

REHABILITATING PRECAST CONCRETE HISTORY USING DESIGN-BUILD DELIVERY

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ABSTRACT

Bridges are an integral part of our transportation system and history. The average age of a bridge in the United States is 42 years, but some are much older. The Port Approach of the Murray Morgan Bridge in Tacoma, Washington is more than 55 years old. The bridge is also historically significant for the city of Tacoma because it was one of the first all precast concrete bridges built in the United States. In 2010, the City of Tacoma began a major \$50 million design-build rehabilitation project for the entire bridge that included a significant amount of precast/prestressed concrete repairs and seismic retrofits.

First the details of the precast/prestressed concrete repairs and seismic retrofits used on the project will be discussed. Some of the repairs/retrofits included prestressed girder strand splicing, hollow column jacketing, and seismic restraint. Additionally some of the methods used to accelerate the project using the design-build delivery method will be discussed. This will include the development of the proposal and the steps that were taken to minimize project risk, while still allowing cost saving innovations, utilizing a careful balance of prescriptive and performance specifications. Finally, the things that went well and lessons learned during the project will be summarized.

Keywords: Design-build, Bridge, Rehabilitation, Performance, Seismic, Retrofit

INTRODUCTION

PROJECT HISTORY

The original Port Approach was constructed in 1911 along with the rest of the Murray Morgan Bridge that crosses the Foss Waterway in Tacoma, Washington. While the City Approach and Center Truss Spans were constructed of built-up steel truss members on concrete footings and concrete river piers, the Port Approach was constructed entirely of timber, primarily because of the abundance and economy of timber in the northwestern part of the United States in the early 20th century. However, after 40 years of service, deterioration of the timber superstructure and substructure had taken its toll and the Port Approach required replacement.

In the early 1950s the Washington Department of Transportation (WSDOT) acquired ownership of the 11th street the corridor. This acquisition added the bridge to the state highway system as part of State Route (SR) 509. Since the bridge was now a critical part of a state route and replacement of the deteriorated Port Approach was warranted, WSDOT decided that a full replacement of the Port Approach was the best option. At that time, in the early 1950s, the recent development of precast concrete components for bridges allowed for a unique innovation to be used for the replacement approach structure. The timber Port Approach was replaced with one of the first all precast concrete bridges in the United States (see Figure 1).



Fig. 1 Precast Port Approach of the Murray Morgan Bridge

The innovative new Port Approach was roughly 900 feet long by 73 feet wide. It featured seven precast concrete girder spans that total 670 feet in length. The substructure consisted of concrete bents, each of which is made up of eight hollow precast/prestressed concrete piles and two sections of precast concrete caps that were all grouted together to form the support bents. The superstructure consisted of concrete spans, each of which is made up of 16 precast/prestressed concrete deck girders. The concrete deck girders were post-tensioned together at five diaphragm locations in each span. The roadway was a 2-inch-thick asphalt wearing surface that covers roughly 33,500 square feet (sf) of deck area, and the sidewalks were raised precast concrete curb segments that are anchored to the concrete deck girders. Prefabricated railings were bolted to the curb segments to complete the prefabricated bridge. The only cast-in-place concrete used was for the abutment and approach wingwalls. The use of precast/prestressed concrete significantly shortened the on-site construction time needed to build the new Port Approach, and it was one of the first uses of Accelerated Bridge Construction (ABC) techniques using precast concrete elements.

Unfortunately, after more than 50 years of service in a marine environment, the Port Approach was beginning to show signs of deterioration. Cracking and spalling were seen in many of the high stress areas, such as at the tops of the hollow precast concrete columns (see Figure 2), and in areas regularly exposed to weather, such as exterior girders. Also, a few of the high strength post-tensioning rods that ran through the precast superstructure diaphragms had failed at the anchorage locations on the fascias of the bridge.



Fig. 2 Deteriorated Port Approach Hollow Precast Concrete Columns

Despite the deterioration in the Port Approach, it was still fully functional. Unlike the steel truss portions of the bridge that required load restrictions, the precast/prestressed concrete Port Approach did not have any load posting restrictions.

PROJECT FUNDING

With the construction of a new SR 509 corridor and a new bridge located about one-half mile south of the Murray Morgan Bridge, the ownership of the bridge was to be turned over to the City of Tacoma (City) in 1997. The Turn-Back Agreement between the City and WSDOT called for the state to rehabilitate the bridge before turning it over to the City. As WSDOT began pre-construction inspections and repair design plans, it became evident that the necessary repair work needed would cost much more than WSDOT had anticipated. The bridge was considered to be unsalvageable (primarily due to the deterioration of the steel truss spans), and WSDOT amended the Turn-Back Agreement to be able to replace the bridge as an option, as opposed to just rehabilitating the bridge. Then in 2007, WSDOT closed the bridge to traffic due to deterioration of the main members in the steel truss spans.

In 2008 a feasibility study was conducted to evaluate the City's options for re-opening the bridge. This study followed the 2003 Tacoma Central Business District to Port Connection Study₁ that studied the local transportation region and 11th Street corridor, but specifically focused on options for the bridge. The anticipated cost to replace the entire bridge with a replica bridge was \$160 million, while the cost to replace the bridge with a non-replica bridge was \$135 million. With WSDOT only willing to contribute roughly \$35 million to the project as part of the Turn-Back Agreement, the City needed to search for other options for re-opening the corridor. The City needed to re-open the corridor because the closing of the bridge significantly constricted commerce and industry, greatly increased commute times, and strained fire/Emergency Medical Service (EMS) response times. The 2008 Feasibility Study₂ found that the bridge could be rehabilitated in phases over time for a total anticipated cost of between \$77 million and \$80 million, significantly less than the replacement cost.

In 2009, the final Turn-Back Agreement was executed, and the City assumed ownership of the bridge. WSDOT gave the City a final total of \$38.7 million dollars as part of the agreement to rehabilitate the bridge. This amount was sufficient to execute the first phase of work that would conduct the critical repairs needed to re-open the bridge to traffic. The second phase would include necessary additional repairs, and the third phase would seismically retrofit the bridge to current seismic standards.

DESIGN-BUILD

In early 2010, the City Council unanimously approved the first phase of the bridge's rehabilitation using the design-build delivery method. The Request for Proposals (RFP) contained a number of performance and prescriptive specifications that also had to be flexible with regard to the variable scope of work for the entire project. The design-build project had two basic scopes of work. The first was the Base Program scope of work, which contained the work elements necessary to return vehicular traffic to the bridge without load restrictions. This scope of work represented the first phase of the rehabilitation project. Of the roughly \$32 million needed to execute the first phase of work, the Port Approach was estimated to need roughly \$1.7 million only. This relatively low cost of repair work attests to

the durability of precast/prestressed concrete compared to the durability of the steel approach and truss span structures.

The second scope of work was the Supplemental Program. This was the second phase of the rehabilitation project and contained the additional repair elements that were necessary to complete the rehabilitation of the bridge. The Supplemental Program was not fully funded at the release of the Request for Qualifications (RFQ) that preceded the RFP, and needed roughly \$15 million to execute the second phase of work. Again, the durability of the Port Approach was demonstrated: It was estimated to need roughly \$1.9 million only during this phase.

The RFP contained the necessary details to complete both scopes of work and contained a mix of prescriptive and performance specifications. The prescriptive specifications were used as the basis of the performance specifications for the larger elements of work, and this approach gave the design-builder controlled flexibility to develop innovative, cost-saving, and time-saving solutions. Additionally, the RFP contained provisions for “Unknown Major Deficiencies” in which certain types of work were identified and unit prices were established by the design-builder as part of its price proposal. For the concrete elements on the project, unit prices were established for concrete patching and concrete girder strand repair. This approach allowed the design-builder to establish a unit price that would be paid upon approval of the change during the project, and gave the City an understanding and baseline regarding potential additional costs related to Unknown Major Deficiencies.

During procurement, the City acquired the additional funding needed for the Supplemental Program work, thus allowing the first two phases of the rehabilitation project to be executed using the design-build delivery method. The City was able to successfully select a design-builder that rated excellent for both its technical proposal and its price proposal. Additionally, the RFP was shown to be a well-written document that proposers were able to clearly understand, because only two amendments were needed during procurement and all price proposals submitted were within 3 percent of the owner’s estimate.

PRECAST/PRESTRESSED CONCRETE REPAIRS

HOLLOW PRECAST/PRESTRESSED CONCRETE COLUMN REPAIRS

Each hollow precast/prestressed concrete column measured 3 feet in diameter and had 5-inch-thick walls. The cracking and spalling that was located at the tops of many of the columns where they connected to the concrete caps presented the unique problem of how to repair the columns without excessively damaging them during repair. One concern was how to prepare the outer surfaces of the hollow columns while not damaging or spalling the inner surfaces of the hollow columns, because such damage would definitely reduce the capacity of the columns. Additionally, the columns were found to not meet current seismic requirements (common in older bridges) and this deficiency also had to be addressed during the project.

The contractor and designer found a unique solution to address both the top-of-column deterioration and the seismic inadequacy of the columns in one repair method. This repair involved filling the tops of the concrete columns with structural foam and then wrapping the tops of the columns with a steel jacket. The two-component rigid polyurethane system of poured structural foam specified for the repair was a 2.0 pound (lb.) density foam that when fully cured had a compressive strength of about 300 pounds per square inch (psi). Note that as the density of the foam increases, the compressive strength can also be increased up to a maximum of about 1,200 psi for the product used on the project. The installation method for the structural foam was first proven in a mock-up test on the project site (see Figure 3) in order to properly calibrate the foam installation to on-site conditions.



Fig. 3 Mock-Up Structural Foam Field Test

Once the installation procedure was properly calibrated to the proper installation temperature, foam chemistry, flow rate, and incremental placement time increments, the production installation of the structural foam was ready to begin. A small, 5-inch hole was drilled in each of the hollow columns about 10 feet below the concrete cap. An inflatable, plastic, heat-resistant plug was inserted into the hole and inflated, sealing off the top 12 feet to 14 feet of the hollow columns. The 5-inch hole was patched and two small, 1-inch-diameter holes were drilled in each column—one at the top of the column and the other about 6 feet below the concrete cap. These holes allowed for the multiple lift placement of the structural foam in the hollow columns. The two-part mixture was first pumped through the lower hole and allowed to set, so that the lower foam was secured to the interior walls of the hollow columns. The mixture was then pumped through the upper hole to ensure that the entire top portion of the column was filled with the structural foam.

After the foam fully cured (after about 72 hours) the inner surfaces of the hollow columns were now supported and the outer surfaces could be properly chipped and prepared for spall repairs. Since all of the hollow column tops were receiving steel jackets, it was decided to use the grout between the steel jacket and column as the spall repair grout, instead of repairing the columns with one type of stiff, self-supporting structural repair mortar and then installing the jacket with a fluid grout. After all of the deteriorated material was removed from the tops of the outer hollow column surfaces, the steel jackets were installed and were externally supported, and the void between the jacket and outer surface of the hollow column was sealed. A flowable grout was pumped in through the bottom valve of the steel jacket. Once clean grout flowed out of the upper valve, both valves were closed and the grout was allowed to cure to full strength (see Figure 4 for completed repair).



Fig. 4 Completed Hollow Precast/Prestressed Column Repair

The jackets were finally painted in a Washington Gray color paint to protect the steel from corrosion. The efficient repair restored capacity to the hollow precast/prestressed concrete columns. In addition, the use of the exterior steel jackets and interior structural foam increased the confinement capacity of the columns and added additional seismic capacity to the columns in the column plastic hinging zones.

PRECAST CONCRETE CAP REPAIRS

Each of the precast concrete caps was made up of two precast segments that are 30 feet long by 4.5 feet wide by 2.5 feet high. They are connected to each other via a shear key and grouted connection at the center of the bent to have a total length of 60 feet. The deterioration of the concrete caps was minor, consisting of random minor to moderate

cracking and edge spalling. The cracks were repaired using traditional non-shrink, two-part epoxy resin that achieved a minimum compressive strength of 10,000 psi and had minimum bond strength of 500 psi using the drill port injection method. The spalls without exposed reinforcing steel were cleaned of all loose and deteriorated material, and a stiff, non-shrink structural repair mortar that achieved 4,000 psi compressive strength after seven days was formed back over the spalled area.

The spalls with exposed reinforcing steel were required to be chipped a minimum of $\frac{3}{4}$ inch behind the reinforcing steel and to sound concrete. Oxidized reinforcing steel had to have all rust removed from the reinforcing bars within the spalled area. Once the surface was properly prepped, the builder applied a traditional, stiff, non-shrink structural repair mortar that achieved 4,000 psi compressive strength after seven days. The repair was finished to the shape of the original concrete cap.

PRECAST/PRESTRESSED CONCRETE GIRDER REPAIRS

The precast/prestressed concrete deck girders were typically 74 feet long by 3.5 feet deep and 3.75 feet wide at the top flanges. The deterioration in the deck girders primarily consisted of spalling at the top flange shear key connections and spalling in the bottom flanges of the exterior girders (see Figure 5). A number of the bottom flange spalls had exposed and severely deteriorated the prestressing strands.



Fig. 5 Damage to Prestressed Girder Top and Bottom Flanges

The project RFP permitted the use of strand couplers, and also gave flexibility to the contractor in deciding which deteriorated strands needed to be coupled and subsequently

have the individual strand capacities restored. Because the lane configuration of bridge was changing and a permanent traffic barrier would now be separating the exterior pedestrian sidewalks from the vehicular traffic lanes, the live loads on the deteriorated exterior girders would be significantly lower than the original configuration design live loads (which did not have a permanent barrier that separated pedestrian and vehicular traffic). This meant that, of the 25 broken strand locations discovered, only 12 broken strands required strand couplers to restore strand capacity and girder capacity. This saved both time and money while still allowing the precast/prestressed concrete girders to meet the minimum required inventory rating factor of 1.25 that was specified in the RFP.

The top flanges shear key connection spalling of the precast/prestressed concrete deck girders were repaired by chipping a minimum of $\frac{3}{4}$ inch behind the exposed reinforcing steel to sound concrete. The oxidized reinforcing steel had to have all rust removed from the reinforcing bars within the spalled area using abrasive blasting or compressed air pressure flushing, which had to produce a minimum 100 psi pressure at the nozzle. Pressure washing was not permitted. Once the surface of the spalled area was properly prepped, the builder applied a traditional stiff non-shrink structural repair mortar that achieved 4,000 psi compressive strength after seven days (see Figure 6 for a completed top flange repair).



Fig. 6 Repaired Precast/Prestressed Girder Top Flange

PRECAST/POST-TENSIONED CONCRETE DIAPHRAGM REPAIRS

The precast/post-tensioned concrete diaphragms were cast with each precast/prestressed concrete deck girder, and each set of diaphragms was post-tensioned together using two $\frac{3}{4}$ -inch-diameter post-tension rods that were stressed to 35 kips each. The post-tensioning rods

were coupled at the centerline of the bridge. Since the deck girders were post-tensioned together roughly every 15 feet, the concrete deck girders acted as a monolithic unit to resist live loads as opposed to individual girders. This greatly increased the efficiency of the superstructure but was contingent on maintaining the post-tensioning force in the diaphragms.

It was discovered during previous and pre-construction inspections that 5 of the 70 diaphragm end anchorage locations had failed. The post-tensioning rods were grouted; however, it was difficult to determine the extent of post-tensioning loss in the diaphragms. Therefore, it was decided to build an exterior concrete diaphragm block that would subsequently be post-tensioned and grouted. This would ensure that any lost post-tensioning force at the fascia locations where the exterior anchorages had failed would be restored. A 3-inch-diameter hole was cored through the girders and a 1-inch-diameter high strength bar was threaded through the newly constructed concrete diaphragm block and cored holes. The high strength bar was tensioned to 70 kips and grouted using a traditional pre-packaged thixotropic grout (see Figure 7 for a completed precast/post-tensioned diaphragm repair).



Fig. 7 Repaired Precast/Post-Tensioned Concrete Diaphragm

CAST-IN-PLACE CONCRETE REPAIRS

MAIN RIVER PIER CAP STRENGTHENING

The four main cast-in-place concrete river piers are roughly 110-feet-tall and measure about 80-feet-long by 16-feet-wide at the base and about 63-feet-long by 10-feet-wide at the pier

cap (see Figure 8 for an overall view of the concrete river piers). The main river pier columns and cap were in good condition after roughly 100 years of service with just minor cracking and spalling. The web walls of the main river piers require repair due to moderate to heavy spalling, but those repairs are scheduled to be completed as part of the phase three seismic retrofit of the bridge that will be completed in the next phase of the project. However, it was decided to enhance the seismic capacity of the pier caps during the second phase of the rehabilitation project to provide intermittent protection between the superstructure and substructure connection points.



Fig. 8 Overall View of Concrete Main River Piers

Calculations showed that the original concrete pier caps could not resist the current AASHTO level earthquake design loads at the main lift tower bearing locations. However, to prevent future rework, the cap strengthening needed to be compatible with a variety of possible seismic retrofit options. It was decided that the rehabilitation of the pier caps would consist of primarily adding radial reinforcing steel bars and a radial steel cap casing at the bearing locations to achieve the necessary seismic capacity at each main lift tower bearing location.

The strengthening construction of the main river pier caps started by chipping back 6 inches of the existing concrete cap at the bearing locations, so that the new steel reinforcing bars could be tied in with the existing reinforcing steel bars. Two layers of #6 Grade 60 steel reinforcing bars were placed and spaced at 6 inches on-center around the existing pier caps at the bearing locations. The ½-inch-thick ASTM Grade 50 radial steel cap casing had steel studs welded to the interior of the casing to ensure composite action with the new cap

concrete. The casing also acted as the concrete form for the cap strengthening, so no additional formwork was needed.

To complete the seismic retrofit, a 2 ½-inch-diameter ASTM F1554 Grade 55 galvanized tie rod was installed through a 3-inch core hole that was drilled through the pier cap. The rod was designed to resist the demand radial forces generated by the lateral seismic demands at the bearing location. Finally, the retrofit was completed by placing a standard concrete mix that achieved a 28-day design strength of 4,000 psi. See Figure 9 for the final main river pier cap seismic retrofit.



Fig. 9 Main River Pier Cap Bearing Seismic Retrofit

CAST-IN-PLACE AND PRECAST/PRESTRESSED CONCRETE SEISMIC RETROFITS

CAST-IN-PLACE CONCRETE ABUTMENT SEISMIC RETROFITS

New seismic design requirements meant that the movable bearings location at the Port Approach Abutment required seismic enhancement to meet current seismic design requirements. Calculations showed that the beam seats did not have sufficient width to meet seismic design/detailing requirements. Therefore, the designer-builder decided to build out the abutment beam seat by building an additional 9-inch-wide corbel on the face of the abutment cap. To handle the transverse requirements, eight new shear keys were built on top of the new corbel.

Construction of the retrofit started with drilling $\frac{3}{4}$ -inch-diameter holes 8 inches into the existing abutment cap and stem wall. Three layers of #5 Grade 60 steel reinforcing bars that were spaced at 18 inches on-center were set in the holes and secured using a standard two-part epoxy resin. The steel reinforcing bars and wood forms were set for the corbel, and the corbel was completed by placing a standard concrete mix that achieved a 28-day design strength of 4,000 psi. Once the corbel achieved strength, the forms for the shear keys were built and the same concrete as for the new corbel was placed for the new shear keys (see Figure 10 for the completed seismic retrofit). The RFP gave controlled flexibility to the design-builder that allowed it to develop an acceptable and simple solution to meet current seismic design requirements.



Fig. 10 New Concrete Girder Stops at the Port Approach Abutment

PRECAST/PRESTRESSED SUPERSTRUCTURE SEISMIC RESTRAINER RETROFITS

Similar to the Port Approach Abutment, two bents in the Port Approach structure supported locations that had expansion joints. The flexibility of the RFP requirements allowed the design-builder to design and build a simple system of longitudinal seismic restrainers to meet the seismic design requirements for the intermediate bents at the expansion joints.

The seismic retrofit called for three pairs of seismic restrainer rods to be installed in each span that was adjacent to an expansion joint intermediate bent. The girder rod connection consisted of a $\frac{5}{8}$ -inch-thick ASTM Grade 50 steel plate bracket that was fabricated for each girder connection location. Four 1-inch-diameter connection holes were drilled through the webs at the girder restrainer rod connection locations. Special care was taken to miss the girder vertical reinforcement and longitudinal draped prestressing strands. The girder

restrainer rod connection bracket was then bolted to the precast/prestressed concrete deck girder. The cap rod connection required a 2-inch-diameter core hole to be drilled through the opposite face of the cap at the correct orientation. A new concrete bearing block and bearing plate were subsequently constructed using a combination of #4 Grade 60 steel reinforcing bars, a 28-day design strength 4,000 psi concrete, and an ASTM Grade 50 1-inch-thick steel bearing plate to make the cap rod connection point.

The 3/4-inch-diameter F1554 Grade 105 seismic restrainer rods were secured between the girder and cap connections with ASTM A563 Grade DH double jam nuts at each end. A 1-inch gap between the double jam nuts and the cap rod connection bearing plate was provided, so that the seismic restrainers would not be engaged during normal thermal movements of the bridge. The seismic restrainer rods were sealed in a 1-inch-diameter P.V.C. sheathing in order to provide long-term protection to the high strength rods (see Figure 11 for the newly completed longitudinal seismic restrainers).



Fig. 11 New Port Approach Longitudinal Seismic Restrainers

LESSONS LEARNED

The project as a whole was executed very well. No major problems occurred during the design or construction of the project; however, there were many smaller problems (typical of large design-build projects) that had to be managed by the stakeholders involved.

One incident that occurred during construction was the sudden rupture of a high-strength steel post-tensioning bar in one of the new precast/post-tensioned concrete diaphragm. The

bar ruptured at the location of the bar coupler, and the subsequent investigation revealed that the steel post-tensioning bar ends were not properly seated in the bar coupler before the stressing operation began. The design-builder took many safety precautions during stressing operations and throughout the project so no injuries resulted from the incident. However, it is important to emphasize that a large amount of force is contained in high-strength steel post-tensioning bars. Bar coupler locations need to be properly marked to ensure that the steel post-tensioning bar ends are properly seated in the bar couplers. The bar and coupler were replaced so that the final permanent work could be completed.

It is important to note an RFP requirement regarding the preconstruction inspection that worked very well. The design-builder was required to complete a pre-construction inspection before beginning work on an element of the bridge. The pre-construction inspection confirmed the deficiencies listed in the RFP documentation, and was also intended to document deficiencies that were not in the RFP documentation. If the undocumented deficiencies affected the final rating of the bridge, then they needed to be repaired in accordance with the Unknown Major Deficiency provisions of the RFP. In the Port Approach, roughly \$300,000 worth of additional work was discovered during the pre-construction inspection. The owner was able to work with the design-builder early on in the project to include the additional work efficiently in the first set of Issue for Construction IFC drawings for the Port Approach. Having very early knowledge of potential and subsequent additional work (which is common on rehabilitation projects) was a very important feature that contributed to the success of the project, and allowed for repairs to the Port Approach to be completed in the fall of 2012 (see Figure 12).



Fig. 12 Rehabilitated Port Approach

CONCLUSION

The total project was able to be completed within the City's project program budget of \$57 million. The bridge was re-opened to traffic on February 1, 2013, and now has a sufficiency rating of 81 (up from 2 in 2007) and the controlling inventory rating factor is 1.30 (up from 0 in 2007). The service life of the bridge was also extended an additional 75 years with the repairs completed during the design-build project, and the maintenance and life cycle costs of the bridge were also substantially reduced.

The careful balance of prescriptive and performance specifications in the RFP and the durability of one of the first all precast concrete girder bridges in the United States allowed for many tried-and-true methods of precast/prestressed, precast/post-tensioned and cast-in-place concrete repairs to be used in innovative, cost-saving, and time-saving ways during the design-build rehabilitation project. Additionally, the use of Unknown Major Deficiency and pre-construction inspection clauses allowed the City to minimize the project risk and allowed the project to be completed in a two-year period. The final product is something that all stakeholders involved can be proud of, and will serve as an example for future rehabilitation projects that will be completed using the design-build deliver method.

REFERENCES

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