Structural Elements

BNSF Bridge 14.2 Live Load Bearing Replacement

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Bridge Background

Burlington Northern Santa Fe (BNSF) Bridge 14.2 over Chambers Creek in Steilacoom, WA was constructed in 1913. The bridge is a 96-foot Pony truss Strauss direct drive vertical lift bridge. This structure is one of only two of its kind still in service today. In fact, this bridge has been classified as a Historical Structure by the State Historic Preservation Office. The lift span is operated by an on-site bridge tender approximately 5 to 10 times per day during navigation season to allow passage of marine traffic, primarily pleasure vessels. BNSF Bridge 14.2 provides an 85-foot-wide navigation channel and 25 feet of vertical clearance for Chambers Creek. The bridge supports double mainline tracks that carry nearly 40 BNSF and Union Pacific Railroad (UPRR) freight trains per day, as well as four Amtrak trains per day in each direction.

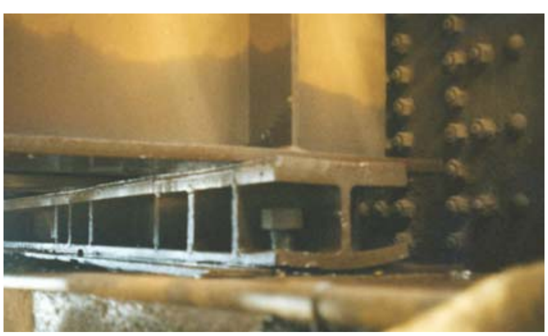
Photograph 1: Bridge 14.2 Elevation

The Problem

Bridge 14.2 became a focal point for the Railroad a few years ago when operational difficulties resulted in a permanent speed restriction of 30 mph being placed on traffic crossing the structure. The structure had exhibited operational difficulties for several years, but the problems were becoming more and more frequent. The most negative consequence of the speed restriction was that it interrupted a run of almost 43 miles at 50 mph for freight trains and 55-79 mph for passenger trains. Because there are eight Amtrak trains that cross this bridge daily and 62 million gross tons of freight that cross this bridge annually, there was significant time loss due to the permanent speed restriction.

In response to growing concerns by operations personnel, the BNSF Railroad decided that this bridge should be the focus of a multi-phased rehabilitation to address each of the operating systems on the structure. The structural, mechanical and electrical systems had all experienced operational difficulties to some degree in the recent past, and it was clear that they would all eventually need to be improved. It was determined that the structural system would be the focus of the first phase, as it was in the worst condition of the three systems. Several items in the structural system needed attention.

The movable span bearings exhibited deterioration as a result of improper seating and severe impact forces transmitted through the structure from the rail joints. The bridge had rider-style rail joints that had very short rider rails because of clearance issues between the span and the tower bracing. Additionally, the approach span rest pedestals had dislocated vertical cracks. Each approach span sat on a bolster beam that rested on relatively slender concrete pedestals which were doweled into the rest pier. Each approach span rest pedestal was made up of two pre-cast concrete blocks doweled together. The bolster beams were in poor condition due to excessive movement of the cracked pedestal supports, as seen in Photograph 2.



Photograph 2: Existing approach span bolster beam. Note the deformation in the bottom flange.

The dislocated vertical cracks in the pre-cast pedestals allowed considerable deflection under live load. The approach span girders actually functioned as struts to hold the pedestals in place. To make matters worse for the approach span support conditions, the pre-cast pedestals were not long enough to extend out to the two exterior approach span girders. The two exterior approach span girders were not supported by the concrete pedestal; the bolster beam carried these outside girders and rested on short H-pile columns, which also experienced significant deflection under live load.

In addition to the deteriorating movable span bearings and approach span pedestals, the end floorbeams were in need of attention as well. The top flange on one of the end floorbeams had a full-depth crack due to the fact that the rail joints had not been adequately secured to it. The severe impact forces from the rail joints eventually took their toll on the top flange.

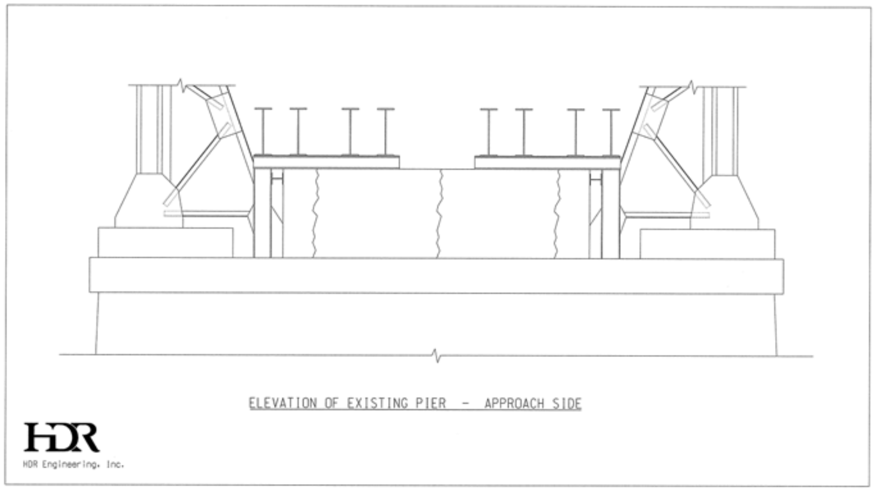


Figure 1: Approach span side of the rest pier before rehabilitation. Note the cracks in the rest pedestal and column supports at the exterior girders.

Identifying each component that needed to be repaired was the first step; the next step was to develop a construction phasing plan to minimize operational disruption during the repairs. As with any busy double track line, the BNSF could not afford to take the bridge out of service for any extended period of time. The challenge was to figure out how to create a new solid level bearing surface under the existing movable span bearings and the approach span bolster beams without taking the bridge out of service. HDR's innovative solution was to build new temporary supports that the span could rest on while the existing bearings were removed and replaced.

Because any shift of the load path away from the original bearing locations would subject the structure to stresses for which it was not originally designed, every effort was made to keep the reactions as close to the original bearing locations as possible. Due to the physical dimensions of the pier and bearings, the center of the end floorbeam was the only reasonable location for the temporary support. The exterior stringer framing location on the end floorbeam provided the best location to carry the end floorbeam with as little modification to the end floorbeam as possible.

The BNSF had anticipated replacing the approach spans to this movable span at some point in the future. HDR investigated replacing these spans in conjunction with the repair scheme described above. However, due to limited program funding and restricted work windows, BNSF and HDR decided to leave the existing approach spans in place and devise a repair plan that would include re-alignment of the approach spans and re-securing them to their supports. The best option to improve the support condition of these approach spans was to install new bolster beams immediately behind the existing bolster beams. New cast-in-place concrete would be poured to support the new movable span bearings and the new approach span bolster beams. One of the challenges of the project was developing a way to pour new concrete to create a solid bearing surface without taking any of the components of either the existing system or the new system out of service.

The Repair

The first task was to build a working platform around each pier. The platform was supported by a wide flange beam bolted to the pier with concrete anchors through an end plate connection. The United States Coast Guard stipulated that only one pier could have the working platform in place in the channel at any given time. This meant that the platform on the channel side of each pier had to be removable. Using an end plate connection allowed the platform and support beams to be easily removed. Given the restriction on the work platforms, the piers were repaired sequentially.

Once the working platforms were in place, the piers could be prepared for repairs. The cracks in the approach span rest pedestals were jacked together and then injected with epoxy. Whaler beams were used to jack the pedestals back into position. The displacement of the cracks was so large that the whaler beam system had to be detailed to accommodate this movement as the blocks were jacked back together. Before the blocks could be jacked together, the approach spans had to be unbolted from the bolster beam atop the blocks. After the blocks were jacked together, the bolster beams were reset and grout was pumped under them to ensure a solid beam seat. With the approach spans re-centered on the bolster beams, they were field-welded to hold the correct position on the bolster beams.

After the approach spans had been re-centered and reattached to the existing bolster beams, the next step was to install the new bolster beam for the approach spans, directly behind the existing bolster beams on the approach span side (See Figure 2). Stiffeners were installed for the web on the approach spans over the locations of the proposed bolster beams. The new bolster beams were installed by bolting them to the approach spans. The new approach span bolster beams would bear on new cast-in-place concrete that would be placed in the next phase of construction. Anchor bolts were hung from the bottom of the new approach span bolster beam. When the new concrete was poured, the anchor bolts would then be embedded in concrete.

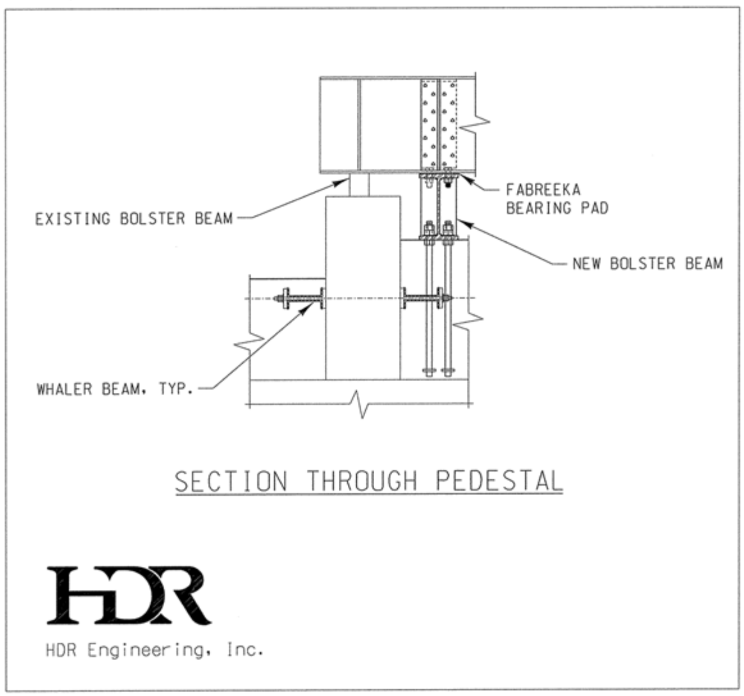


Figure 2: The new approach span bolster beam was positioned behind the existing bolster beam on the approach span side.

Temporary End Floorbeam Bearings

After the repair scenario was identified, the location of the temporary end floorbeam support was determined. These supports were placed under the outside stringers of the bridge to minimize beam stresses and modifications to the existing structure.

As mentioned previously, field investigations had identified a full-depth crack in the top flange of the end floorbeam. The top flange was built up from cover plates, and the crack propagated through every plate. A stress analysis was performed on the existing end floorbeam without the contribution of the top flange, to set a baseline for determining the stress to be utilized in the design of the temporary beam condition. The support location was chosen to allow stiffeners to be placed on the floorbeam and minimize any physical modification to the beam web. Stringer rivets were removed and the new stiffeners were bolted the same location.

The analysis of the existing end floorbeam condition without significant contribution from the top flange indicated that the primary stress in the web was 16.93 ksi. As the design began on the temporary support, the existing condition stress was used as a limiting factor in detailing beam modifications. The temporary condition would change the stress distribution in the beam, but the design ensured that the primary stress would not be exceeded. It was originally assumed that the bending in the beam would not contribute to the maximum stress in the temporary condition because of the framing system of the truss chords and stingers. Out-of-plane bending would be prevented. It was assumed that this beam would carry most of the temporary load as a shear stress.

Discussions with BNSF set some of the operational parameters that would need to be accounted for while the bridge was operating in the temporary condition. The temporary support system was not designed to allow both bearings to be removed simultaneously, in order to ensure that the lateral restraint would be provided by the opposite bearing. BNSF elected not to restrict traffic to a single track during bearing replacement; however, they did allow a speed restriction of 10 mph to be imposed upon traffic crossing the bridge.

As part of the bridge rehabilitation program, new rail joints were planned; a new top flange would be required for installation of those joints. The decision was made to install the top flange prior to the bearing replacement to provide additional capacity to the end floorbeam.

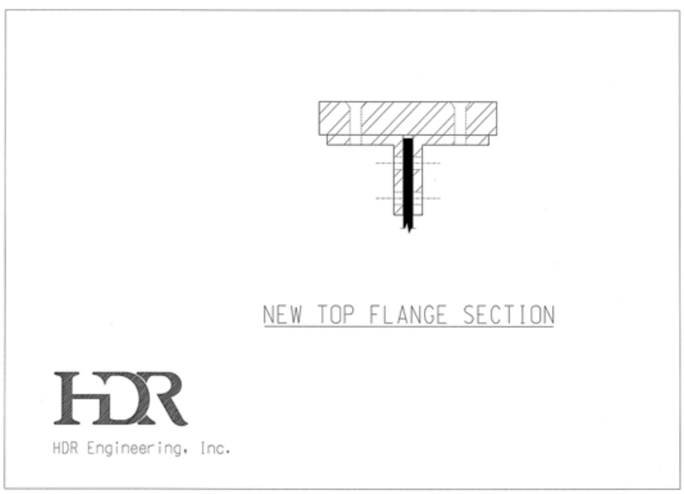


Figure 3: New end floorbeam top flange. The cover plate provides a substantial cross section to resist negative bending stress resulting from the new support condition.

A short construction window was established for the top flange change-out because both tracks over the entire length of the end floorbeam would need to be removed. In order to make the window as short as possible, the new top flange was assembled before the work window so that it could be dropped into place as a single piece. The new end floorbeam top flange was made of two angles bolted to a substantial top cover plate, as seen in Figure 3. The clearance between the two outstanding angle legs was made slightly larger than the thickness of the end floorbeam web to make sure that the new piece could be installed quickly and easily. Once the new top flange assembly was in place, thin shims were installed to ensure a tight fit between the new top flange angles and the end floorbeam web. Once the top flange was in place and shimmed to a tight fit, the bolts were installed. The BNSF elected to use Huck style fasteners to simplify the installation. With Huck fasteners, there was a lower probability of failing to achieving correct bolt torque. Once the top flange was installed and bolted up, the focus turned to replacing dozens of existing rivets with high-strength bolts. The rivet replacements occurred in the floorbeam-to-truss connection and in the floorbeam bottom flange connection outside the new supports. The high-strength bolts were necessary because these connections would experience a significant increase in load once the new H-pile support columns were in place under the end floorbeam.

With this preparatory work completed, the floorbeam was reanalyzed for the temporary condition to ensure that the web would not be stressed beyond its original stress. The analysis indicated a web stress of 11.21 ksi. The stress distribution within the beam changed as a result of the new support condition, but the maximum primary stress was actually reduced by 34 percent.

With the floorbeam top flange replaced and the end connections improved, the new H-pile support columns were then installed. The short column support members were anchored to the pier, and a grout mixture was pumped under the masonry plate to ensure a solid bearing base was achieved, regardless of concrete imperfections on the pier cap. When the grout cured, a shim was installed between the bottom of the floorbeam and the top plate on the short support column to ensure a tight fit; now the end floorbeam would seat firmly in the same spot every time on the new bearings.

Movable Span Bearings

Everything was now set so that the movable span bearings could be removed and the bridge would seat firmly on the temporary bearings installed under the end floorbeams. One of the movable span bearings was removed down to the old concrete pier cap. Trains were still allowed to cross the bridge under a slow order. Holes were drilled for new reinforcing in various locations around the existing pier cap, including the existing approach span pedestals. New reinforcing steel was installed into those holes with epoxy grout. The goal was to pour new concrete up to the bottom of the new movable span bearings and new approach span bolster beam. The new concrete would fully encapsulate the old pier cap, the approach span pedestals, and the existing approach span slender column supports at the outside stringer locations. This would be achieved in three pours. The first pour would include one movable span bearing seat and half of the approach span side of the pedestals, including the new bolster beam seat under the approach spans.

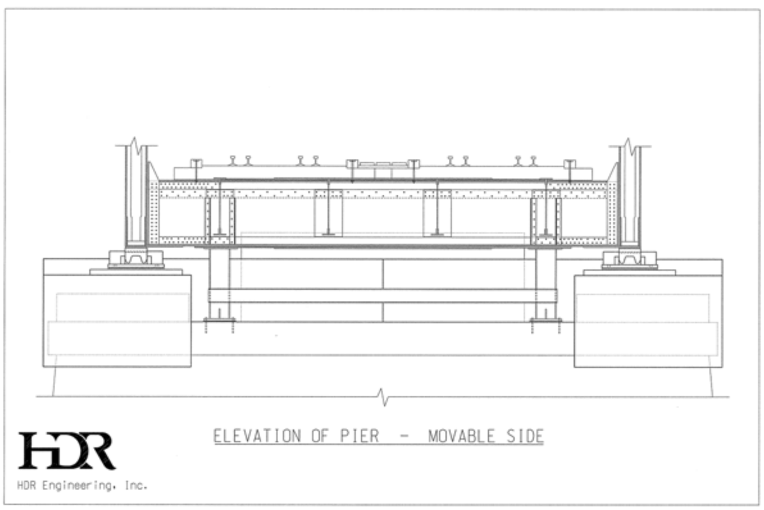


Figure 4: New concrete was poured to support the new movable span bearings. Note the temporary column supports under the end floorbeam.

After all of the prep work, the cap was thoroughly cleaned of all of the dust and debris. The first pour of concrete was placed where the new movable span bearings would be placed. During the short curing time while the concrete reached a minimum compressive strength of 3000 psi, the movable span was fitted with a new bearing yoke. The old yoke was severely worn, so a new yoke was required to fit properly on the new bearing shoe. With the new bearing yoke in place, the new masonry plate and bearing shoe were placed as near to their exact location as possible. The span was lowered to check alignment. After fine-tuning the alignment, the anchor bolt locations were marked, the new masonry plate and bearing shoe were moved, and the anchor bolt holes were drilled. The anchor bolts were set in place with epoxy. After curing, the new masonry plate and bearing shoe were placed over the anchor bolts. The span was lowered onto the new bearing shoe to double-check elevations. Minor leveling adjustments were made to ensure that the bearing yoke was bearing firmly and evenly on the new shoe. At the same time, rail joint seating was inspected for accuracy. After the bearing shoe was positively in the correct vertical position and in the correct alignment with the bearing yoke on the span, epoxy grout was pumped under the masonry plate to achieve a solid uniform bearing surface. Figure 4 shows a view of the structure after the first two concrete pours.

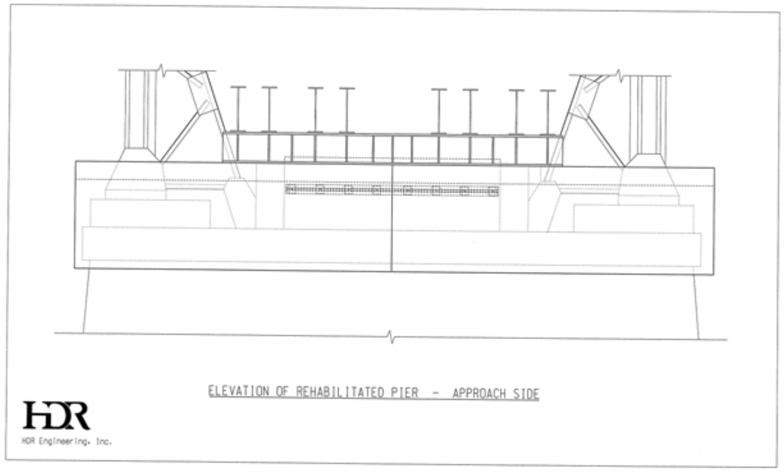


Figure 5: After the pier rehabilitation, the approach span stringers were supported on a single bolster beam which rested on a solid bearing surface of new cast-in-place concrete.

On the approach side, the new bolster beam had been installed with Fabreeka pads between the bolster beam and the approach span bottom flanges. Once the bolster beam anchor bolts were set in the new concrete, those pads were removed to allow for any live load deflection in the approach spans. This was done to ensure that no live load was transmitted to the new concrete before it was fully cured. After all of the epoxy and concrete had cured, the Fabreeka pads were reinstalled to give the approach spans a solid seating surface on the new bolster beam. The second pour followed the same procedure to reset the second new movable span bearing and the second half of the concrete under the new approach span bolster beam. The third and final pour encapsulated the bottom portion of the new short column supports under the end floorbeam and the entire movable span side of the pedestals. Originally, the short column supports under the movable span end floorbeam were going to be removed, but the BNSF was pleased with the tremendously improved ride quality and opted to leave them in. After the third pour was completed, an entirely new pier cap was present. Figure 5 shows a view from the approach span side of the pier after the pier rehabilitation was complete. The new movable span bearings and approach span bolster beam were founded solidly on new concrete. As a result of the construction, the movable span now had four bearings to rest upon, and the approach spans were actually bearing on two bolster beams. After proper curing time, rail traffic was permitted to resume normal speed. The removable pier platform on the channel side of the pier was removed, and the second rest pier was tackled in a similar process.

Rail Joints

Photograph 3: The old rail joints sat atop a steel tie that was supported by several shims of various sizes and thicknesses.

After all of the pier repairs were completed, the railroad replaced the existing rail joints. The new rail joints were of a similar style as the old joints, but they had longer rider rails to ease some of the live load impact to the structure. One of the additional benefits of replacing the end floorbeam top flanges was that the new rail joints now had a uniform surface to bolt to for the entire length of the end floorbeam. The old end floorbeam top flange had been made up of several riveted cover plates of varying lengths, and a collection of shims with a variety of thicknesses had been used to fill the odd gaps between the old end floorbeam top flange and the steel ties supporting the rail joints. The railroad had never been able to keep the joints firmly bolted to the end floorbeams with all of the miscellaneous fill plates under the joints at those locations. In the areas of the rail joints, the new top flange cover plate was bolted to the new top flange angles with countersunk bolts. This detail provided a flat surface to support a single fill plate, eliminating the need for a collection of shims to fill gaps. The multiple layers of shims in the old arrangement could not provide a tight fit, which was part of the reason that the live load impact forces had been magnified to levels that were detrimental to the structure and the pier cap.

Results

Feedback from the railroad indicates that they are very pleased with the final product. The entire structure exhibits much less movement and vibration under live load. In fact, the vastly improved ride quality at the joints has allowed track speeds to be increased by 10 mph.

There were two keys to the success of this phase of the rehabilitation. The first and most important key was that, rather than taking the bridge out of service to reset the movable span bearings, two new bearings were installed under the end floorbeams. This allowed rail traffic to continue uninterrupted, even when the existing movable span bearings were removed.

The new end floorbeam bearing columns did require that the top flanges be replaced, but that had added benefits as well. The new top flanges result in a far superior support condition for the new rail joints than the old end floorbeam top flange provided. The new end floorbeam top flanges are the same thickness over their entire length. The old top flanges were built up with a number of riveted cover plates of varying length across the length of the floorbeam, resulting in a precarious support condition for the rail joints due to the varying number and thickness of shims under each joint. Since the new top flanges are one consistent thickness, they allow for a single shim of the same thickness to be used under all of the new rail joints. Every layer of shimming that can be eliminated reduces the vibration and movement of the joints.

The second key that made this phase of the rehabilitation so successful was that no more of the existing structure was removed than was absolutely required; the old structure was repaired and incorporated into the new structure. This approach eliminated large amounts of demolition and time. The old approach span pedestals were grouted and jacked together to effectively create one new solid piece, and then they were encapsulated within the new pier cap concrete. The old approach span bolster beams were left in place, but re-leveled, grouted and re-connected to the approach spans. The addition of the new bolster beams ensures that the approach span bearings will never again be overloaded with live load impact, causing them to deteriorate to such an ineffective state.

While the bearings and pier caps themselves are holding up better in service than ever could have been hoped for, there are still further benefits to the bearing and pier repairs. Many of the mechanical system woes that had the railroad stymied for so long have settled down significantly. There was so much movement and vibration in the movable span under live load before the repairs that the machinery was being subjected to live load forces. The railroad had attempted to address one specific problem several times with only short-term effectiveness each time. The structure has four racks and four rack pinions, one set on each corner of the bridge. Time and again, the local maintenance personnel had noted that they were not able to keep solid engagement between all of the racks and all of their respective pinions. There were several occasions where there was only one pinion fully engaged with its respective rack at a time. The racks were shimmed and re-shimmed in efforts to maintain rack and pinion engagement. The racks continually needed to be re-tightened to their support structures. After the movable span bearings were reset and the joints were replaced, the railroad did one comprehensive round of adjustments with the racks and pinions, and they have not needed further adjustment since. The racks have all stayed very tightly bolted to their supports and the pinions have maintained engagement because they are no longer subjected to the effects of the live load. This unforeseen benefit has allowed the railroad to slow the pace of the next phase of rehabilitation for this structure, allowing them to redirect resources to other structures that may need more immediate attention.