, mild reinforcement could be used in the nominal moment capacity calculations. Consequently wherever additional moment capacity was required in the floorbeams, mild reinforcement was added to provide the necessary capacity. Shear capacity primarily consisted of ASTM A706, Grade 60 #5 stirrup bars that were spaced between 6 inches and 12 inches apart along the lengths of the floorbeams.

STAY CABLE SHELF DESIGN

As previously mentioned, the offsetting of the stay cable anchorages to the outside faces of the edge beams solved many of the geometric issues for the project; however, this did create additional design challenges that needed to be successfully overcome. The stay cable shelf that was created for this project had to resist the multitude of design forces in a different manner than if the stay cable ran through the center of the edge beam. The design moment between the edge beam and stay cable shelf was a challenge to overcome primarily due to the shallow depth of the stay cable shelf, typically about 2.0 feet, and the fact that significant design moment bent the shelf up (when considered normal strength loads) and down (when considering loss of stay cable loads). The relative narrow depth also presented challenges in resolving the shear in the connection.

Ultimately the significant shear and moment forces created at the stay cable shelf and edge beam connection were resolved with the use of the 9 ksi concrete and a variety of ASTM A706, Grade 60 mild reinforcing bars that ranged in size from #5 to #8. See Figure 13 for the typical mild reinforcing steel used to reinforce the stay cable shelf.

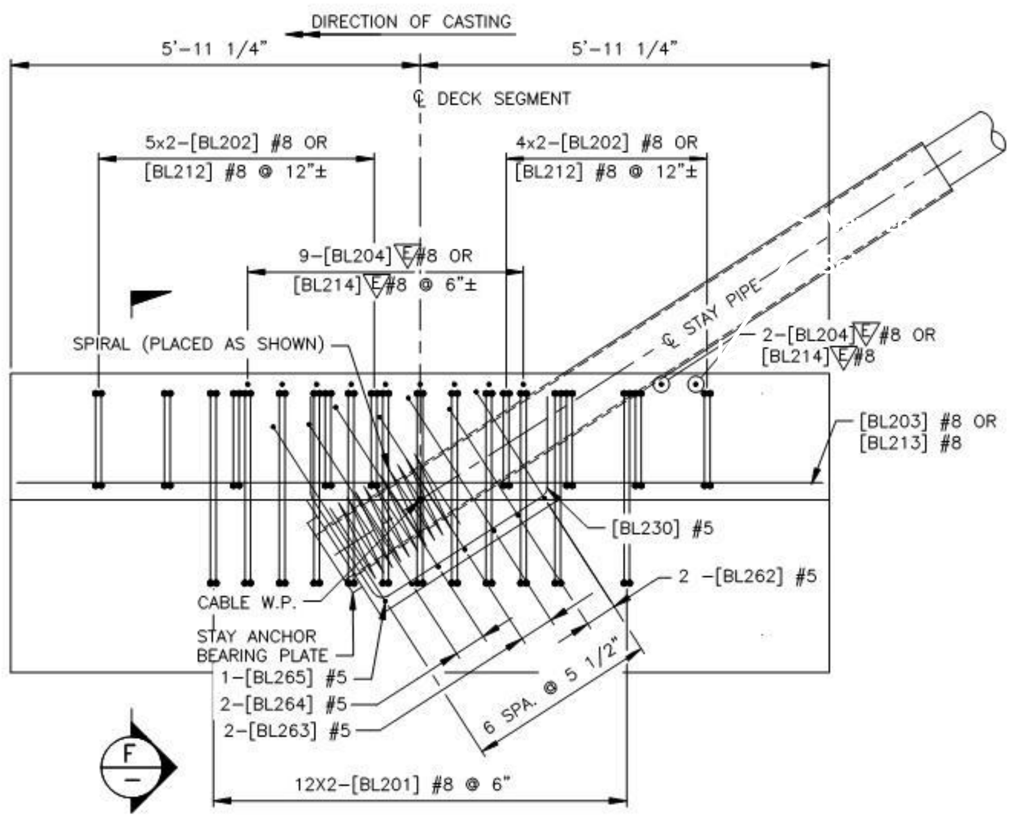

Fig. 13 Typical Stay Cable Shelf Reinforcing Details

Although service forces were checked to confirm proper crack control in the stay cable shelf and edge beam connection, there was not a need to check the previously mentioned concrete stress limits because there was no post tensioning crossing the connection. The concern of normal long term cracking of the reinforced concrete element was evaluated further and adding transverse post-tensioning in the stay cable self was considered during design. However, the mild reinforcement provided to resist the strength and extreme force event requirements was more than sufficient to manage the service force requirements, and adding transverse post-tensioning across the connection was found to only further complicate the design and detailing when considering all aspects of code requirements and available geometry.

SEGMENT FINITE ELEMENT ANALYSIS

Given the unique geometry and complexity of the precast/post-tensioned concrete cable stayed main span superstructure, it was discovered during the design process that a highly detailed finite element model of the segments of the main span superstructure was needed to adequately evaluate all stress and force effects on the superstructure. A finite element model utilizing a typical discretization grillage of 6 inches was created to represent five precast segments in the main span superstructure. See Figure 13 for a typical view of the five segment local finite element analysis model.

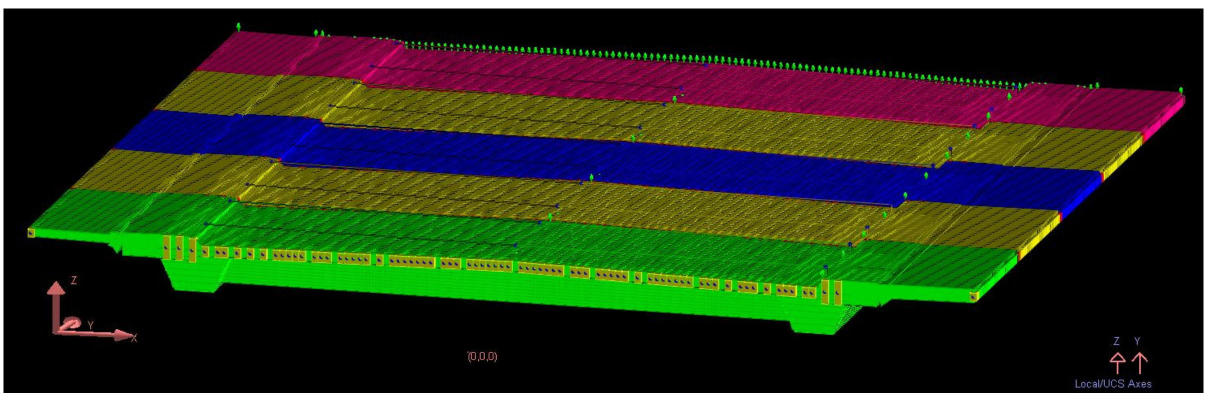

Fig. 13 View of Five Superstructure Segment Local Finite Element Analysis Model

The finite element model required a fair amount of testing to create the proper boundary conditions and proper interaction between the finite elements and post-tensioning elements. Once the finite element model was functioning properly it was used to evaluate a number of conditions that included the following;

- Removal of formwork from the precast concrete segment
- Transport of the precast concrete segment from a proposed casting bed
- Evaluation of a proposed segment storage support arrangement
- Transverse post-tensioning tendon stressing sequence

- Evaluation of a proposed segment transport and segment lifting plan
- A proposed cantilever construction and longitudinal post-tensioning bar and tendon stressing sequence
- Temporary cantilever construction loadings and stresses
- Final configuration design loads

The five segment finite element model proved to be a very valuable tool in the design of the precast/post-tensioned concrete cable stayed main span superstructure because it was able to capture various force and stress effects that could not be captured in the less discretized global analysis model. High stress regions were easily identified in the five segment model and subsequently mitigated by changing localized design details or changing the proposed construction sequence.

CONSTRUCTION SPECIFICATIONS

SUPERSTRUCTURE ERECTION GEOMETRY CONTROL

During the design of the precast/post-tensioned concrete cable stayed main span superstructure it was discovered early on that the geometry of the superstructure during the stage construction analysis had a significant impact on the construction and final design forces and stresses. Additionally, it was discovered that controlling the geometry of the superstructure was relatively difficult task because the slender superstructure is fairly flexible and easily influenced by minor changes such as segment weight, applied erection forces and stay cable jacking forces. These factors and others established superstructure erection geometry control as a very important element that needed to be clearly defined in the bridge special provisions.

Segment creep and shrinkage effects were major factors in controlling the geometry of the precast/post-tensioned concrete segments. In order to better control the effects on segment geometry from creep and shrinkage effects, which can be very difficult to predict during design since many of the variables are estimated values and not actual values, a minimum erection age of 60 days was prescribed in the contract specifications. Testing in both the global analysis and five segment finite element model proved that a 60 day age was an appropriate age to minimizing the undesirable geometric effects that are caused by the creeping and shrinking of post-tensioned concrete. Additional discussions with contractors validated that a 60 day erection age requirement was acceptable and preferable for this specific project.

A number of erection tolerances also needed to be defined to help establish the necessary construction means and methods to build the precast/post-tensioned concrete cable stayed main span superstructure. A number of tolerances were established to check the segments at the cantilever tips, as well as to continuously check segment joints along the length of the cantilevers as balanced cantilever construction progresses. The maximum differential between the outside faces of adjacent segments is 3/16 inch. The maximum transverse angular deviation between successive segment joints is 0.001 radians, while the maximum longitudinal angular deviation is 0.003 radians. The maximum roadway elevation differential between two adjacent segments is established at 1/8 inch. Additionally, the accumulated maximum permissible error is 1/1000 of the span length for both the vertical and horizontal profiles. These established tolerances will ultimately help the contractor successfully erect the main span superstructure. Finally, the contractor is also required to submit an Erection Manual that contains a Geometry Control Plan prior to constructing the precast/post-tensioned concrete cable stayed main span superstructure.

INTEGRATED DRAWINGS

The precast/post-tensioned concrete cable stayed main span superstructure will have a number of non-standard and/or proprietary elements amongst a number of standardized elements. Two major proprietary systems that will be part of the precast concrete segments are the stay cable system and post-tensioning system, and the geometry of various supplier components can vary significantly from supplier to supplier. With the tight segment geometry requirements and with the segments already being congested with mild reinforcing steel, it was not practical to develop design plans that could encompass every possible supplier for every non-standard/proprietary component. However, it is critical that the contractor and designer know all components fit together prior to building the precast concrete segments.

This necessitated the definition and requirement for integrated drawings in the bridge special provisions. When the contractor selects his suppliers for the project and they know the actual geometry of all non-standard and proprietary elements that will be used on the project, the contractor is required to develop the integrated drawings prior to beginning segment construction. The integrated drawings will include all elements in a segment such as mild reinforcing steel, post-tensioning ducts, grout tubs, anchor bolts, drainage systems utility conduits and anchorage reinforcement. All conflicts will need to be resolved and all integrated drawings need to be reviewed by the designer prior to the beginning of precast concrete segment construction.

HIGH PERFORMANCE CONCRETE

With the large volumes of HPC being used on the project and the various bridge elements utilizing HPC, with some that are initially categorized as mass concrete placements, the need for a project bridge special provision for HPC was needed. First, a number of testing requirements for the HPC are defined in the bridge special provisions. Table 1 defines the required concrete properties to be investigated, the required testing method and the test acceptance criteria.

Testing Requirements for HPC		
Property	**Test Method**	**Acceptance Criteria**
Compressive Strength (at 56 days)	AASHTO T22	>9,000 psi (all tests)
Freeze/Thaw Durability (x=relative dynamic modulus of elasticity after 300 cycles)	AASHTO T161 Procedure A	$x \geq 80\%$
Scaling Resistance (y=visual rating of surface after 50 cycles)	ASTM C672	$y \leq 3$
Elasticity (E=Modulus of Elasticity)	ASTM C469 (Note C)	$E > 4.35 \times 10^6$ psi
Shrinkage (s=microstrain)	AASHTO T160-97 (at 56 days)	$s < 600$
Creep (c = microstrain/ pressure unit)	ASTM C512 (at 56 days, 40% fc')	$c \leq 414$ psi
Chloride Penetration (p= increase in percent of chloride ion by weight of concrete)	AASHTO T259 modified (Note A)	$p \leq 0.025\%$ at 1 inch
Air Content	AASHTO T 152	A = % selected by the Contractor, $A \geq 3\%$
Water/Cementitious-Materials ratio (W=mass ratio)	AASTHO TP23-93 (Note B)	Supplier Selects W, $W < 0.40$

Table 1 High Performance Concrete Testing Requirements

Preproduction testing is required to be completed by an independent AASHTO Accredited Testing Laboratory using the preapproved mix design sheets. Upon approval of the HPC, production concrete needs to adhere to placement and curing requirements defined in the bridge special provisions. The maximum steam curing temperature permitted for precast concrete segment casting is 175^{o}F, with a limiting heating and cooling rate of 25^{o}F per hour. HPC that is defined as a mass concrete will be required to follow additional requirements in the bridge special provisions that are required to be incorporated into the Mass Concrete Placement and Curing Plan.

MOCK UP TESTING

With the high density of reinforcing steel, post-tensioning hardware, stay cable hardware, utility hardware and inserts in each segment, combined with a unique precast concrete segment cross section, the need to require a segment mock-up that be completed prior to the production of the precast concrete superstructure segments became clear. The bridge special provisions define the requirements and objectives of the segment mock up test. Given the symmetry of the precast concrete segment about the centerline of the bridge, only a partial full scale segment mock-up is required.

The goal of the mock up is to help confirm that the contractor's chosen precast concrete segment construction means and methods can adequately produce segments that meet the contract requirements prior to full segment production. Additionally, approval of the Casting Manual is contingent on the completion of a success segment mock up test.

CONCLUSION

Currently the design drawings and specifications for phase one of the new Puyallup River Bridge project are complete. The City is currently awaiting the completion of final right-of-way negotiations with adjacent property owners. Once the final right-of-way negotiations are complete, the drawings will be let for bidding. Currently final completion of the phase one portion of the new Puyallup River Bridge project is anticipated to be completed in 2017.

The new Puyallup River Bridge project contains a number of unique and excellent solutions that were needed to overcome the many project challenges. The extensive deterioration of a structure that has long exceeded its original service life generated the need for the bridge replacement project. The limited available funding for the bridge replacement project, combined with the challenging project constraints, generated the need for a two phase bridge replacement project solution. The two phase bridge replacement project solution generated the need for a phase one bridge that would be compatible with both the remaining existing Puyallup River Bridge and the phase two bridge. Finally, the unique requirements of the phase one bridge created a unique and innovative precast/post-tensioned concrete cable stayed bridge that is both a highly functional and a signature bridge structure. See Figure 14 for a view of the final 4-dimensional global structural analysis model.

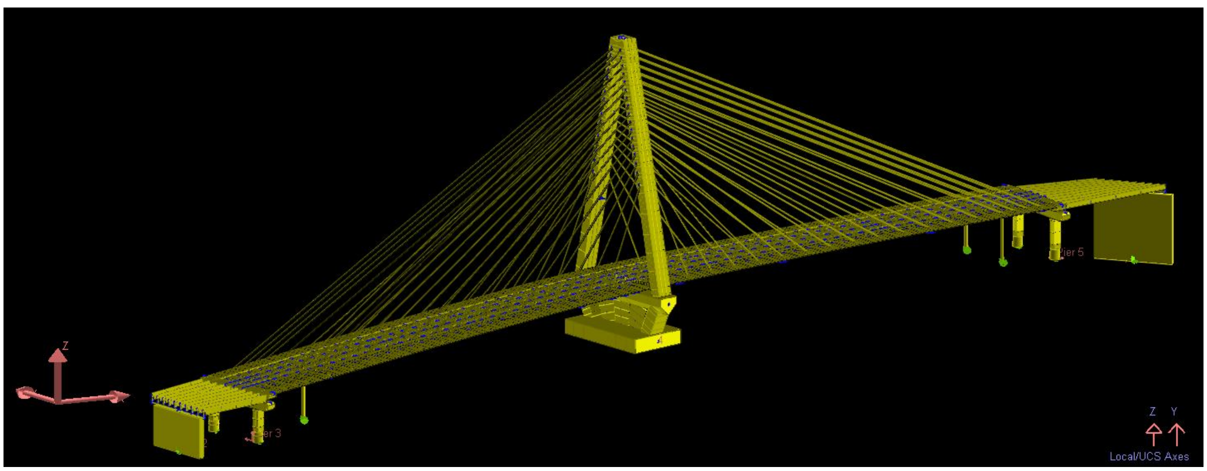

Fig. 14 View of Global Cable Stayed Bridge Analysis Model

This project has created a unique signature cable stayed bridge for the City. A significant amount of analysis, design, industry research, and careful attention to both design details and project specifications has created a complex cable stayed bridge that is not only highly constructible for a contractor but also highly durable for the City. The final new Puyallup River Bridge will be something that all stakeholders involved can be proud of, and will serve as an example for future major bridge replacement projects.

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