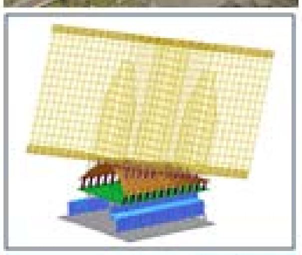
REPORT

Ministry of Transportation Nipigon River Bridge Independent Technical Review

Nipigon River Bridge
Northwestern Region
Contract#: 2013-6000

September 2016

Executive Summary

This Report was prepared by Associated Engineering (Ont.) Ltd. (AE) for the Ministry of Transportation of Ontario (MTO) to provide an independent technical review and commentary on the failure of the Nipigon River Bridge tie-down bearing bolts that occurred on the afternoon of January 10th, 2016.

The bridge is a two-span cable-stayed bridge, designed with a shorter backspan and a longer main span, creating a permanent uplift tension in the bearings at the west abutment. A similar uplift condition occurs in many cable-stayed bridges.

On Sunday afternoon January 10, 2016, the Nipigon River Bridge on Highway 11/17, a section of the TransCanada Highway across northwestern Ontario, became impassable following the complete fracture of all 40 bolts connecting the tie-down bearing to the main girder bottom flange on the north-west bearing of the bridge. The temperature on the afternoon of the failure was -16 ^{o} Celsius with northerly winds of approximately 27 km/h.

Immediately upon the failure of the bolts, the unbalanced weight of the spans, acting through the backstay cables, pulled the north-west end of the bridge deck upwards, coming to rest at approximately 600 mm above the road level. The center-west bearing did not fail, which limited further damage to the bridge.

The new four-lane bridge is being constructed in two stages, with half of the bridge completed and open to traffic at the time of the failure. There were no reported injuries to either the public or to bridge workers. The TransCanada Highway was closed to all traffic for approximately 17 hours while emergency measures were implemented to bring the bridge back down level with the roadway which allowed a return to single lane traffic. During the following weeks, two independent temporary tie-down assemblies were installed at the west abutment to allow two lanes of traffic to safely use the bridge.

Figure 1-1: Nipigon River Bridge, 600mm uplift of north bridge edge at west abutment, January 10th, 2016

The cause of the tie down failure was a progressive fracture of the 40 (ASTM-A490) bolts connecting the bearing to the girder flange over the weeks and months prior to the complete fracture of the bolt group. While the bolt group failure was immediately apparent (as seen in Figure 1-2), the progressive nature of the bolt fractures and the contributing factors have been identified through examination and testing of the failed bolts and by the analyses and assessments described in this report.

Figure 1-2: Fractured bolts (ASTM A490 bolt - connecting shoe plate to girder flange) and deformed shoe plate at north-west bearing. Note four lines of failed flange connection bolts.

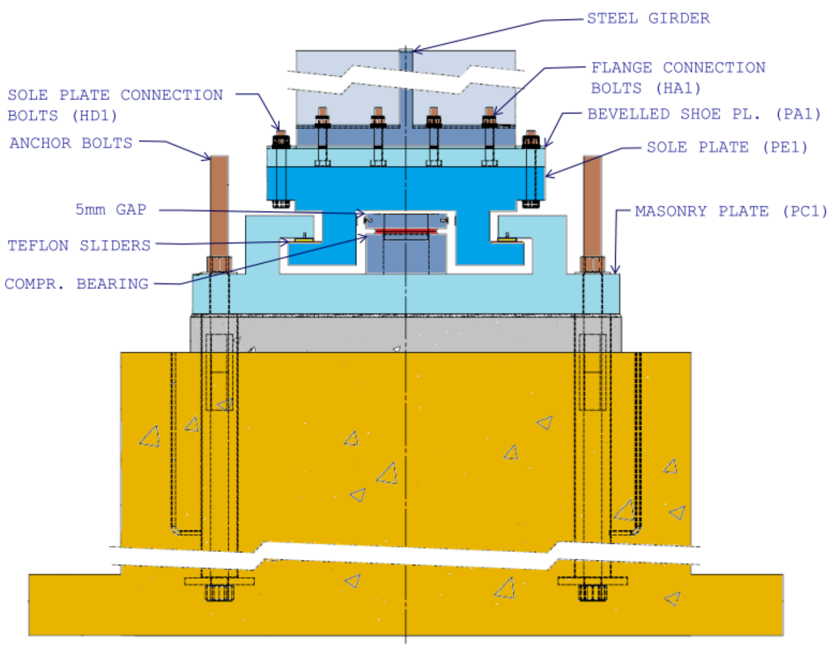


Figure 1-3: Bearing Nomenclature

Figure 1-4: Centre-west bearing in place. Bevelled shoe plate is seen between girder flange and bearing sole plate.

Key findings of this report are:

1. The bolt failure was through a progressive low-cycle fatigue fracture process induced by cyclic changes in bolt tensions in combination with localized bolt bending. The bolts were loaded in tension into the plastic range and cycled repeatedly under relatively heavy but highway legal-weight truck traffic, combined with other demands over the 42-day period following the bridge opening to traffic. As summarized below, the bolt material met specifications and was not embrittled by coatings or cold temperatures.
2. The forces and strains in the outer two lines of bolts were increased significantly by flexibility of the tapered shoe plate and related prying effects between the bearing and the girder flange. In addition to the flexibility of the shoe plate, yielding and plastic strains also occurred in the shoe plate during the bolt fracture propagation. Yielding was evident in the permanent bend of the plate following failure. Both prying effects and plastic behaviour in the shoe plate contributed to increased strains in the bolts and the progression of the bolt fractures.
3. The stiffness and lack of rotation capacity of the tie-down bearings when acting in uplift lead to significantly increased bolt forces and strains arising from the deformations imposed by the bridge superstructure. The resulting force eccentricities both longitudinally and transversely within the bearing contributed to force redistribution within the bolt group. The bearing system could not accommodate the specified or in-service demands, in that it neither isolated the bolt groups from these demands nor provided adequate capacity for the amplification of forces and strains within the flange-to-bearing bolted connection.
4. Two types of failure surface propagation were reported by the National Research Council (NRC) and by Surface Science Western (SSW). One failure surface type had striations progressing from two sides of a bolt towards an eventual ductile fracture of the centre region, and the other had striations progressing from one side, also culminating in a final ductile fracture with the bolts under high tension. These patterns indicate that bending also occurred in many of the bolts. Most

bolts showed clear evidence of low-cycle fatigue cracks. The demands and fracture propagations of the bolts were influenced by several factors at different stages of the failure progression of the 40-bolt group. These factors include high tensile strains, cyclic alternating horizontal bending of the bolts in the outer two lines of bolts, and predominantly one-sided fracture propagation in the inner line of bolts.

5. The ASTM A490 bolts connecting the girder flanges to the west abutment bearings were not pre-tensioned (tightened) during bearing installation. Additionally, the bolts supplied were too long, thread lengths were shorter than standard practice for this length of bolt, and temporary flat washers were used as an interim measure to allow the nuts and bolt threads to match during installation. The Canadian Highway Bridge Design Code requires that high-strength bolts in this type of connection be pre-tensioned (tightened). This would have introduced a large and important 'clamping' pressure between the connected plates, which acts to reduce cyclic axial strains in bolts under service loads. As long as bolt pre-tensions are not exceeded by applied loads, strains in the bolts would remain nearly constant and low-cycle fatigue failure would not occur. The lack of pre-tensioning also allowed the shoe plate and girder flange to slip horizontally, thus allowing horizontal shear forces across the bearings to transfer into the bolts. Polished surfaces on the sides of many bolts show clearly that sliding occurred. These shear forces would be small if the PTFE was functioning smoothly as intended, but increase with an increasing friction of damaged PTFE. Our assessment shows that sufficient bolt bending can be generated from this mechanism even with a modest increase in PTFE friction. This lack of pre-tension also means that initial forces and strains in various bolts are neither uniform nor predictable, and may allow for other secondary effects to occur.

6. Independent testing of the bolts by NRC and SSW concluded that the bolts met the project and Canadian Highway Bridge Design Code (CHBDC) specifications for materials, strength, ductility and low-temperature toughness. The bolts met the most stringent low-temperature toughness requirements required for a bridge located anywhere in Canada. The bolt coating was appropriate for the ASTM-A490 bolts and did not materially affect the bolt ductility or other properties.

7. The global finite element analysis of the bridge confirmed that the uplift force on the north-west bearing before the failure occurred was in the order of 1,720 kN for permanent loads only, which was in general agreement with the design. However, based on our analysis results and design checks, the west abutment bearings were unable to resist either service or ultimate (factored) uplift loads and other demands specified on the bridge contract drawings while meeting the design requirements within the CHBDC. The bearing design did not comply with the requirements of the contract.

8. Passage of heavy but legal trucks over the bridge, combined with the out-of-parallel bearing condition arising from the installation methods, would be sufficient to cause permanent deformations (plastic strains) in bolts from changing uplift reactions and axial forces in the critical bolts at the north-west bearing. Overload permits issued by MTO indicate that just under ninety trucks over 60 tonnes (the weight of a code design truck) potentially crossed the bridge. The passage of these vehicles would be sufficient to contribute to the accumulation of low-cycle fatigue fractures of the bolts. The number of heavy trucks crossing the bridge is similar in magnitude to the number of crack propagation cycles seen in the striations on the fracture surface of some of the bolts.

9. Design-based wind effects (for a relatively short 10-year return period such as typically used for bridge construction demands) were analyzed and found to increase uplift forces by approximately 10% of the reaction caused by bridge self-weight. This is not sufficient to contribute significantly to plastic strain accumulation in the bolts. The bridge was unlikely to have experienced winds of this magnitude prior to the failure. As such, the bearing failure was unlikely to have been influenced materially by wind effects.
10. Cold temperature effects (cable shortening, deck shortening and deformations) were analyzed and also found to increase uplift forces by approximately 10%. These increases would have occurred in combination with truck passage effects, both for uplift and for deformations at the north-west bearing, and may have accelerated but not materially changed the bridge failure mechanism. Thermally induced axial deformations in the bridge superstructure would also have affected shear forces in the northwest bearing as described above, and would have contributed to bolt crack propagations.

The two west abutment bearings supplied had the same design uplift capacities. The north-west bearing failed, while the centre-west bearing did not. The north-west bearing has higher uplift demands at this stage of construction than the centre-west bearing, but the latter will have approximately double the uplift reaction of the north-west bearing in the completed bridge. These higher loads at the centre girder governed the bearing selection. That the northwest bearing failed at demands of only half of their maximum design demands illustrates that the bearing assembly's capacity, as affected by installation, was substantially deficient. Several conditions at the two bearings that affected bolt tensions and bolt bending could have been sufficiently different that the bolt cracking and fracture propagation had not yet occurred at the center bearing. Bolt polish marks were also observed on the intact bolts of the centre bearing suggesting that it was also experience bolt bending and was prone to a fracture similar to the north-west bearing failure.