Investigation and Repair of the Diefenbaker Bridge Fracture

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ABSTRACT

The Diefenbaker Bridge is a 7-span, 304m (1000 ft) long, fracture critical steel structure crossing the North Saskatchewan River in Prince Albert, Saskatchewan, Canada. The bridge is an important regional crossing and was constructed in 1959. On August 29th, 2011 a major fracture was found in one of the two steel girders of the southbound bridge. The crack extended from the bottom flange through the web of the girder, nearly full height of the girder and is one of the largest in-service bridge fractures to occur in Canada. The southbound structure was immediately closed and traffic was diverted to the parallel northbound structure. Full collapse was averted and there were no reported injuries.

This paper describes the resulting fracture investigation and major structural repair implemented on the structure. Based on this investigation, the structure is believed to be susceptible to constraint-induced fracture (CIF). To repair the structure, the fractured bridge was supported on steel towers constructed on a river berm and raised to its original position. A section of the girder was then cut out and replaced with new steel and connected to the temporarily supported horizontal and vertical bracing. This intricate procedure required innovative engineering and construction procedures. Once the structure was reopened to traffic, attention was turned towards rehabilitation including retrofit methods to mitigate approximately 160 CIF susceptible details in the southbound and northbound structures. Construction consisting of retrofit of CIF details along with rehabilitation of joints and rocker bearings was completed between May and November, 2012. The methods used to retrofit the CIF details are described along with lessons learned.

The paper will be of interest to bridge owners, designers, and anyone involved in inspecting and maintaining bridge infrastructure.

INTRODUCTION

The Diefenbaker Bridge is a 7-span, 304m (1000 ft) long, fracture critical steel structure crossing the North Saskatchewan River in Prince Albert, Saskatchewan, Canada (Figure 1). The bridge is an important regional crossing and was constructed in 1959. On August 29th, 2011 a major fracture was found in one of the two steel girders of the southbound bridge.

The crack extended from the bottom flange through the web of the girder, nearly full height of the girder and is one of the largest in-service bridge fractures to occur in Canada. The structure was immediately closed and traffic was diverted to the parallel northbound structure. Full collapse was averted and there were no reported injuries.

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BRIDGE CONSTRUCTION AND HISTORY

The structure was constructed for the Provincial Highways Department (Saskatchewan Department of Highways and Transportation), but ownership had been transferred to the municipality (City of Prince Albert) many years ago. A cost sharing agreement is in place to fund maintenance.

The bridge was designed to AASHO 1957 and A.W.S. D2.0 -1956 Specifications. The design loading was specified as H20 S16-44. The steel grade in the girders at the location of the crack was ASTM A373-56T in the web and flange plates.

The 2 lane northbound and southbound structures are classified as fracture critical structures comprised of two continuous welded plate girders per structure. The superstructures share a common substructure. Horizontal plan cross bracing was provided throughout the structure, floor beams were spaced at 11'-9" (3.581m) and vertical bracing diaphragms were spaced at 23'-6" (7.162m). Girders, stiffeners and gussets were of welded construction, while the rolled shapes in bracing and floor beams used bolted connections. (Figure 2).

The most recent and major rehabilitations were performed in 1988/89 including deck and pier cap strengthening, and deck rehabilitation again in 2003. Based on a recent loading rating the structure was known to be adequate for current legal loadings and was found to be acceptable for single permit trucks.

The structure had been inspected regularly, and maintenance had been recently recommended for deck, barriers, joints, and steel coating.

At no time in past visual inspections were cracks of any kind noted in the girders.

FRACTURE

On August 29th, 2011 a major crack was observed in the southbound structure by a passing canoeist. Police and emergency services were notified and the police closed the bridge to traffic. Stantec was retained to provide advice during the emergency, and to investigate the fracture, determine if the structure would collapse, and if the structure could be repaired. Traffic was re-routed to the northbound structure and the allowable truck weight was reduced as a precaution.

The fracture occurred in the exterior girder of span 4, near mid-span and about mid-length of the structure (Figure 3). The crack extended thru the bottom flange and the web to near the top of the web (approximately 60mm below the top flange).

Photographs were taken from an adjacent railway structure and from the police helicopter as it was unknown whether the structure was safe under its own load having lost one of the two girders supporting the structure. Even from some distance, the crack was seen to be large and was observed to have a bifurcation into a second crack extending from the main crack at about a third of the height (Figures 4 and 5).

The maximum width of the crack was estimated to be approximately 25 mm knowing the bottom flange thickness is 1.5 in. (38mm).

It was observed from photographs taken from the river that the location of the crack coincided with the location of a vertical stiffener, horizontal bracing and vertical bracing connection. This information was used to help determine an inspection and investigation plan to assess the causes of the fracture and safety of the fractured bridge and the unfractured bridge.

EMERGENCY INSPECTION AND FRACTURE INVESTIGATION

A preliminary structural analysis was done to determine stresses in both structures under dead and live loads, and answer the question of how the fractured structure was carrying its own load with one of the two girders virtually lost. This would affect the development of the plan to perform a detailed inspection and non-destructive testing (NDT) program throughout the fractured structure.

It was determined that under dead and superimposed dead loads only, the structure was supporting itself due to structural continuity of adjacent spans and load transfer from the fractured girder to the unfractured girder through the bracing system and the deck. It was decided that the visual and NDT program would proceed immediately using three (3) two-person climbing teams that could access every connection on the structure to look for signs of cracking, map the fracture location, and produce a photographic record of the two cracks and failure surfaces.

The climbing inspection was performed in only three days, which is a fraction of the time it would have taken with conventional articulated inspection truck. Photographs and other information were emailed to the design office for verification during the inspection.

The crack passed vertically through a connection between web, vertical stiffener, horizontal gusset plate, and horizontal cross bracing (Figure 6). The horizontal gusset plate was welded to the girder web on each side of the vertical stiffener and to the vertical stiffener. The horizontal gusset-to-web welds intersected the stiffener-to-web welds.

In each bridge, approximately 90 bottom connections were very similar to this detail and were visually inspected and non-destructively examined in detail. Of these, twenty (20) connections in each bridge were of the identical configuration as the location of the fracture. This raised concerns about the likelihood of similar fracture occurring at the identical 40 locations in the two bridges.

In addition to the 90 lower connection locations, over 300 other connections were also inspected and tested in the top 1/3 of the girders. Since the unfractured structure was of the same configuration as the fractured bridge, a total of approximately 780 connections were inspected and tested in the two structures. In addition, longitudinal stiffeners were welded to the web using intermittent fillet welds, were also inspected.

Although other cracking (suspected fatigue cracking) was found in other locations on the two bridges, no other cracks were found at any of the 360 lower gusset connections.

The fracture was mapped and photographed along the fracture surface of two cracks identified as ‘primary’ and ‘secondary’ cracks (Figure 7).

Initially it was postulated that the crack formed in the lower area i.e. either at the bottom flange or at the gusset connection and as the fracture progressed it then bi-furcated into a secondary crack due to the large amount of energy release associated with the primary crack propagation.

The dimensions of the crack was measured and photographed including width, length, and angle (Figure 8). The terminus of the crack tip was confirmed by magnetic particle examination so that this could be monitored. The fracture surface was photographed in detail (Figure 9).

A close up inspection of the crack surface revealed no signs of fatigue cracking as the crack surface was clean, bare steel without patterns consistent with working cracks (Fisher, 1984). The gusset-to-stiffener weld had failed as the energy and stress from the fracture led to the gusset pulling away from the vertical stiffener.

Indications of the failure surface suggested that the fracture had started at the gusset and propagated towards the bottom flange and upwards into the web and had failed in manner consistent with brittle fracture. The rupture surface chevrons in the web were pointing upwards towards the gusset below the gusset (Figure 10) and downward above the gusset.

At the start of the inspection and continuously throughout the project, the geometry of the structure was monitored by conventional survey techniques. This was essential in ensuring the safety of the structure and inspectors during inspection, and afterwards. It also formed a baseline reference to be used when restoring the superstructure to original geometry.

A few weeks after the inspection was completed, temporary access was provided to a small crew to enable taking of samples of the girder web and bottom flange, re-mapping of the crack tip using NDT, and arresting of the two crack tips by coring. The cracks had not changed from the previous inspection confirming that the structure was stable. The cores provided test coupons for tensile strength and Charpy testing. Additional CVN testing was performed at Purdue University once the fractured girder specimen was sent for examination of the fracture surface.

FINDINGS OF THE ANALYSIS AND ASSESSMENT

The finite element analysis (FEA) performed to assess the safety of the structure was extended to a more detailed three dimensional analysis in which non-linear material properties and geometric non-linearity were included. This enabled the study of the re-distribution of stresses once a crack had formed and detailed study of the re-distribution of stiffness as girders and bracing yielded and as the reinforced concrete deck cracked. The modeling confirmed that the structure was stable and that overload was not occurring other than in bracing elements two bays each way from the fracture. The predicted deformations of the fractured structure matched closely with what was measured by survey.

The fatigue assessment did not provide conclusive results that the stress ranges and estimated cycles were sufficient to exceed the design fatigue life of the connection. In addition, visual and fractographic examination of the of the fracture surface revealed no signs of fatigue crack growth. Further, since no other cracks were found in the many other locations of the same type of detail, it was concluded that fatigue was not the cause of the fracture.

The test coupons taken from the girder web and flange were tested by a certified testing laboratory to determine tensile strength and toughness. The results were consistent with yield and ultimate tensile strengths for the specified steel. The measured CVN impact energy toughness was also consistent with steel of this vintage specification. Though the CVN values of the steel did not meet modern CSA _____ or ASTM A709 requirements, it was concluded that the steel properties were not the main contributing factors to the fracture. As an aside, since the fracture occurred on a warm August day, temperature was also not a factor.

It was determined that the type of fracture was very similar to the brittle fractures that had occurred in the Hoan Bridge on I-794 in Milwaukee, Wisconsin in December 2000 (Wright et al, 2003) and in the US 422 Bridge over the Schuykill River, Pottstown, Pennsylvania in May 2003 (Kaufmann et al 2004). In all cases the brittle fractures had occurred suddenly, without fatigue crack growth.

These fractures were attributed to a relatively new phenomenon in bridges called constraint induced fracture (CIF) and occurred at gusset plate connections which had the following characteristics (Connor et all, 2005, 2007) :

- Crack-like geometric condition of gusset plates on each side of vertical stiffener
- Intersecting welds
- Resulting in highly constrained stress concentration

Since the Diefenbaker Bridge connections met all these conditions the cause of the fracture was thought to be most likely due to constraint induced fracture. A contributing factor relates to the prepared weld specified for the horizontal gusset to vertical stiffener weld. This was called up as a 45 degree bevel partial penetration groove weld with a 1/6th inch gap. A small gap such as this would is not allowed by specifications today because it does not provide enough space to fit the welding electrode. With this small gap, incomplete fusion of the weld is highly probable. The weld shown in Figure 9 at the fracture location clearly showed that fusion was not complete in the bevel weld. This served to increase the stress concentration intensifying the CIF strains and increasing the likelihood of fracture. It was significant that this prepared partial penetration groove weld existed at 180 locations in the two bridges and that these locations have high probability of having incomplete fusion. If the fractured bridge could be repaired, then a risk management solution and mitigative repairs would be required for these 180 locations. Fortunately mitigative repairs had been developed and it was expected that CIF repairs could be implemented in future after the bridge was repaired (Connor et all, 2007). The CIF repairs are described later in this paper.

REPAIR OF FRACTURED BRIDGE

A fast track emergency repair was required due to the load restriction and the serious traffic implications of requiring 4 lane traffic to function as two way traffic in two lanes. Traffic congestions and delays were daily occurrences, and the load restriction meant that truck traffic was being detoured to the next crossing some 300 km away.

A repair scheme was developed in which the structure would be supported on towers from barge or river berm, jacked upwards into position, and the girder repaired. Following this the structure would be lowered back into final position.

The entire procedure was studied using the FEA model in load steps and determined to be feasible. It was decided that a river berm was the most cost effective way to provide a platform for supporting jacking towers. In early winter 2011, a berm was constructed in the North Saskatchewan River and three large towers were erected under the structure - one tower on each side of the fractured girder and a third tower under the unfractured girder (Figure 11).

A jacking system was designed that would apply measured loads and displacements at the three load points and support the span while repairs could be made to the fractured girder. A structural monitoring system was designed and installed which would accomplish several goals:

- Monitor strains in both girders during raising and lowering of the structure
- Monitor displacements throughout
- Double as load testing instrumentation

Instrumentation was installed on the undamaged bridge and a load test was performed to determine a baseline to compare the repaired structure to when the same load test was applied to the repaired structure.

Once the jacking system and instrumentation were in installed and tested, the girders were raised in small steps to a predetermined elevations. The broken girder which had dropped and the unfractured girder which was carrying extra load due to the fracture had to be raised to elevations calculated using the FEA model. The final elevations also had to consider the cross fall in the bridge. The model was used to estimate the forces in vertical and horizontal bracing so that the elevation where the bridge was locked off would have negligible forces in these members. Note that as collapsed condition these bracing elements were estimated to have forces at or above yield so that safety was a primary concern. Strains and displacements were monitored throughout the jacking procedure. Once the girder reached their target elevations it was confirmed that jacking forces very closely matched the calculated values. The structure elevation was adjusted again, locked and a few bolts were removed from bracing elements. The team felt both a sense of satisfaction and relief when one by one the last bolt of each bracing member was removed and each brace did not shift at all which confirmed the calculations of jacking forces and elevations.

Before the damaged girder section could be cut and removed from the structure, the interior and exterior floor beam, and all interior horizontal and vertical bracing needed to be supported. Then, a section of the fractured girder was removed and a new section that had been fabricated to as-built dimensions was installed (Figure 12 and 13). Field drilled, bolted connections were used for all connections for several reasons not the least of which was to avoid welding during winter construction.

The new girder section was installed and bolted into place with web and flange splice plates, bracing elements were installed and all bolts were torqued. Strains in the girders, jacking forces and displacements were monitored throughout the process and adjustments made as necessary.

When complete, the structure was lowered in small steps, strains monitored and finally the structure was released into final position. After a period of inspection and testing, the structure was ready for load testing. The load testing was performed with the same testing truck, measured loads, and truck positions as the northbound structure. The repair passed with ‘flying colours’ and the structure repair was completed successfully on December 20th 2011.

CIF MITIGATIVE REPAIRS

It was determined that the risk associated with the 180 potential CIF locations in the structures was too great, especially in conjunction with high likelihood of incomplete fusion in the gusset to stiffener welds. A repair strategy was developed for the connection locations that had the identical or similar details and at which the girder was in a tension condition at the elevation of the gusset (160 locations). Similar in principle to the repairs in Connor et all, 2007 the repairs proposed consisted of coring (milling) a relieving hole at the location of the intersecting welds (Figure 14). The intent of the repair is to remove the intersecting welds, and points of high stress concentration that lead to CIF. The procedure was to be done with all live load off the bridge and in warm weather.

It is essential to test the jigs and procedure on a full scale mock-up of the girder web and flange geometry to ensure the right position of pilot holes, accuracy and depth of the cutting. The procedure must be performed by qualified machinists. When complete, the entire surface of the hole must be smoothed and clear of any notches or other defects so as not to introduce fatigue concerns.

The repairs were successfully completed in 2012 and at the same time rehabilitation work was carried out on deck joints and bearing. The milling repairs proved to be very effective way to mitigate the potential for constraint induced fracture (Figure 15).

A very significant finding during the repairs was the extent to which the high concern gusset-to-stiffener groove welds had incomplete fusion. Poor weld fusion was observed at a large number of connections suggesting the risk for repeat CIF would have been high had the repairs not been done (Figure 16).

CONCLUSIONS

A method to successfully repair a fractured steel girder bridge is described which included supporting the structure on large towers founded on a river berm, jacking the structure to a predetermined elevation, cutting out a section of the girder and replacing it with a new section of girder. The effectiveness of the repair was verified through a load test and structural monitoring system. The cause of the fracture was determined to be constraint-induced fracture and a method was successfully used to mitigate the 160 other locations of potential CIF in the southbound and northbound structures.

It was shown that the emergency repairs and the mitigative repairs were a cost effective way to restore the structure in to service and manage risk. This was done a cost far that was a fraction of structure replacement, and a construction duration far less than would be the case for new construction.

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FIGURES

Figure 1 Diefenbaker Bridge, Prince Albert, Saskatchewan

Figure 2 Girders, horizontal cross bracing, floor beams and k-brace diaphragms

Figure 3 Location of Fracture

Figure 4 Crack showing bi-furcation

Figure 5 Closer view of crack bi-furcation

Figure 6 Crack location at connection

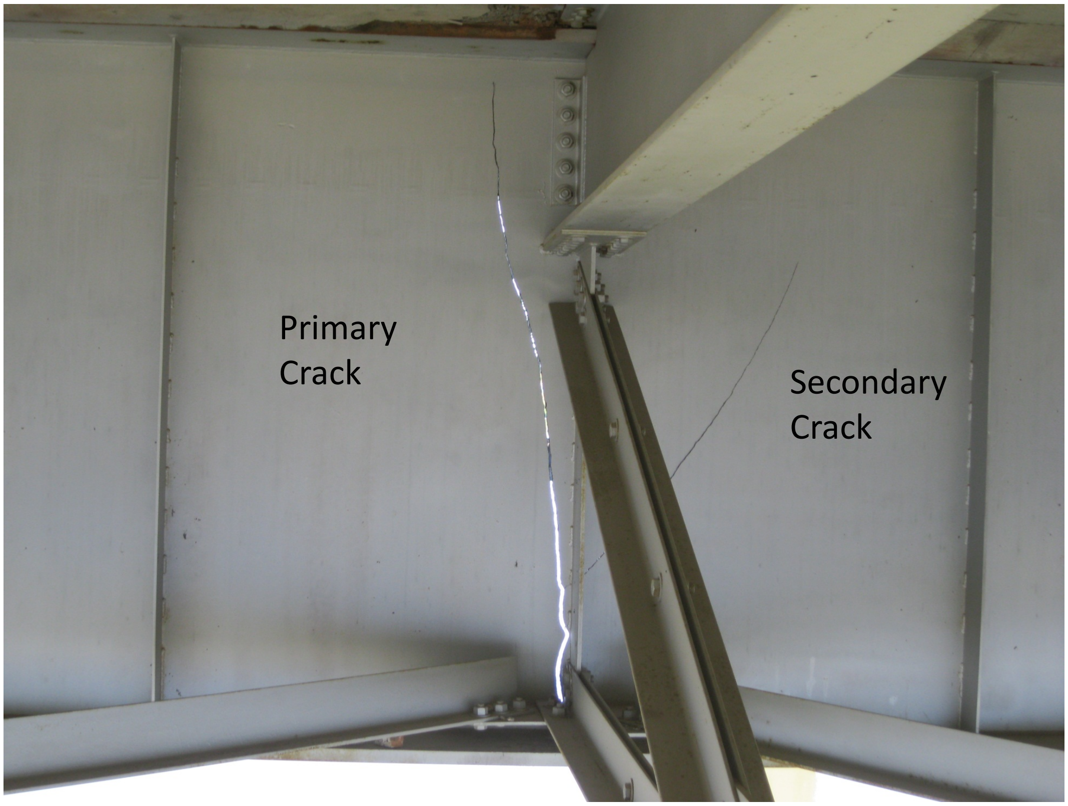

Figure 7 Primary and Secondary Cracks

Figure 8 Crack dimensions

Figure 9 Crack surface and weld fracture (underside of gusset shown)

Figure 10 Crack surface and weld fracture

Figure 11 Steel towers on river berm

Figure 12 Girder section cut out

Figure 13 New girder in place

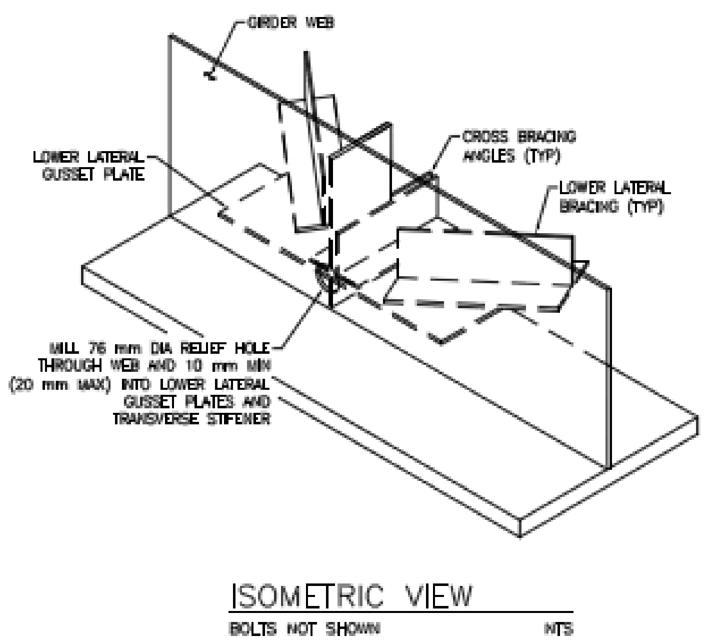

Figure 14 CIF Repair detail

Figure 15 Completed CIF repair

Figure 16 Completed repair showing exposed incomplete fusion.