STRENTHENING OF THE SR 520 EVERGREEN POINT FLOATING BRIDGE BY EXTERNAL POST-TENSIONING

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ABSTRACT

The SR 520 Evergreen Point Floating Bridge serves as a direct route for over 125,000 daily commuters between Seattle and East King County. After a January 1993 storm severely damaged the bridge, WSDOT began a preservation program to prolong the life of the bridge and prevent further damage. KPFF Consulting Engineers was selected to complete the work. Post-tensioning was selected for the long-term preservation of the bridge which involved installing 3,600-foot-long tendons. Tendons of this length had never before been used in bridge post-tensioning. This paper will discuss the floating bridge dynamic analysis; rehabilitation studies, including the full scale "mock-up" constructibility verification study program; final design implementation; and lessons learned from the installation of the external post-tensioning.

Keywords: Floating Bridge, Constructibility / Verification Program, Friction Losses, Long Post-Tensioning Tendons, Repair And Rehabilitation, Long Span Bridges, Post-Tensioning Technology, External Post-Tensioning, Post-Tensioning Anchorages, Unique Design

INTRODUCTION

The Evergreen Point Bridge crosses Lake Washington connecting Seattle with East King County. The floating portion of the bridge is comprised of 33 concrete pontoons forming a 7,578-foot-long structure. Four traffic lanes and one safety walk lie directly on the top slab of the box-shaped pontoons. The pontoons experienced cracking as a result of severe storm wave loading conditions, allowing water to seep into some cells of the pontoons, requiring that they be pumped out on a regular basis.

The Washington State Department of Transportation (WSDOT) initiated a program in 1997 to strengthen the bridge and prevent further storm damage. This paper presents the floating bridge dynamic analysis, rehabilitation study, final design implementation, and the construction of the external post-tensioning.

Driving over the Evergreen Point Floating Bridge can be unsettling in a storm. The bridge rocks and wave sprays hurl over the roadway. During the January 20, 1993, Inauguration Day Storm, the bridge was severely damaged—tearing several of the mooring cables that anchor the bridge to the lake floor. The box-shaped concrete pontoons cracked and water began leaking into some of the cells, eventually requiring that they be pumped out twice weekly.

Spanning 7,600 feet, its length, along with the three-mile long fetch of open water that lies to the south, make the bridge highly susceptible to wind-driven waves. As the longest floating bridge in the world, the 37-year-old structure crosses Lake Washington connecting Seattle with the suburbs of East King County for over 125,000 daily commuters.

Fig. 1A Aerial view of SR 520 bridge.

Fig. 1B SR 520 during a storm.

In 1997, WSDOT began a program to rehabilitate the bridge with the goal of preventing further damage and extending its life. They selected KPFF Consulting Engineers as the prime consultant to design the bridge strengthening and repair. By using external post-tensioning to strengthen the continuous concrete box structure, the design incorporates 3,600-foot-long post-tensioning tendons, substantially reducing costs and minimizing traffic impacts. These tendons are nearly twice as long as any tendons previously used in the world.

CRITERIA

Meeting the rehabilitation criteria for the bridge posed many challenges. The bridge had to remain open with minimal closures limited to nights and weekends, requiring that work proceed in severely confined areas with moving vehicles overhead. Inclement weather restricted construction to the months of April through September over a 2-year period. The structural performance criteria precluded concrete cracking during a 20-year storm event and prevented reinforcing steel yielding during a 100-year event. Finally, the existing freeboard had to be maintained.

It was imperative that the rehabilitation did not further reduce the freeboard since it could jeopardize bridge operations. Previous additions of an asphalt wearing surface and jersey barrier shaved 9 inches off the original design freeboard. Also, the addition of a precast concrete wave deflector caused a 3-inch list on the south side. All of these elements contributed to the south side of the bridge sitting 12 inches lower in the water than specified in the original design.

ELEMENTS OF THE STRUCTURE

Four traffic lanes and one safety walk lie directly on the top slab of the box-shaped pontoons. Located in the middle of the bridge, a draw-span section divides the 7,600-foot-long floating bridge into two parts. The west half is 3,160 feet long and the east half is 3,600 feet long. Mooring cables, which anchor the bridge, extend from the pontoons to the floor of Lake Washington approximately 200 feet below. 360-foot-long individual pontoons are bolted together with 110 high-strength, 1 1/2-inch diameter bolts that are pretensioned to a force of 140 kips each. At several locations, some of these bolts had yielded during the Inaugural Day Storm.

The pontoons that make up the floating bridge are typically 360 feet long by 60 feet wide and 15 feet deep. The pontoons are subdivided into 15-foot by 15-foot by 15-foot cells that have watertight bulkheads every 30 feet in each direction. Access to the interior of the bridge pontoons is permitted through hatches in the top deck. These hatches are typically located immediately adjacent to the roadway on a narrow walkway. It is difficult and potentially dangerous with traffic present to move materials through these hatches. Fortunately, at the east and west ends, the roadway is elevated off the pontoon deck and the deck widens at the center draw span. Here, access through the hatches improves, accommodating a staging area for construction equipment.

ANALYSIS AND DESIGN

A preliminary analysis and feasibility study established the strengthening requirements and a construction scheme that complied with the criteria. This study included conducting a hydrodynamic analysis, developing demand/capacity curves, and investigating 13 design alternatives.

Hydrodynamic Analysis: The effects of wind and waves were combined in the analysis. The Glosten Associates, Inc., and KPFF developed probabilistic combinations of biaxial structural bending demand curves for the bridge, assuming exposure for 20- and 100-year return interval storms. The hydrodynamic analysis was based on a frequency domain approach—considering both the short-crested and incoherent nature of the wave spectrum imposed on this lengthy floating structure.

Demand/Capacity Curves: KPFF developed capacity curves for yield-moment and cracking-moment biaxial bending based on various levels of prestress. These figures were plotted with the hydrodynamic demand values to determine the prestress level required to meet the design criteria. It confirmed that the original prestress of 560 psi was insufficient to meet the criteria requiring no cracking in a 20-year storm and no yielding of steel reinforcing in a 100-year storm. Additional post-tensioning of 390 psi was required to comply with the criteria. To achieve this, an additional compressive force of 8,000 kips needed to be applied to the bridge.

The project team evaluated thirteen alternatives for configuring the post-tensioning force, factoring in any necessary mitigation if additional weight was imposed on the bridge. The selected post-tensioning configuration included these components:

- Sixteen tendons with fifteen 0.6-inch diameter strands each that are centralized in the pontoon cells.
- Full length tendons: 3,160 feet to 3,600 feet long.
- Steel "spider" frame anchorages.
- Replacement of the south side concrete wave deflector.
- HDPE duct.
- Cementitious grout.

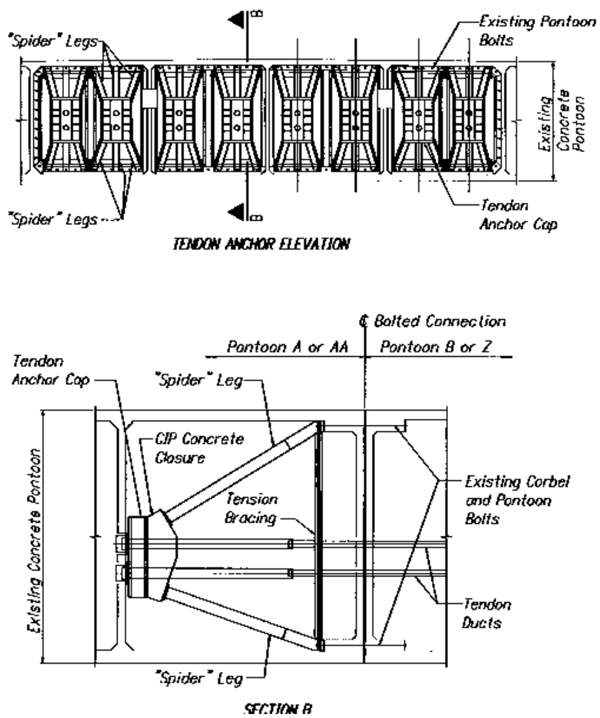

Fig. 2 The top sketch is a full width section of the bridge at the tendon anchor location. The section below shows the "spider" anchor.

Many benefits of the selected design configuration ensued:

- Centralized tendons eliminate conflicts with most of the existing utilities.
- Full length tendons omit the intermediate anchors and the associated costs of weight mitigation to maintain the existing freeboard. This factor alone reduced the rehabilitation cost by more than $5 million. It also eliminates the difficulty of erecting the anchors and installing the post-tensioning strands with no shoulder for construction access to the hatches, thereby substantially reducing construction-related traffic impacts to one of the Northwest's most vital transportation corridors.
- Steel anchors reduce weight and mitigation costs to maintain the freeboard.
- Replacing the concrete wave deflector with a lightweight steel wave deflector maintains the existing freeboard on the critical south side of the bridge.
- HDPE duct reduces the friction forces that arise when steel tendons are installed, thereby eliminating the risks associated with potential construction delays.

VERIFICATION PROGRAM

Although the tendons realized substantial cost savings and minimized traffic impacts, tendons measuring 3,600 feet in length had never been constructed anywhere in the world. WSDOT and KPFF agreed that the project benefits gained from this novel approach warranted a full-scale mockup test in order to demonstrate constructibility to the construction community. The verification program took place at a nearby airport. Contractors were invited to develop post-tensioning methods and to demonstrate how to install the tendons. Selected to implement the program, AVAR Construction Systems installed the strands one at a time, pulling them through with a "tugger" wire rope. Retrieving the tugger wire rope was accomplished with a second wire rope pulled through along with the strand. It took several attempts to iron out the details on how to rig the strands and wire ropes; in the end, an effective method was developed and documented. This effort, prior to construction, resulted in reducing risks to contractors and commanding a construction bid $2.3 million lower than the estimate.

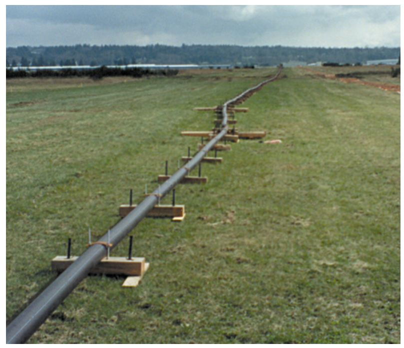

Fig. 3 Test site of the constructibility verification program.

EXECUTING THE DESIGN: YEAR 1

Mowat Construction won the bid to perform the post-tensioning and weight mitigation at $6.5 million. During the first year of construction, 3,600 holes were core-drilled through watertight bulkheads; 108,000 lineal feet of duct was installed; utilities were relocated; and the anchor fabrication was completed.

Accessing the pontoons, while keeping the bridge open, required extreme care. The project team and work crew accessed the interior of the bridge through hatches on the deck of the safety walkway. Located approximately 160 feet apart, the hatches measure 2 feet by 4 feet. Any work performed inside the pontoons, including assembly and fabrication, required entering through these hatches.

The contractor installed a temporary catwalk along the side of the bridge for construction access. During the day, the core drilling, utility relocation, and duct installation crews worked inside the pontoons. At night, one lane of the bridge was closed to allow resupply of the day crews and to remove cores from the cells. A typical day's work for the core drillers involved completing fifty 6-inch diameter by 6-inch long cores. Core hole locations were checked by sighting down the alignment with a laser beam and measuring to the center of each hole. As a precaution, work was halted during the storm season from November to April to ensure that the watertight integrity of the bridge would not be compromised during a storm.

Fig. 4 Core driller removing cores from cells.

The duct installation followed the core drilling. High-density polyethylene (HDPE) ducts were installed in 30-foot lengths and sealed in the 6-inch core holes with an elastomeric sealant. Watertight plugs were installed in the ends of ducts at the end of each workday.

Fig. 5A Watertight plugs are installed in the core holes maintaining integrity.

Fig. 5B Post-tensioning tendons run through the pontoon cells.

At changes in the direction of the tendons, the HDPE pipe was coupled to a steel pipe to form the radius of the angle change, using a grooved coupling system. Since the coupling required a groove in both the steel and HDPE pipe, the assembly was tested to a pressure of 250 psi water.

Modifications to the utilities inside the bridge included relocating fiber optic conduits, reinstalling fiber optic lines, and relocating four 12,470 volt transformers.

EXECUTING THE DESIGN: YEAR 2

The second year of construction included the anchor and strand installation, post-tensioning, grouting, weight mitigation, and bridge reballasting.

SPIDER ANCHORS AND STRAND INSTALLATION

An optimized steel anchor assembly, with multiple lightweight legs, delivered the post-tensioning force to the pontoon's cross-section, reducing the weight penalty to acceptable levels. The multi-legged nature of the assembly led to its name, "spider anchor." The "spider" anchor has a central bearing transfer assembly that transfers the force of two tendons to six legs, braced against the corbel that anchors the existing pontoon bolts. A concrete closure pour was cast between the legs and the tendon anchor cap assembly to provide added fit-up tolerances. The concrete pour also reduced welding in the confined spaces of the pontoon.

Fitting the anchors to the existing pontoons imposed a formidable challenge, requiring separate surveyed dimensions for each of the 32 tendon anchors. As luck would have it, several of the anchor legs were fabricated out-of-tolerance. KPFF reanalyzed the stresses in these assemblies within 24 hours to ensure that they were within acceptable limits. In many cases, the tensioning sequence was revised to limit the differential force to a maximum of 75 kips (out of 615 kips maximum) between the two tendons anchored on one anchor.

Fig. 6 Spider anchors are assembled inside the pontoon cells.

The wide flange top and bottom legs of the tendon anchors were tied together with tension braces. Slider bearings were installed to ensure that the normal force exerted by the anchor at the pontoon interface did not restrain the spreading force in the braces. The required movement of the bearings was only 0.1-inch, but the friction needed to be low to control bearing stresses. After conducting a test at the University of Washington, woven Teflon and stainless steel bearings were selected because they can sustain high bearing stresses.

At the anchor bearing, the stresses in the existing concrete corbels of the pontoon are 6 ksi. KPFF created 3-dimensional solid finite element models of the corbel and pontoon connections to determine the concrete stresses. In some locations, the 1 1/2-inch diameter pontoon bolts were replaced with smaller bolts to reduce the bearing force on the corbels. Since the original design of the bridge called for a 3,650 psi concrete mix, KPFF and Construction Technology Laboratories (CTL) performed an extensive concrete testing program of the corbels. The tested concrete strengths ranged between 10,000 psi and 12,000 psi. This allowed higher design bearing stresses in the concrete.

After the anchor legs and anchor cap were installed, the concrete anchor cap closure was placed. The prepackaged, shrinkage-compensated, and pumpable concrete mix had a 28-day strength of 7,000 psi.

The tendon strands were installed as performed in the mockup test with some modifications. The changes involved delivering the strands on reels to remove the twist that occurs in strand packs, placing the reel on a turntable with a brake to control the rate of installation, and installing the strands two at a time.

POST-TENSIONING AND WEIGHT MITIGATION

Another uncertainty in the scheme was the anticipated friction losses in the 3,600-foot long tendons. The work was scheduled to allow for post-tensioning of both ends of the tendons if the friction losses exceeded the anticipated zero wobble coefficient. The post-tensioning was scheduled for two weekends during July and August 1999. The bridge was closed from 10:00 p.m. Friday night to 4:00 a.m. Monday morning on each weekend to complete the post-tensioning and to replace concrete wave deflectors with lightweight steel wave deflectors.

Fig. 7 Post-tensioning operations with 24 feet of elongation.

As the post-tensioning subcontractor, AVAR used four post-tensioning rams simultaneously to complete the job on the west end in 46 hours. The four center tendons were tensioned and work proceeded to the exterior tendons. Two crews of ironworkers worked 12-hour continuous shifts. The post-tensioning of 16 tendons on the west end, each 3,120-feet-long, was completed by noon on Sunday morning, July 18, 1999. The 3,600-foot long tendons on the east side of the bridge were completed in 20 hours.

CTL supplied, calibrated, and monitored load cells on each end of each tendon. The load cells allowed WSDOT and KPFF to quickly determine if the required post-tensioning forces had been achieved. In fact, the total force applied to the bridge was 9,400 kips or 17 percent more than the minimum required to meet the design criteria.

The friction losses in the tendons were consistent with a friction coefficient of 0.2 on the steel pipes at the angle changes and a wobble coefficient of zero. The zero wobble factor is appropriate for external tendons because if the strands rub against the duct, the duct will move laterally and limit the normal force on the duct. If there is no normal force on the duct, there can be no friction and, thus no tendon force losses due to friction. The typical tendon had 25 kips of friction losses (or 4 percent of the jacking force of 615 kips). The total elongation was 24 feet per tendon or 2 miles of total elongation for all the strands. Interestingly enough, when the load cells were checked 24 hours after post-tensioning was completed, the difference in force between the tensioned end and dead end of the tendons was less than 1 percent. Apparently, the friction losses equilibrated throughout the length of the tendons after one day of service.

Grouting the tendons was performed using one injection port at one end of each tendon, and venting at the high and low points during grouting. Each tendon used 7 cubic yards of grout with 200 sacks of cement, taking more than 2 hours to complete.

Completing the project required removal of the precast wave deflector and replacement with a lighter weight steel deflector to assist mitigation of the bridge freeboard loss.

Fig. 8 Installation of wave deflectors.

PROSPERING WITH INNOVATIVE DESIGN

The use of full-length tendons reduced costs for an extensive rehabilitation by over $5 million, while minimizing traffic impacts to one of Seattle's vital links, the Evergreen Point Floating Bridge. A mockup test demonstrated constructibility of the design. Cooperation between WSDOT, KPFF, and the construction community allowed a creative approach to bid well under budget and to be completed on schedule in November 1999.