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°C 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
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 pretensioned (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.

11. 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.
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<td>A325 bolt</td>
<td>Structural bolt produced in accordance with ASTM material specification A325. This is a commonly used structural bolt.</td>
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<td>A490 bolt</td>
<td>Structural bolt produced in accordance with ASTM material specification A490. This bolt has a higher strength, and less ductility, than an A325 bolt.</td>
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<td>Abutment</td>
<td>Concrete supporting structure between the bridge superstructure and the foundation.</td>
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<td>An addition or revision required to be made to a contract document by its author subsequent to its issue.</td>
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<td>Bolts connecting the steel bearing masonry plate to the concrete abutment.</td>
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<td>Articulation</td>
<td>The arrangement and details of bearings and joints in a bridge that accommodate movements. The articulation also affects how forces are generated or resisted by these movements.</td>
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<td>Beach Mark</td>
<td>A distinct curved line on a fractured surface that delineates a change in condition on the crack propagation pattern.</td>
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<td>Bearing</td>
<td>A support between the bridge girder and concrete abutment. Movements and rotations are inherent in the regular service of a bridge. The purpose of a bearing is to support the bridge, while accommodate these movements without imposing additional stresses on the structure.</td>
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<td>Bearing assembly</td>
<td>The bearing and its connecting parts above and below.</td>
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<td>Cable stay</td>
<td>Cable supporting the bridge deck from the bridge tower.</td>
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<td>Canadian Standards</td>
<td>A not-for-profit organization which develops standards in 57 areas of specialization. CSA, along with experts from practice and academia, produced the Canadian Highway Bridge Design Code.</td>
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<td>Association (CSA)</td>
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<td>Charpy test</td>
<td>A standardized test that determined the amount of energy absorbed by a material during fracture. This can be used to describe the material's ductility.</td>
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Compression  A physical force that pushes the ends of an element outwards its centre. In this context, it generally refers to bolts and bearing surfaces experiencing a compression force.

Contract Administrator  Company or individual responsible for the administration of the contract on behalf of the MTO.

Contract drawings  The bridge design drawings prepared by the Designer for the tendering and construction of the bridge.

Deformation  Physical movements of the bridge elements as a result of imposed forces. This includes linear movements and rotations.

Degrees of Freedom  A direction in which independent motion can occur. These include translations and rotations.

Designer  A PEO-registered professional engineer responsible for the bridge’s structural design.

Ductility  The ability of a solid material to deform beyond its elastic limit while sustaining load. This also allows the material to absorb energy before fracturing.

Eccentricity  The uneven (not centered) application of a force which causes additional stresses on a structure or component.

Elastic  The portion of a strain or change in shape of a structurally loaded element that is fully recovered when the load is removed.

Engineer of Record (EoR)  See Designer

Fabrication drawings  Bearing drawings used for fabrication. Also called working drawings or shop drawings depending on the contractual function.

Finger joint (expansion joint)  Joint in the bridge deck of corresponding metal fingers (teeth) with a gap between them, allowing for the free expansion and contraction of the bridge due to temperature.

Flange  The horizontal top and bottom plates of the girder.

Girder  A main structural supporting member of the bridge. The girder supports the bridge deck, and is in this case supported by cables attached to the bridge tower. The girder rests directly on the bridge bearing.

Global analysis  An analysis of the overall structural behaviour of the bridge. In this context, it was used to determine the forces and deformation imposed at the bearing from the bridge in service.
Issued for Construction (IFC) documents: Documents issued by the bridge Designers, containing instructions, dimensions, and bridge components, from which the bridge is constructed. Also called contract drawings in this context.

Issued for Tender (IFT) documents: Drawings and specifications issued to potential bidders to allow them to prepare financial bids to construct the bridge.

Load path: The physical path of internal forces through and between structural elements, from their point of application to the ground.

Local analysis: A detailed analysis of one component of a structure. In this context, the structural analysis of one bridge bearing.

Low-cycle fatigue: The progressive crack propagation of a material by the repeated application of loads that are large enough to cause plastic strain of the material.

Masonry plate: Bearing component. The steel bottom plate of the bearing, which is in contact with the concrete abutment.

MTO: Ministry of Transportation Ontario

Notch toughness: In the context of the bolts, the ability of the bolt material to remain ductile at cold temperatures.

Phase One: This bridge is being built in two phases. Phase One includes the north and central towers, cable planes, girders, and bearings. The two northern lanes are open to traffic at the conclusion of Phase One, with one West-bound lane and one East-bound lane. This was the service condition at the time of the northwest bearing failure.

Phase Two: This bridge is being built in two phases. The conclusion of Phase Two constitutes the final bridge configuration of four traffic lanes.

OPSS: Ontario Provincial Standard Specification

Piles: Vertical structural elements driven or drilled into the ground that transfer forces from the bridge to the soil.

Plastic: The portion of a strain or change in shape of a structurally loaded element that is not fully recovered when the load is removed.

Pre-tensioned: In this context, high-strength bolts are installed first to a ‘snug tight’ condition, and then are pre-tensioned using impact wrenches to elongate the bolt to a very high stress, at or near it’s initial yield stress. This imparts a high clamping pressure to the connected plates.
<table>
<thead>
<tr>
<th>Term</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prying</td>
<td>A phenomenon in bolted construction that describes additional tensile forces resisted by the bolts as a result of the connected steel plate deformation.</td>
</tr>
<tr>
<td>Pylon</td>
<td>See Tower.</td>
</tr>
<tr>
<td>Quality Verification Engineer (QVE)</td>
<td>means an Engineer qualified to provide the quality verification engineer services specified in the Contract Documents.</td>
</tr>
<tr>
<td>Redundancy</td>
<td>The existence of more than one structural load path. If a component of the structure is damaged or removed, another component will transfer the load.</td>
</tr>
<tr>
<td>Rotation</td>
<td>The circular movement of an element around a central point, relative to another element.</td>
</tr>
<tr>
<td>Shoe Plate</td>
<td>Steel plate between the girder and the bearing. It is bevelled along its length to account for the slope of the bridge roadway.</td>
</tr>
<tr>
<td>Shop Drawings</td>
<td>See Fabrication drawings</td>
</tr>
<tr>
<td>Sole Plate</td>
<td>Bearing component. The steel top plate of the bearing, which is in contact with the shoe plate.</td>
</tr>
<tr>
<td>Staged construction</td>
<td>The construction of the bridge in multiple stages (in this case two), to allow for partial use of the bridge while the remainder is under construction.</td>
</tr>
<tr>
<td>Strain</td>
<td>In this context, a relative change in length of a material due to compression or tension.</td>
</tr>
<tr>
<td>Stress</td>
<td>A measure of the average internal force per unit area of an element, as a result of applied external forces. Each material has its own stress limit, after which point continued application of the external force will result in permanent deformation of the material.</td>
</tr>
<tr>
<td>Substructure</td>
<td>The portion of the bridge that supports the superstructure and transfers the structural loads to the foundation of the bridge.</td>
</tr>
<tr>
<td>Superstructure</td>
<td>The portion of the bridge between supports that directly receives the traffic loads. This includes the bridge deck, girders, and in this case cables.</td>
</tr>
<tr>
<td>Tension</td>
<td>A physical force that pulls the ends of an element outwards from its centre, or pulls two elements apart. In this context, it generally refers to bolts, or cables, experiencing a stretching force.</td>
</tr>
<tr>
<td>Term</td>
<td>Definition</td>
</tr>
<tr>
<td>----------------------</td>
<td>-------------------------------------------------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Tie-down Bearing</td>
<td>A structural <em>bearing</em> that must resist uplift forces in <em>tension</em>, as opposed to the more common compression <em>bearing</em>.</td>
</tr>
<tr>
<td>Tower</td>
<td>The main vertical member near the centre of the bridge from which the cables are supported, which in turn support the bridge deck.</td>
</tr>
<tr>
<td>Translation</td>
<td>Linear movement of structural elements; up, down, or sideways.</td>
</tr>
<tr>
<td>Working drawings</td>
<td>See <em>Fabrication drawings</em>.</td>
</tr>
</tbody>
</table>
1 Introduction

1.1 NIPIGON RIVER BRIDGE PROJECT – CONTRACT #: 2013-6000

The Ministry of Transportation tendered Contract 2013-6000 for the replacement of the Nipigon River Bridge Project on March 6, 2013. This Group “B” project as defined under the Class Environmental Assessment for Provincial Transportation Facilities (2000) included both the replacement of the existing structure as well as highway realignment and upgrades. Highway 11/17 is a strategic link in the TransCanada Highway system and, at the Nipigon River there is no alternate route across Canada. The primary objective of this project was to improve and renew this critical link for the country. The expected fixed completion date for this contract was estimated to be July 28, 2017.

1.2 HIGHWAY REALIGNMENT

The highway component of this project included upgrading the existing cross section to four lanes complete with wider shoulders to enhance traffic operations and safety. The additional tie in to the eastbound Highway 17 truck climbing lane and complete illumination to the 2 lane transition was included as well. Critical design issues included, increasing the median width to 3.8m with future provisions for a barrier, a grade raise of approximately 2.1m across the project site to enhance drainage and staging/constructability as traffic must be maintained throughout the duration of construction. The bridge is on a tangent section with constant 2% cross-fall from the road centreline, and a constant 0.8% longitudinal downhill grade toward the east.

1.3 BRIDGE REPLACEMENT

The centre piece to this contract is the construction of the new Nipigon River Bridge which is a signature cable stayed structure. The cable stayed structure is being constructed in two phases with the northern half built in Phase One and the southern half in Phase Two. Phase One has been in operation since November 7, 2015, and carries two lanes of opposing traffic. Phase Two construction is under way, and will be completed with ongoing traffic service of Phase One lanes. The structure includes 3m shoulders to enhance safety and provide future expansion capacity of the highway as well enhance deck structure drainage.

1.4 UPLIFT EVENT & SUBSEQUENT INTERMEDIATE REMEDIAL WORKS

On the afternoon of Sunday, January 10, 2016, the Nipigon River Bridge on Highway 11/17, the TransCanada Highway across northwestern Ontario, became impassable following the complete fracture of all 40 bolts at the northwest bearing of the bridge. The temperature on the afternoon of the failure was -16°C with winds of approximately 27 kph to the North (as recorded from Katatota Island weather station). Phase One of the four-lane bridge had been constructed at the time of failure. Temporary tie-down assemblies were subsequently installed at the west abutment to allow two lanes of opposing traffic to safely use the bridge. Two independent tie-down systems were installed to provide redundancy and safety. Permanent repairs are currently under design development and will be instituted to facilitate continued construction of the structure’s second stage.
2 Project Scope

The Ministry of Transportation Ontario (MTO) requested the services of Associated Engineering (Ont.) Ltd. in February 2016 to provide an independent technical opinion of the observed uplift event that occurred on the Nipigon River Bridge. The scope of this report is outlined below.

AE was retained to:

i. Undertake an independent inspection of the bridge in its current conditions and to inspect the failed bolts as examined and tested by National Research Council (NRC), Surface Science Western (SSW), and MTO.

ii. Analyze the bridge using an independent finite element model of the global structure, and derive demands exerted on the bearing assembly and abutments from the superstructure.

iii. Analyze and evaluate the local bearing assembly utilizing independently verified demands from the global model.

iv. Determine the causes of failure that resulted in the uplift event based on all available information provided or developed independently.

All of the documents reviewed by AE were provided by the Ministry through the project filing site, MTO RAQS MERX, MTO Emails and Emails from the Designer, Contractor and MTO.
3 Project Approach

AE’s approach to evaluating the cause of the uplift event including the following:

- Visit the Site to make observations of the existing conditions following the uplift event inspect, the bridge, and observe general conditions on site.

- Review design and construction documents produced by the Engineer, Contractor, Contract Administration Consultant and MTO.

- View, compare and inspect the bolts as tested by the National Research Council (NRC), Surface Science Western (SSW), and those retained by MTO.

- Document and summarize all bolt observations and tests and combine bolt observations and testing into our failure assessment.

- Develop a global finite element model of the bridge based on as-constructed records to determine and confirm the demands throughout the structure. These results used for component evaluations and the bearing failure assessment.

- Develop local models of the bearing to evaluate the demands placed on various components of the bearing assembly utilizing both design and erection information from cable stressing records.

- Develop and evaluate bearing failure mechanisms in conjunction with observations and measurements, finite element models, bolt reports, and engineering interpretation.
4 Site Visit Observations

4.1 BEARING ARRANGEMENT

The nomenclature used to describe the bearing components within the site observations, and throughout the report, is summarized in Figure 4-1.

The bridge bearings consist of a masonry plate anchored to the concrete abutment, disc bearing transferring compression forces, sole plate, and bevelled shoe plate. The sole plate is bolted to the bevelled shoe plate, which is bolted to the steel girder bottom flange. Steel guides connect the sole plate and masonry plates, providing guidance and resisting uplift. Since transverse restraints are provided, the bearings are not required to transfer transverse load from the superstructure to the abutment.

Due to the asymmetric bridge span arrangement, the west abutment bridge bearings experience tension (uplift) due to permanent loads, as well as a combination of service level permanent and transitory loads. By inspection of the bearing arrangement, we note that there is no mechanism provided for in the design of the bearing to freely accommodate rotation while subjected to uplift.

4.2 DETAILED STRUCTURE INSPECTION – APRIL 2016

As part of AE’s scope, a detailed site investigation was undertaken in April 2016. The detailed inspection plan and observations are given in Appendix A, together with sketches used to capture measurements on site.

The site visit included the following tasks:
• A brief inspection of the bridge as a whole, which included the bridge deck, central tower, stay cables and anchors and substructure foundations.

• Close up examination of all of the bearings and adjoining bridge components at both bridge abutments

• A study of the shoe plate. Measurements were taken of the deformed shape of the north-west bearing shoe plate, and compared with the MTO measurements.

• A study of the cut-off ends of the bolts at the MTO offices in Thunder Bay which remained in the shoe plate. The exposed fractured ends of the bolts had been cut off to enable the reinstatement of bridge deck levels prior to opening the bridge to traffic.

• Brief discussion with Ministry representatives on site to better understand procedures followed during the installation of the bearings.

4.3 INSTALLED BEARING CONDITION

The bearing installation methods are important as they affect the demands on the bearing assembly. A review of available photographs of the bearing installation indicate that the shoe plate was affixed to the girder flange and the bearing sole plate slid into position, and connected to the underside of the shoe plate. (Figure 4-2). The bearing sole plate was set level onto the concrete abutment.

The fixed dimensions of the pre-fabricated shoe plate, and its attachment to the girder flange in this sequence, introduced dimensional imperfections between the upper and lower portions of the bearing when it was subsequently connected to the level set masonry plate.

![Figure 4-2: Construction photo of bearing installation showing the shoe plate bolted to the girder flange](image)

The predicted slope and elevation of the girder on the contract drawings at the time of bearing installation is based on an idealized condition. Considering the complexities of segmental cable stay bridge construction and expected dimensional imperfections in the girder bottom flange during manufacturing, the perfect position of the bearing would need to be accounted for during its installation by adjustments in the field, either to the shoe plate dimensions or grouting below the bearing in its neutral position. Due to the
installation sequence used at this bearing, these variations were not accounted for during bearing construction. Any dimensional variations in slope and elevation would have been imposed on the bearing assembly.

Measurements of the relative alignment of bearing components were taken on site to gain an understanding of the installed condition. The following points are relevant considering when the measurements were taken:

- The bearings were disconnected from the deck superstructure at the west abutment
- The deck was being held down by temporary holding down devices installed after the bearing failure
- We assume that the girder bottom flanges and bearing masonry plate are as they were prior to the failure when measuring the difference between the four corners of the shoe plates along the girder bottom flanges

Measurements were taken on all four corners of each installed bearing, from the top of the bottom flange to the top of the masonry plate. See Figure 4-3 and Table 4-1.

![Figure 4-3: Measurements taken at the north-west bearing to establish variations in the alignment](image)

<table>
<thead>
<tr>
<th>Date</th>
<th>Temp °C</th>
<th>Bearing numbering as per Figure 4-3</th>
<th>NW</th>
<th>SW</th>
<th>NE</th>
<th>SE</th>
<th>Length of shoe plate (l) mm</th>
<th>Width of bottom flange (B) mm</th>
<th>West edge thickness of shoe plate mm</th>
<th>East edge thickness of shoe plate mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>15th April</td>
<td>18</td>
<td>1</td>
<td>425</td>
<td>428</td>
<td>416</td>
<td>413</td>
<td>1000</td>
<td>630</td>
<td>60</td>
<td>52</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>427</td>
<td>432</td>
<td>415</td>
<td>420</td>
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<td>3</td>
<td>181</td>
<td>180</td>
<td>180</td>
<td>180</td>
<td>800</td>
<td>630</td>
<td>60</td>
<td>54</td>
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<td>4</td>
<td>185</td>
<td>190</td>
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<td>190</td>
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</tr>
<tr>
<td>20th April</td>
<td>6</td>
<td>1</td>
<td>424</td>
<td>429</td>
<td>415</td>
<td>413</td>
<td>1000</td>
<td>630</td>
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<td>52</td>
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</table>
This translates to measured vertical variations along the length of the shoe plate and across the width of the bottom flange at each of the four abutment bearings as shown in Figure 4-4. Since the measurements were taken after the failure, with the temporary tie down system in place bypassing the bearing, all measurements should have been zero had the bearings been installed in a perfectly neutral position. The variations indicate the out-of-parallel condition imposed on the bearing during construction.

Figure 4-4: Plan view schematic of the bearings with measured vertical variations. A negative value (-) is down, and a positive value (+) is up.
The corner with the largest positive (+) difference would have been the first corner of the sliding path to be subjected to wear. Based on the measured variations at the north-west bearing, we would expect the PTFE along the lower sliding surface to experience the highest pressure and to wear first at the southwest corner of the bearing. This is consistent with our observations of the distorted PTFE in this location, as shown in Figure 4-5.

![Distorted PTFE at the southwest corner of the north-west bearing](image)

**Figure 4-5: Distorted PTFE at the southwest corner of the north-west bearing. These photos were taken of the unloaded bearing after failure.**

Given these measurements and observations of the bearing’s installed condition, we expect that significant eccentric forces and high local compression stresses to the PTFE would have been imposed on the bearing from the time of installation. This would have resulted in an uneven force distribution among the flange connection bolts under an uplift condition, prior to the application of any traffic or climatic loads.

4.4 **FLANGE CONNECTION BOLT INSTALLATION**

Based on observations made from available bearing installation photographs, and a review of construction documentation, we note changes made between the flange connection bolt design and the as-installed condition. This section summarizes these changes and their impacts to the bearing performance.

4.4.1 **Bolt Length**

The flange connection bolts were erroneously specified to be too long for the north-west and centre-west bearings. Additional flat washers were installed to accommodate the longer bolts and the elevated threaded portions.
4.4.2 Lack of Bevelled Washers

The contract drawings show bevelled washers for flange connection bolts, to account for the slope of the girder bottom flange in relation to the bearing sole plate. The bevelled washers were not installed. Instead flat washers were used.

The installation of the flat washers is not in accordance with the requirements of the CHBDC, which states that “in the case of ASTM A490 bolts, bevelled washers shall be used to compensate for any lack of parallelism due to the slope of outer faces.”

The installation of flat washers instead of the specified bevelled washers would have introduced a local eccentric force to the bolts as a result of bolt tightening or from axial loads from the uplift force in service. A sketch showing this local force eccentricity is shown in Figure 4-6. At some locations, when considered together with the local deformations of the shoe plates and the girder bottom flange in uplift, the local force eccentricity accentuates bending in the bolt.

![Figure 4-6: Eccentric force application on the bolt as a result of the use of flat washers](image)

4.4.3 Bolt Pre-Tension

Pre-tensioning of bolts, by tightening of the nut to a specified level, provides an important clamping force that affects the connection behaviour. This force must be overcome before bolt deformations are able to occur. Any tension forces in the service of the connection will not begin to affect the bolt until the pre-tension level has been reached. Pre-tensioning is especially important for bolted connections on bridges due to the cyclic nature of the loads. A lack of pre-tension increases the bolt’s potential vulnerability to fatigue.
Available construction correspondence documents indicate that the bolts were not pre-tensioned as required by the CHBDC and MTO specifications. This requirement is described further in Section 6.

Furthermore, a close-up bolt photo during construction (see Figure 4-7), shows variations in the observed bolt protrusion above the nut, which are not related to the bevel of the shoe plate.

![Figure 4-7: Photo of the Flange Connection Bolts During Construction](image)

The variations of bolt protrusion above the nut between the outer and inner lines of bolts (for example $\delta_5 > \delta_3$ labeled in Figure 4-7) indicates that, at the time this photo was taken, some outer line bolts may not have been fully tightened. Since the sole plate had already been slid into position on the abutment, access to the bolt head would not have been possible, and consequently, difficulties may have been encountered that prevented further bolt tightening. Measurements of the bolts during our investigation show that some M22 bolt heads have a minimum dimension of 39.55mm across their corners. Since the recess for the bolt heads at the underside of the shoe plate are dimensioned as 40mm x 44mm, it is possible that bolt heads would have slipped in the recess, preventing further tightening.

Immediately after failure of the north-west bearing, many of the bolts at the centre-west bearing needed at least a quarter turn of the nut to reinstate snug tightness in the bolts (see Figure 4-8). This indicated that many of the bolts were loose prior to and at the time of failure which could be interpreted that the same condition existed at the north-west bearing.

The lack of documented pre-tension and observed variations in bolt protrusion indicate that there would have been a significant variation in the tightness of the flange connection bolts in their installed condition. This would have contributed to uneven force distribution among bolts, with the tighter bolts initially experiencing higher load. Other implications are described throughout this report.
4.5 SHOE PLATE OBSERVATIONS

Permanent deformation of the north-west bearing shoe plate was observed after the failure. (as seen in Figures 1-2 and 5-5)

MTO provided drawings prepared internally that included detailed information of the measurements of the permanently deformed shape of the north-west bearing shoe plate. Figure 4-9 shows the drawing and the measurements (shown in black) of the post-failure deformed shape as prepared by MTO. These measurements were verified independently during our study of the plate and are given in red. Both sets of measurements are in agreement, and indicate the gap between the shoe plate bottom face and a level placed across the shorter dimension of the shoe plate.

Figure 4-8: Note the markings on the centre-west bearing connection bolts. Close up of markings in the insert and general view of the connection bolts.
The following is noted from the shoe plate measurements:

- The permanent deformations observed in the plate provides evidence of yielding of the shoe plate.
- The maximum deformation of the shoe plate occurs at its thinnest edge and is equal to 7.5 mm. The maximum deformation along the thicker edge of the shoe plate is 5.7 mm. This difference may be due to the reduced stiffness of the thinner section of the plate.

- The point of maximum deformation is located just to the left of the plate’s centre. This could indicate that the bolts in the inner line in the northeast quadrant were the last bolts to fail. Bolts in the southwest quadrant, with the least measured plate deformation, may have been the first to fail.

- The recesses were measured to be generally 40 mm x 44 mm x 18 mm which confirmed the dimensions given on the fabrications drawings.

- The measured diameter of the bolt holes also confirmed the dimension of 25.4 mm given on the fabrication drawing.

- Measurements at each plate cross section indicated a single-curvature plastic deformed shape.

The deformed shape of the shoe plate is consistent with the outer line of bolts having higher initial tensions and likely failing before the inner line. This can be explained by a mechanism called prying, which is shown schematically in Figure 4-10 below. This sketch shows how the outer line of flange connection bolts would have been subjected to a higher portion of the uplift force due to the flexibility of the shoe plate and the placement of the outer line of bolts connecting it to the sole plate.

**Figure 4-10: Load path of an applied uplift force showing an exaggerated shoe plate deformation**

The permanent deformation of the shoe plate confirms that the design of the shoe plate and bolted connection arrangement were inadequate for the transfer of service loads in uplift from the girders to the bearings. This is consistent with the findings of our bearing analysis discussed in further sections of the report.
5 Bolt Investigation Review

As part of the investigation, an independent in-depth review of the available documentation was undertaken.

The following Technical Reports were produced from the evaluation of the failed bolts:

1. Evaluation of Failed Nipigon River Bridge Bolts by NRC Construction, National Research Council Canada (NRC), Ontario Ministry of Transportation in Ottawa

2. Report on the Evaluation of the Bolts provided by MTO by Surface Science Western (SSW) at the University of Western Ontario

Upon investigation of the failed bearing immediately after it happened it was found that 39 of the 40 bolts holding the girder to the northwest shoe-plate had failed by fracture in the threaded portion at or near the nut, while only one had failed at the bolt head.

Not all of the bolts recovered from the site after the bearing failure could be matched and located on the shoe plate as the nuts were found to be loose on the bridge or had fallen into the snow around the bearing. The shaft ends of the bolts still contained within the shoe plate, which were still attached to the bearing sole plate with the matching fracture surfaces, were cut off to allow the reinstatement of the roadway level and enable the bridge to be re-opened to traffic. These cut off sections of bolts were also not properly marked such that they could be relocated on the shoe plate.

Photographs were taken of the tops of the lower portion of bolts left behind on the shoe plate immediately after the failure. These photographs together with detailed close up photographs from the forensic investigation of the fracture surfaces of the failed bolts were used to match the bolts and relocate them onto the shoe plate; 29 of the 40 bolts (73%) that failed were relocated on the shoe plate. This is sufficient to establish the patterns of failure expected to have developed during failure.

Of the total 40 bolts that failed 14 were sent to NRC, 14 to SSW and 12 remained at the St Catherine’s MTO offices for study by the MTO Bridge Office staff.

5.1 NATIONAL RESEARCH COUNCIL (NRC) REPORT SUMMARY

The National Research Council (NRC) developed a report for the Ministry to evaluate the failed bolts and outline particular failure modes. The following is a summary of the pertinent conclusions from the NRC Report:

a. 14 failed bolts from the NW bearing plus 10 intact bolts from the CW bearing were received from MTO for testing and analysis.

b. Bolts met the chemical composition and mechanical performance requirements of the standards
c. Cracks were found in the crest and flanks of both the intact and failed bolts, but no evidence was found that they contributed to the bolt failure. They were likely present at the time of installation.
d. Cracks that led to the bolt failures initiated at the thread roots
e. Neither bolt composition, bolt mechanical performance nor the coating appeared to be responsible for the bolt failures
f. Analysis of the fracture surface showed the following:
   i. Bolts failed due to low-cycle (ductile) fatigue, with between 50 and 140 cycles occurring between crack initiation and final fracture. See NRC Report Table 14 on Page 57
   ii. Low-cycle fatigue occurs when plastic behaviour predominates during fatigue and is controlled by changing strain levels instead of stress levels
g. Bolts failed due to experiencing high cyclic loads, which caused fatigue cracks to initiate and propagate until the individual bolt’s load capacity was exceeded and the final fractures occur
h. As the bolts did not fail simultaneously, it is likely that full extent of the high loads initially affected some bolts only. Once those failed, the high loads were then transferred to other bolts until all failed.

5.2 SURFACE SCIENCE WESTERN (SSW) REPORT

Surface Science Western (SSW) at Western University developed a report for the Ministry to evaluate the failed bolts and outline particular failure modes. The following is a summary of the pertinent conclusions from the SSW Report:

a. 14 fractured bolts from the NW bearing plus 10 intact bolts from the SW bearing were delivered by MTO.
b. All bolts satisfy the chemical composition requirements for Type 1 and Type 3 bolts as specified in ASTM A490-14a.
c. The metal undergoes ductile failure when loaded at temperatures of -20°C to -30°C.
d. Four intact bolts were tested with a 10° wedge under the head and four more with a 1° tapered washer under the round washer and nut. All met strength requirements of ASTM A490-14a.
e. The intact bolts fractured under monotonically increasing tensile load. All regions of the fracture surface, flat or angled, displayed cup-and-cone features characteristic of a ductile fracture process.
f. Both the intact bolts and the fractured bolts fractured by crack initiation and growth from the root of the thread nearest to the nut.
   The majority of the fractured bolts displayed:
   i. predominantly flat regions, comprised of flat and highly cracked features, extending from the thread root toward the center of the bolt
   ii. The central region of the fracture surface usually contained a region with cup-and-cone features characteristic of a ductile failure process distinctly different than the flat regions
   iii. Where the flat regions were symmetrically opposed about the mid-plane of the bolt suggested that cracks nucleated on opposite sides of the bolt and grew together as a result of alternating cyclic tensile loading until final ductile fracture of the central region occurred
iv. These fracture surface features are consistent with unidirectional cyclic bending failure (SSW Report: Page F-12)

Figure 5-1 places the bolts tested by NRC and SSW and those allocated to MTO for their own investigation on the shoe plate and allows the interpretation of the results from the fracture surface analyses for understanding possible sequences and patterns of failure.

The division of the failed bolts into three portions was done such that both laboratories and the MTO had sufficient coverage of the shoe plate. Bolt samples from all four quadrants of the shoe plate were allocated to each of them to enable their own interpretation of the results from their own analyses, but also to ensure that bolts tested represented all of the key features of the failure of the bearing.

Figure 5-2 summarizes the results of the bolt fracture surface analyses on the shoe plate. Bolts that had been subjected to fatigue and bending are shown in green, those that had been subjected to fatigue and tension in white and when torsion was present at the time of crack initiation they are shown on the sketch in yellow. Presenting the different types of bolt failures in this way was done with the purpose of possibly revealing patterns of failure of the bolts which may have existed prior to and during the final failure of the bearing. A discussion of Figures 5-1 and 5-2 is provided in Section 5.4.
The letters were allocated to bolts by MTO, prior to them being sent to the laboratories.

Figure 5-1: Bolts allocated to the testing laboratories
Figure 5-2 shows the type of bolt failures as observed by the laboratories and interpreted for the remaining bolts.

- Fatigue with tension, crack growth from one side only, cyclic tension
- Fatigue with bending, crack growth from two sides, alternating cyclic tension
- Fatigue with tension, crack growth from one side only, cause of crack is torsion,
- Confirmed or potential early failure evident on bolt fracture surface after failure

Figure 5-2: Bolt fracture surface types
5.3 CONSIDERATION OF PHOTOGRAPHS TAKEN IMMEDIATELY AFTER THE FAILURE

A more detailed analysis of the fracture surfaces was done using the photographs taken of the shoe plate immediately after the failure and prior to the fractured bolts being cut off (see Figure 5-3 below). The results of the fracture surface analyses from the NRC and SSW reports, together with our own photographs and interpretation, were summarized and illustrated on Figure 5-3. The background photograph of the four bearing quadrants were collated by MTO and presented in such a way that they show the whole of the shoe plate on this figure.

The following is necessary when observing the illustrations and photos in Figure 5-3.

1. Crack initiation is shown as small yellow arrows. They locate the point where the crack propagated from and the direction the crack propagation followed
2. The purple arrow at bolt 21/H and 23/BB indicate an additional crack which existed on the fracture surface of these two bolts, but which did not interfere with the fatigue with tension mode of failure
3. When cracks are initiated from opposite sides of the fracture surface of bolts this is shown as two yellow arrows pointing in opposite directions
4. Lines of four different colours indicate the angle of crack propagation. Note that the yellow arrows are always perpendicular to these lines
   a. The white lines are derived from the NRC report
   b. The blue lines are derived and interpreted from the SSW report
   c. The green lines are our own interpretation of the untested bolts at St Catherine’s MTO offices
   d. The pink lines have been interpreted directly from these and other photos of the bolts after failure. It was not possible to allocate a bolt to any of these locations.
5. The width of striations and the number of cycles to failure is taken from the NRC report and is given in yellow
6. The bolts circled in red depict those with confirmed early failure and those circled in red dashed lines are those with potential early failure
7. The green circular arrows indicate that the bolt has experienced torsional forces at the time of crack initiation
8. Information is also given on the fracture pattern observed on the bolts in grey

The following table is presented here for completeness. It includes all additions and changes to bolt allocation on the shoe plate from a previous sketch provided to AE by MTO. The allocation of bolts on Figure 5-3 has taken these changes into account.
<table>
<thead>
<tr>
<th>Bolt</th>
<th>Locations 1 to 40</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Old</td>
</tr>
<tr>
<td>BB</td>
<td>-</td>
</tr>
<tr>
<td>G</td>
<td>23</td>
</tr>
<tr>
<td>GG</td>
<td>-</td>
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<tr>
<td>I</td>
<td>4</td>
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<tr>
<td>II</td>
<td>-</td>
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<td>J</td>
<td>-</td>
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<td>K</td>
<td>6</td>
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<td>L</td>
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<td>LL</td>
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<td>R</td>
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<td>T</td>
<td>18</td>
</tr>
<tr>
<td>W</td>
<td>5</td>
</tr>
<tr>
<td>X</td>
<td>-</td>
</tr>
<tr>
<td>NN</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 5-1: Updated Bolt Locations
Figure 5-3: Top View of bolt fracture surfaces at the north-west bearing immediately after failure on January 10, 2016 with information on the fracture surfaces and a sketch of the dimension of the shoe plate below.
5.4 DISCUSSIONS ON BOLT OBSERVATIONS

In order to interpret Figure 5-3, it is necessary to consider the following:

- The condition of the connection bolts prior to failure.
- The effects of a flexible shoe plate on the increase in loads in some of the connection bolts and on the consequential eccentric forces applied to the nut and bolt head in uplift.
- Important considerations from the NRC and SSW reports.

5.4.1 Condition of The Connection Bolts Prior to Failure

The bolts were supplied and installed with a number of deficiencies which contributed to the bearing behaviour as discussed in Section 4.4 of this report. In summary:

1. The installed bolts were too long and therefore needed additional washers. These were not bevelled as required by the code for the A490 bolts. Additional eccentric forces on the bolts during tightening or in the uplift condition would have occurred.
2. Bolts were not pre-tensioned as confirmed by construction documentation.
3. The recess provided in the shoe plate could not prevent the bolt heads from slipping during tightening, which would further hinder the tightening of the bolts once the shoe plate was installed.
4. The shoe plate may have separated from the underside of the bottom flange of the girder during uplift, potentially due to loose bolts and the deflections in the shoe plate before and during the gradual failure of the bolts.
5. Shearing between plates and “tilting” of the bolts in service may have occurred and could have resulted in cyclic alternating tensile forces being applied to the bolts. This may have resulted in the two-sided crack propagation observed on some bolt fracture surfaces.
6. Shearing effects are expected in the longitudinal direction of the bridge and this effect is observed in the direction of the striations of the bolt fracture surfaces.

The bolts connecting the girder bottom flange to the bearing were significantly impeded from fulfilling their function as a result of these limitations.

5.4.2 The Effects of a Flexible Shoe Plate

1. The flexibility of the connection between the bearing and the girder was partly due to the chosen configuration of the shoe plate, the method used to connect it to the girder bottom flange (see contract drawing A-11) and the method chosen by the bearing manufacturer to connect the bearing sole plate to the shoe plate (see fabricators drawings sheet 13 of 15).
2. Figure 5-4 shows an exaggerated view of the expected deformation of both the shoe plate and the girder bottom flange when subjected to an uplift load from the bridge and Figure 5-5 shows the shape of the shoe plate after failure.
3. The form that the flexible shoe plate takes in uplift due to how it is connected to the girder and sole plate will cause a redistribution of forces to stiffer areas of the plate and also apply localized eccentric forces to the bolts.

4. The prying effects transfer the uplift loads to the outer line of bolts which elongate under load and result in the shoe plate possibly separating from the underside of the bottom flange along. The implications of these bolt investigations coupled with bearing design and installation, the demands on the bearing described in section 7, are interpreted in section 8.
5. The conditions under which slip between the shoe plate and the girder bottom flange when the bearing is subject to deck translations and rotations could occur are discussed in Sections 8.7, with evidence supporting this in Section 5.4.3.

The following photos show clear evidence of rub marks on the sides of bolts which provide evidence of slip between the shoe plate and the girder bottom flange.

<table>
<thead>
<tr>
<th>Figure 5-6: Top half of bolt 20/I</th>
<th>Figure 5-7: Bottom portion of bolt 20/I</th>
<th>Figure 5-8: Bolt 40/EE failed at the head</th>
<th>Figure 5-9: Bottom portion of bolt 19/NN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Figure 5-10: Top portion of bolt 11/HH</td>
<td>Figure 5-11: Bottom half of bolt 10</td>
<td>Figure 5-12: Bolts taken from the CW bearing</td>
<td>Figure 5-13: Bolt 39 from the CW bearing</td>
</tr>
</tbody>
</table>
5.4.3 Interpretation of Figure 5-3

The following are observations taken from the information summarised on Figure 5-3, which presents various patterns of failures of the connection bolts, with consideration of the arguments presented above.

1. The fatigue cracks propagate predominantly in the longitudinal direction (west-east direction) of the bridge. The cracks on the fracture surfaces of the bolts are mostly perpendicular (north-south direction) to the girders. The following are pertinent points:

   **Evidence**
   - i. The axis of crack propagation is generally closer to the vertical axis than to the horizontal axis.
   - ii. The direction of propagation of the fatigue cracks and the location where the cracks have initiated point generally in the longitudinal direction of the bridge.

   **Reasons**
   - i. Bending introduced in the bolts from a possible shearing of the shoe plate in relation to the girder flange. Movements are likely to be primarily in the longitudinal direction of the bridge and would have resulted from the movements of the bridge at the bearing when subjected to traffic, temperature or other effects.
   - ii. The inability of the bearing to accommodate rotations in all directions, but in this case particularly in the longitudinal direction would have accentuated the forces in the connection bolts from the eccentricities generated along the lower slide paths.
   - iii. Bending of the flexible shoe plate under the permanent uplift loads in service.
   - iv. The axes about which the fatigue cracks propagate will vary between bolts depending on, but not limited to, the following factors:
     - Location of the bolts on the shoe plate.
     - Translations and rotations on the bearing.
     - The variation in pre-tensioning loads applied to the prior to failure.
     - The variation in snug tightness that bolts could have been subjected to prior to failure.
     - The possibility of self-loosening of the bolts in service.
     - The effect of the separation of the shoe plate from the bottom flange.
     - The effect of the flat washers.
     - The constructed out of tolerance of the sole plate in relation to the masonry plate of the bearing.
     - The effect of the progressive failure of the bolts on the deformation of the shoe plate and the girder bottom flange and the gradual redistribution of the uplift loads to the remaining bolts prior to final failure of all of the bolts.

2. The outer lines of bolts have been predominantly subjected to unidirectional cyclic bending forces (alternating cyclic tensile loading) when compared to the inner line of bolts which, apart from 2 bolts in the SW quadrant, have been subjected only to cyclic tensile forces.
Evidence.

i. At least 9 bolts along the outer lines have evidence of fatigue cracks on the fracture surface having initiated on opposite sides of the bolts shown on the figure as two opposing yellow arrows.

ii. Rub marks on bolt shanks indicate shear movements across bolt lengths. Note in Figure 5-12 that the rub marks were also seen in the centre-west bearing, which had not failed.

Reasons.

i. The prying effects from the transverse bending of the shoe plate and increased loads in the outer line of bolts.

ii. A separation of the shoe plate from the girder flange will therefore first occur along the outer line of bolts as these bolts elongate under load.

iii. Shearing between plates, if this does occur, would impose bending effects to the loaded outer line of bolts.

iv. Prior to the failure of the outer line of bolts the inner line would be essentially unloaded but could also be subjected to shearing between plates. However, this would not be transferred through the bolt heads and nuts as would occur along the loaded outer line of bolts.

v. After the gradual failure of the outer line of bolts, the inner line of bolts would feel the effects of the uplift loads, translations and rotations from the bridge deck. As portions of the outer line of bolts fail, prying effects on the shoe plate are reduced causing the inner line of bolts to predominantly fail as a result of fatigue and tension only.

3. The failure appears to have begun in the SW quadrant of the shoe plate and ending in the NE quadrant or generally on the eastern quadrants of the shoe plate

Evidence

i. 6 of the 10 bolts in the SW quadrant of the shoe plate exhibited either confirmed or potential early failure from the evidence of corrosion on the fracture surfaces observed immediately after the bearing failure.

ii. Unique to this quadrant, the fracture surfaces of the majority of the bolts exhibited alternating cyclic tensile loading.

iii. The permanent deformation of the shoe plate observed after failure is the least in this quadrant of the shoe plate.

Reasons

i. Our measurements of gaps between the top of the girder bottom flange and the top of the masonry plate has indicated that the south-west corner of the bearing would come into contact first during uplift. We measured a 7 mm difference from the expected designed elevations along the southern guide.

ii. The PTFE was also found along the sliding surfaces of the longitudinal guides to be more deformed and damaged at the south-west corner of the bearing.

iii. Higher loads were applied to this quadrant in service and we expect that the bolts in this quadrant would have resisted the majority of the translations and rotations in uplift from the traffic on the bridge.

4. The propagation of the cracks on the bolts subjected to cyclic tensile loading appear to be directed towards the central load bearing stiffener.
Evidence
i. The direction of crack propagation on the majority of the bolts that failed with fatigue and tension only, generally point towards the centre of the shoe plate and at the central bearing stiffener
ii. This is more pronounced in the east quadrants.

Reasons
i. The alternating longitudinal translations and rotations from the bridge deck results in fatigue cracks propagating in opposite directions.
ii. Cracks propagate towards the stiffer part of the shoe plate and the girder bottom flange. The local effects require additional investigation.

5. Steep angled fracture surfaces are primarily located along the inner line of bolts and in particular along both inner lines in the north-east and south-east quadrants. These appear to be last group of bolts to have failed.

Evidence
i. Figures 5-14, 5-15, and 5-17 show in the background the inner line of bolts in the eastern quadrants with steep fracture surfaces.
ii. Measurements taken of the shoe plate after failure show that the section with the most deformation coincides with the inner line of bolts in the east quadrants (maximum deflection of 7.2 mm).

Reasons
i. The two inner lines of bolts in the eastern quadrants of the shoe plate are located on the more flexible portion of the shoe plate and closest to the thinnest edge of the shoe plate.
ii. These bolts would have been the least loaded during the gradual failure process of connection bolts and consequently would have been the last bolts to fail.

iii. The inner lines of bolts would have failed as a result of cyclic tensile loading once prying effects on the shoe plate were reduced.

The weakest link in the bolts was generally the first thread at the underside of the nut. Cracks typically initiated there and sometimes ended at the root of the first thread (the thread run-out)

Evidence
i. Both testing laboratories show the initiation of cracks to be located primarily at the first thread at the nut.

Reasons
i. The first thread is closest to the where the load is applied to the nut, forms a crack induced deformity in the bolt and has a reduced cross sectional area when compared to the shank.

5.5 SUMMARY AND CONCLUSIONS

1. The study of the fracture surfaces of the failed bolts confirm the following:

a. Bolts failed due to experiencing high cyclic loads which caused low cycle fatigue cracks to initiate and propagate until the individual bolt’s load capacity was exceeded and the final fracture occurs.

b. Some bolts experienced fatigue crack propagation from one side only indicating direct tension whilst others showed fatigue cracks that propagated from two sides which indicated bending effects or also referred to as cyclic alternating tensile loading.

c. The existence of corrosion on some of the fracture surface of bolts indicates that bolts fractures happened gradually and also provided conclusive evidence that some bolts had already failed before the time of the final bearing failure.

d. The fracture surfaces show evidence of different types of loading, likely caused by the decrease in the number of bolts taking up the load applied by the girder as the failure progressed.

2. Locating the connection bolts where they were on the shoe plate during the failure confirms the following:

a. Crack propagation is predominantly in the longitudinal direction of the bridge and likely influenced by translations and longitudinal rotations of the bridge from heavy vehicles, slip between the shoe plate and the girder bottom flange and the inability of the bearing and its connecting components to accommodate these rotations in uplift.

b. The number of permit vehicles which may have crossed over the bridge before the failure is consistent with the approximate stress cycles to failure observed on the fracture surfaces of bolts.

c. There exists a clear pattern of failure on the shoe plate when observing the fracture surface of the bolts.

i. A large proportion of the bolts in the south-west quadrant exhibit either potential or confirmed early failure of the bolts. This is consistent with the measured construction tolerances which indicate that this quadrant would have been the most heavily loaded section of the shoe plate in uplift.
ii. The outer lines of bolts have flat fracture surfaces and a predominance of crack propagation from both sides of the bolt which is consistent with these lines being more subject to the prying effects of the shoe plate and the consequential higher loads.

iii. The inner lines of bolts have steep fracture surfaces and a predominance of crack propagation from one side only which is consistent with the expected failure of bolts in direct tension but not subject to the prying effects of the plate. The prying effects would have been largely removed after the failure of the higher loaded outer line of bolts.

iv. Bolts with cracks which have propagated from one side only point mostly towards the centre of the shoe plate, towards the central load bearing stiffener and the stiffer part of the plate, supports the contribution that the alternating longitudinal and rotations have on the propagation of cracks in the bolts.

d. The observations on the fracture surfaces support failure of the bearing that likely originated within the outer line of bolts and ended with a remaining group of inner line bolts.

3. The connection bolts have been loaded beyond their designed limits because:

   a. Rotations of the bridge deck applied to the bearing which was not designed to rotate in uplift.

   b. The force in the outer line of bolts more than doubled by the prying action of the flexible shoe plate.

   c. The concentration of uplift loads on the SW quadrant of the shoe plate resulted from installation procedures and construction tolerances.

   d. Loose and/or un-tensioned connection bolts have contributed to un-equal and eccentric loads applied to individual bolts.

   e. Independent lateral shifting of the shoe plate in relation to the girder bottom flange in shear may have resulted from the presence of loose connection bolts.

4. The pattern of bolt failures observed on the shoe plate supports the following reasons for the failure of the bearing:

   a. The inadequacy of the bearing connection details to the shoe plate and to the girder bottom flange.

   b. The inability of the bearing to accommodate rotation.

   c. The inability of the bearing to accommodate the construction out-of-parallel.
6 Review of Documents

6.1 BEARING DRAWINGS & SPECIFICATIONS

Our independent evaluation of the bearing failure included the review of contract drawings, bearing fabrication drawing, and available construction documentation. The purpose of this review was to determine the intended function and requirements of the bearing and aspects of the details, fabrication and construction that may have contributed to the failure. The information and discussion in this section is intended to be factual and from observation, coupled with engineering considerations for the intended functions of the bearings.

The following documents were found to be most relevant for understanding the bearing design and installation:

- Bridge contract drawing: Bearing Plan and Design Data. Sheet 218-1-R2. Dated 02/13 & Sealed 11/07/13
- Bearing fabrication drawings by the bearing supplier. Sheets 1 to 15. Revisions 7 and 8. Rev 8 was dated 21/10/15 & sealed as "As-built" details on 24/11/15
- Superstructure Erection Manual

The contract drawings, specifically 218-1-R2, contains details of the connection of the bearings to the girders bottom flange and to the bearing pedestal. Included in these details are the dimension of the shoe plate, its bolted connection to the girder, and the anchoring system to the pedestal. Shaded and labeled excerpts from drawing 218-1-R2 are shown in Figure 6-1. The bearing fabrication drawings include details

![Figure 6-1: Shaded and labeled excerpts from contract drawing 218-1-R2](image)
of the “rotational bearing” specified on the contract drawings, as well as the shoe plate and its bolted connection to the girder flange.

Our review of the contract and fabrication drawings produced the following key points:

- The shoe plate dimensions in plan and elevation given on the contract drawings have been repeated on the bearing fabrication drawings. However, fabrication drawings specify the shoe plate as ASTM A36 (yield strength 248MPa), which is a change or deviation from the specified 350W (yield strength 350MPa) grade steel on the contract drawings.

- Instead of 8 rows of 4 M22 (7/8") ASTM A325 flange connection bolts shown on the contract drawings, 10 rows of 4 M22 (7/8"), ASTM A490 bolts are provided on the as-built fabrication drawings.

- The contract drawings do not specify the method for connection of the shoe plate to the bearing assembly. This was left up to the contractor (and consequently the bearing supplier) to determine. The bearing supplier specified two lines of 12 x 1" connection bolts along the longitudinal edges (parallel to the girder) for the connection of the shoe plate to the sole plate. The shoe and sole plate dimensions on the contract drawings provided adequate space for these bolts to be accommodated. The contract drawings precluded the use of welds in this connection. This arrangement of bolts contributed to the prying action of the plate on the flange connection bolts, as was described in Section 4.

- The contract drawings show bevelled washers on flange connection bolts to account for the slope of the girder bottom flange in relation to the bearing sole plate. Despite the bearing being specified to be level as per installation Note 6 on fabrication drawing sheet 1 of 15, bevelled washers were not specified on
the fabrication drawings. Flat washers were installed on site, as discussed in Section 4.4.2, introducing an initial local bolt force eccentricity.

- The flange connection bolts on the fabrication drawings were also erroneously specified too long for the north-west and centre-west bearings. Additional flat washers were installed on site to accommodate the distance needed to the threaded portions of the bolts.
- Since the long bolts could not be readily removed once the bearing was installed and in service, level solid plates were proposed by the bearing supplier as a permanent solution to replace the additional flat washers. These were not installed at the time of the bearing failure. Bevelled washers would still be necessary to meet the specifications.

6.1.1 Bearing Design Rotational Requirements

The bearing supplier was responsible for designing the bearing to accommodate the design loads, rotations and displacements given on the contract drawings. Contract drawings provided design rotations about the horizontal and vertical axes. Note 14 of drawing 218-B-R1 stated that the rotational bearings shall accommodate rotation about horizontal axis as indicated on the bearing design table in all directions.

The maximum serviceability limit state (SLS) rotation specified for the north-west bearing about the horizontal axis is 0.8º. This maximum horizontal design rotation is 1.3º at the ultimate limit state (ULS). Specifications OPSS 922 and 1203 also include a requirement to accommodate an additional 1.2º of rotation about the horizontal axis for the ULS. These requirements were not repeated on either the contract or fabrication drawings.

Based on the function of the bearing, as discussed in Section 4.1, the bearings were not designed to freely accommodate the specified rotations when subjected to uplift loads. The rotations in either direction would impose significant unintended demands on the bearing assembly components, despite the uplift loads and corresponding rotations being clearly laid out on the contract drawings.

6.1.2 Additional Bearing Constraints

A physical limitation to the allowable bearing rotation is the vertical gap above the lower sliding area of the bearing. Based on the dimensions taken from the fabrication drawings, we have calculated a remaining gap of 17.6 mm available for rotation in the uplift condition, when the lower PTFE is in contact with the lower stainless steel plate (see Figure 6-2). This equates to a physical rotation limit of 1º, which is less than the required ULS design rotation, even without consideration of the additional OPSS requirements.
The bearing appears to have been designed to be primarily in compression, given that the top PTFE was recessed, a polytron disk was provided that would only have accommodated rotation under bearing compression, and the side and lower PTFE was unrecessed. OPSS 1203 requires the PTFE to be recessed by at least ½ of its depth into the upper surface of the masonry plate along the lower sliding surface to accommodate the continual uplift forces and corresponding compression loads on the PTFE.

As noted on the contract drawings, the north-west bearing would experience tension (uplift) in service for all load cases (min uplift force of -140 kN and maximum uplift force of -2540 kN) due to the bridge configuration. The polytron disk would have been unloaded and have no function for forces or rotations while in uplift. The lower PTFE would remain in (partial) contact with the lower stainless steel at all times in service.

6.2 BEARING INSTALLATION AND CONSTRUCTION TOLERANCES

A review of the bearing installation procedure was conducted based on available photographs and the Superstructure Erection Manual. Information from site personnel was not available regarding the bearing installation at either the west and east abutments.

The pertinent steps of the installation procedure for the west abutment bearings, as specified by the Erection Engineer, are as follows:

- Install girders NG110 and CG110 and all floor beams except the floorbeam at the centerline of bearing.
- Jack girders NG110 and CG110 vertically to the specified elevations.
- Install the floorbeam and the north permanent lateral restraint at the centerline of bearing.
- Slide the bearings into place and install the post-tensioning bars connecting the bearing to the abutment.
- Jack the structure transversely from the shear block to align the girders with the bearings. Estimated horizontal jacking distances were provided.
- Release horizontal jack and transfer the lateral load to the kicker beam assembly, installed between the shear block and the girders.
- Verify alignment of bearing then engage the vertical jacks, remove cribbing under floorbeam, and then transfer vertical load to the bearings.
- Complete the bearing installation by installing the bolts connecting the bearing to the bottom flange of the girders.

The following photos were taken on October 5 and 14, 2015, when the north-west bearing was installed. Key points associated with each photograph are provided.

**Photo 1. Date: 10/05/2015**

A portion of the deck close to the west abutment, deck segment 110, and the last four stay cables, cables CC111, NC111, CC110 and NC110, had not yet been constructed and installed when the bearings were installed.

The early phases of the north-west bearing installation at the west abutment can be seen in this photo.
Photo 2. Date: 10/05/2015

The shoe plate has been installed onto the girder bottom flange and is not yet connected to the bearing sole plate, which is being slid into place.

The masonry plate has been installed and already grouted onto the bearing pedestal. The sole plate is later slid into place and connected to the shoe plate after the masonry plate is installed.

As shown in the close-up detail of the bolts, the varying bolt protrusions between the inner and outer line of bolts ($\delta_2 > \delta_1$), combined with our bolt measurements indicates uneven bolt tightening. The installation of the sole plate prevents access to the connection bolt heads for bolt tightening purposes from this point onwards.

Photo 3. Date: 10/05/2015

The shoe plate is connected to the girder bottom flange. The shoe plate to sole plate connection bolts have been inserted into the sole plate (seen being installed from the left of the photo).
Nine days later, the bearing installation has been completed. The sole plate is connected to the shoe plate and the masonry plate anchor bolts are installed. The floor beam at the bearing has also been installed. The bearing is now securely in place and uplift load will be transferred from the deck to the west abutment through the bearings after the deck panels are installed and stressing of the last four stay cables is completed. These operations would have introduced some deformations and force effects onto the bearing assembly.

The north-west and centre-west bearings are in place and last pre-cast deck segment needs to be constructed and the last four cables installed.

The available construction photos show two clear variances from the specified installation sequence:

- The floor beam connected to the centre load bearing stiffener immediately above the bearing was installed after the bearings were slid into place. It was uncertain how that may have affected the performance of the bearing in service.

- The shoe plate was bolted to the girder flange prior to connection with the bearing. The Erection Manual installation procedure specifies this as a final step, although it is unclear how this recommendation would have improved the performance of the bearing.
In accordance with OPSS 922 the construction tolerances defined in 922.07.06 were not achieved. The elevations with respect to the top of the bearing as previously noted in Chapter 4, and measured on site, exceed all of the allowable construction tolerances outlined in this specification.

6.3 FINDINGS OF BEARING DESIGN AND INSTALLATION REVIEW

A summary of the key findings from our review is as follows:

1. The shoe plate was manufactured to the dimensions given in the contract drawings without provisions for field tolerances. For this installation approach to meet installation specifications in the field, the masonry plates would have had to be installed parallel to the shoe plate rather than level.

2. The fabrication drawings of the shoe plate introduced dimensional variations from the values given in the contract drawings between the upper and lower portions of the bearings.

3. Any variations in slope (both longitudinal and transverse) or elevation from that given in the contract drawings introduced between the girder bottom flange and masonry plates from any cause, would now cause unintended force effects on the bearing assembly and connections.

4. The location where any rotations could be facilitated would be between the sliding surfaces of the bottom and top longitudinal guides, because the upper and lower portions of the bearing are disconnected along these sliding surfaces.

Given the rigid bearing sliding surfaces, which were not designed to accommodate deck rotations nor to accommodate out-of-parallel tolerances, we expect the functioning of the bearing and of its connection bolts to be significantly impacted in service. These effects are discussed further in Section 8.
7 Independent Analysis and West Abutment Bearing Evaluation

We have conducted independent global and local (bearing assembly) analyses of the Nipigon Bridge to achieve the following two goals:

1. Evaluate the design and specifications of the west abutment bearings as shown on the contract drawings.
2. Develop global demands, and local effects on bearing assemblies from the global bridge model, to help explain the failure of the north bearing on the west abutment.

To achieve our objectives, we created a global finite element model of the bridge, as well as local models of west bridge bearings. Our models, modeling assumptions, analysis results and conclusions are provided in this report.

7.1 GLOBAL MODEL – PHASE ONE

An independent global model analysis of the Nipigon Bridge was completed using the commercially available Midas finite element program. We have modeled the bridge for the first phase of construction was with the bridge in service as a two-lane structure. Our Midas model of the Nipigon Bridge after Phase One is shown in Figure 7-1.

![Figure 7-1: Global Phase One bridge model](image)

Our global bridge analyses focused on the west abutment bearing reactions and deformations. The objective of our model is to confirm the abutment bearing reaction forces, and in particular uplift reaction on the north-west bearing after the first phase of construction.
7.1.1 Applied Loads in Global Model

To determine bearing reactions on the west abutment bearings, we have applied the following loads to our Phase One model:

- **Permanent loads:**
  - Phase One of the construction is complete; bridge operates as a two-lane structure. Barriers are installed on the bridge deck.
  - Steel weight: 77 kN/m³. We have added a 25% allowance to the calculated weighs of steel girder and floor beams to account for miscellaneous components such as splice plates, stiffeners, studs, bolts and anchor plates. In calculating the weight of bridge cables, we have included a 10% allowance to allow for cable sheeting, spacers and anchor plates.
  - Reinforced Concrete weight: 25 kN/m³. Calculated weights of concrete components were compared to the weights shown on the shop drawings prepared by the pre-caster. While we could normally assume reinforced concrete weight of 24.0 to 24.5 kN/m³, we have applied the weight of 25 kN/m³.
  - No overlay on deck at this time.
  - Cable forces are assumed to be equal to cable lift-off forces provided by the Contractor.

- **Counter Weight Load:** Following the north-west bearing failure, concrete Temporary Concrete Barriers (TCBs) were placed on the bridge deck to bring the north bridge girder in contact with the bearing. The weight placed on the deck consisted of (for illustration see Figure 7-2):
  - 9 Barriers on the sidewalk – 220.68 kN
  - 101 Barriers on the Deck – 2476.53 kN
  - 12 Single Blocks on the Deck – 120 kN
  - 16 Single Blocks – 160 kN

- **Vehicular load:** Live load on the bridge was assumed to consist of one CL-625 ONT truck with GWV of 63,700 kg as confirmed by the vehicle permit information available. As this analysis is focused on the north-west bearing, we assume that the truck is using the westbound lane, with no other loads on the bridge. The truck is assumed to travel at highway speed; truck load is increased by 25% to account for dynamic impact. We understand just less than 90 similar vehicles have crossed the bridge from the opening to traffic until the bearing failure occurred. Therefore, this loading is appropriate for a low cycle fatigue analysis.

- **Wind load on the structure was conducted for the wind load along the bridge (from west to east) and perpendicular to the bridge; a 1/10-year wind load was applied on the structure. As this load case was considered to assess the potential effects on the western bearings and whether wind may have played a role in the bearing failure. We have based our wind analysis on the methodology described in the bridge code, rather than an in-depth wind study taking into account specifics of the location and the bridge structure.

- **Differential Temperature:** The thermal inertia of the bridge cables is much smaller than that of the concrete pylons and deck. Therefore, there will be times when the bridge cables are warmer or
colder than the rest of the structure. If the cables are colder than the structure, the uplift reaction at the abutment will increase, and conversely uplift reaction will be reduced if the cables are warmer than the concrete structure. Based on our field measurements, we have made an assumption that the difference in temperature of cables and concrete structure can be up to 15°C. This is similar to cable temperature differenced outlined in the Post-Tensioning Institute (PTI) for cable stayed bridges. This case was also considered to determine the possible role of thermal effects on the failure.

The weight of ballast on the deck caused the north girder to touch down on the bearing; we therefore know that the uplift reaction due to permanent load is not greater than the compression reaction due to barrier weights. The weight was subsequently removed but this information was a valuable control for our permanent load reaction calculation as a means to confirm behaviour of the model. A sketch showing barriers on the deck, and a model rendering showing applied loads and resulting west abutment bearing reactions is shown on Figure 7-2.

7.1.2 Global Analysis Results

Global analysis results are as follows:

- Permanent Loads: We have calculated the bearing reaction due to permanent load (including cable forces) to be equal to:
  - \(-1,719\text{kN (uplift)}\) at the north bearing.
  - \(-1,064\text{kN (uplift)}\) at the centre bearing.

The result for the north bearing compares well with the calculated reaction for the counterweight on the deck, which is 2,051kN (compression) at the north bearing and 648kN (compression) at the centre bearing, see Figure 7-2 for illustration. The calculated bearing reaction on the north bearing
also compares well with the ‘Dead Load (MIN)’ reaction shown on the design drawings, which is equal to 1,900 kN.

- Counter Weight Reactions: Calculated reaction for the counterweight on the deck is equal to:
  - 2,051kN (compression) at the north bearing.
  - 648kN (compression) at the centre bearing.

See Figure 7-2 for illustration. As stated above, the calculated reaction for the north bearing is slightly greater than the permanent load uplift, which can be expected since the counterweight was sufficient to bring the north bearing into compression.

- Vehicular Loads: Bearing reaction on the west abutment due to one CL-625 ONT truck on the west-bound lane bridge deck are as follows:
  - At the north bearing: max. uplift = -154kN; max compression = 466kN
  - At the centre bearing: max. uplift = -118kN; max compression = 213kN

The above reactions include the dynamic impact of 25%. In addition to maximum bearing reactions, we have determined the maximum rotation of the north bearing on the west abutment during the truck crossings. The maximum girder rotation at the north-west bearing caused by the truck is equal to approximately 0.19° (in longitudinal direction); this reaction corresponds with north-west bearing compressive reaction of 275kN.

- Wind Load: Additional uplift reaction on the north bearing on the west abutment due to wind load is in the order of -160 kN on the northwest bearing and 80 kN on the centre bearing, combined with a longitudinal rotation of 0.06°.

- Differential Temperature: Additional uplift reaction due to bridge cables being 15°C colder than the rest of the structure is in the order of -150kN on north bearing of the west abutment, and 230kN on the center bearing. If the cables are warmer by 15°C, the two uplift reactions will be reduced by the same amount. This differential temperature was determined utilizing field measurements undertaken using an infrared thermometer during the site visit outlined in Section 4. It is also noted that 15°C is well within the anticipated recommendations for cable-deck thermal differential as outlined by the Post-Tensioning Institute Recommendations for Stay Cable Design, Testing and Installation publication.

7.2 BEARING DESIGN EVALUATION

7.2.1 West Abutment Bearing Design

We have analyzed the West Abutment bearings as designed for Phase One construction.

While the bridge design drawings do not include a detailed design of the abutment bearings, performance requirements for the bearings are provided on drawing 218-B-R1, attached to this report. The designer’s performance requirements for the bearings include minimum compressive and tensile demands, with coinciding translation and rotation demands about two horizontal axes and the vertical axis. The bearing performance requirements for Centre and North bearings on the West Abutment, installed in Phase One of the bridge construction, are shown in Figure 7-3. Bearing detailed and other design requirements were
described previously in Section 6.1. Our design reviews were based on the contract and fabrication drawings provided.

Note 14 on drawing 218-B-R1 states that the “rotational bearings shall accommodate rotation about horizontal axis as indicated on the bearing design table in all directions”

The contract drawings also include the design of the bevelled shoe plate and the bolts connecting the shoe plate to the girder bottom flange. The bevelled shoe plate is specified, on contract drawing 218-1-R2, to be 1000 mm long and 800 wide with thickness varying from 52 to 60 mm, to compensate for girder grade and provide a level bearing. For illustration see Figure 6-2. The bevelled shoe plate steel grade is specified on drawing number 218 as grade 350W to CAN/CSA G40.21-M04; with yield strength of $F_y = 350$ MPa. The contract drawings show the shoe plate connected to the girder bottom flange with 32-Ø22 (7/8”) bolts, grade ASTM A325.

Bearing fabrication drawings include: masonry plate, sole plate, compression bearing, bevelled shoe plate, as well as the bolted connection between sole plate and bevelled shoe plate, and the bolted connection between bevelled shoe plate and girder flange.

Figure 7-3: West abutment bearing performance requirements - contract drawing 218-B-R1
The detailed design of the bevelled shoe plate and the bolted connection to the girder flange by the bearing supplier is based on the contract drawings; the size of the bevelled plate is as shown on the contract drawings (800x1000 plate, thickness varies 52-60mm).

The Bearing Capacity Table shown on the fabrication drawings Sheet 3 of 14, asserts that the bearing meets the performance requirement specified on contract drawing Sheet 218. Specifically, the Bearing Capacity Table notes that the bearing meets requirements for:

- compressive and uplift capacity,
- longitudinal displacement,
- the ability to accommodate a ‘service rotation’ of 0.0175rad (1.0°), and
- the ability to accommodate a ‘strength rotation’ of 0.0244rad (1.4°).

While the bearing is in compression, the rotation occurs about the Polytron disk supporting the bridge. However, when the bearing is in tension (uplift), the rigid steel restrainers of the upper and lower blocks are in contact, separated by a thin sheet of PTFE, and are therefore unable to accommodate rotation about any horizontal axis. Due to the horizontal clearance between the upper and lower block restrainers, the bearing is capable of accommodating some rotation about the vertical axis. Neither the contract drawings nor fabrication drawings specify construction tolerances for bearing placement.
7.2.2 West Abutment Bearing Finite Element Model

The objectives of the analysis were to determine the adequacy of the bearing design and explain the North girder bearing failure. To achieve the objectives, a finite element (FE) model of the west abutment bearing using Midas Civil analysis software was created. The FE model of the West Abutment Bearing reflects the bearing design shown on the bridge contract drawings and the bearing fabrication drawings.

The 3D bearing model was built using shell elements to represent the masonry plate, sole plate, shoe plate, and girder flanges, web and stiffeners (see Figure 7-5). All plates are modeled as linearly elastic. However, in cases where calculated stresses in plates exceeded yield strength, plastic deformation of the steel was considered. The bolts were modeled as non-linear frame elements. The modeled bolt non-linearity enabled the simulation of inelastic deformation of bolts for a more realistic prediction of ultimate bolt failure. The PTFE contact between lower and upper guide bars was modeled using non-linear frame members; linking the upper and lower block under compressive load and releasing at tensile load, allowing the upper and lower guide bars to separate and form a gap.

Figure 7-5: Midas model views of west abutment bearing (plate 3D and 2D)
To realistically determine the distribution of stresses on the plates and bolts, a section of steel plate girder, with stiffeners was included in the model. For convenience, the girder top flange was restrained along the girder web line and a tensile (downward) load was applied on the nodes representing the 8 anchors connecting the masonry plate to the concrete abutment. For illustration of modeled frame elements and load application, see Figure 7-6.

7.2.2.1 Modeling Assumptions

Modelling assumptions have been included that idealise material properties, fabrication and installation of components. Specifically, it has been assumed that all the materials conform to specifications, and that all bolts are installed in a way that maximises the capacity of the bolted connection. The model will allow the bearing assembly and connected plates to deform and rotate to the degree that they are physically capable, notwithstanding that the bearing itself is rigid and not designed to rotate freely.

The FE model geometry and section properties are based on the bearing fabrication drawings, discussed above, except that it has been assumed that bevelled shoe plate steel grade is 350W as per contract drawings, instead of A36 as shown on fabrication drawings. This enables commentary to be provided on the adequacy of the design.

The steel plates have been modelled with a linear elastic material having a Young’s Modulus of $E=200,000\,\text{MPa}$. However, in cases where calculated steel stresses in the bearing plates exceeded their specified yield strength, the material stiffness was locally reduced to simulate plastic hinging.
With FE modeling, plates are represented by a 2D element at the plate centreline. To model the potential for plate contact pressure, which may result in prying, axially rigid compression only frame elements were modeled in a grid pattern between plate centres where required.

All bearing bolts were modeled as non-linear elements, with a tensile stress-strain relationship representing tested bolt behaviour. Local bolt bending was not modeled directly in these analyses. The initial bolt axial stiffness is a function of the length of the bolt between nuts, and the cross-sectional area at the bolt threads. The ultimate elongation was assumed to be 3.5% (3.5mm for a 100mm long bolt) based on published ASTM A490 bolt tensile test results in “Guide to Design Criteria for Bolted and Riveted Joints”, by Kulak, Fisher, and Struik. This bolt behaviour curve was developed with monotonic direct tension, but has been assumed to also generally represent the stress-strain envelope resulting from cyclic tension testing. The published results indicated an average maximum bolt tensile resistance of 110% of the specified nominal bolt capacity.

The bolts connecting the beveled shoe plate with the girder flange were modeled as 40-Ø(7/8” (22.2mm), grade ASTM A490 bolts, which was the installed configuration. For bearing design checks, bolt tensile capacities for each type of bolt (HA1 and HD1) were assumed to be equal to the maximum design tensile capacity for A490 bolts of their diameter, calculated in accordance with CSA S6-14 Canadian Highway Bridge Design Code. The bolt tensile capacities, of 241kN for HA1 bolts and 315kN for HD1 bolts, include a material resistance factor of 0.80. The nonlinear tensile behaviour of the bolts was idealized with a trilinear curve and was scaled to represent the bolt design tensile behaviour, as shown in Figure 7-7.

![Figure 7-7: Tensile load vs. elongation bolt behaviour (Reference “Guide to Design Criteria for Bolted and Riveted Joints”, by Kulak, Fisher, and Struik)](image)
Clause A10.1.6.4. of CHBDC specifies that the bolts shall be pre-tensioned to at least 70% of the minimum tensile strength specified in applicable ASTM standard; Canadian Steel Code CSA S16 specifies a minimum bolt pretension of 218kN for \(\frac{7}{8}\) A490 bolts. However, it is understood based on construction correspondence that the bolts connecting the bearings and girders at the west abutment were not pre-tensioned, but installed generally snug-tight. The analysis is based on the assumption that all bolts were installed snug-tight in a consistent manner, but it is noted that tightening some bolts more than others would lead to unequal force distribution. A uniform snug-tight bolt group is a ‘best-case-scenario’ for static capacity of the bolted connection, as it results in maximized bolt ductility, most favourable distribution of force, and consequently the greatest possible uplift and rotation capacity of the bolted connection and bearing assembly. The bolt tensile force, and corresponding elongation, is assumed to increase linearly until a first yield point, where its stiffness is reduced. The reduced stiffness is maintained until the bolt reaches its full bolt tensile capacity. After exceeding its tensile capacity, the bolt will inelastically elongate to a total of 3.5% of the initial bolt length, at which point the bolt is expected to fail.

As noted, the bolt force was limited to tension only. The potential effect of local bolt bending due to the missing beveled washers, shoe plate deformation under uplift, and the shoe plate bevel have been omitted. At elastic strain levels these would cause uneven bearing below the bolt nut, leading to stress concentration at extreme bolt fibres, which could further reduce bolt ductility and potentially affect the fatigue life of the bolts, considering that the bolts are not pre-tensioned. This idealization is consistent with the analysis approach to maximize the potential for positive analysis results.

7.2.2.2 Applied Loads for Bearing Design Evaluation

The bearings have been analyzed for the following effects:

- Tensile bearing reactions (uplift), centered on a bearing with no eccentricity and rotation
- Bearing rotation (longitudinal and transverse)
- Longitudinal translation

Self weight of bearing components is small relative to uplift reactions and can be ignored in this analysis. Other effects on the bearings were also analyzed as part of our failure assessment. These results are also identified in this section.

7.2.2.2.1 Design Bearing Uplift

Tensile (uplift) reaction for the west abutment bearings is caused by the asymmetric bridge configuration and applied cable loads, as well as transitory loads. The design loads for the three bearings on the west abutment are shown on contract drawing 218-B-R1; bearing reactions for centre-west and north-west bearing are also shown in the table in Figure 7-3. The contract drawings do not provide information to indicate the phase of bridge construction for which minimum and maximum dead load reactions occur.

The key bearing reactions shown on the design drawings are as follows:

- Reactions of the south bearing (Phase Two):
  - Serviceability dead load uplift (min) = -1,020kN
  - Serviceability dead load + transitory load uplift (min) = -1,640kN
• Ultimate dead load + transitory load uplift (min) = -2,240kN

Critical bearing reactions occur at the centre bearing:
• Serviceability dead load uplift (min) = -3,530kN
• Serviceability dead load + transitory load uplift (min) = -4,410kN
• Ultimate dead load + transitory load uplift (min) = -5,300kN

Reactions of the north bearing, which failed last January, are as follows:
• Serviceability dead load uplift (min) = -1,900kN
• Serviceability dead load + transitory load uplift (min) = -2,540kN
• Ultimate dead load + transitory load uplift (min) = -3,370kN

As stated above, as part of this report an independent global analysis of the bridge was conducted. Based on the global analysis results, the uplift loads on the west abutment bearings shown on the design drawings are reasonable and can be used for a detailed stress check of the bearings. Therefore, the bearing reactions specified on the contract drawings were used to check the bearing design, and investigate the north-west bearing failure.

In the FE analysis, the following uplift reactions were applied to the finite element model:

• 1,900 kN (uplift). This is the minimum design serviceability dead load reaction, specified on the contract drawings.
• 3,370 kN (uplift). This load is considered as the ultimate bearing uplift on north bearing to determine if the bearing design is adequate. In the absence of details on the contract drawings it has been assumed that this reaction includes dead and live load effect, as well as temperature and wind load effects.
• 5,300 kN (uplift). This load is considered as the ultimate bearing uplift on centre bearing, which is the greatest specified design uplift force. The design of the three West Abutment bearings is similar, so the ULS uplift force on the centre bearing should govern its design.

7.2.2.2.2 Design Bearing Rotation

In addition to applying the bearing uplift loads, we have considered corresponding bearing rotations as a result of the change of angle between steel girders and the concrete abutment. Bearing rotations may be a result of several factors:

• Girder rotation due to applied load dead and live load.
• Construction tolerance and imperfections. We assume that bearing rotation requirement specified on the design drawings include a reasonable allowance for possible imperfections in both installation of the bearing base on the abutment and the fabrication of steel girders.
• Temperature change and temperature gradient.

The design drawings specify 0.2° rotation coinciding with dead load only at north-west bearing, as well as 0.8° rotation for serviceability dead and live load, as well as up to 1.3° of horizontal rotation coinciding with
the maximum ULS uplift. OPSS 922 1203 require the bearings to accommodate an additional rotation about any horizontal axis of 1.2° for the ULS condition. The specified horizontal rotation applies in all directions. The physical dimensions of the bearing do not allow free rotations to meet the requirements.

Based on the field measurements outlined in Chapter 4, construction imperfection of the north abutment bearing placement is in the order of 7mm over the 1,000mm long bearing, or 0.4° and 3mm over the 800mm wide bearing or 0.23°

The design drawings also require 1.0° of rotation capacity about the vertical axis; it is assumed that this means that the bearings are required to ‘twist’ 1.0°, corresponding with girders rotating in horizontal plane. Upon inspection there does not appear to be realistic potential for significant girder rotation in the horizontal plane and as such bearing rotation about the vertical axis was not considered.

The bearing design used at the west abutment does not readily facilitate girder rotation coinciding with uplift. Practically, the upper and lower bearing guide bars will only transfer uplift force when in contact; for an uplift force centered on the bearing, the entire length of guide bars need to be in contact. In the case of longitudinal bearing rotation, upper and masonry plate guide bars will be in contact only at one end, resulting in an eccentricity of uplift reaction (see Figure 7-8). The longitudinal eccentricity limit is 500mm, corresponding with half the length of the bearing blocks; 500mm eccentricity would cause the upper and lower bearing blocks to be in ‘knife-edge’ contact, effectively creating a hinge, provided that the bolts and steel plates do not fail.

Eccentricity of the uplift reaction may cause overstress of the connecting bolts or result in overstressing and plastic deformation of the plates. The effect of girder rotation relative to the abutment on bearing component stresses are partially mitigated by elastic bending of the girder flange, shoe plate and bearing blocks, as well as stretching of the bolts. Figure 7-9 illustrates an exaggerated rotation of the girder (with uplift reaction) relative to the abutment as a result of bending of the plates and bolt elongation.
To determine the effects of rotation under SLS and ULS uplift, the specified uplift force was applied with a progressively increasing girder rotation until reaching the capacity criteria for each analysis.

To determine capacity of the bearings to tolerate rotation under loads, the following two conditions were analyzed:

- Uplift load of 1,900 kN, corresponding with the maximum unfactored permanent load uplift on the north bearing, combined with rotations up to 0.2° longitudinally or transversely.
- Uplift load of 3,370 kN, corresponding with the maximum ULS uplift on the north bearing, combined with rotations up to 1.3° longitudinally or transversely.

As noted previously, bearing geometric limitations did not allow for full rotation requirements.

7.2.2.2.3 Design Bearing Longitudinal Translation

In addition to uplift force eccentricity caused by bearing rotation, as discussed above, the bearing is expected to experience additional uplift force eccentricity caused by bearing longitudinal translation, or
sliding. In addition, longitudinal movement will cause a friction force, resulting in further increased tension on some of the bolts; see Figure 7-10 for illustration.

It was intended to consider effects of bearing longitudinal movement in case the analysis results indicated that the bearing may be able to resist design uplift force and rotation. However, based on the analysis described below, the bearing does not have the capacity to resist the design uplift force combined with bearing rotation. Analysis of longitudinal movement effects was therefore not required.

Figure 7-10: Bearing longitudinal movement
7.2.3 West Abutment Bearing Analysis Results – Design Evaluation

To check bearing design adequacy, the analysis includes design SLS and ULS uplift forces, with and without their corresponding bearing rotations. A summary of the considered design evaluation cases, and their results, is provided in Table 7-1 and Figure 7-11 below. Each case is discussed in detail within the following sections.

The design evaluation results are reported in terms of maximum flange connection bolt (HA1) demand, with one of the following outcomes:

1) Maximum bolt tensile force and elongation remain within the elastic range.
2) Bolts elongate past their first yield point (0.32mm), and into their inelastic range. Due to the cyclic nature of bridge traffic loads together with the imperfections of the bearing this presents a potential for low-cycle fatigue of the bolts, which would eventually lead to bolt failure.
3) Bolts elongate to failure (3.5mm) with the initial application of the load.

<table>
<thead>
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<th>Limit State</th>
<th>Design Demands (Force/Rotation)</th>
<th>Shoe Plate Yield?</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>1900kN</td>
<td>SLS 1900kN/0°</td>
<td>No</td>
</tr>
<tr>
<td>Case 2</td>
<td>3570kN</td>
<td>ULS 3570kN/0°</td>
<td>No</td>
</tr>
<tr>
<td>Case 3</td>
<td>5300kN</td>
<td>ULS 5300kN/0°</td>
<td>Yes</td>
</tr>
<tr>
<td>Case 4</td>
<td>1900kN + Longitudinal Rotation</td>
<td>SLS 1900kN/0.2°L</td>
<td>No</td>
</tr>
<tr>
<td>Case 5</td>
<td>1900kN + Transverse Rotation</td>
<td>SLS 1900kN/0.2°T</td>
<td>No</td>
</tr>
<tr>
<td>Case 6</td>
<td>3370kN + Longitudinal Rotation</td>
<td>ULS 3370kN/1.3°L</td>
<td>Yes</td>
</tr>
</tbody>
</table>

Table 7-1: Analysis cases and results summary for design evaluation
7.2.3.1 Center Uplift Reaction

The following uplift forces have been applied to the FE model:

- **Case 1** - 1,900KN uplift, corresponding with the dead load uplift on the north-west bearing, used in failure analysis.
- **Case 2** - 3,370KN uplift, corresponding with the minimum ultimate demand on the north-west bearing, used for the design check.
- **Case 3** - 5,300KN uplift, corresponding with the minimum ultimate demand on the center-west bearing, used for the design check.

The uplift forces are applied centered to the bearing, see Figure 7-12 for illustration.
The exaggerated deflected shape of bearing plates resisting a centered uplift reaction is shown on Figure 7-13. From the deformed shape of the bearing significant relative bending of the shoe plate is evident, as well as local bending of the girder bottom flange between girder stiffeners, which results in uneven distribution of uplift forces on bolts connecting the shoe plate and the girder flange.
7.2.3.1.1 Case 1 Design Evaluation: 1,900 kN Uplift

Model results for Case 1, a centered maximum dead load uplift reaction (1,900kN), at the north-west abutment bearing, indicate the following:

1. The Von Mises (effective) stress in the lower and sole plates does not exceed 248MPa, which is the yield strength of ASTM A36 used to fabricate these components. See Figure 7-14.

2. The stress in the beveled shoe plate does not exceed the yield strength (for 350W and A36 steel). The following is a hand check of bending moments in the shoe plate:
   - Shoe plate 100x800; thickness varies 52 to 60mm; average thickness = 56mm
   - Factored bending moment at outside bolts: \( M_f = 1900 \text{kN}/2 \times 0.12 \text{m} = 114 \text{kNm} \)
   - Stress in shoe plate \( f = \frac{114 \times 10^6}{(1000 \times 56^2/6)} = 218 \text{ MPa} < F_y \)
   - Plastic capacity of the plate assuming 350W steel: \( M_{pl} = 0.95 \times 350 \times 1000 \times 56^2/4/10^6 = 264 \text{kNm} \)
   - Plastic capacity of the plate assuming A36 steel: \( M_{pl} = 0.95 \times 248 \times 1000 \times 56^2/4/10^6 = 185 \text{kNm} \)

3. Tensile loads in the bolts connecting the shoe plate and the flange are shown on Figure 7-15. The force distribution is highly uneven; the tensile loads are the greatest in outside line bolts closest to the bearing stiffeners, indicating that the stiffeners are creating hard points that attract force.

Figure 7-14: Von Mises stress [MPa] in bearing elements due to 1,900kN uplift
Figure 7-15: Bolt tension [kN] due to 1,900kN uplift

The greatest resulting force in a ∅7/8" bolt connecting the shoe plate and girder flange is 161kN; which is less than the design bolt tensile resistance of 241kN. Therefore, the bolt capacity is not exceeded.

The maximum tensile force is also lower than the first assumed yield tension force of 217kN, indicating that the bolts do not experience plastic tensile behavior with this applied load.

The sum of forces in the front rows of bolts is equal to: 4*(161+98+77+93+148) =2,308kN, which is greater than the applied force of 1,900kN. The difference of 408kN represents the calculated prying force in the connection. The prying force is equal to:408/1900 = 21% of the externally applied load, which is less than 30% limit, prescribed in Cl. 10.17.2.6 of CHBDC S6-14.

Figure 7-16 shows elongations in bolts connecting the beveled shoe plate and the girder flange. Based on the model results, elongation of critical corner bolts are in the order of 0.2%.

4. Stresses in bolts connecting shoe plate and sole plate are within the allowable limits as well. Tensile demand in these bolts is 131 kN; the design capacity of ∅25.4 (1") A490 bolt is 315 kN.
7.2.3.1.2 Case 2 Design Evaluation: 3,370 kN Uplift

An uplift force of 3,370kN is the greatest factored (ULS) reaction which occurs at the north-west bearing. Model results for a centered uplift of 3,370kN indicate the following:

1. The Von Mises (effective) stress in the lower and sole plates does not exceed 248MPa, which is the yield strength of ASTM A36 used to fabricate these components. See Figure 7-17.
2. The maximum stress in the bevelled shoe plate is near 350MPa at the outer line of bolts, which is the yield strength of grade 350W steel (specified on contract drawings). This far exceeds the yield strength of A36 steel, specified on fabrication drawings. See Figure 7-17 for illustration. As noted, the shoe plate yielding effects were accounted for by incrementally softening these elements during the analysis. As such, the residual plastic strains and shape will not be evident in the plots.

To confirm the FE analysis results, a hand calculation was carried out to evaluate the bending stresses in shoe plate at the outside line of bolts connecting it to the girder flange (see Figure 7-18 for illustration):

Figure 7-17: Von Mises stress [MPa] in bearing elements due to 3,370kN uplift
Shoe plate 100x800; thickness varies 52 to 60mm; average thickness = 56mm
Factored bending moment at outside bolts: \( M_f = \frac{3370 \text{kN}}{2} \times 0.12 \text{m} = 202.2 \text{kNm} \)
Stress in shoe plate \( f = \frac{202.2 \times 10^6}{1000 \times 56^2/6} = 387 \text{ MPa} > F_y \)
Therefore, it is expected the shoe plate to be near yield.
Plastic capacity of the plate assuming 350W steel: \( M_{pl} = 0.95 \times 350 \times 1000 \times 56^2/4/10^6 = 264 \text{kNm} \)
Plastic capacity of the plate assuming A36 steel: \( M_{pl}'' = 0.95 \times 248 \times 1000 \times 56^2/4/10^6 = 185 \text{kNm} \)

This hand calculation confirms that the shoe plate does not have the capacity to support the applied loads. A factored load of 3,370kN will cause the plate stress to be near yield; if the plate yield strength were 248MPa (as specified for A36 steel), full plastic hinges would form at the outside line of bolts connecting shoe plate and the girder flange, resulting in a mechanism and ultimately connection failure. Note that the above stress calculation is unconservative, as the 10-\(\frac{2}{5}\)25.4 bolt holes were not included, which would significantly reduce the plate’s bending capacity.

3. Tensile loads in the bolts connecting the shoe plate and the flange exceed the elastic capacity of the bolts, causing faying surfaces to lose contact and bolts to elongate inelastically. See Figure 7-19.
Stresses in bolts connecting the shoe plate and the sole plate are within the allowable limits. Tensile demand in these bolts is 227 kN; the capacity of \( \varnothing 25.4 \) (1") A490 bolt is 315 kN.

4. Figure 7-20 shows elongations in bolts connecting the bevelled shoe plate and the girder flange. Based on the model results, elongation on critical corner bolts (in the corners) in are in the order of 0.5%, which is less than the critical elongation of 3.5%. Therefore, the analysis results indicate that the bolts will yield under this static load, but not fail. Repeat application of this load could result in low-cycle fatigue of the bolts.
Figure 7-20: Bolt elongations [mm] due to 3,370kN uplift

7.2.3.1.3 Case 3 Design Evaluation: 5,300 kN Uplift

An uplift force of 5,300 kN corresponds with the maximum factored (ULS) uplift on the centre bearing of the West Abutment. This is the critical ULS reaction. Model results for a centered uplift of 5,300kN indicate the following:

1. The uplift force is applied to the model in increments equal to 1/20 of the target force of 5,300kN. Monitoring the response of connection bolts at each increment of uplift load increase, it was found that the bearing capacity corresponds with an uplift force of 5035kN, past which point progressive failure would onset. The steps leading up to failure are as follow:
   a) As the uplift load was incrementally increased, the bolt capacity was exceeded, causing the bolts to yield.
   b) Since the shoe plate does not have sufficient capacity to transfer the tension to the inside line of bolts, bolts in the outside line continued to elongate with increasing load until the first (corner) bolt elongation reached 3.5% at 5035kN, resulting in its failure.
   c) Following the failure of the first (corner) bolt, forces redistribute to surrounding bolts, causing progressive bolt failures and ultimately connection collapse.
2. The Von Mises (effective) stresses in the bearing plates exposed to the greatest possible uplift force of 5035kN are shown in Figure 7-21.

![Figure 7-21: Von Mises stress [MPa] in bearing elements due to maximum uplift (5035kN)](image)

The Von Mises (effective) stress in the lower plates (bearing sole plates and masonry plates) does not exceed 248MPa, which is the yield strength of ASTM A36 used to fabricate these components. In certain location, such as corners of the guide bar angles, the model shows local stresses exceeding 248MPa; however, these locations are not critical and have been neglected.

3. The maximum stress in the beveled shoe plate exceeds 350MPa, which is the yield strength of grade 350W steel (specified on contract drawings) and far exceeds the yield strength of A36 steel, specified on fabrication drawings. See Figure 7-21 for illustration. To confirm the FE analysis results, we have conducted a hand calculation of bending stresses in shoe plate at the outside line of bolts connecting it to the girder flange:

Shoe plate 100x800; thickness varies 52 to 60mm; average thickness = 56mm
Factored bending moment at outside bolts: \( M_f = 4240kN/2 \times 0.12m = 254kNm \)
Elastic Stress in shoe plate \( f = 254\times10^6 / (1000 \times 56^2/6) = 486 \) MPa >> Fy
Therefore, it is expected that the shoe plate will yield.

Plastic capacity of the plate assuming 350W steel: \( M_p' = 0.95\times350\times1000\times56^2/4/10^6 = 264kNm \)
Plastic capacity of the plate assuming A36 steel: \( M_p'' = 0.95\times248\times1000\times56^2/4/10^6 = 185kNm \)
The hand calculation confirms that the shoe plate does not have the capacity to support the applied loads; if the plate yield strength is 248MPa (as specified for A36 steel), full plastic hinges will form at the outside line of bolts connecting shoe plate and the girder flange, resulting in a mechanism and ultimately connection failure. Note that the above stress calculation is unconservative, as the 10-Ø25.4 bolt holes were not included, which would significantly reduce the plate’s bending capacity.

4. Stresses in the bolts connecting the shoe plate and the flange reach the bolt tensile capacity causing faying surfaces to lose contact and bolts to elongate. The stress distribution is highly uneven; the bulk of the total uplift load is carried by the outer lines of flange connection bolts. For illustration see Figure 7-22.

![Figure 7-22: Bolt tension [kN] due to maximum uplift (5035kN)](image)

Stresses in bolts connecting shoe plate and sole plate would exceed the allowable limits in this case. Tensile demand in these bolts would exceed the design capacity 315kN for Ø25.4 (1”) A490 bolts.

5. Figure 7-23 shows elongations in bolts connecting the beveled shoe plate and the girder flange. The applied centered uplift force of 5035kN causes the elongation in critical bolts to exceed 3.5%, resulting in the first bolt failure, which would be followed by progressive connection failure.
7.2.3.2 Results for Bearing Rotation

The bearing’s ability to tolerate horizontal rotation combined with uplift reaction was analyzed, considering both longitudinal and transverse horizontal rotation. Longitudinal rotation is defined as rotation about an axis perpendicular to the bridge girders. Transverse rotation is defined as rotation about an axis parallel to the bridge girders. See Figure 7-24 and 7-25 for illustration.

The bearing design does not readily allow bearings to rotate in either direction when combined with an uplift force, other than through plate bending and bolt elongation. Thus the rotational capacity of the bearings under uplift forces is limited primarily by bolt ductility and plate bending strength. Separation of the PTFE from the stainless steel along the slide path will result in damage to the PTFE and further additional eccentric forces applied to the bearing in uplift.

As a consequence of the bearing design, a constant uplift reaction (Zi) combined with girder rotation (θ) results in a specific longitudinal eccentricity (e) of the reaction. Thus it follows that an eccentricity of the uplift reaction corresponds with an applied bearing rotation θ.
Figure 7-24: Bearing longitudinal rotation schematic (exaggerated deformation)

Figure 7-25: Transverse bearing rotation schematic (exaggerated deformation)
7.2.3.2.1 Case 4 Design Evaluation: Bearing Longitudinal Rotation With Uplift Load of 1,900 kN

Based on design demands, an uplift load of 1,900 kN (max dead load uplift in northwest bearing) was considered in combination with 0.2° of longitudinal rotation, corresponding with a bearing uplift eccentricity of 180 mm. A longitudinal bearing rotation of 0.2° is specified on the contract drawings. With the application of a 1,900 kN uplift force and bearing rotation of 0.2°, tension in the critical (corner) bolt will reach its capacity, causing the bolt to yield and inelastically elongate, as shown on Figure 7-26.

![Figure 7-26: Bolt elongation [mm] for uplift of 1,900 kN and longitudinal rotation of 0.2°](image)

Since analysis results show corner bolts yielding under dead load, vehicle passage on the bridge deck would cause inelastic elongation of 0.6 mm of the corner bolts. Ultimate bolt elongations for this analysis case are well under the 3.5% limit. Although bolts would not immediately fail, they would be exposed to low cycle fatigue, leading to possible eventual progressive connection failure.
Figure 7-27: Bolt tension [kN] for uplift of 1,900kN and longitudinal rotation of 0.2°
Prior to low-cycle fatigue failure of the bolt observed on the fractures surfaces of the failed bolts by NRC and SSW, stresses in the beveled shoe plate will be below the design yield of 350MPa (see Figure 7-28). As the bolts in the outer line of bolts fail due to low-cycle fatigue, the load would be transferred to the centre bolt lines, dramatically increasing bending moment in the shoe plate, which would result in overstressing and yielding of the shoe plate.

Figure 7-28: Von Mises stress [MPa] in plates for uplift of 1,900kN and longitudinal rotation of 0.2°

7.2.3.2.2 Case 5 Design Evaluation: Bearing Transverse Rotation With Uplift Load of 1,900 kN

In addition to investigating the combined effects of uplift and longitudinal rotation, uplift load of 1,900kN in combination with transverse rotation of 0.2° was evaluated, as specified in the contract drawings.

An exaggerated deformed shape of the bearing exposed to transverse rotation is shown on Figure 7-29. As a consequence of the transverse rotation; distribution of the uplift force on the restrainer angles becomes highly uneven.
As a result of these demands, bolt tension in the critical (center and corner) bolts will exceed their theoretical yield force (as shown on Figure 7-30), causing the bolt to elongate plastically with the passage of vehicular traffic.

Figure 7-30: Bolt tension [kN] for uplift of 1,900kN and transverse rotation of 0.2°
Since analysis results show corner bolts yielding under dead load, vehicle passage on the bridge deck would cause inelastic elongation of the corner bolts. Ultimate bolt elongations for this analysis case are well under the 3.5% limit (see Figure 7-31). Although bolts would not immediately fail, they would be exposed to low-cycle fatigue, leading to possible eventual progressive connection failure.

![Diagram of bolt elongation](image)

**Figure 7-31:** Bolt elongation [mm] for uplift of 1,900kN and transverse rotation of 0.2°

Prior to potential bolt low-cycle fatigue failure, stresses in the bevelled shoe plate would be below the design yield of 350MPa (see Figure 7-32). However, the stress in some locations exceeds 248MPa, which would result in shoe plate local yielding if it were A36 steel, as shown on bearing shop drawings. As the bolts in the outer line fail in low-cycle fatigue, the load would be transferred to the inner line bolts, dramatically increasing bending moment in the shoe plate, resulting in overstressing and yielding of the shoe plate.
To determine the maximum rotation that the bearing can tolerate in combination with maximum ULS uplift at the north bearing, we have applied constant uplift load $Z_f$ with increasing eccentricity $e$, until the bolts reach maximum elongation of 3.5%, and recorded girder rotation in each step. The greatest rotation, obtained just before first bolt failure at maximum eccentricity is the maximum bearing rotation under the specified uplift load.

Finite element analysis results for the uplift load of 3,370kN (max ULS uplift in north bearing) are shown below. Based on our analysis results, the bearing can withstand up to 0.8° of rotation, combined with uplift force of 3,370kN; this rotation corresponds with uplift force longitudinal eccentricity of 200mm. As the uplift force eccentricity increases from, axial tension on bolts connecting the beveled shoe plate and girder flange exceeds the bolt capacity and bolts begin to yield. Elongation in critical (corner) bolts increases with rotation. When rotation reaches 0.8° (corresponding with $e=200$mm), elongation in the corner bolts exceeds 3.5%, causing the corner bolts to fail, as shown in Figure 7-34. Failure of the corner bolts triggers a progressive failure of the bolted connection.

**Figure 7-32:** Von Mises stress [MPa] in plates for uplift of 1,900kN and transverse rotation of 0.2°

**7.2.3.2.3 Case 6 Design Evaluation: Bearing Longitudinal Rotation with Uplift Load of 3,370 kN**

To determine the maximum rotation that the bearing can tolerate in combination with maximum ULS uplift at the north bearing, we have applied constant uplift load $Z_f$ with increasing eccentricity $e$, until the bolts reach maximum elongation of 3.5%, and recorded girder rotation in each step. The greatest rotation, obtained just before first bolt failure at maximum eccentricity is the maximum bearing rotation under the specified uplift load.

Finite element analysis results for the uplift load of 3,370kN (max ULS uplift in north bearing) are shown below. Based on our analysis results, the bearing can withstand up to 0.8° of rotation, combined with uplift force of 3,370kN; this rotation corresponds with uplift force longitudinal eccentricity of 200mm. As the uplift force eccentricity increases from, axial tension on bolts connecting the beveled shoe plate and girder flange exceeds the bolt capacity and bolts begin to yield. Elongation in critical (corner) bolts increases with rotation. When rotation reaches 0.8° (corresponding with $e=200$mm), elongation in the corner bolts exceeds 3.5%, causing the corner bolts to fail, as shown in Figure 7-34. Failure of the corner bolts triggers a progressive failure of the bolted connection.
In addition to bolt elongation, the stresses in the bearing plates were examined. Stresses in upper and lower bearing blocks were determined to be generally under the yield stress of 248MPa, except for a few localized areas. However, stress in bevelled shoe plate was near its yield (assumed to be 350MPa) for centered load (e=0); the stress further increased with uplift eccentricity, causing plastic deformation of the shoe plate. To account for inelastic plate deformation, the effective bending stiffness of the shoe plate was reduced in the zones where resulting bending moments exceed plate plastic capacity, effectively modeling yielding of the plate. Von Mises stresses in bearing plates for the critical stage are shown in Figure 7-34.

Figure 7-33: Bolt elongation [mm] for Z=3,370kN and e=100mm. Note that maximum elongation is 3.7mm (>3.5mm), indicating bolt failure
7.2.4 West Abutment Bearing Design Evaluation Discussion

The bearings at the west abutment do not have sufficient capacity to resist the uplift loads shown in the contract drawings, in particular the factored (ULS) uplift load of 5,300kN on the centre girder. Using the most optimistic assumption regarding construction tolerances, materials and installation, the tensile capacity of the bearing was calculated to be in the order of 5,035kN. The bearing tensile capacity is limited by the capacity of the 7/8”-A490 flange connection, as well as capacity of the shoe plate in bending. This limiting uplift force was calculated under the assumption that the bearing and the bolts are ‘perfectly’ installed (with zero tolerances), and that there is no corresponding girder rotation. The bearing capacity would be further reduced by any girder rotation, construction tolerance in bearing placement, lack of bevelled washers under bolt nuts, and variation in installed bolt tightness.

The bearing design does not facilitate rotation when combined with uplift force; any girder rotation relative to the abutment significantly increases stresses in the shoe plate and its connection bolts. Consequently, the north-west bearing design is not able to tolerate a specified bearing rotation of 1.3° in combination with ULS uplift force of 3,370kN. With an uplift force of 3,370kN, a bearing rotation of only 0.8° causes critical bolts to fail in tension, contributing to the potential for progressive connection failure.

The bearing design was evaluated for a service load, including 1,900kN dead load reaction and bearing rotation of 0.2°, longitudinally or transversely. Based on this analysis, this load combination will cause

Figure 7-34: Von Mises stress [MPa] in bearing plates; Zf=3,370kN’ e=100mm; Longitudinal Rotation =0.8°
some of the bolts to yield, also consistent with conditions necessary for progressive low-cycle fatigue failure.

7.3 BEARING FAILURE ANALYSIS

In addition to the analysis focused on bearing design evaluation described above, the following analysis was conducted to assist in understanding the low-cycle fatigue failure of the northwest flange connection bolts. For this purpose, loads present on the bridge from opening until failure were considered, as well as geometric imperfections measured on site during the site investigation outlined in Section 4.

7.3.1 Applied Loads

To explain the north-west bearing failure, the following is considered:

1. Bridge dead load for structure in Phase One. Dead load analysis is described above.
2. Imperfections in bearing placement, measured in the field after bearing failure.
3. Live load consisting of one CL-625-ONT truck in the westbound lane obtained from the global FE bridge model.
4. The possibility of slip (shear deformations) between the shoe plate and the girder flange plate.

Based on the global FE model analysis results, neither cold weather nor wind was a material cause of the bearing failure.

Measurements taken of the neutral north-west bearing position post-failure, as described in Section 4, indicate an average vertical difference of 4mm (7mm difference along the south guide bar and 1mm along the north guide bar) in the longitudinal direction over the length of the shoe plate, with a slope toward the centre pier. This difference has been included in the model as a corresponding longitudinal girder rotation of $+0.23^\circ$ to represent the initial installation condition.

The governing force and rotation demands on the north-west bearing from the passage of a truck correspond with truck Position A and Position B as shown in Figure 7-35. Truck Position A results in a maximum north-west bearing uplift load, and truck Position B results in the maximum positive longitudinal rotation at the bearing. A summary of the applied truck demands, combined with the dead load uplift and initial rotation, is shown in Table 7-2.
For the purpose of presenting realistic low cycle fatigue potential, the assumed flange connection bolt (HA1) trilinear tensile stress-strain curve was modified to represent the higher nominal capacity of 301 kN and associated yield elongations (see Figure 7-7).
7.3.2 Analysis Results

7.3.2.1 Girder to Shoe Plate Contact

A Phase One dead load only case, in combination with an average measured longitudinal out-of-tolerance, was considered to provide an indication of the prying-induced contact force between the girder bottom flange and shoe plate prior to the north-west bearing failure. This contact force was quantified by the summation of the individual forces in rigid ‘compression-only’ elements between the girder bottom flange and the shoe plate, as shown in Figure 7-36, resulting in a total of 513 kN of compression. This figure displays only the modeled contact elements that experience compression forces; all other elements are hidden from view, but the net contact forces are included.

![Figure 7-36: Compressive forces between plates from Phase One dead load from](image_url)

The total contact force on the girder flange from prying of the shoe plate is 513kN. Frictional forces between steel surfaces are known to be in the order of 20% of the applied force at the start of slip and reduce to 12% once slip has occurred. The resistance to slip of the shoe plate could then be in the order of 62kN to 102kN. When compared to the resistance to slip along the lower sliding surface between PTFE
and stainless steel of 5% to 10% of the total uplift force of 1719kN equates to 86kN to 172kN, this analysis and assessment that the possibility exists for slip to occur.

Slip of the shoe plate below the girder flange would introduce additional bending moments to the bolts which have not been quantified in our analyses and will contribute significantly to the low-cycle-fatigue cracks observed on the fracture surfaces of the bolts

7.3.2.2 Bolt Strain

Due to the effects discussed in previous sections, the flange connection bolts experience an uneven distribution of forces, with a maximum force experienced by the west corner bolts. The bolt elongations from the load combinations with the truck at Position A and Position B are summarized in Figure 7-37.
Figure 7-37: Flange Connection Bolt Elongations [mm]
Results of our analysis show that the most heavily loaded flange connection bolt (HA1), in the west corners of the shoe plate, which would yield and inelastically elongate when a CL-625-Ont truck was in Position A. The demands on the bolt would be reduced when the same truck is in Position B. This inelastic cycling behaviour of the bolt indicates plastic bolt strains and the potential for low-cycle fatigue failure initiated in a west corner of the shoe plate (Figure 7-38).

![Diagram of Demands from Truck Passage on Flange Connection Bolt (HA1)](image)

Figure 7-38: Results of single truck live load analysis

This analysis result is based on an average measured longitudinal out-of-parallel initial bearing condition. Considering that the southwest corner of the lower guide is measured to be the highest, combined with the evidence of the PTFE crushing in this corner, this analysis supports the conclusion that the highest initial bolt strains were experienced in the southwest corner.
7.3.2.3 North-west Bearing Failure Analysis Discussion

Based on the analysis, the repeated passages of single CL-625-ONT legal trucks along the bridge would cause cyclic plastic strains and in the bolts connecting the bevelled base plate and girder flange.

With the combined Phase One dead load, installation imperfections, and the passage of a truck, the analysis shows that the west corner bolts likely experienced cyclic inelastic elongation first. This is consistent with an observation from site made of PTFE distortion in the south-west corner after bearing failure, indicating high load concentration. Each passing truck over the bridge deck caused a cycle of reduced and increased uplift reaction and translation in the longitudinal direction of the bridge, leading to low-cycle fatigue of the highest loaded bolt. This process progressed, and ultimately resulted in complete connection failure.

Other factors, such as wind effects, additional uplift force due to cable thermal loads, or bearing translation, may have increased the uplift force and expedited the failure of the bearing assembly; but those effects are relatively minor.

A lack of bolt pretension and variation in bolt tightness may have accelerated the bearing connection failure, due to further uneven force distribution among the bolts. The local bending of the bolts, due to a lack of beveled washers and transverse prying deformations of the shoe plate, further accelerated the bolt failure.
8 Discussion

Previous sections of this Report described information assembled and generated regarding the circumstances contributing to the failure. They emphasized factual information, modelling and analysis results, and assessments and interpretations of information within sections. This section integrates our findings into the factors contributing to the failure of the bearing.

These findings are based on the Site Visit Observations, Bolt Investigation Review, Document Review, West Abutment Bearing Evaluation and engineering interpretations by Associated Engineering (Ont.) Ltd. (AE) The bolt testing reports by NRC and SSW, and the fracture surface interpretations by NRC, provided critical physical information and material behaviour assessments that were considered in our investigation and contributed to our conclusions.

The Site Visit Observations undertaken by Associated Engineering staff revealed key information including:

- The shoe plate had undergone plastic deformation, as evident from the permanent deflected shape of this plate.
- The bearings were observed to be unable to freely accommodate rotation about either major axis.
- The masonry and sole plates were misaligned at all bearings.
- Bolts supplied to site were too long requiring multiple flat washers to achieve a snug tight condition. This snug-tight condition appears not to have been achieved consistently for all of the bolts on the north-west bearing. The bolts were not pre-tensioned.
- The final detailing of the shoe plate and the machined recesses made it difficult to tighten the bolts. Access to the bolt heads became difficult following the upper bearing assembly installation, and recess dimensions were too large to effectively prevent bolts from rotating, but too small to allow a socket to be inserted to hold the bolt heads.
- The centre-west bearing required ¼ to ½ turns of the nut after failure of the north-west bearing indicating the bolts were possibly loose and/or un-tensioned at the time of failure.
- Clear evidence of sliding of plates on bolt shanks was observed. These demonstrate conclusively that horizontal movements occurred at the interface of the shoe plate and girder flange.

A detailed review of the NRC and SSW reports combined with an independent visual review of all the bolts, provided the following critical information and findings:

- Bolts failed progressively due to cyclic loads combined with high axial tensions that caused low-cycle fatigue cracks.
- Crack propagation from one vs two sides indicates different types of loading causing failure.
- Corrosion on bolt fracture surfaces indicate early failure of some bolts. The south-west bearing quadrant included five bolts with evidence of early failure.
The outer lines of bolts exhibit flat failure surfaces and two-sided crack propagation consistent with high axial tension coupled with alternating flexural tension causing low-cycle fatigue, while the inner lines of bolts generally show steep failure surfaces consistent with one-sided crack propagation with high axial tension causing low-cycle fatigue.

All bolts displayed striations consistent with low-cycle fatigue ending in ductile fractures in high tension.

The Review of Documents yielded several observations that influenced the bearing behaviour. These observations include:

- Shoe plate and bearing installation was undertaken in a manner unable to meet construction tolerances.
- The installed bearing assembly did not satisfy construction tolerances outlined in OPSS 922.
- The bearing cannot freely accommodate rotation while subjected to uplift.

In addition to the site investigations, bolt observations and document review, AE engineers undertook a series of global and local finite element analyses to determine demands on the bearings and other components and to investigate the local behaviour of the bearing assembly. The local finite element analyses subjected the bearing assembly to a number of demand combinations including uplift and rotation both in the longitudinal and transverse directions. These analyses were used to both evaluate the design of the bearings and to investigate the behaviour of the bearings in it’s “as-built” condition when subjected to service loads. Key observations from this series of analysis include:

- The north-west bearing design evaluation showed that the bearing assembly was inadequate for all design evaluation cases except for service dead load only with no rotation.
- Bolt tensions were amplified significantly from prying effects of the relatively flexible shoe plate and the arrangement of bolts.
- Shoe plate flexure coupled with loose bolts cause conditions where contact pressured between the shoe plate and flange were reduced. This created the potential for sliding displacements across this interface.
- Design evaluation cases confirmed that the bearing assembly could not freely accommodate specified rotational demands while subjected to uplift, without causing unintended demands in bolts and plates comprising the bearing assembly.
- Design evaluation cases indicated yielding of the flange connection bolts resulting in either low-cycle fatigue potential or tensile failures.
- Shear stresses would occur from traffic loads and temperature effects at the interface beneath the girder flange and shoe plate. These stresses would be limited by PTFE friction within the bearing.
These key findings are discussed below.

8.1 PHYSICAL CAUSE OF FAILURE

The north-west bearing bolt group failure was through a progressive low-cycle, plastic-strain fatigue fracture under high axial tension. The bolts failed progressively over time rather than in a single, sudden event. Nine bolts were assessed as failing early as evident from corrosion seen on the fracture surfaces. All of the failed bolts showed striations consistent with low-cycle fatigue at high strains. Some of the bolts fractured primarily in high axial tension under cyclic loading. Other bolts fractured in high axial tension combined with cyclic bending. The contributing causes to the individual bolt failures and to the bolt group failure overall are described throughout this report and discussed together in this Section.

Twelve of the 14 bolts examined by NRC displayed progression fracture surfaces propagating from one side of the bolts consistent with cyclic direct tension. Two of the bolts displayed fatigue crack growth from two sides and NRC reported that they likely also experienced bending during loading. Four bolts examined by SSW showed that the failure surfaces were more consistent with cyclic bending combined with high axial tension. SSW also reported some bolts as failing before others based again on the corrosion product seen on some of the bolts.

Our analyses of the various demands on the shoe plate / bearing/ bolt system, and of the local deformations, stresses and strains in the bolts and shoe plate, are consistent with the failure surfaces observed and a finding of progressive bolt failure. Analyses show that strains in the bolts arising from heavy but highway-legal trucks, when combined with permanent dead load uplift, with the bearing’s inability to freely rotate while in uplift, and with the out-of-parallel installation of the bearings would have been within their post-elastic (plastic) range and sufficient to cause low-cycle fatigue fractures. Horizontal bending stresses were introduced into some bolts, typically in the outer two lines of bolts having the highest initial tensions, which contributed to their fracture. Bending effects arose from length changes in the bridge deck under traffic and thermal effects causing longitudinal displacements at the bearing. The bearings were intended to accommodate these deformations through sliding along low-friction PTFE sliding surfaces. However, local damage to the PTFE caused an increase in friction and allowed bolts to experience bending.

The bolts were not pre-tensioned on installation, and this also contributed materially to the failure including to the bending effects initially in outer lines of bolts. The beveled shoe plate was too thin and flexible, and this also contributed to increases in bolt forces that contributed to the failure.

That there were several important contributing causes and mechanisms is important, and each are discussed in this Report and described below. The occurrence of all of these in combination is unique and was highly improbable, however a confluence of events and conditions is typical of failures within structures.
8.2 PRYING EFFECTS

The forces and strains in the outer two lines of bolts (“lines” of bolts are oriented parallel to the roadway) were increased significantly by transverse prying effects arising from flexibility of the bevelled shoe plate between the bearing and the girder flange. The prying effects cause the majority of the uplift load to be carried by the outer line of bolts.

In addition to the elastic flexibility of the shoe plates, yielding and plastic strains also occurred in the shoe plate during the bolt fracture propagation which allowed the shoe plate to deflect further. Yielding was clearly evident in the permanent deformation of the plate following failure. Both elastic prying effects and plastic behaviour in the shoe plate contributed to increased strains in the bolts. Both effects contributed materially to the progression of the bolt fractures.

Bolts were countersunk into the shoe plates which would have weakened and softened the shoe plates. The analysis in this report did not include the removal of base metal from the shoe plates since the design review demonstrated that the shoe plate, even without this weakening effect, was inadequate for the demands specified for the tie-down bearings. A thicker, stiffer and stronger shoe plate would not have reduced the risk of bolt failure to acceptable levels, but could have delayed the fracture of the bolts within the group.

8.3 BEARING DEFICIENCIES

The inability of the tie-down bearings to freely accommodate rotation in an uplift condition caused increased bolt forces and contributed to plastic strains in the bolts. These increases arise from the deformations imposed on the bolts and shoe plate by the bridge superstructure. The bearing system design neither isolated the bolt groups from these deformation demands nor accommodated the specified demands in the flange-to-bearing bolted connection. Based on analysis, observation, our understanding of the bearing behaviour, and measurements, the tie-down bearing arrangement was inadequate and inappropriate for the requirements of this bridge.

In addition, there were a number of variations on the fabrication drawings relative to the contract drawings. These are described in detail in Section 6. Important variations from the construction drawings included the use of A36 steel (minimum 248 MPa) where 350 MPa yield stress was specified, an apparent error in shoe plate bolt lengths, bevelled washers were not included, and changes were made to the grade, number and spacing of bolts. These are variations, and while some are not necessarily errors, all of these affected the bearing behaviour and required consideration in the failure assessment. There were also variations noted in the bearing design relative to OPSS specifications, as outlined in Section 6.

8.4 BOLT INSTALLATION

Observations on bolt dimensions and installation were provided in Section 4.4. Implications are described below. The review of the documents, construction records and discussions with site staff confirm that the 40 flange bolts in each of the two west abutment bearings were not pre-tensioned (tightened) during bearing installation. The bolts supplied were too long and did not have sufficient thread length to allow
tensioning without installing washers or shim plates. Discussions on site with project representatives from the Ministry revealed that the bolts of the centre-west bearing required a ¼ to ½ turn of the nut to reinstate tension. It is expected a similar condition existed in bolts of the failed north-west bearing. In addition, flat washers were installed during bearing installation, and a permanent solution comprising galvanized shim plates was proposed and accepted but had not been installed. Bevelled washers would still have been necessary as part of this retrofit. Five washers were provided on each bolt, although the shoe plate bevel required that an increasing number of washers were needed to provide consistent nut purchase onto threads on all of the bolts. Figure 4-7 showed also that nut purchase onto the bolts did not vary in the uniform manner expected, and as such the bolts were not snug-tightened or pre-tensioned consistently along the length of the bolt group. Axial tensions within the bolts would have varied significantly contributing to unequal bolt forces, fewer bolts contributing to uplift resistance, and affecting flexural effects within the shoe plate and girder flange.

The lack of bolt pre-tension also meant that there was inadequate ‘clamping’ or pre-tensioning force between the plates in the connection. Properly pre-tensioned bolts experience tensile strains close to or beyond their initial yield strain. As long as the bolt pre-strain is not exceeded by applied force or deformation demands then the strains in the bolts remain nearly constant, and low-cycle fatigue failure does not occur. In addition, the lack of pre-tensioning implies that initial forces and strains in various bolts are neither uniform or predictable. As such many non-linear analysis scenarios would be needed to characterize the patterns of progression of bolt fracture. The lack of pre-tension also reduced shear friction resistance at the shoe plate to flange plate interface as a result of bolt elongations. This introduced the potential for bending strains into the bolts.

The flat washers caused initial, internal bolt force eccentricity and bending. Owing to the high bolt axial tensions, this would have caused bolts to yield and rotate immediately below the nuts causing internal bolt force eccentricities to reduce. This would have subsequently allowed for alternating flexural tensions and symmetric crack propagation as discussed in Section 8.7. As such, the flat washer profiles do not appear to be a primary factor in the bolt group failure overall. However, some or all of these factors would have contributed to either a decision or an omission to tension the 40 (ASTM-A490) bolts in each west bearing.

While the flat washer profile is not assessed as a primary fracture in the failure, the lack of bolt pre-tension on installation was found to have contributed materially to the failure of the north-west bearing in a number of ways. These include:

- Non-uniform bolt forces and hence reduced load sharing among bolts.
- Lack of clamping pressure which allowed repeated, high bolt axial strains to occur.
- Lack of friction between the shoe plate and girder flange which allowed shearing across this interface, and consequently created a condition in which bolt bending could occur.

8.5 BEARING INSTALLATION

Sections 8.3 and 8.4 discussed important bearing attributes and bolt installation factors that influenced the behaviour of the bearing assembly overall. Importantly, the site installation of the west abutment bearings
also affected the behaviour of the bearing and contributed to the conditions allowing failure to occur. Factual site observations on the bearing installation were included in Section 4.3 – *Installed Bearing Condition*. As part of our review of documentation related to the bearings in Section 6.2 – *Bearing Installation and Construction Tolerances*, important aspects of the in situ bearing installation were also described. This section integrates these factors and their contribution to the bearing behaviour.

The bearing assemblies were installed out-of-parallel relative to the shoe plates and the abutments. The rotations induced in the north-west bearing assembly from these out-of-parallel dimensions, as seen in Figure 4-4, were 0.17° transversely (corresponding to an out-of-parallel condition of 3 mm) and 0.40° longitudinally (out-of-parallel 7mm). In combination with the rigidity of the bearing, i.e. its inability to freely accommodate rotations while in uplift, these imposed rotations caused bolt forces to redistribute within the bolt group. The resultant of the 40 bolt forces would have been eccentric to the bearing center. Some bolt forces therefore increased and load sharing among the bolts was significantly less uniform. This increase in bolt forces was significant when combined with the lack of bolt pre-tensioning and with the passage of heavy trucks. Similar but larger out-of-parallel dimensions were recorded in the center-west bearing.

This shifting of the uplift force resultant, and the re-distribution of forces within bolts, occurred in both primary directions. The distribution of forces between the contact pressures on the upper and lower guide bars was also significantly affected. Transverse rotations caused more of the uplift force to be resisted on one guide bar more than the other. The longitudinal rotations lead to a gradient of contact pressures along each guide bar, with higher pressures at one end. Both of these also affected the contact pressures between the PTFE (Teflon) strips on the upper and lower guide bars. For the north-west bearing, contact pressures would have been highest at the southwest corner of the assembly as discussed in Section 4.3 and as seen in the photographs of Figure 4-5. Those photos show the bearing in an unloaded condition following failure. While in service, these high local pressures caused local damage to the PTFE, also as seen in Figure 4-5. Under these conditions, the co-efficient of friction of the PTFE would have increased from the very low values intended for proper bearing function. An increase in the friction between the PTFE layers of the guide bars was necessary to allow significant shearing forces to be generated in the flange connection bolts, which was the key factor in the two-sided fracture propagation seen in these bolts. This out-of-parallel installation therefore is assessed as contributing materially to the fracture of the bolts connecting the bearing assembly to the girder flange. Additional discussion on the friction, bearing demands, sequence and interaction of this and other factors is summarized in Section 8.7.

### 8.6 STRUCTURAL DEMANDS ON BEARINGS

The global FE analysis of the bridge confirms that the uplift force on the northwest bearing before the failure occurred was at or near 1,720 kN, which was in general agreement with the contract drawings for this bearing and condition. At that time, the uplift on the north-west bearing was significantly higher than the uplift on the centre-west bearing. It is noted that the centre-west bearing will have an uplift force significantly higher than the northwest bearing in the final, four-lane finished condition. It was this larger design force that governed the bearing design requirements, and the north-west bearing could have been designed for a lower uplift force, if refined accordingly. The fact that the north-west bearing failed at demands far lower than its specified, or apparent capacity, demonstrates the significant degree of structural inadequacy of the bearing components.
Analyses indicate that bolts on the north-west bearing would have yielded under demands from legal highway trucks which place additional uplift and rotational demands on the west bearings. The analyses undertaken shows that these demands are sufficient to cause axial plastic strains in the bolts. 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 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.

In conjunction to the analysis of the dead and live load demands, wind effects on the structure were evaluated and found to increase uplift forces by approximately 10% of the reaction caused by bridge’s 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 during the short period that the west bearings were installed.

Thermal effects on the structure were also investigated for their effect on relative demands placed on the north-west bearing. It was found that cold temperature effects, specifically relative changes in cable temperatures increased the uplift forces by approximately 10% of the reaction caused by bridge self-weight. This load would have occurred simultaneously with the effects from truck traffic and while this would have accelerated the failure of the bolt group it would have not materially changed the failure mechanism.

Charpy impact testing on the bolt steel was conducted at -27.9°C Celsius and returned minimum average energy values (J) meeting all code minima required by CHBDC. This suggests that significant cold-temperature embrittlement of bolts would not be expected. Failure surfaces were seen to be ductile, which directly supports this conclusion.

8.7 BOLT FRACTURE AND CONTRIBUTING FACTORS

Shoe plate to flange bolts failed under high tensions with cyclic, alternating tensions. Accompanying several bolt fractures was clear evidence of striations propagating from both sides towards the bolt centers. This pattern indicates that alternating flexural effects were present. Discussions with the lead NRC investigator confirmed that direct bending effects in the bolts would be necessary for this pattern of striations to occur. Therefore, a mechanism to explain this final fracture behaviour is necessary and is provided below. The following discussion combines and extends observations and partial behaviour mechanisms descriptions by NRC, MTO and AE engineers.

Fundamentally, the amplified bolt forces from shoe plate flexure, prying and eccentricities of the bolt group resultant reactions (arising from the bearing’s inability to rotate freely in uplift) as well as the lack of bolt pretension, were fundamental to the tie-down bolt failures. The bearing assembly, connections, interface elements and installation were materially deficient as evidenced in this report. However, without additional effects, the bolt fractures and bearing failure would likely have been delayed or prevented and the two-sided fracture patterns could not have occurred.

Several observations are important to the bolt fracture patterns seen, including:

- The evidence of very high bolt tensions from the shoe plate permanent yield deformations.
Figure 5-3 shows that the two-sided crack propagation patterns generally orient transversely to the roadway, suggesting longitudinal alternating horizontal bending demands. Our independent examination of the failed bolts and each bolt's fracture patterns allowed us to locate or confirm the location of most of the bolts onto the bolt group ‘template’ as seen in Figure 5-3.

Figure 5-3 illustrates two or more patterns of behaviour, with patterns changing as bolt failures progressed.

Site observations and measurements of out-of-parallel bearing guide bars and significant local damage to the PTFE, in particular at the southwest corner of the failed bearing. The damage observed was seen following failure, with the uplift loads removed. The PTFE was shown to be highly and locally over-stressed and damaged. In service under uplift the PTFE would have been crushed, with less capacity to allow for a smooth, low-friction sliding behaviour as was necessary.

Bolts supplied were measured and confirmed as too long with thread lengths shorter than standards for this length.

At least two bolts in the outer line of the southeast quadrant were observed to be loose, as seen in fewer than expected exposed threads.

Rub marks were seen on the bolt shakes which clearly show shear movements at the shoe plate interface.

A mechanism and sequence of bolt failures that is consistent with the observations, forensic investigations and analyses is provided below:

Bolts in the outer two lines are heavily loaded. Some of these bolts may have been snug tight; some in these lines were not. The bolts that were tightened, and also having the highest tension from prying, yielded in tension under dead loads and imposed deformations. Additional demands from truck loads have been shown to be sufficient to lead to tensions beyond yield in outer bolts. These highly loaded bolts would have reached the plastic strain plateau of the stress-strain curve.

Owing to the use of flat washers and the bearing surface slope at the nuts, these bolts would also have yielded in flexure, in as little as one load cycle, which would bring the nuts ‘flat’ onto the washers. This yielding was from the applied eccentric bolt tension at the 0.8% out-of-parallel of the flat washers, but also from the out-of-parallel of the bearing installation. Following this yield event, the underside of the nuts sit flat and the bolts tend towards uniaxial, concentric tension. The initial local eccentricities are reduced or eliminated by plastic redistribution.

With these bolts in this condition, they are in a ‘fix-fix’ condition for any added flexure from horizontal loads. Because they are on or near the yield plateau, any small additional cyclic, horizontal shear forces can cause bolt flexure in double-curvature and additional plastic strains on one side of the bolt thread or the other.

With several bolts in this condition, in fix-fix flexure and high tension (tension stiffened), they are relatively stiff and resist horizontal loads with little horizontal deformation. This shear flow load path at the interface between the shoe plate and flange plate would experience these effects if friction between the shoe plate and flange plate was overcome. The shear-oriented movements could be very small, and would not necessarily produce ‘scratch’ patterns on either of the sliding plates. However, clear evidence of sliding movements was seen in polishing marks on the bolt shafts.
- Bolts in the inner lines were shown by non-linear analyses to be under much lower tensions in this phase, and as such would be ‘loose’ and not participating in resisting shear deformations. Some may have experienced shear flows through a different mechanism, but which would not contribute to bolt cracking, nor prevent shears in the bolts with the tension-stiffening.

- Horizontal shear forces in the bolts would be small when the PTFE guides were smooth and functioning with low friction. However, with local crushing of the PTFE the friction force would increase significantly, increasing bolt shear forces. The out-of-parallel bearing installation, and observations of damage to the PTFE in the southwest corner of the bearing, implies that the frictional force would be greater than intended, and the bearing would have been effectively locked (frozen) at the time.

- A portion of the prying effects arise from internal contact pressures between the shoe and sole plates. Analyses show that flexural deformation of the shoe plate was a more significant contributor to bolt force increases. Analyses in Section 7 indicate that under service load conditions, internal contact forces of approximately 510 kN under service conditions may occur for a loose bolt condition. For a friction co-efficient between the shoe and sole plates of approximately 0.2 (as an indicative value) then a longitudinal shear force as low as 0.2 x 500 = ~100 kN would be sufficient to overcome this friction and allow shear forces to be generated in the tension-stiffened bolts. A tie-down reaction of at least 1720 kN (dead load only, greater under DL+ LL) would be experienced by the PTFE surfaces. A co-efficient of friction as low as 6% in the damaged PTFE would be sufficient to allow bolt shears to occur. Frictional forces between stainless steel and PTFE could be in the range of 5% but never more than 10% for filled, undamaged PTFE. Horizontal longitudinal displacements at the northwest bearing under truck loads cycle between approximately -1 mm and + 2 mm. This would increase further if secondary bearing displacements from girder rotations and the eccentricity of the bearing from girder’s neutral axis were added. Even at these small, conservatively derived displacements, significant shear forces would develop in the bolts. A supporting analysis with the north-west bearing assumed to be ‘locked’ indicates bearing shear forces ranging from approximately -590 kN to +820 kN caused by the passage of a heavy, legal truck.

- These horizontal forces (or displacements) would be sufficient to generate shears, moments, plastic strains and low cycle fatigue crack propagation in the bolts from alternating directions.

- In addition to truck loads, temperature effects cause horizontal forces (with bearings locked) as the deck length changes under thermal changes. These effects would occur both alone and in combinations with truck passages.

- These alternating strains in the tension-stiffened bolts cause cyclic horizontal shears consistent with cracks propagating from two sides until the eventual ductile tensile fracture occurs.

- Failure of each bolt in turn causes uplift loads to shed to other bolts (some in outer lines, some inner). Bolt crack propagations don’t increase axial strains in adjacent bolts until bolt failure.

- As the outer bolts fracture, the forces shift to the inner bolts. Prying becomes much reduced because the system has changed, but the share of the total uplift load per bolt increases.

The crack patterns seen in Figure 5-3 show that the inner bolts fail in tension but predominantly with one-sided crack propagations. This indicates that the horizontal shears and bolt bending are reduced. A one-sided crack propagation under high axial tension and incremental plastic strains from increased loads,
imposed deformations and local plate deformations was sufficient to cause the remaining bolts to fail in this manner.

The source of the high cyclic loads felt by the bearing in service may be explained by the large number of heavy vehicles which have travelled on the bridge since the day it was opened to traffic. Approximately 90 overweight vehicles with weights in excess of 60 tons possibly used the bridge up until it failed on January 10th, 2016. This number coincides well with the approximate stress cycles to failure observed on the fracture surfaces of the bolts. Additional shearing demands in the bolts would have also occurred from thermal length change of the deck and girders.
9 Conclusion

This report outlines both the observations and analysis undertaken by Associated Engineering (Ont.) Ltd to outline the key factors that contributed to the failure. These key factors resulted in the failure of the tie-down bearing:

- The failure of the tie-down bearing was caused by the complete fracture under cyclic loading of all 40 A490 bolts connecting the north-west bearing assembly to the main girder of the bridge deck. Individual bolts within the group failed at different times over a period of several weeks.
- The fracture mechanism was a progressive low-cycle fatigue caused by high bolt forces in tension arising from bridge uplift at the west abutment, coupled with, in some cases, cyclic bending of the bolts. The crack propagation patterns on fracture surfaces indicate that bolt bending occurred and was strongly oriented in a longitudinal direction. All bolts showed evidence of low-cycle fatigue.
- The bolts failed gradually while experiencing repeated cycles of axial and bending effects from heavy but legal truck traffic. Temperature effects in the bridge deck and cables also contributed, to a minor degree, to bolt demands.
- The high tensile forces and strains in the outer two lines of bolts were caused by flexibility of the shoe plate and related prying effects between the shoe plate and the girder flange. Yielding of the shoe plate from the high bolt forces was evident in the permanent bend of the shoe plate.
- The stiffness and lack of rotation capacity of the tie-down bearings, when acting in uplift, contributed significantly to increased bolt forces and strains.
- The out-of-parallel installation of the bearing, coupled with the bearing’s inability to freely rotate, contributed to a non-uniform bolt force distribution and to local damage and increased friction in the PTFE (teflon) guides within the bearing guide bars.
- The flange connection bolts were not pre-tensioned (tightly) during bearing installation. This lack of pre-tensioning, in combination with effects noted above, allowed the shoe plate and girder flange to slide against each other, which in turn allowed horizontal shear forces and bending to occur in the bolts.
- The bolt material met all specifications and bolts were not embrittled by either the coating or by cold temperatures.

The north-west bearing failed on January 10, 2016. The centre-west bearing did not fail and so limited damage to the bridge which provided the opportunity for MTO to implement emergency repairs and re-instate single lane traffic on the Trans Canada Highway within a day. Both bearings were subsequently bypassed with two sets of redundant, temporary tie-down assemblies to allow the bridge to re-open to two lanes of normal unrestricted traffic.
10 Closure

This report was prepared for the Ministry of Transportation Assistant Deputy Minister's Office to investigate and evaluate the northwest bearing failure that occurred at the Nipigon Bridge.

The services provided by Associated Engineering (Ont.) Ltd. in the preparation of this report were conducted in a manner consistent with the level of skill ordinarily exercised by members of the profession currently practicing under similar conditions. No other warranty expressed or implied is made.

Respectfully submitted,
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