HEAVY MOVABLE STRUCTURES, INC.
FIFTEEN BIENNIAL SYMPOSIUM

September 15 – 18, 2014

Sheave Trunnion Fatigue and Replacement at Snohomish River Bridge in Everett, Washington

Krishna H. Mehta, P.E.
(Stafford Bandlow Engineering, Inc.)
Scott Snelling, P.E.
(Parsons Brinkerhoff)

NEW ORLEANS FRENCH QUARTER MARRIOTT HOTEL
NEW ORLEANS, LOUISIANA

Introduction

The Snohomish River Bridges are located near the town of Everett, Washington and carry Route 529 over the Snohomish River. There are two bridges, one carries northbound traffic from Everett to Marysville and the other carries southbound traffic from Marysville to Everett. This paper is going to concentrate on the rehabilitation performed on the bridge (529/10W) that carries southbound traffic. This bridge carries two vehicular lanes and a pedestrian walkway over the Snohomish River. The movable portion of the bridge is a tower drive vertical lift span with a length of 180 ft. and a width of 36ft. between the live load supports. The width of the channel is 105ft. The weight of the lift span is approximately 800 kips. The bridge was constructed in 1953.

The above photo shows three movable bridges. The subject bridge, 529/10W Snohomish River Bridge is in the middle and has an enclosed room on top of the tower. A railroad swing bridge is in the foreground. The 529/10E Snohomish River Bridge is in the background.

In the closed position, the movable span provides 35 ft. of clearance for marine traffic. The movable span can be raised 40 ft. to provide 75 ft. of clearance for marine traffic in the fully open position. Bridge operation is controlled from the operator's house on the adjacent 529/10E bridge. The point of operation for both bridges is combined due to the close proximity of the bridges. These bridges are owned and maintained by the Washington State Department of Transportation (WSDOT).

WSDOT performs in-depth inspections of the mechanical and electrical systems of movable bridges on a six year cycle, including disassembly and measurement of key components. Less intensive, routine inspections are performed on an annual basis. As part of an in-depth inspection, special inspections of the main sheave trunnion were recommended due to the original design of the trunnion and the time in service. Through these special inspections cracks in the trunnion fillet were discovered and it was determined that a rehabilitation was required. To perform this rehabilitation, the scope of the rehabilitation was developed based on the marine and vehicular outage requirements, feasibility of reuse of components, redesign required to meet current AASHTO requirements and cost. The rehabilitation consisted of , mechanical work, mechanical support work and electrical work. The work consisted of jacking the counterweight, replacing the sheaves, trunnions, trunnion bearings, ring gears and pinions, counterweight wire ropes, installing temporary dead load plus live load supports and rehabilitating the existing live load shoes, and supporting electrical work. Construction support services were also provided in the form of reviewing shop drawings, installation and alignment procedures, wire rope tensioning report and alignment measurements. As of the writing of this paper the construction is substantially completed and the machinery alignment is on-going.

Timeline

The following timeline summarizes the significant findings and rehabilitation work at the Snohomish River Bridge West – 529/10W:

- **August 2002**: An in-depth mechanical inspection was performed by Stafford Bandlow Engineering, Inc. (SBE), including visual and dye penetrant inspection of the sheave trunnion fillets. The fillet at the trunnion shoulder is a critical area on the sheave trunnions. This area is subject to the maximum stress as a result of the applied load and the stress riser created by the radius at the transition from the journal diameter to the larger sheave fit diameter. In addition, the trunnions are subject to complete stress reversal during operation of the bridge. The combination of high stress and stress reversal is a concern with regard to fatigue cracking and ultimately the development of cracks at this location.

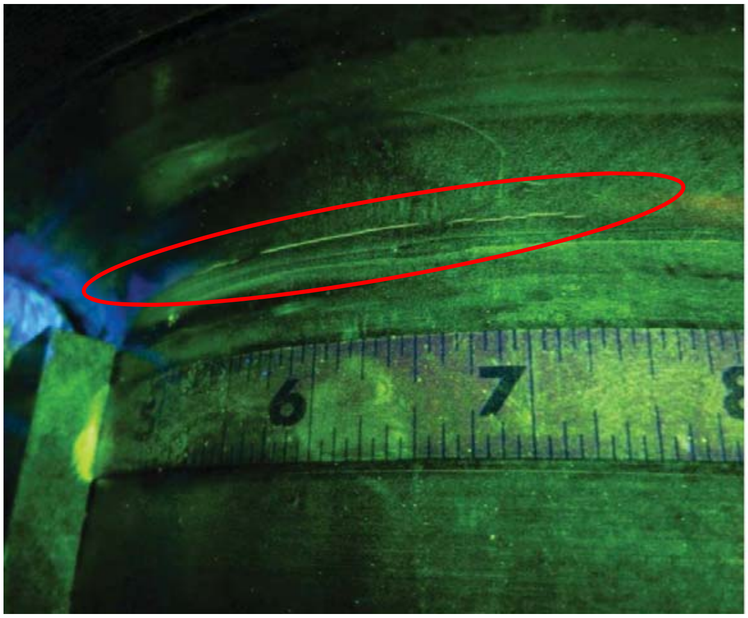
  To verify the integrity of the trunnions at the fillet area, the top 180 degrees of the fillet area was subjected to a dye penetrant test and close visual examination and the bottom 180 degrees was subjected to a close visual examination at all of the opened trunnion bearings. Access to the bottom 180 degrees of the fillet area requires raising the span and closing it to vehicular traffic. Traffic considerations did not allow for a dye check of the bottom 180 degrees of the fillet area. No cracks were found in the fillet area as part of this inspection

  Although dye penetrant and visual inspection provide some level of surety with regard to the condition of the fillet area, it is possible that cracks exist that were not picked up through these inspection methods. A more sensitive method of crack detection is wet fluorescent magnetic particle examination. This method of inspection is recommended to identify cracks in the early stages of development. Wet magnetic particle inspection was beyond the scope of this inspection.

  Calculations were prepared to determine the fatigue life of the counterweight sheave trunnions. The number of bridge operations over the life of the bridge was projected using the average number of openings per year based on data for the years 1979-2001 that was provided by WSDOT engineering personnel. All other information required to prepare the calculations was obtained from the original design drawings. The calculations indicate that the trunnions have a fatigue life of 19,732 cycles and that the actual number of cycles due to bridge operations is 34,632. Therefore the trunnions are well beyond their calculated fatigue life. Based on this information and our experience on other bridges we thought there was a high probability of finding fatigue cracks in these trunnions.

  The conclusion of this report was that although no cracks were found at the counterweight trunnions, using the methods employed during the 2002 inspection, **fatigue theoretical calculations indicate that the trunnions have exceeded the number of cycles to failure by 76%.** These calculations and our knowledge of calculated fatigue life and cracks in trunnions on other vertical lift bridges suggest that cracks are likely on this bridge. It was strongly recommended to perform follow-up inspection with more sensitive non-destructive examination such as ultrasonic testing of the fillet areas, including wet-magnetic particle inspection.

- **July 2008**: An in-depth mechanical inspection was performed by Scott Snelling, P.E. with non-destructive inspection, including through-bore ultra-sonic inspection and wet-magnetic particle inspection, of selected sheave trunnion fillets by Rob Gessel of Wiss Janney Elstner (WJE). Fatigue cracking in the sheave trunnion fillets was discovered. Several potential repair options were presented, including replacement of the sheaves, post-tensioning the sheave shafts, or in-place machining to excavate the cracks and increase the fillet radius, followed by peening. Calculations indicated that rehabilitation of the existing sheave shafts could add approximately 50 years of life before cracking would recur, but that infinite fatigue life was not possible with the existing trunnions.



CAPTION: Close up photo of fatigue cracks in the trunnion fillets as indicated during wet magnetic particle type non-destructive examination. Photo by R.Gessel of WJE

The original design of the subject sheave shafts, circa 1952 predated the application of stress concentration, metal fatigue and fracture mechanic principles within the AASHTO design codes. The current 2008 AASHTO LRFD specification is based on Soderberg fatigue failure theory. Fatigue calculations indicated a nominal bending stress at the fillet of 21 ksi. The 1938 AASHO, applicable at the time of design, has an allowable stress of 15 ksi for sheave shafts fabricated from heat-treated alloy steel. Therefore, the trunnions were apparently not designed in compliance with the standards in effect at the time. One explanation might be that the designer of the subject sheave shafts applied the 25 ksi allowable stress recommended by O.E. Hovey in his two-volume treatise on movable bridges, published in 1927.

The shafts in question were fabricated from ASTM A235 Class G, which is equivalent to ASTM A668 Class F, alloy steel forging with a heat treatment of quenched and tempered, with an ultimate stress of 82ksi. (For reference, using a typical safety factor of 1/5 used in the AASHTO 1988 specifications for other trunnion materials, the approximate allowable stress would be 16.4 ksi). Looking at the geometry of the shaft (D/d = 1.22, r/d = 0.055, r = 1/2 in) a stress concentration factor of 1.95 for a shaft with a shoulder fillet in bending (Kt) can be derived. From this the factored bending stress at the fillet was determined to be 42 ksi. Since sheave shaft rotations cause complete reversals in bending, the factored stress range is 84 ksi. Therefore the trunnions of the Snohomish River Bridge were the most overstressed, when compared to the eleven other known vertical lift bridges at which trunnion cracks have been discovered. Below is a list of a few of these bridges with known cracking, along with their factored stress ranges :

Shippingsport Bridge in Illinois	75.7ksi	Trunnion Collapse occurred
Valleyfield Bridge in Quebec	56.2 ksi	Trunnion Collapse occurred
Carlton Bridge in Maine	72.8 ksi	
Duluth Aerial Bridge in Minnesota	55.2 ksi	
Calument River Bridge in Illinois	53.4 ksi	
PATH-Hackensack Bridge in New Jersey	44.7 ksi	

Note that the stress ranges cited above include the stress concentration factor for both tension and compression. This is the typical practice when performing fatigue calculations. However, once the cracks have initiated and fracture analysis is being performed, standard practice is to only apply the stress concentration factor to the tensile stresses in the fillet and not the compressive stresses. Also, fracture analysis typically accounts for the decay of the stress concentration factor as the crack deepens.

The trunnions for Shippingsport Bridge in Illinois and Valleyfield Bridge in Quebec collapsed due to fracture induced by fatigue cracking at the trunnion fillet. At Valleyfield Bridge the factored stress range is lower than at Carlton Bridge, however the trunnion at Carlton Bridge did not collapse. An explanation for this can be that Valleyfield Bridge trunnions experienced more fatigue cycles than Carlton Bridge. Also note that temperature can affect the crack propogation if a crack has initiated. This is because the toughness of steel decreases with temperature.

The consequences of trunnion failure and collapse are very serious. As a worst case, risks include complete collapse of the bridge and associated potential loss of human lives. As a best case, the vertical-lift bridge risks being rendered inoperable for a months or longer while new sheaves are fabricated, most likely with the bridge closed to highway and marine traffic for the duration of the required repairs. The economic impacts of bridge closures to the local economy can be serious, depending on the location of the bridge and the availability of feasible detours. A failure of this serious nature would likely have political ramifications as well.

- **March 2010:** SBE worked with WJE to perform a complete non-destructive examination of all eight sheave trunnion fillets. The Northeast Inboard trunnion fillet location had the most advanced cracking, with continuous and intermittent cracking over 70% of its circumference. The maximum crack depth was estimated to be 0.125 inches. WSDOT maintenance staff used a “flapper wheel” to attempt to excavate the cracks, but the cracks were deeper than the amount of material that could practically be removed with a “flapper wheel” of approximately 1/16 inch. Dr. John Fisher performed material coupon testing and fracture analysis which concluded that if any trunnion cracks were allowed to extend to 0.5 inches deep, this would result in a safety factor of less than two against brittle fracture, rendering the movable span unsafe for operation. Dr. Fisher calculated that 4041 trunnion stress cycles, equivalent to 1585 movable span full openings (2.55 cycles per opening) could result in the cracks extending to thedepth of 0.5in. The average number of movable span openings in recent years was 600 each year, with a maximum number of

843 openings having occurred in 2005. In other words, WSDOT had about 2.5 years to design and implement a repair before it would be forced to lock the bridge in the open position and close the highway.

- **October 2010**: SBE worked with WJE to perform a follow-up non-destructive examination of the sheave trunnion fillets. No perceptible crack growth was indicated since the inspection six months earlier.

- **May 2011**: Scott Snelling, P.E. of Parsons Brinckerhoff and Norm Duke performed an in-depth inspection of the wire ropes. The 24 counterweight ropes dated from when the bridge was originally constructed in 1954. The wire ropes were 1-5/8 inch diameter 6x41 Warrington Seale construction with a fiber core. The ropes had moderate deterioration. The ropes had significant crown wear due to abrasion resulting in an estimated 9% reduction in ultimate breaking strength. Crown wear had increased measurably since the previous rope inspection in 2004. The calculated rope safety factors of 6.1 for direct static loads and 3.6 for dynamic loads did not comply with current AASHTO recommendations of 8.0 and 4.5, respectively. Light corrosion was found on the ropes underneath the old, hardened, accumulated lubricant. It was recommend to replace the counterweight ropes concurrently with upcoming sheave trunnion replacement work. Combining the rope replacement with the sheave trunnion replacement resulted in significant savings due to shared costs of the jacking and temporary support of the counterweights.

- **July 2011:** Parsons Brinckerhoff and SBE teamed to develop a set of plans and specifications to replace the sheaves, trunnions, trunnion bearings and wire ropes. The scope also included designing the temporary counterweight supports, temporary live load shoes, and evaluating the temporary stresses on the existing structure imposed by the temporary counterweight support. The new trunnions were designed such that they will have infinite fatigue life with a generous fillet radius, larger diameter and made with higher strength material. This dictated a redesign of the trunnion bearings to accommodate a larger diameter trunnion on the existing supports. As part of this design the sheaves were replaced with a new fabricated weldement design versus the previous cast steel sheave design. Also new ring gears and pinions were provided and the ring gears were pressfit into the fabricated sheave and the pinion was press fit onto the cross shaft.

- **March 2012:** Parsons Brinckerhoff worked with WJE to perform another non-destructive examination of the trunnion fillets using through-bore ultrasonic examination and wet-magnetic particle examination. Several new cracks were found, compared with the previous examination in October, 2010. The deepest crack was estimated to be 0.165 inches deep. Based on the March 2010 fracture analysis, there were 548 bridge openings remaining before a calculated crack depth of 0.5 inches was reached and the movable span would be categorized as “unsafe for operation.”

- **April, 2012:** The contract documents were put out to bid. The engineers cost estimate for the rehabilitation was $2.9 Million.

- **June, 2012:** PCL was awarded the contract with a low bid of $1.7 Million.

- **May, 2013:** The threshold of maximum movable span operations, based on the 2010 fracture analysis was reached. Parsons Brinckerhoff worked with WJE to perform yet another non-destructive examination. Parsons Brinckerhoff also worked with Dr. Fischer to update the fracture analysis, which estimated that an additional 1680 bridge openings were available before the cracks propagated to 0.5 inches deep and the movable span was designated "unsafe for operation." Note that the estimated number of remaining bridge openings was larger after the 2013 analysis versus the 2010 analysis, this is due to difficulties with regards to estimating the rate of decay of the stress concentration factor with the depth of the crack. In 2010, Dr. Fisher used a conservative assumption. In 2013, the stress concentration decay could be less-conservatively calibrated based on the actual crack propagation rates measured in the intervening years.

- **September 2013:** Existing sheaves and ropes were removed and new sheaves and ropes were installed.

- **October 2013:** Construction was substantially completed and the machinery alignment is on-going.

Scope of the Rehabilitation

The scope of the rehabilitation was separated into three categories: mechanical support work, mechanical work, and electrical work.

Mechanical Work – Scope:
- Span Support Machinery: Replace the existing trunnions, sheaves, and trunnion bearings.
- Span Drive Machinery: Replace the existing pinion, ring gear, and coupling grids.
- Counterweight Ropes: Replace the existing counterweight ropes and pins, adjust tension in the new counterweight ropes.

Mechanical Support Work – Scope:
- Remove, rehabilitate and reinstall the live load supports.
- Provide and install temporary dead load plus live load (DL + LL) supports.
- Provide temporary counterweight supports
- Temporarily remove and reinstall existing machinery roof sections as needed.

Electrical Work – Scope:
- Create as-found electrical wiring diagrams for the rotary limit switch and position transmitter.
- Temporarily remove the rotary limit switch and position transmitter before the existing sheaves are removed.
- Replace the rotary limit switch and position transmitter after the new sheaves are installed.
- Protect electrical equipment, wiring, and conduits from physical damage during construction and damage due to weather while the machinery room roof is removed.

There were multiple factors that contributed to determining the scope of the replacement of the trunnions. These factors were the required marine and vehicular outage, the required redesign required to meet current AASHTO requirements and costs associated with all options. The option of replacing the trunnions only and salvaging the counterweight sheave and ring gear was considered. The advantage of this option is that the cost for manufacturing the parts would be less as the sheave, ring gear and pinion would be reused. The disadvantages of this option were as follows:

- The existing sheave hub thickness over the trunnion would be much smaller than recommended by AASHTO. This is because the new trunnion diameter through the sheave is larger by 2 ¼" then the existing.
- The removal of the existing trunnion from the existing sheave would add additional risk to the project as they have an FN2 fit and the removal may damage the bore in the sheave. Also a larger marine outage is required for replacing the new trunnions in the existing sheaves.
- Lastly a drop in replacement of the sheave assembly will require a smaller marine outage as the parts can be shop assembled and ready for installation prior to the start of the vehicular and marine outage.

Therefore the selected alternative was to replace the existing trunnions, sheaves, ring gear, pinons and trunnion bearings with new components of new design. This allowed the assembly to be assembled in the shop and essentially be a drop in replacement. With this approach the trunnions and sheaves can be fabricated ahead of time and the trunnions can be press fit into the sheave. Simultaneously, the ring gear can be fabricated and press fit into the sheave. Also the trunnion bearings and the pinions can be fabricated and stored prior to any interruptions to traffic at the bridge. Once all the parts are fabricated the bridge can be closed to vehicular and marine traffic as needed to remove the existing components and install the new components.

The main counterweight ropes were the original ropes installed when the bridge was construted. As part of this rehabilitation the counterweight ropes needed to be removed in order to facilitate the replacement of the sheaves. Considering that there is a potential of damaging the existing ropes in the removal and replacement process, wear found on the ropes and the service life of over 50 years it made sense to provide new wire ropes. Therefore the counterweight wire ropes and pin replacement was added to this rehabilitation. Once the new wire ropes were installed it was necessary to measure and adjust the tensions in the wire ropes such that they shared the load evenly and therefore this requirement was added to the scope.

In order to perform this sheave replacement work, it was necessary to temporarily support the counterweights in order to remove the dead load from the ropes and sheaves. Modern vertical lift bridge designs typically include provisions to temporarily support the counterweights directly from the bridge towers, typically using a steel pin and hydraulic jacks. The Snohomish River Bridge had no such provision. Therefore, the design of temporary counterweight supports was included in the scope of this project.

The four live load shoes for the lift span are all rocker-type with curved bearing surfaces. However, once the counterweight was to be jacked and the ropes removed, four rocker-type bearings is no longer a stable configuration. In addition, the shoes would temporarily be required to support the dead load of the movable span, in addition to the live load. The line-contact portion of the live load shoes would be overstressed by the added dead load. Therefore, the design of temporary dead load plus live load shoes was included in the scope of the project. In addition, the existing live load shoes were seized and in need of rehabilitation to free the shoes to allow rotation and provide for improved lubrication details to the pins to prevent recurrence of the seizing issue.

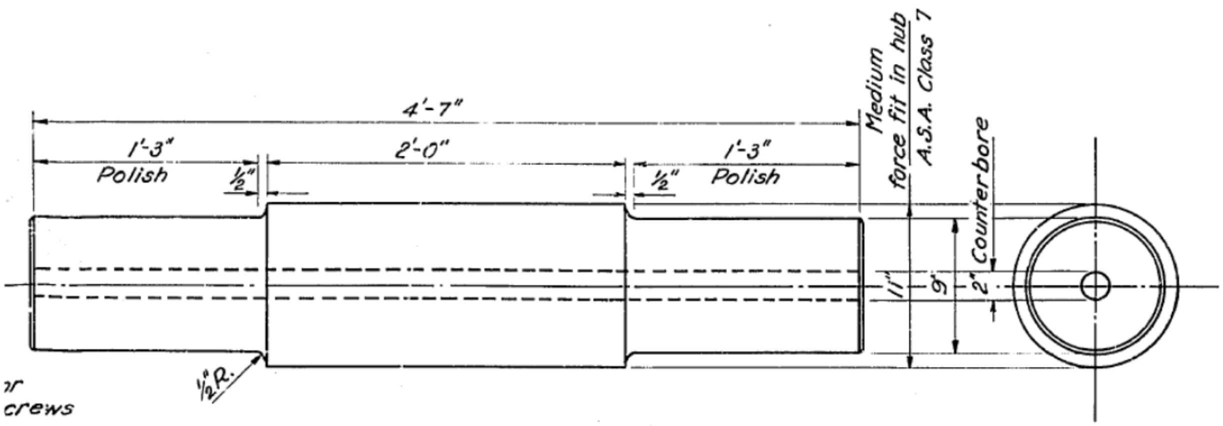
To allow for the removal of the existing sheaves and installation of the new sheaves, the temporary removal of the machinery room roofs and electrical limit switches were also required to be included in the contract documents.

Design Plans and Specifications

The development of the design plans and specifications began once the scope of the rehabilitation was finalized. This part of the work was a collaborative process with WSDOT. Milestones were established for various stages of development of the design plans and specifications. At each stage the design plans and specifications were reviewed by WSDOT and commented on and these comments were incorporated into the submittal. Below is a brief description of the components of this rehabilitation project and the improvements made from existing components.

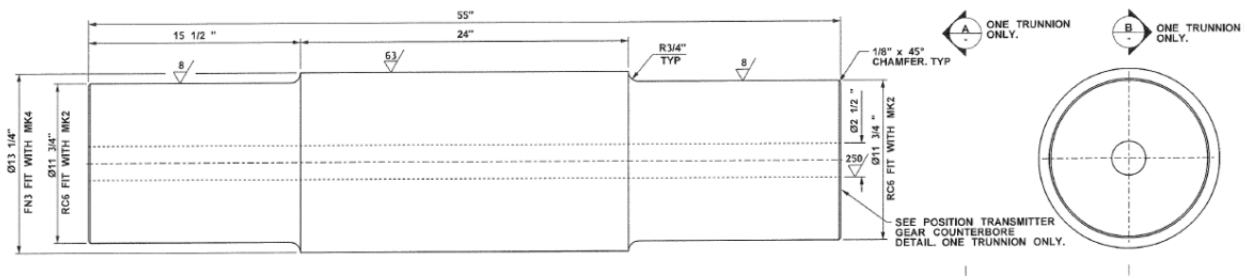
Span Support Machinery – Sheaves, Trunnions and Bearings

The rehabilitation of the span support machinery consisted of replacing the existing trunnion, sheave and trunnion bearings. The existing trunnions had fatigue cracks at the radius at which the trunnion transitions from the sheave fit to the bearing journal.



CAPTION: Design drawing of the original trunnion from 1953. The trunnion fillet radius is ½".

To prevent the new trunnions from cracking due to fatigue the new trunnion were designed to be larger in diameter,have a more generous radius at the transition from the sheave shrink fit to the bearing journal and were made of stronger material. The new trunnions were sized to meet the current AASHTO standards. The current AASHTO standard states that the trunnion shaft should have an infinite fatigue life. The new trunnion needed to be able to fit in the space constraints dicatated by the existing trunnion bearing supports. The new trunnion needed to have a 24in long trunnion hub and needed to have the same trunnion bearing spacing as existing. The new trunnion also needed to have an FN3 fit with the sheave to meet AASHTO requrements. All of these requirements have been added to the new trunnion as shown below.



CAPTION: Design drawing of the new trunnion. The trunnion fillet radius is ¾" and the trunnion is larger 13 ¼" vs 11" dia. and 11 ¾" vs 9" dia.

New sheaves were provided and meet the requirements of the current AASHTO standard and fit in the existing space constraints. The existing sheaves were made from steel casting; however few foundries remain in the USA capable of producing a ten-foot diameter casting. However, many fabricators continue to have the capability to fabricate a sheave of welded construction. Therefore a welded sheave design was developed to fit into the available space and match the critical geometric demission of the existing structure. The new sheave is designed to take the loads imparted on it, accommodates the internal gear ring gear and has a larger hub than the existing sheave to meet AASHTO requirement. It also has an FN3 fit with the trunnion and three 1 ½" pressfit dowels.

CAPTION: Original sheave, trunnion, and trunnion bearing.

CAPTION: New sheave, trunnion, and trunnion bearing.

The sheave trunnions are simply supported by two sheave trunnion plain bearings. To accommodate the new trunnion new trunnion bearings were developed that fit on the existing trunnion support and fit the new trunnions. The trunnion bearing base bolts were oversized so that the new bolts will have a turned bolt fit with the trunnion bearing support. The bearing bushing was designed to have axial lubrication grooves on the bottom half of the bushing and spherical lubrication grooves on the top half. The bottom lubrication grooves had a lube port and a purge port to allow for the old lubrication to be flushed through the bearing. The inboard edge of the bushing was provided with a larger chamfer to accommodate the larger radius on the trunnion.

Span Drive Machinery

To facilitate the drop-in replacement of the sheaves and trunnions, a new ring gear and pinion were added to the rehabilitation scope. The new ring gear and pinion were designed to meet the current AASHTO standards. AGMA spur gear design calculations in bending and pitting were performed to design the gearing as required by AASHTO. The gearing was designed to have a tip relief that will compensate for any deflection of the teeth at load and any manufacturing errors. The ring gear was designed to have a 0.005" to 0.010" interference fit with the sheave and have 32 – 1" diameter turned bolts that secure it to the sheave. The ring gear was made out of a ring forging that was 115" in diameter. The pinion was made out of a solid steel forging and was shrink fit on to the existing cross shaft. The existing pinion was removed from the existing cross shaft by torch cutting the pinion at the keyway to avoid damage to the shaft. At the other end of this cross shaft a grid coupling connects it to the output shaft of the reducer. The grids of this coupling were replaced as part of this rehabilitation.

A critical part of this rehabilitation was aligning the new machinery without moving the high sped end of the existing drive machinery. The intent was to return the pinions to the existing location and move the sheaves as required to obtain the desired alignment. As always the task of aligning large machinery is difficult and was challenging on this project for the following reasons:-

1) The alignment of both the pinion and rack are affected by the deflection of the tower under load and this deflection is not known until the full dead load is applied to the sheaves at which time adjustment to correct alignment are not practical. Therefore making alignment adjustments is an iterative process that involves loading and unloading the sheaves to obtain an acceptable alignment.

2) Due to the design of the ring gear and pinion, access to measure tip clearance and backlash at one end of the pinion is very limited and precludes conventional measurement methods.

3) As mentioned above, due to the limited scope of this rehabilitation, the Contractor was not allowed to move any of the existing remaining machinery. This necessitated that the Contractor control the alignment of the sheave trunnion assembly as well as the alignment of the ring gear to the cross shaft pinion without moving the position of the cross shaft pinion.

To help the Contractor align the new machinery with existing machinery given the above noted issues the following specification requirements were added to the contract:

1) The specification stated that "the alignment of the counterweight sheave trunnions relative to each other and relative to the survey line establishing the position of the existing trunnions is of secondary importance to the trunnion bearing alignment requirements and to the ring gear and pinion alignment requirements provided hearin. As such the alignment of the trunnions shall be recorded but will not dictate the final position of the trunnions." This requirement prioritized the alignment of the trunnion bearing and the ring gear and pinion over the alignment of the counterweight sheave trunnions relative to each other and relative to the survey lines, hence giving the Contractor some leeway in aligning the sheaves.

2) The Contractor was advised that deflection of the bearing supports due to the counterweight load transferring from the temporary counterweight supports to the sheaves will affect the

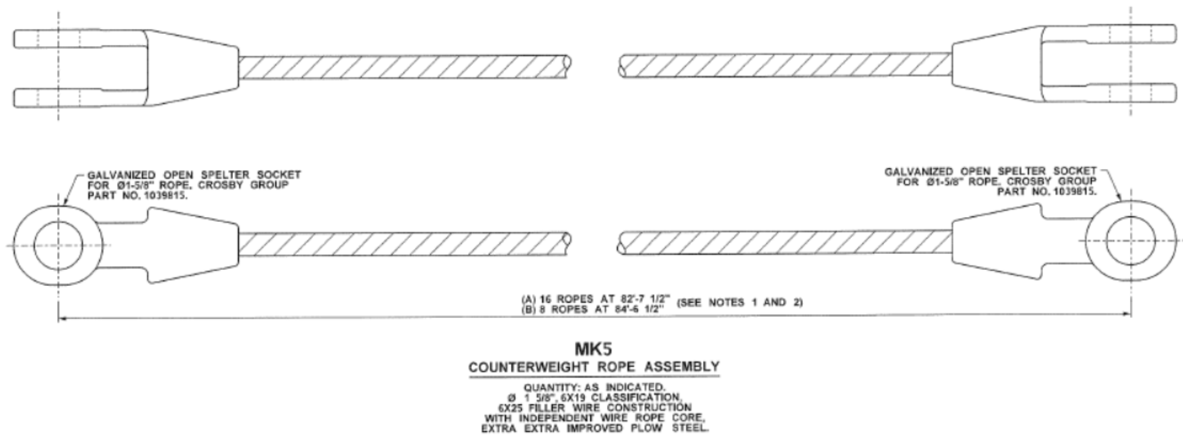
trunnion bearing alignment and that it may be necessary to jack the counterweights using the temporary counterweight supports and adjust the alignment or shims of the trunnion bearings multiple times to achieve the indicated alignment.

3) The specifications also required the Contractor to perform a detailed survey to locate the position of the existing machinery before and after the counterweight was jacked. This survey helps the Contractor predict the effect of the change in position of the machinery due to the counterweight being jacked and unjacked. This allows the Contractor to compensate for deflection when installing the machinery. The following were the requirements of the survey:-

- The alignment of the existing cross shaft, existing trunnion shafts and the existing ring gear and pinion shall be established.
- Permanent reference marks shall be established at the outboard side of each trunnion that can be relocated after the new trunnions are installed. This will be the reference line.
- The trunnion centerlines shall be located at both the inboard and the outboard side of each trunnion to a measurement accuracy of 0.002”.
- The cross shaft shall be located with reference to the reference line at two locations 100 ft apart and measured to an accuracy of a 1/32”.
- The ring gear and the pinion alignment shall be measured as follows:-
  - Tip clearance measured to an accuracy of ±1/64”
  - Axial alignment measured to an accuracy of ±1/64”
  - Backlash measured to an accuracy of ±0.002”
  - Gear tooth contact measured as determined by bluing at 4 locations 90 degrees apart and the length of contact shall be measured to ±1/16”
  - The trunnion bearing alignment with respect to level shall be measured. The bearing caps were removed at this bridge and hence this was possible by measuring at the bearing split. The level of the bearing base was measured in two directions. One is in the axial direction and the other is on the direction perpendicutlar to the axial direction. Precision blocks and level were used to step over the journal and the key in the base.

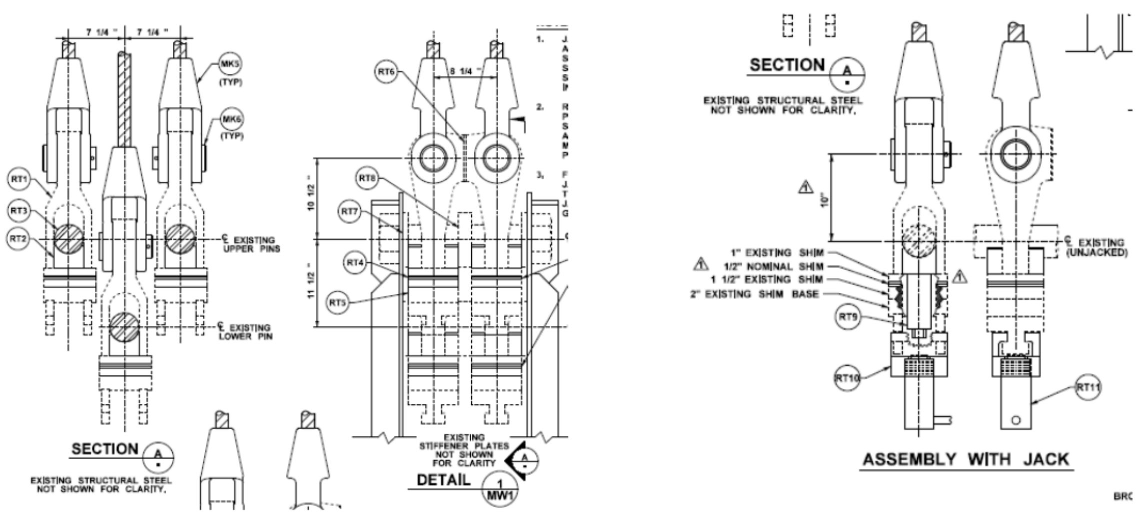
Counterweight Ropes

As part of the inspection of the counterweight wire ropes it was determined that they exhibited moderate wear. The existing counterweight ropes were the ropes originally install on the bridge in 1954. These factors combined with the savings associated with replacing the counterweight ropes along with the sheaves versus replacing the ropes as part of a separate contract dictated that the rope be replaced as part of this work. This bridge has a total of 24 counterweight wire ropes and 6 wire ropes on each sheave located at each corner of the bridge. The existing wire ropes were 1 5/8” diameter 6x41 “M”, Purple, Regular Lay, Fiber Core ropes with a breaking strength of 214,000 lbs per rope. The replacement ropes were 1 5/8” 6x25 filler wire construction with independent wire rope core extra extra improved plow steel with a breaking strength of 292,000 lbs per rope. These new ropes are 36% stronger than the existing ropes which meets the direct load requirement of 2007 AASHTO. The ends of the ropes were fitted with Crosby Group galvanized open spelter socket. See below drawing.



CAPTION: The new counterweight wire rope.

The existing wire rope terminations were reused and included a system where adjustments to the effective length of the ropes can be made by adding or removing shims. See below.



CAPTION: The counterweight wire rope terminations which include a system of adjusting the effective length of the ropes.

The system works by jacking individual ropes using a hydraulic jack against a fixed pin on the structure. This removes the load from the shims and allows for the removal or addition of shims which effectively changes the length of the rope. Through this system the load taken by each rope can be changed and the load can be distributed evenly on all the ropes. This method of adjusting the load taken by each rope along with a method of measuring the rope tensions using their fundamental frequency was required for this project. The specification required that the rope tension at each rope group in all four corners shall be within ±5% of the average tension in each rope group. This work would ensure that all the ropes will equally share the load and decrease the changes of premature wear due to overloading.

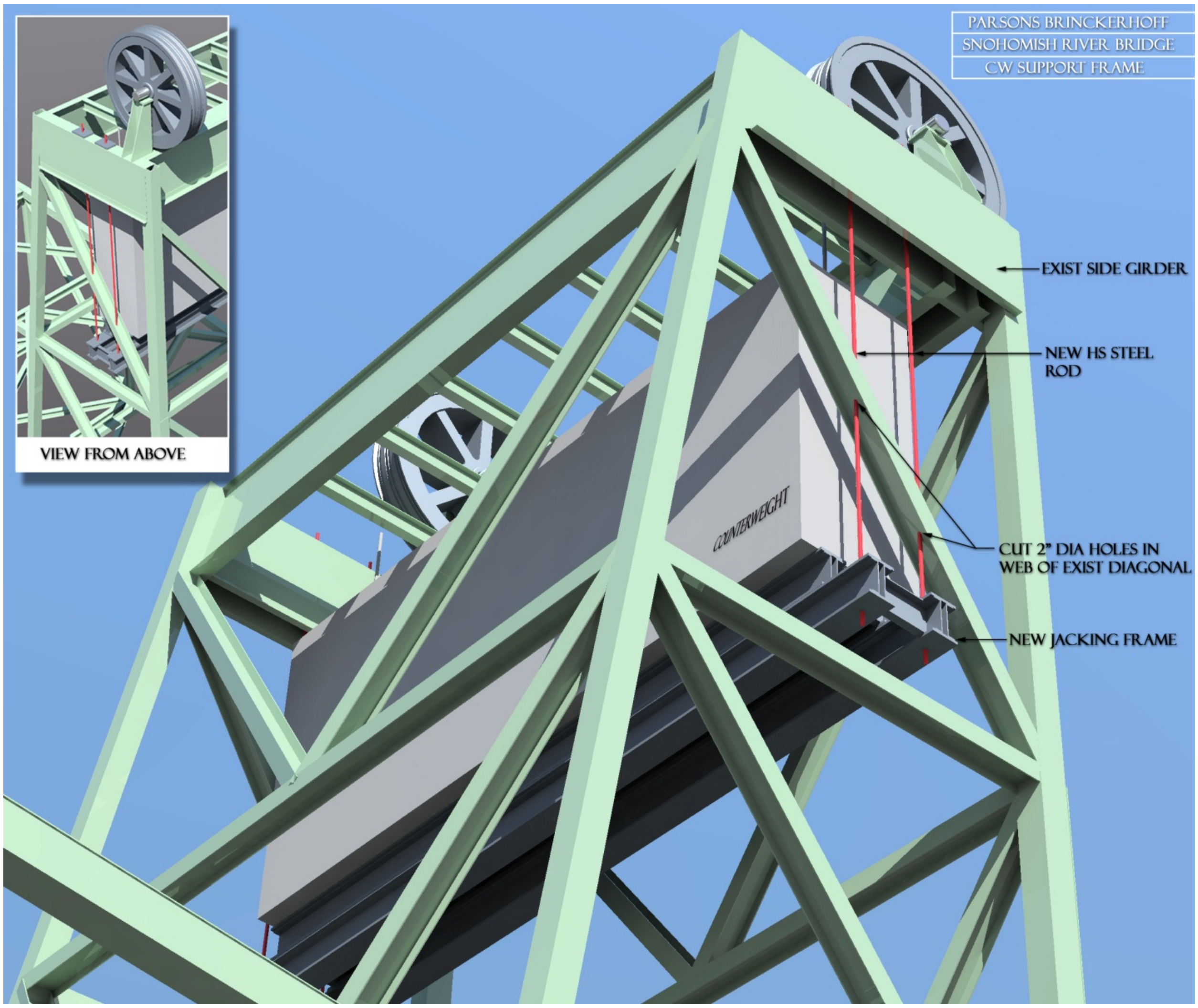
Determining the length of the new wire ropes that will replace the existing wire ropes should be considered carefully. This is because the new wire ropes have a different constructional and elastic stretch as they are of different construction than existing. Constructional rope stretch is the permanent increase in rope length that occurs over years of service. Elastic rope stretch is the increase in rope length that occurs while under load. The Contractor measured the lengths of these ropes and they were on average 4 ¼" longer than as noted on the original shop drawings. This stretch is constructional stretch as the original ropes were measured under load to the lengths noted on the shop drawings. Therefore the temporary counterweight jacking system needs to accommodate the difference in constructional and elastic stretch and the stretch due to service. Once it was determined that the jacking system could accommodate the required momement without any interferences, the new rope lengths were recommended to be the same as the length of the existing ropes noted on the shop drawings.

Temporary Counterweight Supports

The original design of the bridge did not include any provisions for the temporary support of the counterweight in order to facilitate unloading the existing ropes and sheaves for replacement. Therefore, the design of new temporary counterweight supports was included in the scope of this project.

Each of the two counterweights weighs approximately 400 kips. The counterweights are concrete with a structural steel frame inside. Since there is a sidewalk within the lift span through-truss, there are corresponding hollow-cavities on the West side of the counterweights.

Parsons Brinckerhoff developed a 3D model of the existing tower in order to facilitate the preliminary design process for the new temporary counterweight supports. After evaluating the geometry and capacities of the existing tower members, it became apparent that the existing side girder members had the capacity to temporarily support the counterweight, with the ability to transfer the temporary loads to the existing tower columns without any strengthening. See the image below showing the configuration that was proposed as a preliminary design.



CAPTION: Preliminary 3D Model of the Temporary Counterweight Support Design – Early in the Design Process (Existing Wind Guides and Temporary Lateral Support Not Shown)

Key attributes of the temporary counterweight support design included redundancy of the high strength rods and the ability to distribute the point loads imposed by the high strength rods. The quantity of four high strength rods used to provide temporary support, was the maximum feasible quantity, due to the limited geometric space envelope provided due to the close proximity of the tower diagonal and horizontal truss members. Given the opportunity, additional rods would have been used to provide additional redundancy. However, even with only four rods, it was possible to design for redundancy even in the extreme case of the complete failure of one rod. Note that the failure of one rod would result in zero load in the rod on the opposing corner and a doubling of the load in the other two remaining rods.

CAPTION: Counterweight temporarily supported during construction with ropes removed. From top to bottom, note: (1) machinery room roof partially removed with tarp for temporary weather protection, (2) machinery room wall cut to make room for bar jacks, (3) temporary lateral support, (4) temporary counterweight support beam.

Due to the tall and slender profile of the existing counterweight, as well as potential instability related to the considered rod failure case, it was necessary to also provide temporary lateral supports to prevent overturning of the counterweight. The lateral supports were conservatively designed to support 25% of the vertical dead load, which approximately translates to a rotation of 14 degrees from plumb.

The lateral supports were required to provide this overturning restraint while the counterweight was being jacked up, while the existing ropes were removed, and while jacking the counterweight down to transfer the lift span dead load to the new ropes. The total required vertical movement of the counterweight supports was conservatively on the order of two feet, to accommodate the constructional stretch of the existing ropes, as well as the elastic stretch of both the existing and new ropes.

The lateral support design was a steel weldment that was anchored the side of the counterweight and had slots allowing the high strength rods to pass through. The permanent wind guides for the counterweight were left in place and continued to provide additional support. The temporary counterweight supports did not directly utilize any of the adjacent existing tower diagonals to support loads. However, designing around the adjacent existing tower diagonals to avoid interferences was a key component of the design process.

As an additional measure to ensure that the counterweights remained plumb during jacking and were not allowed to rotate, the specifications required that the Contractor monitor and record, at one-minute intervals, the hydraulic pressures and correlated load in each jack, as well as the elevation and levelness of the counterweight.

During design, it was considered that the selected Contractor would be likely to propose design changes to the temporary supports in order to use on-hand materials. In fact, during the shop drawing phase, the Contractor proposed to use an alternative configuration for the jacking beam, while retaining the general configuration with four high strength rods operated by four hydraulic bar jacks. When evaluating the Contractor's proposed alternative design for the jacking beam, there was a focus on maintaining the key attributes of redundancy and the ability to distribute the high point loads imposed by the high strength rods.

Permanent Live Load Supports

CAPTION: Live Load Shoes disassembled in the shop

Unlike a typical simple span bridge with two fixed bearings and two free expansion bearings, each of the four existing live load shoes for the Snohomish River Bridge were rocker type to allow for free expansion. However, the pins were seized and no longer allowed for expansion. In addition, the grease grooves were plugged and were no longer accepting lubrication. Therefore, the scope of this project included removing the existing live load shoes, disassembling, improving the bearing and lubrication details, and re-installing the refurbished shoes.

Temporary DL + LL Supports

The existing live load shoes did not have the capacity to support the dead load of the lift span when the counterweights were jacket and the ropes were removed. Specifically, the overload that occurred was in the line contact created where the curved contact surface of the live load shoes meet the strike plates. The suggested construction sequence in the contract documents was to complete the rehabilitation work at one tower before proceeding to the next tower. Therefore, the new temporary DL+LL shoes were designed as a simple pedestal, with no capacity for free expansion.

CAPTION: Temporary DL+LL Shoe installed on the bridge.

During the shop drawing phase of the project, the Contractor instead proposed to simultaneously perform work at both towers, which necessitated alterations to the DL+LL shoes to allow for free-expansion. In the end, this was accomplished using PTFE (aka. Teflon) sheeting between the shoes and strike plates.

Temporary Removal of Existing Machinery Roof Section

Incidental to the sheave replacement work, it was necessary to temporarily, partially remove portions of the steel machinery house roof and walls to provide access.

Electrical Work

Incidental to the sheave replacement work, it was necessary to temporarily relocate existing limit switches and provide protection to existing electrical equipment.

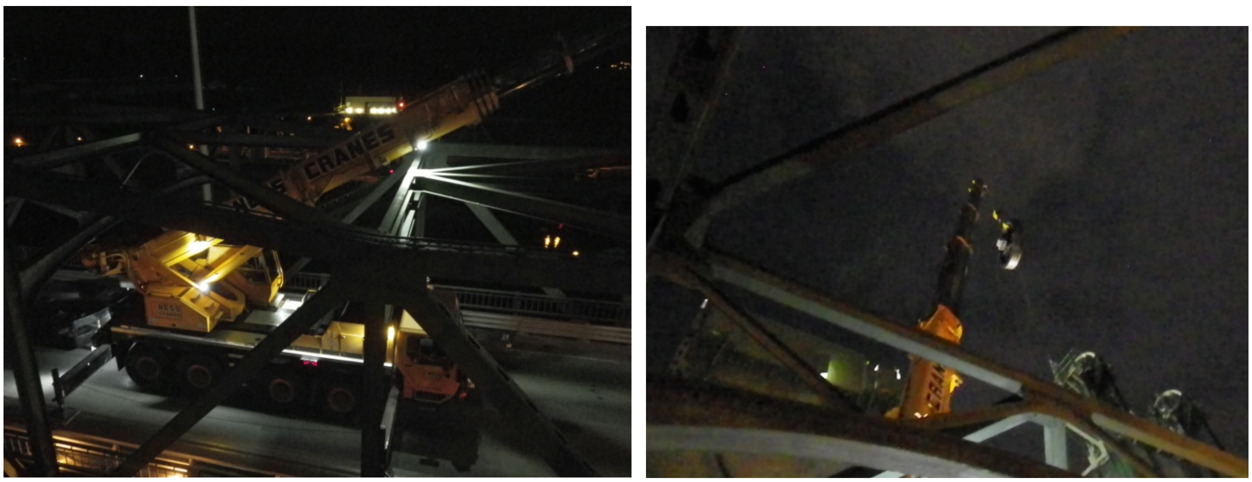
Construction Services and Highlights

This rehabilitation included replacement of large components, supporting heavy counterweights, precession manufacturing and installation of machinery and related work. To help contribute to the success of this project Parsons Brinckerhoff and SBE provided construction services for reviewing shop drawings, requests for deviation, request for information, work procedures, survey procedure and data and other submittals as requested by WSDOT. The goal of this review was to ensure that the Contractor was meeting the contract drawings and specifications and will be able to achieve the objectives of the project.

To perform the construction work, the contract documents allowed for two closure periods of one-weekend-long each to both vehicular traffic and marine traffic. In addition, two one-week-long closures to marine traffic were allowed. Balancing the acceptable length and season for the bridge closure periods was a key component of the project. Shorter closure periods would be physically possible to construct, but would increase the risk to the Contractor and therefore would be expected to increase the bid prices and cost to the State. Longer closure periods would place added burden on the traveling public.

Early in the design phase, WSDOT performed public outreach to bridge users, including the local mariner association as part of the effort to determine what length of closure periods, as well as which seasons were acceptable for closure. The peak mariner summer season, as well as falcon nesting periods in spring were determined to be un-acceptable seasons to perform the work. Winter season allowed for longer bridge closure periods without unduly interfering with marine traffic. An Incentive/Disincentive clause in the contract documents provided for financial rewards if the Contractor completed the work with shorter closure periods and financial penalties if the Contractor was not able to complete the work within the allowed windows. For this project, the Contractor performed the work within the allotted time, with no time related penalty or rewards realized.

During the design phase it was assumed that the replacement of these sheave trunnion assemblies will require the rental of a large barge mounted crane. This was a large part of the engineering cost estimate. However the Contractor was able to prove that this work can also be done using a boom crane that is setup on the span. See below.



CAPTION: The Contractor used a boom crane that is setup on the span to replace the sheave trunnion assemblies.

Being able to lift the sheave trunnion assemblies from roadway to tower level by the use of a rubber-tired boom crane from the bridge deck instead of a barge mounted crane allowed the Contractor to save a significant amount of construction cost.

Conclusion

The Snohomish River Bridges (529/10W) is a tower drive vertical lift bridge located in Everett, Washington and was built in 1954. This bridge is owned and maintained by the Washington State Department of Transportation (WSDOT). As part of maintaining the bridge WSDOT performs periodic in-depth inspections of this bridge. During one of these inspections, non-destructive testing was used and it was found that the main counterweight sheave trunnions exhibited cracks at the fillet area where the trunnion transitions from the sheave hub shrinkfit to the journal. These cracks form because the trunnions are subject to heavy loads and experience full reversal of those loads and the original design parameters did not include the consideration of fatigue. These cracks are due to fatigue and can increase in severity as the bridge is operated and can cause the trunnion to fail.

As these fatigue cracks were discovered, they were further documented by performing nondestructive testing. Dr. John Fisher was consulted to determine the severity of these cracks and he predicted that the cracks will grow and corrective action is necessary in the next couple of years. Several potential repair options were presented, including replacement of the sheaves, post-tensioning the sheave shafts, or in-place machining to excavate the cracks and increase the fillet radius, followed by peening. Calculations indicated that rehabilitation of the existing sheave shafts could add approximately 50 years of life before cracking would recur, but that infinite fatigue life was not possible with the existing trunnions. Therefore a set of contract plans and specifications were developed for the replacement of the main counterweight sheave assembly, counterweight ropes and other supporting work.

The new main counterweight trunnion was designed to fit in the same space envelope, however it was designed to meet 2007 AASHTO standards and has an infinite fatigue life. To accomplish this, the trunnion diameter was increased, stronger material was used and larger fillet radius was provided. This necessitated new sheave, ring gear and pinion. The partial replacement of the machinery required that the new machinery be aligned with the existing remaining machinery without moving it. Also the replacement of the main counterweight sheaves required that the main counterweights be jacked and supported from a different location than the sheaves. This causes the defection of the towers to change moving the sheave assemblies depending on if the load is transferred through the sheaves vs if the load is transferred through the temporary counterweight supports. To help the Contractor achieve the proper alignment in these challenges, the Contractor was asked to perform an extensive survey of the existing machinery prior to and after jacking of the counterweight. The Contractor was also advised that achieving the proper alignment of the machinery may require jacking the counterweight multiple times.

The main counterweight ropes were of original construction and had moderate wear and hence were also replaced at this time due to the savings in replacing them along with the sheaves. The new ropes are 36% stronger than the existing ropes which meets the direct load requirement of 2007 AASHTO. The existing wire rope terminations were reused and included a system where adjustments to the effective length of the ropes can be made by adding or removing shims. Using this adjustment method and measuring the rope tensions allowed the adjustments of the rope tensions at each rope group in all four corners to be within ±5% of the average tension in each rope group.

To facilitate this work, supporting structural and electrical work was necessary. This included installation of temporary support of the counterweight in order to facilitate unloading the existing ropes and sheaves for replacement, temporarily removing sections of the tower house, and temporarily removing electrical equipment and conduit. It was also necessary to remove the existing live load supports and send them to the shop for rehabilitation. Therefore a set of temporary DL+LL supports were provided that would take the loads when the counterweights were jacked.