

Johnson Street Bridge Condition Assessment Report



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EXECUTIVE SUMMARY

Introduction

The Johnson Street Bridge was opened to traffic 85 years ago and, although maintained well over the years, is nearing the end of its design life. It was designed at a time when earthquake engineering was not well understood and is therefore vulnerable to seismic loads.

This study was carried out to establish scope and costs associated with repair and replacement options for the bridge and to weigh which of the two options is the best course of action. The objectives of the study were to:

- Establish the existing condition of the structural, electrical and mechanical components of the bridge;
- Establish the seismic vulnerability of the bridge in its current configuration;
- Develop conceptual seismic retrofit solutions to allow the bridge to function in a seismic event in accordance with the Canadian Highway Bridge Design Code;
- Establish costs associated with seismic retrofits;
- Establish costs associated with rehabilitating the bridge to provide an additional 40 years of service;
- Establish costs for a replacement structure; and
- Establish a repair/replacement strategy based on life cycle costing.

Bridge Connectivity and Context

The Johnson Street Bridge connects the east and west shores of Victoria's inner harbour at the interface with the outer harbour. Access across the inner harbour is also provided by the Point Ellice Bridge which represents a 3 km detour if the Johnson Street Bridge is closed to roadway traffic.

Given the bridge's location between the inner and outer harbours, there is significant marine traffic at the site in the form of barges, motor boats and sailboats.

The Johnson Street bridge is considered to be of historic interest although it is not a registered historic structure. The interest stems from the following characteristics:

- It is one of few remaining bascule bridges designed by Joseph Strauss, the designer of the Golden Gate Bridge.
- The bridge has been a local icon and part of the Victoria harbour's skyline since 1924. It is locally known as "Big Blue".

Bridge Condition

Based on a comprehensive inspection of the bridge's structural, mechanical and electrical systems, the following were noted:

- Corrosion is pervasive and the coating system has failed. Pack rust is forming between plates of built-up members;
- The mechanical system is in relatively good condition but needs specific repairs. Many of the mechanical elements are obsolete and it may be difficult to find replacement parts;
- The motor brake system should be replaced; and
- The electrical system is obsolete and should be replaced to avoid unscheduled bridge closures.

Seismic Vulnerability and Retrofit

Analysis of the bridge in its existing configuration shows that the bridge will experience failure of its foundations and collapse of the counterweight towers under loads from a seismic event with a 35% probability of exceedance in 50 years. This corresponds to a peak ground acceleration of about $PGA = 0.18\text{ g}$ (18% of gravity acceleration). This represents an earthquake that is considerably less than any design earthquake specified in the design codes. This is also larger than any of the earthquakes recorded in Victoria over the past 85 years.

Therefore, the bridge requires a seismic retrofit to meet all the requirements of the Canadian Highway Bridge Design Code (CHBDC). Given that the existing timber piles are weak in a seismic event, the retrofit should include underpinning of the bridge with new piles. This will allow the natural frequency of the bridge to be modified and effectively isolate the bridge superstructure from the existing foundations. Additional seismic retrofit measures are noted below.

OPTION 1: Repair Bridge

Based on a comprehensive inspection of the structural, electrical and mechanical elements of the Johnson Street Bridge as well as a preliminary review of the bridge's seismic vulnerability, a repair program has been developed to extend the life of the existing structure by approximately 40 years.

The repair program recommended in this regard consists of the following scope:

- Repair of corrosion-damaged steel;
- Complete re-coating of the bridge;
- Various repairs to the bridge's mechanical system;

- Replacement of the bridge's electrical system;
- Seismic retrofit.

As noted, the bridge is vulnerable to major damage that is likely not repairable and could be life threatening under earthquakes with a 35% chance of exceedance in 50 years. This represents an earthquake with accelerations that are less than any design earthquake. This happens to be greater than any earthquake the bridge has experienced in its lifetime so far, which would explain why the bridge has not sustained any seismic damage so far. A seismic retrofit of the bridge is recommended to allow the bridge to perform in any of the three recommended categories: 'lifeline', 'emergency route', or 'other' as defined in the CHBDC. This retrofit would include the following scope:

- Installation of energy dissipation bracing and strengthening of selected members;
- Installation of new piles/substructure to relieve the existing foundations/substructure; and
- Replacement of lacing with cover plates in built-up members;
- Installation of bracing to prevent pounding between the highway and roadway bridges;
- Extension of bearing seats or provision of restrainers for the approach spans and at the rest pier end of the bascule span;
- Provision of lateral restraint at the rest pier for the bascule span;
- Provision of hold-down devices at the rest pier for the bascule span;
- Improvement of shear capacity of cross beams; and
- Modification of gusset plates to ensure ductile connections between truss members.

The estimated cost of the rehabilitation and seismic retrofit works is \$23.6M including engineering and contingencies as defined in **Sections 5** and **6**. It has been assumed that the repair option would be implemented within 3 years to minimize risks associated with the existing bridge's seismic vulnerability.

OPTION 2: Replacement of Bridge

Replacement of the Johnson Street Bridge would require consideration of the following:

- The Bridge is considered an icon in the Victoria area and as such, consideration would need to be given to replacement with an equally remarkable structure;
- Staging of the replacement would likely require closure of the bridge

to vehicles for a period of time;

- Vertical clearance under the bridge cannot be increased significantly and as such a new movable bridge would be required to ensure continued access for marine traffic;
- The new bridge would need to carry 2 lanes of traffic, the commuter train and improved sidewalk capacity. As such, the structure would be in the order of 20 m wide; and
- A total bridge length of about 120 m would be required.

A replacement cost of \$35.26M was estimated including engineering and contingencies. If replacement was to be undertaken, it has been assumed that this would occur within 3 years in order to mitigate risks associated with the seismic vulnerability of the existing bridge.

Comparison of Repair and Replacement Options

A 40 year maintenance program with corresponding costs was established for both the repair and replacement options in order to allow a life cycle comparison. Using a discount rate of 2.1%, as specified by the City, lower initial costs are expected for the repair option while there is no significant difference between the total life cycle costs of the two options and the 40 year period considered in the analysis.

Recommended Approach

Based on the findings of this study, either a repair or a replacement option could be justified from a cost perspective. To address the seismic vulnerability of the existing bridge given that it is heavily trafficked and located in the most seismically active city in Canada, one of these options is necessary, in our opinion. The do nothing option is not acceptable. In this report we have suggested that this vulnerability should be addressed within 2 to 3 years by implementing a seismic retrofit or by replacing the bridge.

In order to select either the repair or replacement approach, value needs to be placed on elements of the project that are beyond the scope of this study and are not associated with structural engineering.

Therefore, don't assign costs to all these things. In particular the following could be considered and valued:

- Benefits derived from the improved access provided by a new bridge;
- Value associated with preserving the historical elements of the existing bridge; and
- Aesthetic value of a new landmark bridge.

1. STUDY OBJECTIVES

1.1 Introduction

The Johnson Street Bridge is a single leaf, heel trunnion bascule bridge in which one end rises while a counter weight lowers on the opposite end. The bridge comprises two separate bascules – a railway bridge and a roadway bridge – which can operate independently.

The bridge, shown below in **Figure 1.1**, was constructed in 1922 and was opened in January 1924 at a cost of around \$918,000. The bridge superstructure was designed by Joseph Strauss, the designer of the Golden Gate Bridge, while the substructure was designed by the City's engineering department. The bridge provides a crossing for pedestrians, cyclists, a commuter train and over 30,000 vehicles daily. It is a prominent landmark of the Victoria Inner Harbour.



Figure 1.1 – Johnson Street Bridge

Delcan Corporation was retained by the City of Victoria to carry out a comprehensive assessment of the Johnson Street Bridge in order to investigate strategies and costs associated with maintaining the crossing with particular attention to the structure's seismic vulnerability, its electrical / mechanical equipment, questions surrounding its foundation configuration/condition and the corrosion of the steelwork.

1.2 Study Objectives

The objectives of the study were to:

- Establish the existing condition of the structural, electrical and mechanical components of the bridge;
- Establish the seismic vulnerability of the bridge in its current configuration;
- Develop conceptual seismic retrofit solutions to allow the bridge to function in a seismic event in accordance with the Canadian Highway Bridge Design Code;
- Establish costs associated with seismic retrofits;
- Establish costs associated with rehabilitating the bridge to provide an additional 40 years of service;
- Establish costs for a replacement structure; and
- Establish a repair/replacement strategy based on life cycle costing.

2. BRIDGE CONNECTIVITY AND CONTEXT

2.1 Bridge Site

The Johnson Street Bridge is located between the inner and outer harbours of Victoria, connecting the older part of the City on the west side with the more recently developed and traditionally industrial west side of the harbour. Access across the inner harbour is also provided by the Point Ellice Bridge which represents a 3 km detour if the Johnson Street Bridge is closed to roadway traffic.

Given the bridge's location between the inner and outer harbours, there is significant vessel traffic at the site in the form of barges, motor boats and sailboats. The topography on either end of the bridge is constrained by development and as such building approach embankments to service a high level fixed span appears to be undesirable. As such, a movable bridge is required at the site and needs to be operational at all times to allow unimpeded access to vessel traffic and emergency response vehicles. **Figure 2.1** illustrates the bridge location.

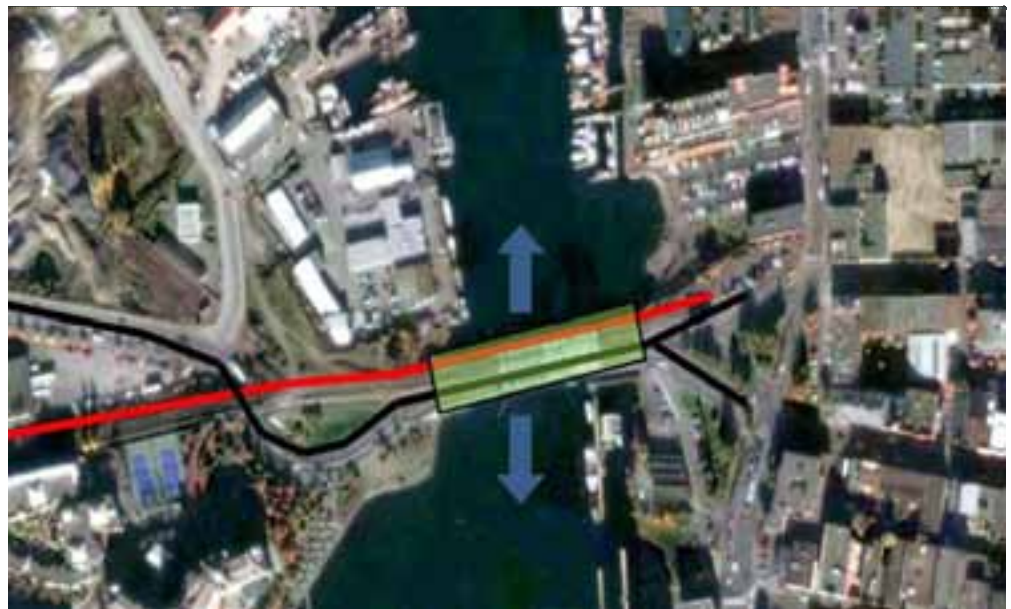


Figure 2.1 - Location of Johnson Street Bridge

[Note: red line indicates the commuter rail track, black line indicates Johnson Street, north is upwards]

2.2 Historical Characteristics of the Johnson Street Bridge

The Johnson Street bridge is considered to be of historic interest although it is not a listed historic structure. The interest stems from the following characteristics:

- It is one of few remaining bascule bridges designed by Joseph Strauss, the designer of the Golden Gate Bridge.
- The bridge has been a local icon and part of the Victoria harbour's skyline since 1924. It is locally known as "Big Blue".

3. EXISTING CONFIGURATION AND CONDITION

3.1 Bridge Configuration

3.1.1 Structural

The arrangement of the Johnson Street Bridge is shown in **Figure 3.1**.

The bridge substructure consists of an east and west abutment as well as three piers. As shown in **Figure 3.1**, piers have been numbered from west to east as Piers 1, 2 and 3. Both abutments and Pier 1 are thought to be founded on bedrock whereas Piers 2 and 3 are supported on timber piles driven through the silty overburden to rock.

The bridge superstructure consists of fixed east and west approach spans and a central, moveable bascule span. Two piers (Piers 2 and 3) are provided at the east end of the bascule span to provide support to the east end of the bascule span as well as to support the counterweight tower and the electrical/mechanical equipment. The span between Piers 2 and 3 is referred to in this document as the Counterweight span.

In plan the bridge consists of two bridges – a north bridge that services the commuter rail and a south bridge that provides access for vehicles, pedestrians and cyclists. As such there are two bascule bridges that can be operated independently.

As shown in **Figure 3.1** key dimensions of the Johnson Street Bridge are:

- Bridge length: 115.15 m
- West approach span: 22.25 m
- Bascule span: 45.7 m
- Counterweight span (Pier 2 to Pier 3): 13.7 m
- East approach span: 33.5 m



Abutments, Piers and Foundations

Abutments and piers were designed and built by the City's Engineering Department. The West and East abutments and Pier 1 bear on rock whereas Piers 2 and 3 are supported on timber piles driven through soft soils to bear on rock.

Existing information indicates that below the tide level, the piers were constructed inside a timber cofferdam. In one pier, this cofferdam was driven to irregular shallow bedrock and difficulty arose with sealing. In this cofferdam and possibly others, concrete was placed under water. Segregation of aggregate occurred and there are extensive bands where aggregate loss has occurred.

Previous repairs have involved installing a concrete jacket to a section of Pier 1 and stacked placement of cement-filled bags. No underwater repairs have been undertaken although periodic underwater surveys have been made.

West and East Approach Spans

Both approaches consist of riveted steel floor beam and stringer configurations. A reinforced concrete deck is used for the roadway while a timber deck is used for the railway. The existing concrete roadway deck was constructed in 1999 as part of a rehabilitation program that also included removal and replacement of secondary cross beams. This was carried out by Formula Pile and Bridge Co.

Bascule Span

The bascule spans consist of Warren trusses with members fabricated using riveted, built-up sections. The original deck of the south bascule span (roadway) was constructed of wooden timbers. Besides being slippery in wet weather, the timber absorbed water and became heavier which placed excessive loads on the opening machinery. In 1966, open steel grid decking, of constant weight, replaced the road deck timbers.

In 1995, abnormally high temperatures caused the steel decking to expand to the point that the bridge would not open or close properly. This necessitated the removal of about 25mm of decking.

The removal, replacement and strengthening of corroded steelwork on the bascule structure was carried out in 1999. Formula Pile and Bridge Co. also completed this work.

Counterweights

The counterweight block on the highway span is a hollow concrete structure and contains a number of smaller concrete weights that collectively weigh over

780-tons. It balances the 350-ton opening span. Two large racks, each driven by a 75 horsepower electric motor, move the linkage.

3.1.2 Electrical/Mechanical System Configuration

The Johnson Street Bridge has provided over 80 years of satisfactory service. For the most part, the mechanical machinery dates to original construction and is similar for both bridges. The only significant exception is the span drive brakes on the vehicular bridge which have been retrofitted with an actuating device that we have not seen used on movable bridges. Overall the machinery was found reasonably well maintained with light wear and only isolated areas of corrosion on the machinery located outside of the machinery room. This machinery is maintenance-intensive and is not adequately guarded to protect maintenance and inspection personnel from injury.

Span Drive

The span drive for each bridge is normally driven by two electric motors. The motors provide power to a gear train that consists of open spur and bevel gears, sleeve type pillow block bearings, and shafts. A differential is provided at the center of the drive train to allow for equal load sharing of the operating struts which are located on opposite sides of the bridge. The operating struts support a rack (straight gear) that is driven by the final pinion in the drive train. The operating struts connect to the structure at the 2nd link pins. The drive machinery pulls the strut back through a guide assembly and causes the span to rotate about the main trunnion bearing.

Each of the span drives is provided with two motor brakes and two emergency brakes. The motor brakes are located on the non-driven end of the motor and the machinery brakes are mounted on a lower speed shaft in the drivetrain. All of the brakes are located on the high speed end of the differential which is not desirable because a failure in any component on the low speed side of the differential could result in loss of span control.

Each bridge is equipped with an auxiliary drive. The auxiliary drive is powered by an internal combustion engine that drives a gear train that engages with the normal drive via a reversing clutch.

A schematic of the normal and auxiliary span drives with component designations for each bridge is presented in Figures 1 and 2, Appendix E. The component designations are consistent with those found on the schematics in the machinery houses.

3.2 Bridge Condition

3.2.1 Inspection Procedure

A comprehensive visual inspection on the bridge was carried out to assess and establish the current condition and seismic risk profile. The inspection results were documented using digital photography and inspections sheets, which are in **Appendices A** and **B**. Each component of the steel spans was evaluated for coating condition, corrosion, section loss, fatigue details, and overall structural condition.

The structural, mechanical and electrical inspections carried out were to provide information and furthermore help present recommendations for either rehabilitation or replacement of the Bridge.

The inspection team for the bridge included:

Structural – Delcan Corporation

- Stan Reimer, P.Eng. – Inspection Team Manager;
- Joost Meyboom, P.Eng. (substructures);
- Dawn Taylor E.I.T.; and
- Rebecca Huang, E.I.T.

Mechanical and Electrical – Stafford Bandlow Engineering Inc.

- Paul Bandlow P.E – Mechanical; and
- Rod Harris P.E – Electrical.

A 2-man crew to operate the below-deck traveler system were provided by the City of Victoria and led by Hector Furtado.

Available information including previous inspection reports and drawings were reviewed prior to the inspection. Previous inspection reports included:

- Electrical/Mechanical Condition Report by Robert Freundlich & Associates in 1990;
- Structural Condition Report by Graeme & Murray Consultants in 1998;
- South Coast Diving Inspection Report by Stantec Consulting in 2004;
- A video documentation of previous underwater inspections was provided to allow us to gain an understanding of the extent of deterioration of the bridge substructure and foundations.

The bulk of the inspection work was carried out over a two (2) day period from June 10th to June 11th 2008. The inspection consisted of a comprehensive detailed visual inspection from shore, deck, available traveling gantries (traveler) and inspection walkways.

Inspection of the substructures was carried out during a 50 year low tide. A visual inspection was carried out from a boat as well as from under the west approach structures. Pier 1 was accessible in the dry during this low tide event.

All structural and non-structural components above ground and water were inspected. These components included concrete soffit, floor beams, girders, abutments, piers, bearings, structural steel members, built-up girders, bracing members, stairs and concrete decking. Conditions of other non-structural components such as hand-railings, road barriers, embankment and sidewalks were also assessed. Field and photographic records of the inspection are provided in **Appendix A** and **Appendix B**.

There are four (4) under-deck travelers on the bridges – three (3) on the road bridge located on the east, west and center of the bridge and one (1) on the rail bridge located in the centre span. Each traveler runs linearly beneath the deck. From the traveler platform, a visual inspection of all accessible areas was carried out. Areas that could not be visually inspected up close, due to limited access, were the railway bridge bearings at the east abutment, the main girders, and bracing of the railway bridge approach spans. The railway bridge approach spans girders and bracing were inspected from the west abutment and piers, through the open-deck ties and from the roadway bridge travelers.

A digital camera with voice recording capabilities was used to record all noticeable deficiencies. Each photographed picture was noted on the relevant bridge plan and/or elevation. All pictures are labeled in accordance with the locations and orientation on the bridge with a small description.

The inspections began above-deck on the east side of the railway bridge systematically moving west along the bridge inspecting vertical and diagonal members, connections, member plates, gussets, rivets and bolts. The same technique was carried out for the road bridge above-deck inspection. Following the completion of above-deck, inspection of the below-deck concrete soffit, girders, and floor beams on the road and rail bridge began. The traveler was utilized during this part of the inspection. Inspections started on the road bridge at the east side of the bridge and systematically moved west. The traveler system was only available beneath the railway bridge between piers 1 and 2.

3.2.2 Condition Rating Scheme

Standard inspections forms and condition rating, adopted from the format used by the BC Ministry of Transportation, were used. Each span, between substructure elements, was evaluated using the standard condition rating scheme. Inspection forms summarizing this work are provided in **Appendix B**.

3.2.3 Inspection Nomenclature

Original erection drawings indicate a member identification scheme with node numbers increasing from west to east. The 1998 Condition Report, by Graeme & Murray Consultants, introduced a panel numbering scheme in the opposite directions (increasing panel numbers from east to west). The orientation convention used for this report respects the original numbering scheme. Thus, the west end of the bridge is considered to be the beginning of the bridge and the numbering scheme given previously in **Figure 3.1** was used.

The numbering scheme for the floor system members was adopted from the original erection drawings.

3.3 Structural Condition

The results of the structural inspection are reported in this section under the sub-headings Substructures, Road Bridge and Rail Bridge. The inspections carried out on the road bridge focused on the deck beams, diagonals, verticals, lateral bracing, gusset plates and bearings. The rail bridge inspection focused on the diagonals, verticals, lateral bracing, gusset plates, bearings and the timber decking. However, limited access caused the east and west approaches to not be fully inspected up close.

3.3.1 Substructures

As noted above, the bridge substructures were inspected under a 50 year low tide condition. Generally the substructures are in reasonable condition given the structure's age although there has been considerable erosion of the concrete on Pier 2, see **Figure 3.3.1**.

Several substructures have minor cracks, see **Figure 3.3.2**. The substructures were not designed to withstand current earthquakes conditions. Substructures have minor cracking and efflorescence but almost no spalling. All bearing seats were inspected and found to be in good condition.



Figure 3.3.1 – Substructures at low tide (a) Pier 1 looking west (b) Pier 1 looking east (c) Pier 2 looking west (d) Pier 2 showing erosion of concrete on the south side



Figure 3.3.2 –Pier 3, West face showing minor cracking and efflorescence

It was noticed that the north east bearing on Pier 3 has reduced anchor bolt stiffeners, so it should be noted that thorough cleaning and a weld repair to replace the existing stiffeners should be carried out.

3.3.2 Road Bridge

Extensive member corrosion was observed in many areas along the bridge – especially in the floor beams and stringers. The joints along the bridge are in good condition and minimal water and debris is coming through. Corrosion and paint deterioration is visible throughout the bridge on most members. However, most of the bridge components were found to be in fair to good condition. Member corrosion was observed in many areas along the bottom flange of the stringers and floor beams of the road bridge shown below in **Figure 3.3.3**, **3.3.4** and **Appendix A**.

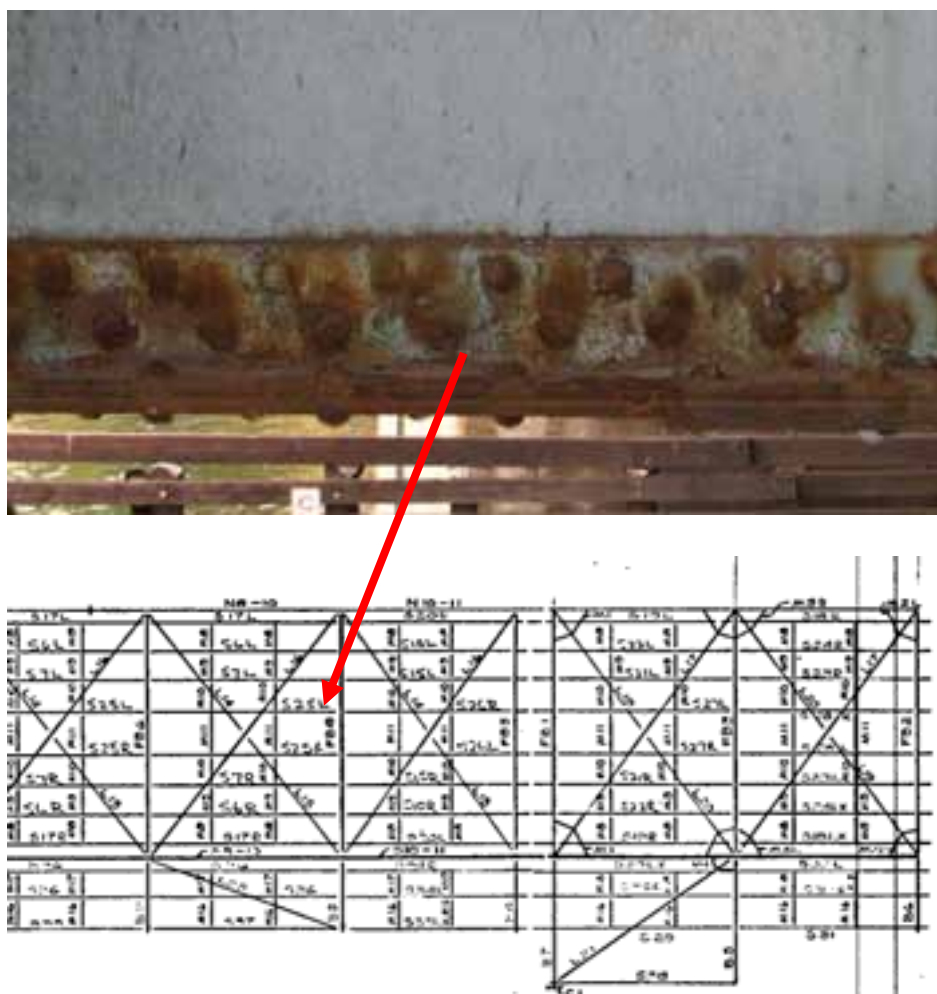


Figure 3.3.3 – Typical corrosion on bottom flange of bascule span (Member S25L)

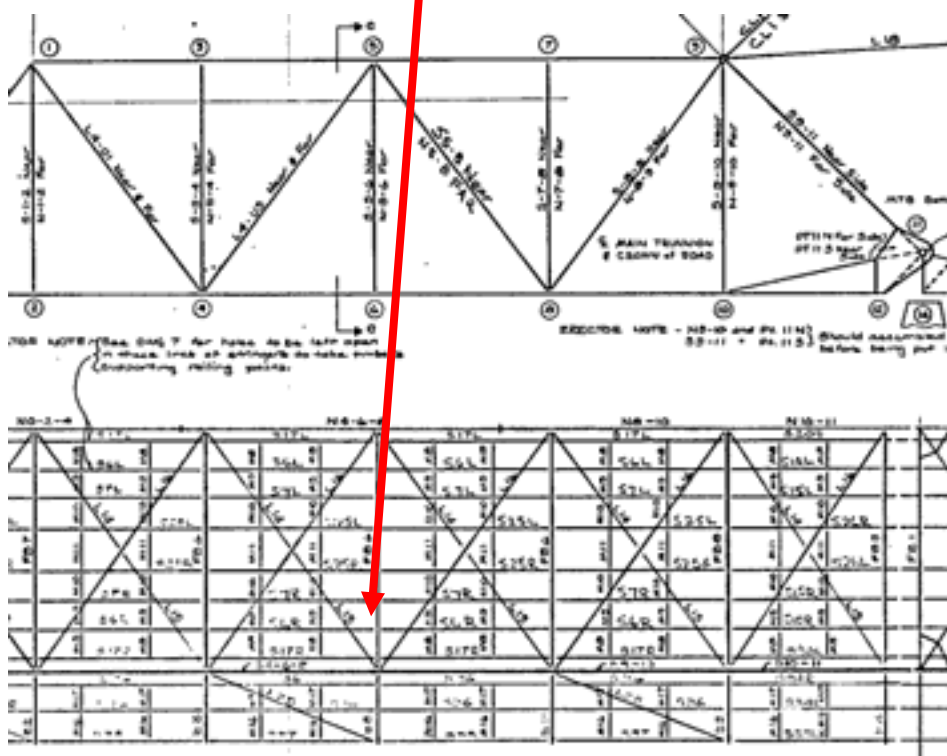


Figure 3.3.4 – Typical corrosion on bottom flange and gusset plate (Bracing Member L15 to Floor Beam FB6)

The structural integrity of the bridge appears to be intact and performing as originally intended with no signs of major distress. However, it was noticed that the paint coating in many areas along the bridge has failed or is beginning to fail. Other than the obvious corroded areas, the paint coating is peeling in areas on the top and bottom and interior surfaces of the diagonals, verticals and girders. Paint coating deterioration is estimated to cover approximately 30-35% of the bridge. Typical interior section paint deterioration is shown in **Figure 3.3.5** below.



Figure 3.3.5 – Typical condition inside built-up truss member

The vertical and diagonal truss members are in generally good condition. There are some localized areas where the steel coating has deteriorated and pack rust is forming between the members built-up plates. This can be seen in **Figure 3.6.4**. The pack rust, over time, can induce undesirable tensile stresses on the rivets and if the resultant stress becomes too high, rivets can break or tear through the parent material. Pack rust can typically not be eliminated although its rate of growth can be reduced.

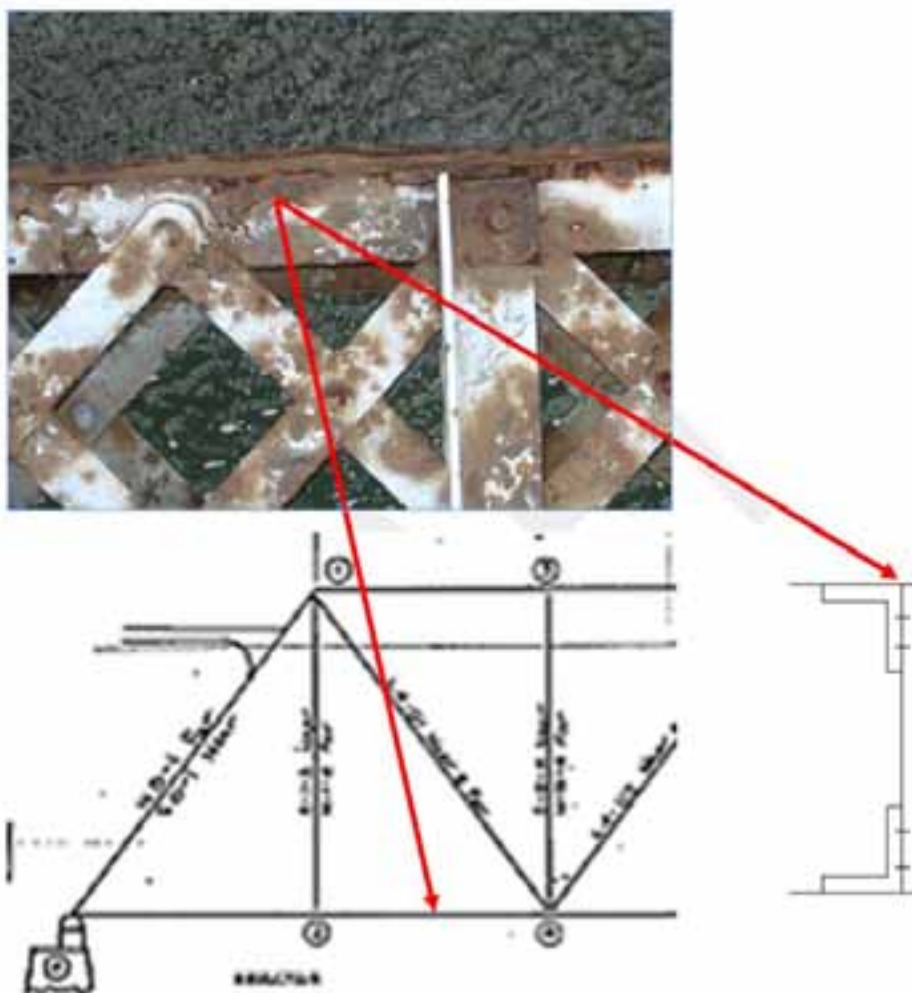


Figure 3.3.6 – Typical condition on top surface of laced member.

Members with 100% section loss were observed in areas where debris and moisture were trapped such as at gusset plates and bracing connections, see **Figures 3.3.7** and **3.3.8**. These affected areas are the result of severe corrosion due to paint loss and/or water accumulation on the member surfaces. It should be noted that this level of deterioration was found in localized areas and is not widespread on the bridge.



Figure 3.3.7 – 100% section loss on bearing stiffener, Northeast bearing, Span 1

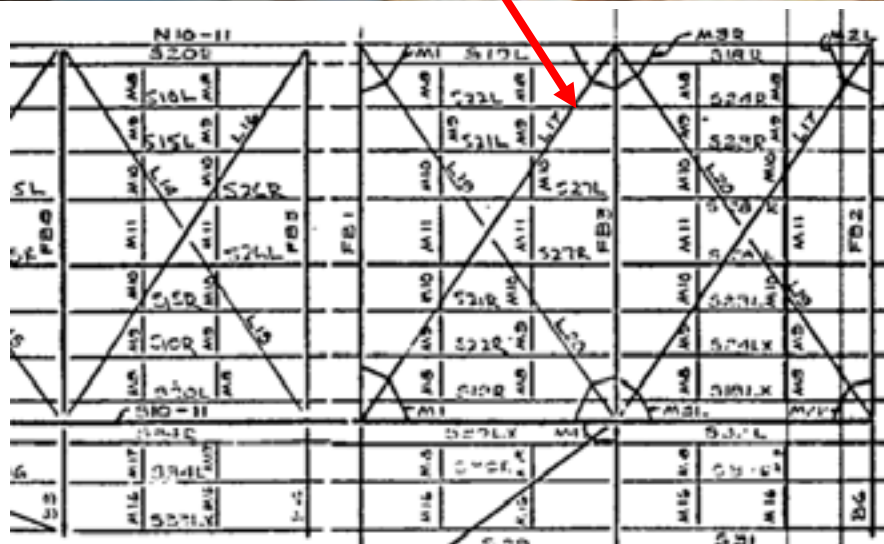


Figure 3.3.8 – 100% section loss, underside of lateral cross bracing, Span 1

Although there is only minor dirt and debris accumulation on the bridge bearing seats, significant corrosion of the bearing plates was observed as shown in **Figure 3.3.9**.



Figure 3.3.9 – Bearing Plate Deterioration

Corrosion and section loss in the deck beams for the road and pedestrian walkways were observed, see **Figure 3.3.10** and **Figure 3.3.11**. It appears that run-off from the deck passes through the open steel grid and gets trapped along the top flanges and the bottom flanges of the supporting beams. Consequently, extensive corrosion has developed on the flanges, webs and rivets on a majority of the beams underneath the lift span, see **Figure 3.3.12**. The webs on the beams in this span have also experienced up to 60% or more paint loss.

Figure 3.3.13 shows the aerial view of the lift span. There is minimal corrosion on upper lateral bracing. However, the horizontal gusset plates have extensive corrosion and should be cleaned or restored, as the paint has failed there.

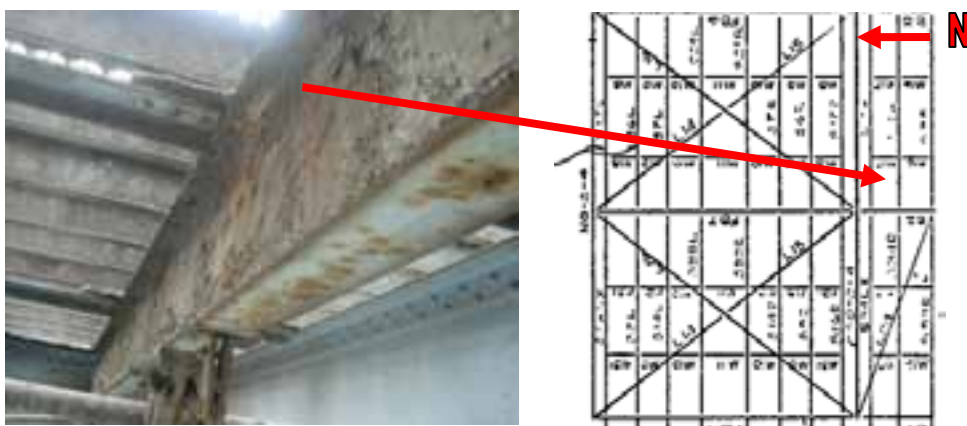


Figure 3.3.10 – North side of a typical floor beam



Figure 3.3.11 – Girder under pedestrian walkway, North face



Figure 3.3.12 – Typical corroded bottom flange and rivets on floorbeam



Figure 3.3.13 – Aerial view of lift span, (looking west)

3.3.2 Rail Bridge

Moderate to severe corrosion was observed in many areas along the bottom flanges of the girders and floor beams of the rail bridge as shown below in **Figure 3.3.14**. No major section loss, however, over any continuous areas was found on the bridge. The worst defect observed was a holed/reduced vertical in the south truss of the lift span, see **Figure 3.6.15**. There was at least one other vertical that also showed significant section loss (see **Appendix A**).

Section loss was also observed in the bottom chord of the lift span, see **Photos P065** and **P046** in **Appendix A**. Several horizontal gusset plates that have been perforated by corrosion are visible throughout the bridge and can be seen in **Photos P049, P051, P053, P058, P061, P064** and **P066** in **Appendix A**, and some anchor bolt stiffeners on one bearing, see **Photo P092** in **Appendix A**.

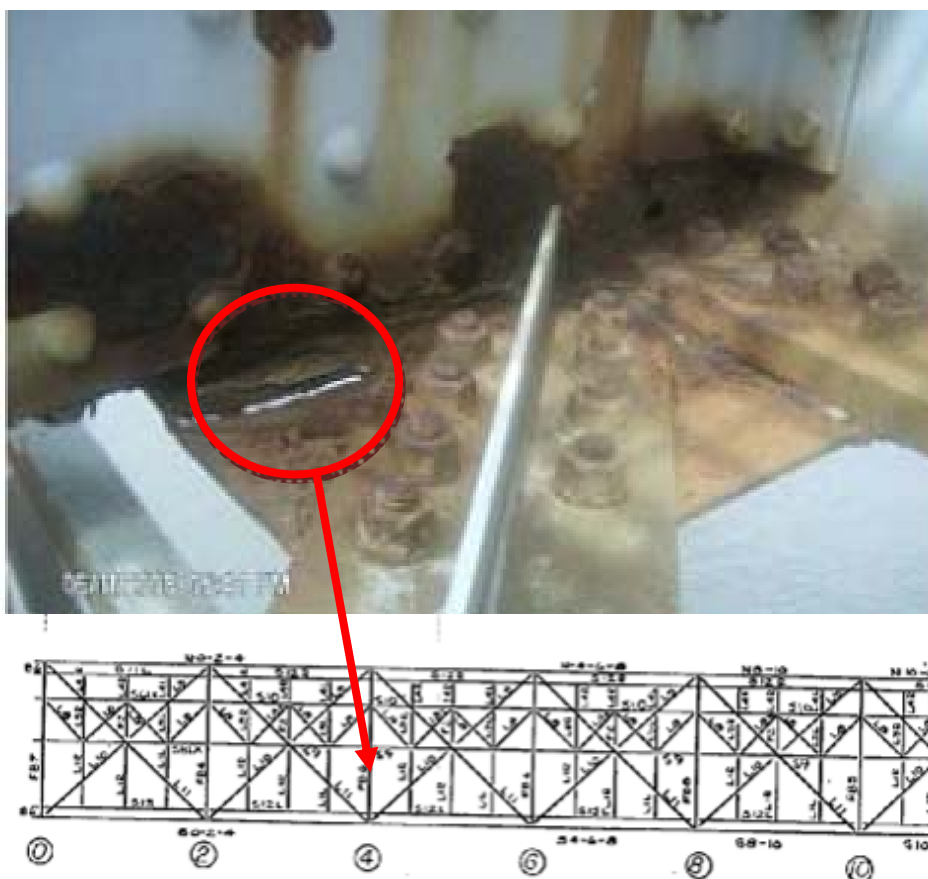


Figure 3.3.14 – Perforation in horizontal gusset plate between bottom chord and lateral bracing

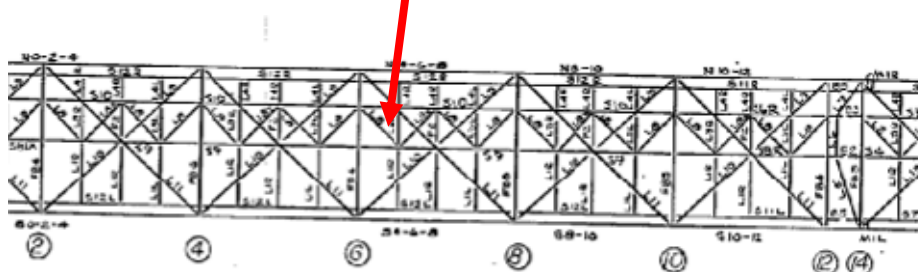


Figure 3.3.15 – South vertical at east end of Panel 3, 50% section loss

The floor system was replaced in all four (4) spans of the railway bridge since initial construction. Replaced members include all floor beams, all stringers and associated bracing. The following were observed with regard to the replacement members:

- They are in good condition;
- The connections for these members are bolted rather than riveted;
- The web stiffeners of floor beams and stringers are welded;
- The stringer and floor beam webs are welded to the associated flanges; and
- Some of the new bracing connections are welded.

The paint coating is generally in poor condition and has failed, see **Figure 3.3.16**.



Figure 3.3.16 – Typical cracks in paint system, Span 3

The truss members are all from the original construction. The largest concern is the vertical member and angle shown above. These members should be replaced in kind within 1 year if possible. There is a lot of debris on horizontal surfaces on all spans including bottom girder flanges, horizontal gusset plates and inside bottom chords, see **Figure 3.3.17**. Corrosion was observed on these surfaces.

Deck ties are in good condition. Rail joints are all tight with no missing bolts and are also in good condition, see **Figure 3.3.18**.



Figure 3.3.17 – Typical debris build-up inside of truss bottom chord at verticals



Figure 3.3.18 – Rail way deck

3.4 Mechanical and Electrical Inspection

3.4.1 Mechanical

A visual inspection with limited measurements was made of the mechanical machinery systems on the bridges on June 10 and 11, 2008 by Stafford Bandlow Engineering Inc. (SBE). This level of inspection provides for an overall assessment of the mechanical systems on the bridge, however many of the wearing surface are not accessible for inspection without disassembly, which was not conducted as part of this inspection. Latent defects may exist that were not revealed as part of this inspection.

The following sections of this report provide:

- Identification of the primary machinery systems and an explanation of the scope of work at each system.
- A brief description of each mechanical system.
- Documentation and discussion of the conditions found at each component.
- Conclusions as to current condition of the mechanical systems.
- Recommendations for repairs.

Schematic diagrams of the span drive machinery, span lock machinery and span support machinery are presented in **Appendix E**.

Color photographs were taken of conditions of interest during the inspection. Color copies of the photographs with captions are presented in **Appendix F**.

Bearings

In general the bearings were found in fair condition with evidence of recent lubrication at all bearings and little or no corrosion at the bearings in the machinery houses. Varying degrees of corrosion exist at the bearings outside of the machinery room. Clearances were measured at a representative number of bearings to evaluate wear versus an ANSI RC9 and RC6 fit. An RC9 fit is our basis for rehabilitation or adjustment to reduce clearance. An ANSI RC6 fit is the specified fit for new bearings of this type according to the Canadian Highway Bridge Design Code (CHBDC) and The American Railway Engineering and Maintenance-of-Way Association (AREMA). The measured bearings on the highway bridge typically have light wear with only one bearing (B3) approaching an ANSI RC9 fit.

On the railroad bridge the bearing wear is far greater than on the highway bridge with 7 of 10 measured bearings having clearance greater than an ANSI RC9 fit. None of the measured bearings on either bridge have clearance within the limits of an RC6 fit.

The following specific deficiencies were noted during the inspection. Referenced photos are in **Appendix F**.

Highway Bridge

- Bearing B3 – There is slight deformation of the bearing cap at the contact point with the shaft of the pinion side of the bearing. There is also degradation of the shaft that has resulted from the contact. See **Photo M-1**.
- Bearing B4 - There is a ½" recess in the bearing prior to the start of the bushing material. The gap between the bearing housing and the shaft is 0.040" at this recess. The clearance between the shaft and the bushing material was 0.014" beyond the recess.
- Bearing B12 – The bearing oscillates during operation of the machinery. The top of the bearing moves approximately 3/16" to 1/4" along the axis of the shaft. The bearing movement is causing deflection of the flange of the supporting beam. The source of the movement was not determined as part of the inspection. It is possible that a bent shaft is causing the movement. At least 3 and probably all four of the bearing base bolts have been replaced indicating that this may be a long standing problem. There is evidence of movement between the base and the support indicating that the base bolts are not adequately securing the bearing. The loose bolts are the result of, and not the source of, the bearing oscillation. See **Photo M-2**.
- Bearing B13 – All four bearing cap bolts and 2 of the 4 bearing base bolts could be moved by hand indicating that the bolts are loose. See **Photo M-3**.
- Bearings B14 and B16 – These bearings are located outside of the machinery house adjacent to the operating strut guide. The mounting bolts for these bearings are difficult to access and only portions of some of the bolts were observed during the inspection. Significant corrosion and debris was observed in the vicinity of the upper inboard mounting bolts at both bearings. The body of these bolts may have significant corrosion. Additional cleaning and better access is required to provide a complete assessment of these bolts. See **Photos M-4 and M-5**.

Railroad Bridge

- Bearing B4 – The clearance at the bearing (0.025") is 0.001" greater than the maximum clearance for an RC9 fit.
- Bearing B6 - The clearance at the bearing (0.041") is 0.017" greater than the maximum clearance for an RC9 fit.
- Bearing B8 – The clearance at the bearing (0.043") is 0.015" greater than the maximum clearance for an RC9 fit.
- Bearing B10 – Movement was observed between the bearing base and the bearing support during operation of the bridge. This movement indicates that the bearing mounting bolts are not adequately tightened.
- Bearing B11 – The clearance at the bearing (0.054") is 0.020" greater than the maximum clearance for an RC9 fit.
- Bearing B-12 – The clearance at the bearing (0.045") is 0.007" greater than the maximum clearance for an RC9 fit. The bearing housing has small areas of paint deterioration and light to moderate corrosion.
- Bearing B13 – The clearance at the bearing (0.039") is 0.005" greater than the maximum clearance for an RC9 fit. Four of the 6 bearing cap bolts are hand loose. Three of 4 bearing base bolts have a second nut that is not original. The addition of a second nut may be an indication of a problem with the bearing. There is significant movement at this bearing during operation at both the bearing base and the bearing cap.
- Bearing B-14 – The clearance at the bearing (0.045") is 0.007" greater than the maximum clearance for an RC9 fit. The bearing housing has small areas of paint deterioration and light to moderate corrosion. One of the lube fittings for the bearing cap has been replaced with a pipe cap. See **Photo M-6**.

Brakes

The brakes are in poor condition overall and no longer suitable for long term use. The brakes are solenoid operated and are provided with dampers to increase the brake set time and thereby reduce impact loading of the machinery. The dampers are no longer effective either due to poor performance or leakage. Brakes of this type have not been manufactured for many years and replacement parts are no longer available.

The actuating mechanism for the motor brakes on the highway bridge apparently failed at some point and these brakes have been retrofit with an actuating mechanism that is typically found in the crane industry according to

maintenance personnel. See **Photo M-8**. We have not seen this type of actuator used in movable bridge applications.

The brakes are not provided with covers to protect maintenance personnel from the rotating brake wheels. The brakes are also not equipped with limit switches that are typically used to provide brake position indication and to provide electrical interlocks for increased safety.

The following specific deficiencies were noted during the inspection:

Highway Bridge

- North Emergency Brake – There is lubricant and light corrosion on the brake wheel.
- North Motor Brake – The brake shoes are provided with a lubrication fitting. This fitting requires a very small amount of lube on an infrequent basis. There is excess lube at this location. The brake wheel is contaminated with lubricant. See **Photo M-7**.
- South Motor Brake – There are a few deep scores on the brake wheel. The scores do not require corrective action. A cotter pin is missing at one of the brake linkage pins. See **Photo M-8**.

Railroad Bridge

- North Emergency Brake – The solenoid housing bolts are loose and the housing is not secure as a result. See **Photo M-9**.
- South Emergency Brake – The west brake shoe is not in contact with the brake wheel when the brake is set. Therefore the brake is not providing torque. See **Photo M-10**.
- South Motor Brake - There is lubricant on the brake wheel.

Open Gears

The open gearing is generally in fair condition. The gears are generally well lubricated, wear is light and corrosion is not an issue on any of the gears in the machinery room. The racks and rack pinions are the only gears located outside of the machinery room. These gears are not as well lubricated and there are areas of corrosion due to insufficient lubrication. The differential assemblies have bearings that are not provided with a method for lubrication. Although the movement at these bearings is limited, it is typical to provide a method of lubricating the sliding surfaces. No assessment of the function of the differential was made as part of this inspection.

Backlash was measured at a representative number of gearsets. In addition, the wearing surfaces of the gears were checked for unusual wear. Backlash was not excessive at any of the measured gears and no unusual wear was observed.

The following specific deficiencies were noted during the inspection:

Highway Bridge

- Gear G2 – The keys for this gear is loose and can be removed by hand. See **Photo M-11**.
- Gearset G7/G8 – There are hardened lubrication deposits in the roots of both the pinion and the gear. The deposits in the roots of the pinion teeth (G7) are heavy. See **Photo M-12**.
- Gearset G10/G11 – There are hardened lubrication deposits in the roots of both the pinion and the gear teeth.
- Gear G10 – The key for gear has deformation that appears to be the result of driving the key into position. This key may be working its way out over time.

Railroad Bridge

- Gear G2 and Gear G4 – The keys for these gears are restrained by a second key that is secured with a hose clamp. At Gear G4 the side fit of the key in the keyway allowed for a small amount of key movement and there is fretting corrosion indicating that the gear is moving relative to the shaft during operation. See **Photo M-13**.
- Gearset G5/G6 – The abrasive wear and scoring on this gear set is moderate however the tooth thickness has not been reduced appreciably as a result of the wear. This was the most severe wear found at any of the gears on both bridges. See **Photo M-14**.
- Gearset G7/G8 – There is a significant accumulation of lubricant at this gearset. See **Photo M-15**.
- Gear G11 – There is a light accumulation of hardened lubrication deposits in the roots of the gear teeth.

Operating Strut and Guide Assembly

The operating strut guide was designed to guide the operating strut and to keep the rack and rack pinion at the correct center distance. Each operating strut guide is equipped with two pair of lower guide wheels and one pair of upper guide wheels. The lower guide wheels support the dead weight of the rack and operating strut when unloaded. The upper guide wheels limit separation between the rack and rack pinion during span operation due to the separating force generated by the gear teeth under load.

The construction of the assembly severely limits inspection as the bearings and rack pinion are inaccessible for the most part. In addition there is no access to the outboard side of the assemblies.

The guides and operating struts appear in fair external condition with paint deterioration and light to moderate corrosion the noted deficiencies. See **Photos M-16** and **M-17**. Lubrication fittings were found in place and the lubricant appeared recent and adequate at the guide assemblies. All of the upper rollers rotated freely by hand and the lower rollers rotated during operation of the bridge.

Lubrication at the span end of the strut appeared marginal, especially at the bottom of the bearing. This may be the result of limited bearing rotation and or insufficient lubrication. Clearance was measured at the one accessible bearing at the span end of each operating strut. All measured clearances were within the limits of an RC9. The clearance at the north operating strut on the highway bridge measured 0.025" which is approaching the upper limit of an RC9 fit (0.028"). When clearances exceed an RC9 fit, we recommend adjustment or rehabilitation.

Debris accumulates on top of the racks between the structural steel channels that form the operating strut. The north rack on the highway bridge had the most significant accumulation of debris however there is debris present at all operating struts. See **Photo M-18**.

The original shop drawings indicate that the clearance between the operating strut and the upper guide wheels was 1/8" when originally constructed. The following measurements were taken during the inspection.

Gap between Operating Strut and Upper Guide Roller – Original Gap = 1/8"		
Location	Highway Bridge	Railroad Bridge
North Guide – Inboard Roller	.240"	.215"
North Guide – Outboard Roller	.220"	.115"
South Guide – Inboard Roller	> .350" < .475"	.165"
South Guide – Outboard Roller	> .350" < .475"	.218"

The measurements indicate that the highway bridge rollers have experienced far greater wear than the railroad bridge. In the case of the south guide on the highway bridge the wear has resulted in clearance that is

3 to 4 times as great as originally intended. See **Photo M-19**. The wear is either due to wear of the rollers or wear of the bearings that support the rollers. The rollers do not appear heavily worn and therefore the likely source of wear is the bearings that support the rollers. Since the bottom rollers support the weight of the operating strut it is likely that these rollers have worn. One other possibility is that the clearance was greater at original construction than shown on the drawings.

The apparent wear can cause two problems, neither of which was observed during the inspection. First, wear of the bottom roller bearings would result in a decrease in the rack and pinion center distance which could lead to interference wear of the gear teeth. Second, wear of the bearings for the top rollers would allow for greater separation of the rack and pinion during operation of the bridge due to the separating force that is generated by the gear tooth loads. The greater separation could result in severe impact loading of the machinery when the operating strut drops onto the lower rollers when load is removed from the system. Loading and unloading of the gear teeth is a normal occurrence during operation of the bridge.

Span Drive Motors

The span drive motors operated satisfactorily throughout the inspection and appear in fair condition externally. All motor mounting bolts appear tight and the motors are secure on their supports. The motors are provided with oil lubricated bearings that require regular maintenance and are subject to leakage.

The following specific deficiencies were noted:

Highway Bridge

- Motor M2 - There is an oil puddle on the support between the motor and the motor brake. The oil lubricated motor bearings are the likely source of the oil although the oil in all of the motor bearings was at the proper level. See **Photo M-20**.

Railroad Bridge

- Motor M1 – The cover for the oil level check fitting is missing and both bearings appear to be leaking oil. See **Photo M-21**.
- Motor M2 – The babbitt in the north motor bearing appears to have overheated and melted as result. Access for inspection is difficult and the damage could not be photographed. This bearing also appears to be leaking oil although the oil was at the proper level at the time of the inspection. See **Photo M-22**.

Auxiliary Drive Machinery

The auxiliary drive machinery was cursorily inspected and no attempt was made to operate either bridge using the auxiliary drive. Maintenance personnel report that the auxiliary drive operates satisfactorily. Overall, the equipment appears in fair condition.

The machinery is not protected against the operation of both the auxiliary and normal span drive machinery simultaneously. Operation of both systems simultaneously is potentially dangerous to maintenance and operating personnel and could result in severe damage to the machinery.

In summary, the following specific deficiencies were noted:

Highway Bridge

- The hand brake assembly is coated with lubricant and there is lubricant on the brake wheel. See **Photo M-23**.
- There is a significant oil leak at the auxiliary drive engine. See **Photo M-24**.

Railroad Bridge

- There is a significant oil leak at the auxiliary drive engine.
- Span Support System (Main Trunnion Bearings, Counterweight Trunnion Bearings, 1st Link Pins and 2nd Link Pins).

The dead weight of the movable leaf is supported by two main trunnions located at the heel of the span; live load is supported by two live load supports at the toe end of the span. The counterweight is located directly above the roadway. It is supported by two counterweight trunnion bearings mounted on a tower above the east approach. The steel counterweight frame is connected to the bascule leaf by a counterweight link utilizing four link pins. There are two link pins (1st link pins) and bearings at the connecting points between the counterweight frame and the counterweight link and two additional link pins (2nd link pins) and bearings that connect the counterweight link to the upper chord of the bascule span. The trunnions and pins are arranged such that they form a parallelogram in a vertical plane. As the span opens the first and second link pin bearing assemblies move while the main and counterweight trunnion bearing assemblies remain stationary with the exception of the rotational movement within the assembly. The movement of the link pins causes the shape of the parallelogram to change however the four sides of the parallelogram remain parallel regardless of the span position. **Figure 3, Appendix E** identifies the primary span support components. **Figure 4, Appendix E** illustrates the change in geometry of the span support system during movable leaf operation.

All of the bearings are located in confined or inaccessible areas. Each bearing is mounted between structural steel plates and as a result, clearances are inaccessible for measurement. Due to the bearing construction, only the main and counterweight trunnions can be opened (bearing cap removed) for inspection of the wearing surfaces. The caps were not removed as part of this inspection, however consideration should be given to removing the caps to observe the condition of the bearing journals. When the caps are removed it is also possible to measure the bearing clearance using a "Plastigage".

Lubrication of the bearings generally appeared to be recent and adequate. All of the grease grooves should be cleaned out on an annual basis. A wire should be run through all grooves to prevent the buildup of hardened deposits. The grooves should then be flushed with clean grease following the wire cleanout. There is no evidence that these bearings are being cleaned out regularly. See **Photo M-25**. At three of four 2nd link pins (both locations on the railroad bridge and the north location on the highway bridge) there is accumulated lubricant at the inboard side of the bearing at one of the lube fittings. The lube piping may be loose at this location. Access to this location is poor. See **Photo M-26**.

All of the bearings were in fair condition externally. No broken bolts were found and there were no obvious signs of poor alignment. There are areas of built up debris, paint deterioration and light to moderate corrosion where the paint system has failed. See **Photos M-27** and **M-28**.

Span Locks

Each bridge has two span locks. The locking mechanism consists of a lock bar supported in roller assembly that is mounted in the bottom cord of each truss at the toe end of the bridge. Each lock bar engages a receiver that is mounted on the channel side of the live load support casting. The lock bars are driven by an electric motor through a series of gears, bearings, shafts, levers, and links. The low speed machinery shaft extends across the bridge to drive the lock bar on the opposite side of the bridge. **Figure 5, Appendix E** shows a schematic of the span drive machinery.

The span drive machinery operated satisfactorily throughout the inspection and overall the machinery is in fair condition. The gears and bearings appeared to have recent and adequate lubrication. No gear tooth or bearing clearance measurements were taken as part of this inspection.

The locks are provided with a crank for manual operation in the event that electric power is not available. No attempt was made to operate the machinery manually. The hand crank is not protected by electrical controls

to prevent the span lock motor from energizing when the hand crank is engaged. Operation of the span lock motor when the hand crank is engaged is extremely dangerous.

To summarize, the following specific deficiencies were noted:

Highway Bridge

- The key for G1 contacts the machinery enclosure. See **Photo M-29**.
- The west nut for the upper rod end connection was loose and the connecting rod is bent. See **Photo M-30**.
- The thrust collar on the hand crank shaft is not properly positioned. In the current position the hand crank can be engaged. This is a safety concern. See **Photo M-30**.
- The machinery enclosure is in poor condition. See **Photo M-30**.

Railroad Bridge

- The key for G1 contacts the machinery enclosure. See **Photo M-29**.
- The nut for one of the base bolts for bearing B1 has nearly complete section loss. See **Photo M-31**.
- The thrust collar on the hand crank shaft is not properly positioned. In the current position the hand crank can be engaged. This is a safety concern. See **Photo M-30**.
- The machinery enclosure is in poor condition. See **Photo M-32**.

Air Buffers

Each bridge is provided with a single air buffer. The buffer is located on the centerline of the bridge at the toe end of the bridge. The buffer rod engages a strike plate that is mounted on the west pier. The highway bridge buffer was observed during operation. Operation of the railroad buffer was not observed. The buffers are not provided with pressure gages.

The following specific deficiencies were noted.

Highway Bridge

- The buffer piston and rod descended rapidly as the bridge opened. This is an indication that the buffer requires rehabilitation.
- The four mounting bolts for the bottom bearing were loose. See **Photo M-33**.
- The buffer is coated with lubricant.

Railroad Bridge

- The buffer is coated with lubricant.

Live Load Supports and Centering Devices

The movable leaf is equipped with two live load supports. One live load support is provided at each corner at the toe end of the leaf. Each live load support consists of a live load shoe and a live load strike plate. The live load shoes are located under the toe end of the bottom chords of the trusses. The live load strike plates are located on the pier.

A centering device casting is bolted to the outboard side of the strike plate casting. The live load shoe engages the centering device and forces the span into the centered position if required.

There was only slight movement at the live load support under the live load of traffic although maintenance personnel indicated that this can vary depending on the bridge operator. There was no indication of hard contact at any of the centering devices.

Three of four anchor bolts at the south live load strike plate for the railroad bridge are loose. See **Photo M-34**.

There are varying degrees of paint deterioration and corrosion at the live load supports and centering devices. The corrosion has not affected the integrity of the components.

Summary of Mechanical Inspection

The mechanical machinery systems on the Johnson Street Bridge are in fair condition overall. It appears that the machinery components are being lubricated on a regular basis however there is in general excess lubrication on and around the machinery that adversely affects the operation of the brakes and increases the risk of injury due to slipping.

Corrosion has not been a problem for the span drive machinery located within the machinery room. The machinery outside of the machinery room has suffered paint deterioration and corrosion to varying degrees. The corrosion that was observed is not severe enough to affect the integrity of the components with a few exceptions that have been identified in the report.

The span drive and span lock machinery are maintenance intensive and in the case of the span drive machinery not adequately guarded to protect maintenance and inspection personnel from injury. This machinery is obsolete by current machinery design practice for movable bridges. A current design would employ the use of enclosed gearing and rolling element bearings with open gearing limited to the rack and rack pinion for the span drive machinery.

The advantages of a current machinery design are increased efficiency, reduced risk of injury and a significant reduction in maintenance requirements.

The span drive machinery brakes are of obsolete construction and replacement parts are no longer available. The brakes do not provide the necessary time delay on setting for smooth application and as a result there is significant shock loading of the machinery with each application of the brakes.

One of the bearings for the span drive machinery on the highway bridge oscillates during operation with resultant flexure of the supporting structural steel. The oscillation may be the result of a bent shaft and there is the potential for shaft or support failure due to fatigue.

The north motor bearing for the north motor on the railroad bridge appears to be severely damaged. This condition may affect the reliable operation of the bridge in the near term.

With the implementation of the recommendations in Section 6, the machinery on this bridge can be expected to provide reliable service for many years. Still it is recommended that strong consideration be given to upgrading span drive and span lock machinery to current standards to improve system efficiency, reduce the risk of injury to maintenance and inspection personnel and to reduce maintenance and operating costs.

3.4.2 Electrical

General

The majority of the electrical equipment in use on the Johnson Street Bridge is obsolete and/or in poor condition and rapidly reaching the end of its useful life.

The bridge control system is typical of an older installation where minimal controls are utilized. Interlock tests were performed to verify protection of the traveling public and protection of machinery. The performed interlock tests found that the control system meets minimum accepted standards.

There is no stand-by electrical power source. To perform a bridge opening during a power loss situation, traffic is regulated with the use of hand paddles by bridge personnel. The traffic gates and span locks are manually operated by hand cranks. In the machinery room, the operator is required to engage a mechanical clutch to allow a gasoline engine to raise and lower the bridge.

This inspection uncovered several deficiencies ranging from minor issues related to the degradation of the installed equipment to major concerns regarding reliability and accessibility of equipment, and safety, all of which are detailed within this report. In general, the aging equipment installed on the bridge is in need of major rehabilitation to increase safety and operational reliability of the bridge.

The electrical power and control systems were evaluated with respect to compliance with the following applicable codes and standards:

- American Association of State Highway and Transportation Officials (AASHTO) LRFD Movable Highway Bridge Design Specifications, 2007 edition;
- NFPA 70, National Electric Code (NEC)-2005;
- NFPA 101, Life Safety Code (LSC);
- National Electrical Manufacturers Association (NEMA);
- Manual on Uniform Traffic Control Devices (MUTCD)-2003 edition;
- Code of Federal Regulation, Title 33 (CFR).

Collection of electrical voltage, current, and resistance measurements were conducted utilizing the following equipment:

- Megger Model MIT 430 (Insulation Resistance Meter);
- Fluke Model 336 (AC/DC Clamp-on Current Meter); and
- Fluke model 87 (Voltage, Current, and Frequency Meter).

The electrical service to the bridge is routed from a utility building located at the southeast bridge approach. Service voltage is 480 VAC, three-phase. The service is routed from the utility building located at the southeast end of the bridge to the control house via conventional electrical conduit.

The incoming service is fed into an electrical panel on the east wall inside the control house. This electrical panel does not provide a way to lock out the service power to allow qualified personnel to perform maintenance. The north wall contains the motor controls for the traffic gates and span locks.

Incoming service voltage and frequency was measured with the results listed below:

- | | |
|-------------------------------|----------|
| • Phase A to Phase B Voltage: | 474 VAC |
| • Phase B to Phase C Voltage: | 473 VAC |
| • Phase C to Phase A Voltage: | 478 VAC |
| • Phase A to Ground: | 270 VAC |
| • Phase B to Ground: | 297 VAC |
| • Phase C to Ground: | 260 VAC |
| • Frequency: | 60.01 Hz |

Distribution of single-phase and three-phase power is accomplished through

a series of distribution panel boards and three-phase breaker panels located in the control house, highway machinery room, and the railway machinery room.

The panels and circuit breakers are all in fair condition. Although the preferred method of distribution would be a dedicated motor control center and lighting panel, this collection of different types of equipment provides the necessary distribution and over-current protection for the bridge electrical equipment.

The bridge is not equipped with any means of backup power in the event of loss of the main service. If main service is lost, the bridge will be without any power for drive equipment or lighting. In the event of a power failure, each bascule leaf is equipped with a manual start gasoline engine. The engines are coupled with a manual clutch that can raise and lower the span. Note that manual operation of the span locks and warning gates need to first be performed prior to use of the gasoline engine to raise or lower the bridge.

Motor Control Devices

All feeder over-current protection devices and full voltage starters (both CEMA and IEC types) for traffic gate motors and span lock motors are located on the north wall of the control house. All feeder over-current protection devices and full voltage starters (both CEMA and IEC types) for the highway span drive motors, motor brakes, and emergency brakes are located on the south and east walls of the machine/electrical room. All feeder over-current protection devices and full voltage starters (both CEMA and IEC types) for the railway span drive motors, motor brakes, and emergency brakes are located on the south and east walls of the machine/electrical room.

The motor control devices are obsolete and the equipment components are in fair to poor condition.

Drive Motors

Each movable bridge span is provided with two AC wound rotor drive motors. The drive motors are of General Electric manufacture and have the following nameplate data:

Highway Bascule Drive Motor

Type:	ITC
Power:	75 Horsepower
Speed:	600 RPM
Amp:	105 Amps
Volts:	440 Volts AC
Hertz:	60

Railway Bascule Drive Motor

Type:	ITC
Power:	37 Horsepower
Speed:	600 RPM
Amp:	56 Amps
Volts:	440 Volts AC
Hertz:	60

The drive motors for each bascule leaf are located in the corresponding machinery room. Each drive motor was inspected and found to be in fair physical condition electrically and obsolete due to age. The railway drive motors produced significantly lower insulation resistance readings. See table below. Accessibility to the drive motors by qualified maintenance personnel is difficult due to crowding in the machinery areas. See Photo E1 in **Appendix G**.

The current draw for the four drive motors was measured during operation and found to be within acceptable limits. The measured values are provided in the following tables.

Highway North Drive Motor (Amps)

Phase	Raise	Lower
A	50	50
B	71	37
C	85	55

Highway South Drive Motor (Amps)

Phase	Raise	Lower
A	81	80
B	82	75
C	81	71

Railway North Drive Motor (Amps)

Phase	Raise	Lower
A	42	41
B	41	42
C	42	42

Railway South Drive Motor (Amps)

Phase	Raise	Lower
A	42	40
B	42	39
C	49	38

The insulation resistance values for each drive motor were collected and are provided in the following tables. Measurements were taken at the disconnect switch.

Highway North Drive Motor (meg Ohms)		Highway South Drive Motor (meg Ohms)	
Winding	Result	Winding	Result
Stator	>200	Stator	>200
Railway North Drive Motor (meg Ohms)		Railway South Drive Motor (meg Ohms)	
Winding	Result	Winding	Result
Stator	.9	Stator	6

Deficiencies regarding the drive motors were noted as follows. Referenced photos are in **Appendix G**.

- There is improper color coding of the highway south drive motor contactor conductors. The red-blue-black combination represents a 240 VAC system. Accepted electrical industry practice for a 480 VAC system is brown-orange-yellow. See **Photo E2**
- There is improper color coding of the highway north drive motor contactor conductors. The red-blue-black combination represents a 240 VAC system. Accepted electrical industry practice for a 480 VAC system is brown-orange-yellow. See **Photo E3**
- There is improper color coding of the railway south drive motor contactor conductors. The red-blue-black combination represents a 240 VAC system. Accepted electrical industry practice for a 480 VAC system is brown-orange-yellow. See **Photo E4**
- There is improper color coding of the railway north drive motor conductors. The red-blue-black combination represents a 240 VAC system. Accepted electrical industry practice for a 480 VAC system is brown-orange-yellow. See **Photo E5**

Brakes

Each drive motor is provided with a single solenoid brake located on the motor tail shaft. The brakes are “fail safe” with a spring holding the brakes in the “Set” position. The brakes release when energized through the “Brake Release” pushbutton on the control desk.

The highway north and south “Motor” brakes are equipped with motors and solenoids to help operate the brakes. The highway north and south “Emergency” brakes are not equipped with motors. Instead, the conductors from the contactor panel are fed to a solenoid on the brake assembly that actuates the brake.

The railway north and south “Motor” and “Emergency” brakes are not equipped with motors. Instead, the conductors from the contactor panel are fed to a solenoid on the brake assembly that actuates the brake.

Only the highway north and south “Motor” brakes were equipped with motor data nameplates. The motors are of Leeson manufacture and have the following nameplate data:

Highway Bascule North and South Motor Brake Nameplates:

Horsepower: .5
 Volts: 208-230 / 460 VAC
 Amps: 1.8 / .9
 Service Factor: 1.15
 RPM: 1725
 Frequency: 60 Hz

Each motor and emergency brake was inspected and found to be in fair condition and functioning as intended. The current draw for the all brakes was measured and found to be within acceptable limits.

The north and south motor brakes are connected in a parallel configuration. There are no individual disconnect switches provided for each brake, therefore the measured amp readings is of both brakes. This configuration is typical for the emergency brakes.

Measurements were taken at the contactor panel and the measured values are provided in the following tables.

Highway Motor Brake (Amps)		Highway Emergency Brake (Amps)	
Phase	Amps	Phase	Amps
A	.6	A	2.6
B	.6	B	3.8
C	.6	C	3.1

Railway Motor Brake (Amps)		Railway Emergency Brake (Amps)	
Phase	Amps	Phase	Amps
A	2.6	A	6.7
B	3.0	B	6.5
C	2.8	C	7.1

The north and south motor brakes are connected in a parallel configuration. There are no individual disconnect switches provided for each brake, therefore the measured insulation resistance is of both brakes. This configuration is typical for the emergency brakes.

Measurements were taken at the contactor panel and the measured values are provided in the following tables.

Highway Motor Brake (meg Ohms)		Highway Emergency Brake (meg Ohms)	
	Result		Result
Winding	>200	Winding	>200
Railway Motor Brake (meg Ohms)		Railway Emergency Brake (meg Ohms)	
	Result		Result
Winding	95	Winding	0

Deficiencies regarding the brakes were recorded as follows. Referenced photos are in **Appendix G**.

- There is improper color coding of the highway and railway motor brake conductors. See **Photo E6**
- There is improper color coding of the highway and railway emergency brake conductors. See **Photo E7**
- Conductors for both the highway and railway motor and emergency brakes are poorly protected from accidental contact. See **Photo E8**
- The highway motor and emergency brakes are not equipped with position limit switches. See **Photo E9**

Span Locks

Each bascule span is equipped with two span locks. Each span lock was found to be operating correctly at the time of inspection. In the event of a power loss the span locks can be manually operated via a crank arm.

Operator's Control Desk

There is a single, free standing control desk and a wall-mounted control panel located inside the control room providing all necessary operator interface devices for control of both structures. See Photos E10 and E11 in Appendix G. The control desk and wall-mounted control panel are constructed from painted steel. The control desk faces west, providing the operator with a good vantage point for bridge operation. The wall-mounted control panel is located on the north wall of the control house. Windows are provided at all four sides of the control house providing an acceptable amount of visibility to the bridge operator for all marine and vehicular traffic.

Due to the age of the control desk, the desk and all components are obsolete and approaching the end of their useful lives.

Deficiencies relating to the operator's control desk were recorded as follows. Referenced photos are in **Appendix G**.

- Flammable materials are being stored inside the control desk. See **Photo E12**
- Flammable materials are being stored inside the wall mounted control panel. See **Photo E13**

Bridge Control System

In general, all bridge functions are initiated by the bridge operator. The operational state of the bridge control system at the time of this inspection utilizes interlocks for traffic gates and span locks.

The existing control system utilizes drum control switches for both highway and railway bascule spans. See **Photo E14** in **Appendix G**. To perform a raise cycle of the bridge, the operator is required to place the drum switch in the "UP" position and simultaneously press and maintain the "Bridge Free Up" button located next to the drum controller. The "Bridge Free Up" button releases the brakes. To perform a lower cycle of the bridge, the operator is required to place the drum switch in the "DOWN" position and simultaneously press and maintain the "Bridge Free Down" button located next to the drum controller. The "Bridge Free Down" button also releases the brakes. A release of either the "Bridge Free Up" button or "Bridge Free Down" button will set the brakes.

The control system has one keyed bypass switch, located on the wall-mounted control panel, for gate-bypass that provides a short circuit path around the interlock relays to defeat their purpose in the event of a limit switch failure.

The control system has two additional keyed bypass switches, "Emergency Flash" and "Flash or Steady". Both are located on the wall-mount control panel. In the event of an emergency, the keyed "Emergency Flash" switch will change the traffic signals to red. The "Flash or Steady" keyed switch is used to determine if the red traffic signals are to flash or maintain on when the "Emergency Flash" keyed switch is required. At time of inspection, the keyed position of the switch was in the "Flash" position.

During this inspection, a test of the bridge interlocking system was performed with the following results:

- The traffic gates could not be lowered until the traffic signals were red.
- The span locks were not functional until the traffic gates were in the lowered/closed position.

- The span could not be raised while the traffic gates were in the raised/open position.
- The span could not be raised while the traffic signals were green.
- The span could not be raised until the span locks were pulled.
- The bridge control systems for both the highway and railway bridges are obsolete.

Field Feedback Devices

Field feedback devices are typically provided on movable bridges to aid the bridge operator with position information for gates, locks, brakes, and the movable span during operation. These devices also provide the necessary input to the bridge control system for interlocking controls and for automation of various operations.

The traffic warning gate limit switches appear to be functioning properly. A blue "Gate Down Enabled" indicator light on the wall mounted control panel provides indication to operator that the traffic warning gates are fully lowered. At the time of inspection, the traffic warning gate indicator lamp was functioning.

The span position limit switches appear to be functioning properly, but are in fair to poor condition. See Photo E15 in Appendix G, which is typical for all limit switches. The control desk is provided with span position indicator lamps for "Bridge Down", "Bridge Nearly Down", "Bridge Nearly Up", and "Bridge Up". See Photo E16 in Appendix G. At the time of inspection, all span position indicator lamps were functioning.

The span lock limit switches appear to be functioning properly, but are in fair to poor condition. The control desk is provided with green and red indicator lights to provide indication to the operator that the span locks are pulled or driven. See Photo E16 in Appendix G. At the time of inspection all span lock indicator lamps were functioning.

Traffic Gates

The highway bascule approach is equipped with a four conventional drop-arm type traffic gates, one combination traffic/pedestrian gate at the south side of the west approach, one gate at the north side of the west exit, one gate at the north side of the east approach and one combination traffic/pedestrian gate at the south side of the east exit. The gates are not equipped with arm lighting or reflective striping to provide visual direction to approaching traffic.

The railway bascule approach is equipped with a two conventional drop-arm type traffic gates, one gate at the north side of the west approach and one gate at the north side of the east approach. The gates are not equipped with arm lighting or reflective striping to provide visual direction to approaching traffic.

The electrical enclosures for the highway and railway bascule gates were in fair condition. Each electrical enclosure contains two contactors, one for “Raise” operation and the other for “Lower” operation, and one overload with manual “Reset”. The southeast and northwest highway gates are wired in the enclosure such that they are operated by one group of contactors. The northeast and southwest highway gates are wired in the enclosure such that they are operated by one group of contactors. The railway gates are wired in the enclosure such that they are operated by one group of contactors.

The motors and interior components were functioning properly at the time of this inspection. The motors are in fair condition and are obsolete. All gate assemblies for both highway and railway bridges are in poor condition and approaching the end of their useful lives.

The following tables provide the data for the highway and railway traffic gate motor nameplates:

Highway Northwest Traffic Motor Nameplate:

Manufacturer:	Century Electric
Horsepower:	illegible
Volts:	440 VAC
FLA:	illegible
RPM:	1136
Frequency:	60 Hz

Highway Southwest Traffic/Pedestrian Motor Nameplate:

Manufacturer:	Century Electric
Type:	SC
Horsepower:	.5
Volts:	440 VAC
Amps per Line:	.95
RPM:	1135
Frequency:	60 Hz

Highway Northeast Traffic Motor Nameplate:

Manufacturer:	English Electric Company
Type:	CD
Horsepower:	.5
Volts:	440 VAC
FLA:	1.3
RPM:	1160
Frequency:	60 Hz

Highway Southeast Traffic/Pedestrian Motor Nameplate:

Manufacturer:	Century Electric
Type:	SC
Horsepower:	.5
Volts:	440 VAC
Amps per Line:	.95
RPM:	1135
Frequency:	60 Hz

Railway Northeast Pedestrian Motor Nameplate:

Manufacturer:	Century Electric
Type:	SC
Horsepower:	.5
Volts:	440 VAC
Amps per Line:	.95
RPM:	1136
Frequency:	60 Hz

Railway Northwest Pedestrian Motor Nameplate:

Manufacturer:	General Electric
Type:	SC
Horsepower:	.5
Volts:	460 VAC
Amps:	1.3
RPM:	1140
Frequency:	60 Hz

The highway northwest and southeast traffic gate motors are connected in a parallel configuration. There are no individual disconnect switches provided for each motor, therefore the measured amp readings is of both motors. This configuration is typical for the highway northwest and southeast traffic gates and for the railway northwest and northeast traffic gates. The current draw for the traffic gate motors was within acceptable limits based on available nameplate data.

Measurements were taken at the contactor panel and the measured values are provided in the following tables.

Highway Northwest and Southeast Traffic Gate Motor (Amps)			Highway Northeast and Southwest Traffic Gate Motor (Amps)		
Phase	Raise	Lower	Phase	Raise	Lower
A	1.3	1.1	A	2.1	1.9
B	1.7	1.3	B	1.8	2.1
C	1.2	1.1	C	2.1	1.9

Railway Northwest and Northeast Pedestrian Gate Motor (Amps)		
Phase	Raise	Lower
A	1.6	2.0
B	1.2	2.0
C	2.0	1.8

The insulation resistance values for the “Northwest and Southeast”, “Northeast and Southwest”, and railway pedestrian gates are provided in the following tables. Measurements were taken at the traffic gate electrical enclosures located in the control house.

Northwest and Southeast Highway Traffic Gates (meg Ohms)		Northeast and Southwest Highway Traffic Gates (meg Ohms)	
	Result		Result
Winding	10.2	Winding	13.3

Northwest and Northeast Railway Pedestrian Gates (meg Ohms)	
	Result
Winding	>200

The highway gate heights are provided in the following tables.

<u>Location</u>	<u>cm (in)</u>
Northwest Traffic	61 (24)
Southwest Traffic	152 (60)
Southwest Pedestrian	86 (34)
Northeast Traffic	119 (47)
Southeast Traffic	137 (54)
Southeast Pedestrian	69 (27)

The railway gate heights are provided in the following tables.

<u>Location</u>	<u>cm (in)</u>
Northwest Pedestrian	129 (51)
Northeast Pedestrian	99 (39)

Specific deficiencies relating to the traffic gates were recorded as follows. Referenced photos are in **Appendix G**.

- The southeast traffic gate arm does not set at 90 degrees vertical. See **Photo E17**
- The southeast traffic/pedestrian gate housings and arms are damaged. See **Photo E18**
- The northeast traffic gate arm does not set at 90 degrees vertical. See **Photo E19**
- The bottom conductor cable feeding the conduit box mounted to side of the bridge guard rail has a damaged water-tight connection. See **Photo E20**
- The northwest traffic gate arm shows evidence of being damaged. The damaged gate arm has been spliced together using bolts. See **Photo E21**
- The northeast railway pedestrian gate is over 90 degrees vertical. See **Photo E22**
- The water-tight fitting protecting the SO cord entering the northwest railway gate arm switch box shows evidence of cracking. See **Photo E23**
- The conduit LB water-tight seal for the northeast railway gate is damaged. See **Photo E24**
- The northwest railway gate shows evidence of being damaged. The damaged gate arm has been spliced together using bolts. See **Photo E25**
- Minor surface corrosion is present on the northwest railway gate housing. Typical at all locations. See **Photo E26**

- The motor housing for the southeast traffic gate was not secured to the traffic gate housing allowing access by non-qualified personnel to the electrical and mechanical equipment. Typical at all traffic gates. See **Photo E27**

Traffic Signals and Signage

Both approaches of the highway bascule are provided with two red over amber traffic signals. During bridge operation, the red signal remains on.

No deficiencies were noted regarding traffic signals and signage.

Aids to Navigation

The Johnson Street Bridge is provided with the following aids to navigation in the form of pier lights and center span clearance lights:

- One pier light with dual white lamps at the southwest pier.
- One pier light with dual white lamps at the southeast pier.
- One multi-sectional (green/red) center span clearance light located on the outside center of the highway bascule span.
- One multi-sectional (green/red) center span clearance light located on the inside center of the highway bascule span.
- One pier light with dual white lamps at the northwest pier.
- One pier light with dual white lamps at the northeast pier.
- One multi-sectional (green/red) center span clearance light located on the outside center of the railway bascule span.
- One multi-sectional (green/red) center span clearance light located on the inside center of the railway bascule span.

In general, the pier lights and center span clearance lights are in good to fair condition.

Specific deficiencies relating to the aids to navigation were recorded as follows:

- Access by qualified personnel to the pier lights is unsafe.

Lighting, Receptacles and Heating

The lighting in the control house consists of two-tube, four foot fluorescent ceiling mounted fixtures. These lighting fixtures appear to meet the minimum accepted requirements for luminance intensity.

120 VAC power receptacles have been installed throughout the control house. They have been evenly distributed and are of sufficient quantity.

The lighting in the highway machinery room consists of two-tube, four foot fluorescent ceiling mounted fixtures. These lighting fixtures appear to meet the minimum accepted requirements for luminance intensity.

The lighting in the railway machinery room consists of two-tube, four foot fluorescent ceiling mounted fixtures. These lighting fixtures appear to meet the minimum accepted requirements for luminance intensity.

Heating is provided within the control house through a thermostatically controlled forced air space heater. This heater provides adequate space heating for bridge operating personnel.

All lighting, receptacles and heaters were found to be in good condition and functioning properly during the time of this inspection.

Submarine Cables

The submarine cables consist of two cables that run from the near side of the bridge to the far side. The submarine cables enter the pier-mounted "Far Side Submarine Cable" junction box located on the southwest side of the highway bascule. See **Photo E28**

The submarine cables provide service power to the traffic gates and navigation lights for the far side of the bridge. It appears that, based upon the meg Ohm readings, the insulation resistance levels are acceptable, but are showing signs of degradation.

Specific deficiencies relating to the submarine cables were recorded as follows:

- The "Far Side Submarine Cable" junction box is in poor condition.
- The conduit LB feeding the "Far Side Submarine Cable" junction box is missing cover exposing conductors and junction box to environment.

General Electrical Installation

The majority of the conduit on the bridge is rigid galvanized steel that was installed as part of an electrical rehabilitation or has been installed as needed over time. The conduit feeding the electrical panels in both the highway and railway machinery rooms was found to be in good condition and had recently been painted.

Limit switches in the bridge span areas are installed with flexible conduit to facilitate adjustments of the devices as needed. Overall, the conduit throughout the exposed areas of the spans was observed to be in poor condition.

Junction boxes located throughout the highway and railway bascules spans are in fair to poor condition.

SO cable is provided at various areas of the highway and railway bascules spans where movement occurs during bridge operation. The majority of

these cables and their associated boxes are in poor condition. See **Photo E29**

The following specific deficiencies regarding the general electrical installation were recorded as follows. Referenced photos are in **Appendix G**.

- A tee conduit body is missing its cover adjacent to the highway starter panels located in the highway machinery room. See **Photo E30**
- Solid conductors are used for power circuits at the distribution equipment located on the south and east walls of the highway machinery room. See **Photo E31**. Accepted bridge building codes explicitly prohibit the use of solid conductors for bridge power and control wiring.
- Solid conductors are used for power circuits at the distribution equipment located on the south and east walls of the railway machinery room. See **Photo E32**. Accepted bridge building codes explicitly prohibit the use of solid conductors for bridge power and control wiring.
- Solid conductors are used for power circuits at the distribution equipment located on the north wall of the control house. See **Photo E33**. Accepted bridge building codes explicitly prohibit the use of solid conductors for bridge power and control wiring.

Summary of Electrical Inspection

The electrical condition of the Johnson Street Bridge is in overall fair to poor condition at both the highway and railway structures. The equipment has degraded over the years and has not had any recent significant upgrades or rehabilitations. As noted in the report, much of the equipment is obsolete and nearing the end of its useful service life. Failures of various electrical components of the bridge are likely in the short term and will result in delays of the needed repairs due to inaccessibility of replacement parts.

Most of the major systems of the bridge lack redundancy. During this inspection, failure of a span drive motor feeder rendered the movable span temporarily inoperable. At a minimum, this bridge should be provided with redundancy for the span drive machinery and electrical service.

This report details recommendations for noted deficiencies. The recommendations should be performed to preserve the basic operational reliability of the bridge as well as restoring the bridge to a higher level of operational reliability and to bring the electrical systems up to current applicable codes.

4. SEISMIC VULNERABILITY OF EXISTING BRIDGE

The Johnson Street Bridge was designed in accordance to the Specification of the Engineering Institute of Canada for Highway Bridges, 1918. Due to the level of knowledge of seismic engineering at that time, seismic loading was most likely not accounted for in the design. As such, the bridge has a number of obvious seismic deficiencies which will be discussed in this section.

According to the Canadian Highway Bridge Design Code (CSA-S6-06), seismic load on a particular structure is characterized by considering the required performance of a bridge, zonal acceleration ratio of the site, soil condition of the site, and vibration frequencies of the bridge.

4.1 Seismic Performance

As part of the study, the City requested that the bridge be evaluated in accordance with the three categories of bridge listed in CSA-S6-06. These categories are:

- Other Bridge – A bridge that is designed/retrofitted to not collapse in an earthquake with a 10% chance of exceedance in 50 years;
- Emergency Route Bridge – A bridge that is designed/retrofitted to be repairable without loss of service for emergency vehicles after an earthquake with a 10% chance of exceedance in 50 years and not collapse in an occurrence with a 5% chance of exceedance in 50 years; and
- Lifeline Bridge – A bridge that is designed/retrofitted to have no loss of service after an earthquake with a 10% exceedance in 50 years, be repairable without loss of service for emergency vehicles after an occurrence with a 5% chance of exceedance in 50 years, and to not collapse in an earthquake with a 2% chance of exceedance in 50 years.

In addition, analysis was carried out to determine what level of earthquake the bridge, in its current configuration, can resist without significant damage. As will be discussed in Section 4.3, this corresponds to a seismic event with a 35% chance of exceedance in 50 years.

There is no correlation between the specific return period earthquakes that we use in this report and Richter scale magnitude. Seismologists use a scale called Moment Magnitude to describe a fault's potential. Engineers use the ground motion data such as acceleration to design structures accordingly. Ground motions are characterized quantitatively in terms of peak ground accelerations (PGA) relative to the acceleration of gravity, g.

To determine how a fault will affect a given area, equations were developed to calculate the PGA at a site or a region. These equations are called attenuation equations and there are several of them. They consider the specific soil types to 'translate' energy to acceleration. Other factors that affect the PGA are the depth of the earthquake, the distance from the earthquake at which the PGA is measured, and the type of fault that the earthquake originated from. Because of all these factors, the scales of Richter and PGA cannot be directly converted.

However, historical examples of earthquakes can be shown to give some idea of how Richter magnitudes relate to measured PGAs at the ground surface. For example, a PGA of 0.18g was measured in different magnitudes: Peru Jan. 5, 1974 (M=6.6), Montenegro April 15, 1979 (M=6.9), Mexico Sept 19, 1985 (M=8), Romania March 4, 1977 (M=7.5).

4.2 Site Specific Seismic Input

The magnitude of an earthquake is defined by a number of parameters including the peak horizontal ground acceleration as measured at bed rock or till. The geographic Cartesian coordinates of Johnson Street Bridge is 48.428° North and 123.3718° West. This results in a peak horizontal ground acceleration of 0.336g at firm ground (NBCC 2005 soil class C) for 10% probability of exceedance in 50 years. This corresponds to a Zonal Acceleration Ratio of 0.4 in accordance with CSA-S6-06.

Site specific peak ground acceleration values were obtained from Geological Survey of Canada according to the site coordinates.

Three earthquake spectra of various probabilities were provided for the site:

- 10% probability of exceedance in 50 years (return period of 475 years).
- 5% probability of exceedance in 50 years (return period of 1000 years).
- 2% probability of exceedance in 50 years (return period of 2500 years).

*The response spectra defined by the above parameters are shown in **Figure 4.1**.*

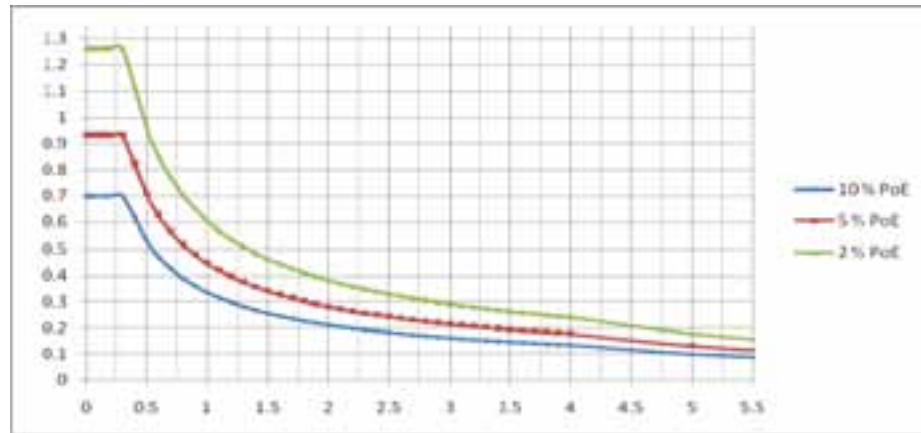


Figure 4.1 – Response Spectra at the Johnson Street Bridge Site for Three Seismic Events

4.3 Structural Modeling and Analysis

A 3-D finite element computer model was created to investigate the seismic vulnerability of the Johnson Street Bridge in its existing configuration. The model includes all structural components on the two approach spans, bascule span, and the counterweight span for both the highway bridge and railway bridge. An outline of the computer model is shown in **Figure 4.2**. Seismic analysis was carried out in accordance with:

- CAN/CSA S6-06 (CHBDC).
- BC MoT supplement to CHBDC S6-06.
- BC MoT Seismic Retrofit Design Criteria.

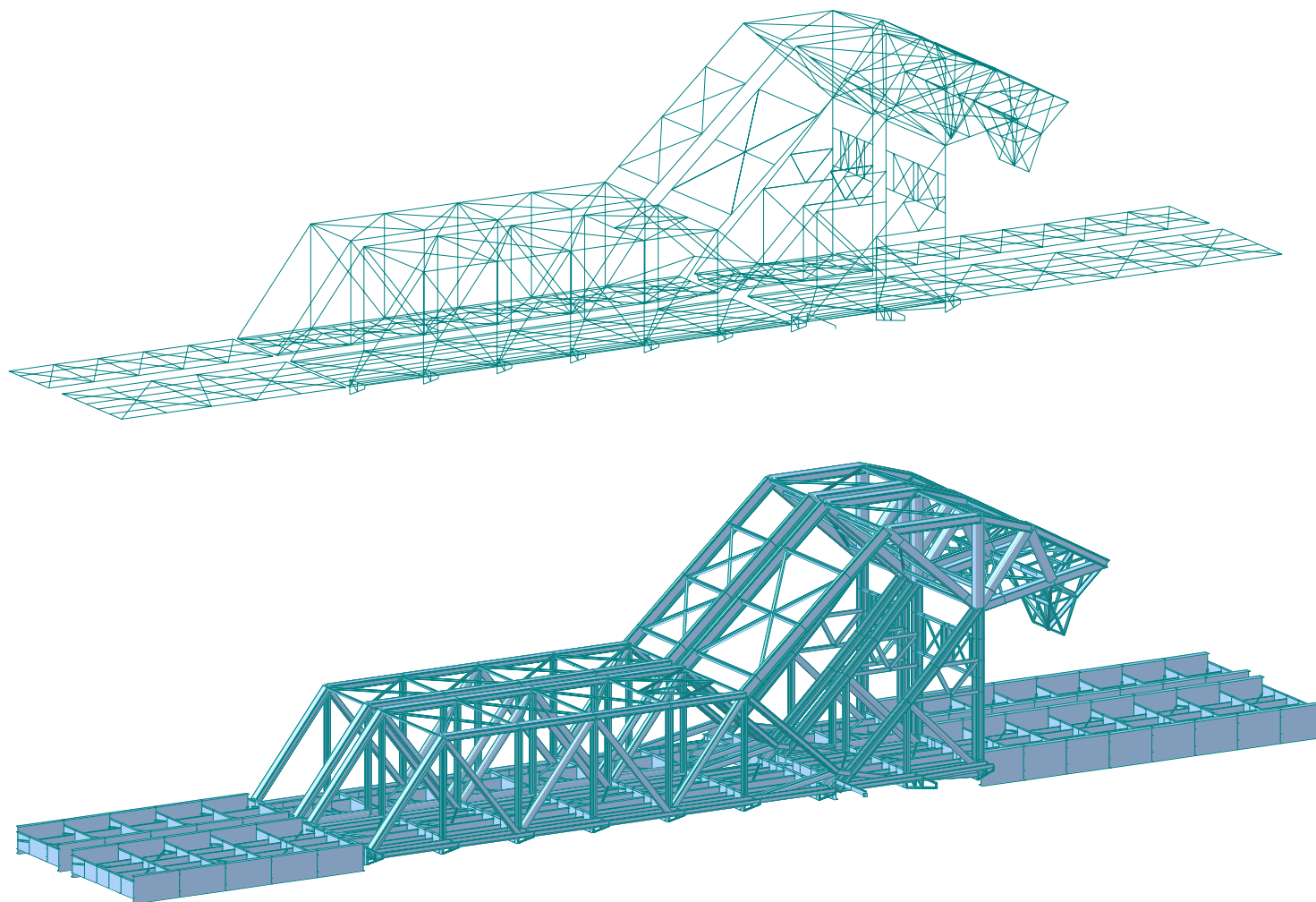


Figure 4.2 –Johnson Street Bridge Finite Element Model

Modeling of the Bridge was carried out using MIDAS and included:

- Definition of the geometry of the structural system and careful consideration of the support elevations, as well as the correct weight of the masses and their centroids.
- Definition of section properties, taking into account their behavior during major earthquakes in both elastic and plastic phases.
- General and rigid links simulating the member section properties laterally or longitudinally, to achieve the proper stiffness of the whole bridge.
- Benefits from soil-structure interaction have been discounted to account for the possibility of liquefaction down to bedrock.

Three kinds of analysis were considered in the analysis – static, modal (free vibration eigenvalue analysis) and dynamic response spectra analysis – to investigate the seismic performance of the bridge.

4.5 Seismic Capacity of Existing Bridge (Basic “Do Nothing” Case)

Analysis was carried out to determine the maximum earthquake that the bridge can resist in its existing configuration. This analysis was used at a global level to determine demand/capacity ratios. This analysis indicates that the bridge, in its current configuration can withstand a seismic event with a probability of exceedance of 35% in 50 years. Failure under this event will occur by failure of the wooden piles leading to unstable structural system. The peak ground acceleration associated with this event is about 0.18 g. This is about half of the peak rock acceleration of an event with intensity having 10% probability of exceedance in 50 years.

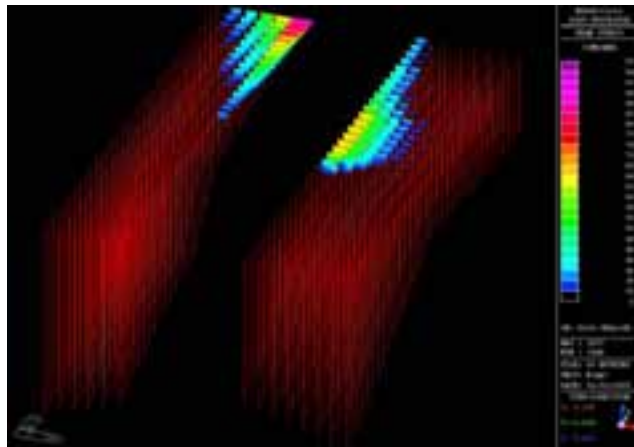


Figure 4.3 – Stresses in the Timber Pile that Exceeds the Ultimate Capacity

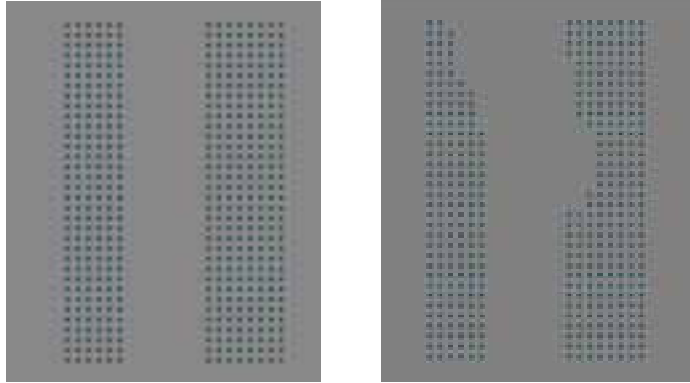


Figure 4.4 – Critical Case, the Timber Piles that Might be Lost During 35% Poe Earthquake

Under the minimum seismic event for design (10% exceedance in 50 years), bracing and connecting elements in the counterweight tower of the existing bridge will be severely overstressed while the bascule span will not be significantly affected. This condition could lead to collapse of the counterweight tower.

Given the age of the bridge and the large number of unknowns in the existing structure it should be pointed out that the bridge could fail during a lesser seismic event given the following:

- The bridge was not designed to any seismic standard;
- Reinforcement details in Piers P2 and P3 could result in brittle failure;
- Laced, built-up, riveted members perform poorly under seismic loads (see **Figure 4.5** that shows the result of tests carried out for the Bay Bridge seismic retrofit);



Figure 4.5 – Photographs of Failures in Laced Members Caused by Cyclic Loads

- Riveted gusset plates may not be able to transfer load during an earthquake;
- Brittle behavior of corroded steel/rivets under cyclic loads;

- Some crack injections and jacketing retrofits were made to the piers in the past due to erosion damage. These retrofits likely define planes of weakness that could affect the lateral strength of the piers; and
- Pounding between road and railway bridges will occur and cause damage.

The deflected shape of the bridge in its current configuration under seismic loads is shown in **Figure 4.6**. Because of the weak bracing used in the structure, much of the bridge's strength is not engaged and consequently the bascule span and the counterweight tower act as discrete units rather than as a continuum.

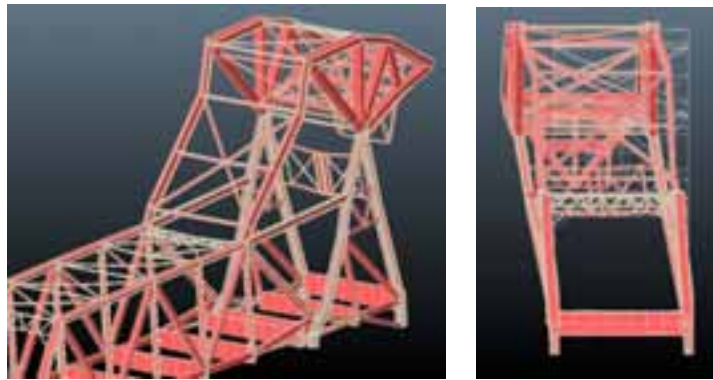


Figure 4.6 - Deflected Shape of Existing Bridge Under Seismic Loading

It can be seen from the **Figure 4.7** that the main bascule trusses are not heavily stressed in a seismic event whereas the cross bracing in the counterweight tower is stressed well beyond failure.

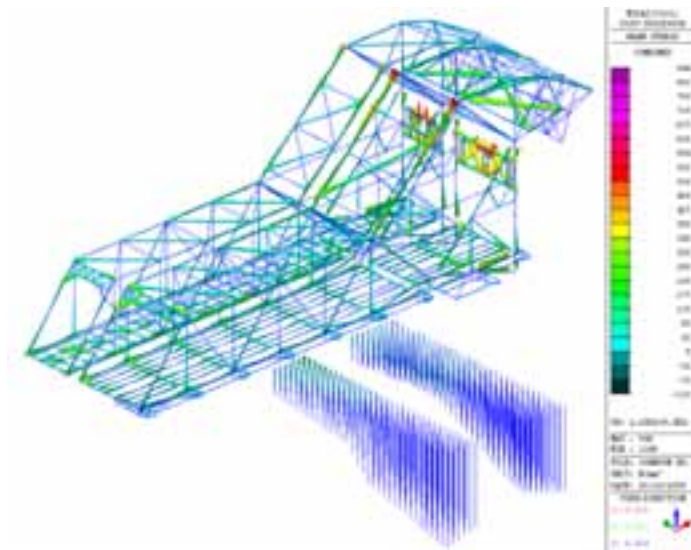


Figure 4.7 - Stresses in the Existing Bridge Under Seismic Loading

The road and railway portions of the Bridge may pound against each other during a seismic event leading to considerable damage. Bracing should be added to let the two bridges work together laterally (coupling). Although such a retrofit would make the entire bridge stiffer and therefore more vulnerable to seismic loads, it is necessary to control the structural behavior during major earthquakes and minimize damage caused by pounding.

5. SEISMIC RETROFIT STRATEGIES

In addition to the basic Do-Nothing case, 5 retrofit strategies have been developed and examined. These strategies are based on either reducing the distribution of mass in the structure or by changing the dynamic characteristics of the existing structure. The best alternative will provide a well-defined load path with predetermined plastic hinge locations and utilize redundancy. The seismic vulnerability of the approach embankments and related retaining walls has not been considered in this study.

5.1 Common Elements of All Retrofit Alternatives

It is noted that given the vulnerability and unknowns associated with the existing foundations, underpinning of the existing foundations would be required to some extent for all alternatives. Other measures common to all alternatives include:

- Replacement of lacing with cover plates in some built-up members;
- Installation of bracing to prevent pounding between the highway and roadway bridge;
- Extension of bearing seats or provision of restrainers for the approach spans and at the rest pier end of the bascule span;
- Improvement of shear capacity of cross beams at Piers P2 and P3;
- Modification of gusset plates to ensure strong connections between truss members;
- Foundation underpinning and Pier Jacketing.

5.2 Retrofit Options

The “do-nothing” option is used as a base case. The 5 retrofit options considered to improve the base case are:

- Option 1 (mass reduction): Reduction of the counterweight mass and replacement of the existing electrical/mechanical system to provide a more powerful drive motor;
- Option 2 (mass relocation): Replacement of the existing counterweights with new counterweights located in a cavity under the deck. This would also require replacement of the existing electrical/mechanical system.
- Option 3 (structural strengthening): Improvement of the seismic performance by changing the structural behavior and mode shapes;
- Option 4 (seismic isolation): Reduction of seismic forces applied to the structure during earthquake by shifting the fundamental periods and increasing the damping which in turn reduces the response spectral acceleration.

- Alternative 5 (Substructure upgrade): By increasing the flexibility of the bridge foundations, the period of the structure will be increased and lower seismic forces will be attracted.

Do Nothing

As discussed in **Section 4 and Appendix D**, the bridge can withstand an earthquake with a probability of exceedance of 35% in 50 years. Based on the review of historical earthquakes given in **Appendix H**, it is not likely that an earthquake of this magnitude has occurred since the bridge was constructed although there was a 50:50 chance it would have happened in the 85 year life of the bridge.

Option 1 – Reduction of Counterweight Mass

In this alternative, the existing bridge was considered without any modification except a reduction in the counterweight mass. The mass of the counterweight was reduced by 60% to represent the weight of the counterweights with all the concrete blocks taken out. In this alternative the electrical/mechanical system of the bridge would need to be replaced to be substantially more powerful. Operational costs would also increase significantly.

Option 2 – Relocation of Counterweight

The height at which the counterweight is located above the foundations increases the load effects caused by the acceleration of this considerable mass. By relocating the counterweight in a cavity below the deck, the seismic effects caused by this mass would be greatly reduced. Significant, if not complete, re-building of the electrical/mechanical system would be required for this alternative, as well as modifications to the approach spans. This approach was taken to retrofit the Fourth Street Bridge in San Francisco which is a similar bascule also designed by Joseph Strauss in the early 1900's. We understand that the Fourth Street Bridge Retrofit cost in the order of \$34M.

Option 3 – Strengthening and Energy Dissipating Bracing

In this alternative, the existing lateral bracing in the counterweight tower is replaced with eccentrically braced frames to improve energy dissipation during an earthquake and as such protect other elements in the bridge. Eccentrically braced frames possess considerable stiffness in the elastic range and have demonstrated an excellent ductility capacity in the inelastic range. The high elastic stiffness provided by the braces, and the high ductility capacity is achieved by transmitting one brace force to another brace or to column through shear in a short beam segment designed by the dimension "e" in **Figure 5.1**.

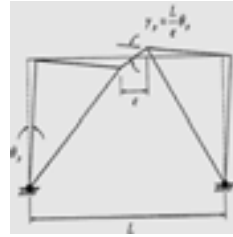


Figure 5.1 – Energy dissipating eccentric bracing

The bracing modifications proposed for this alternative are shown below in **Figure 5.2**.

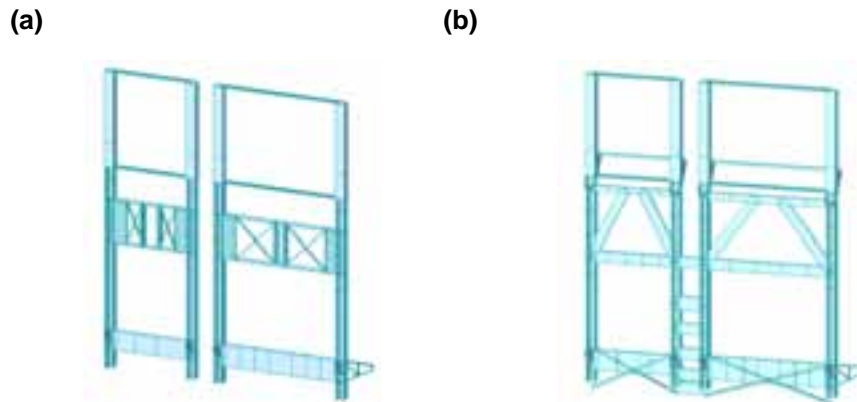


Figure 5.2 – (a) existing bracing; (b) energy dissipating bracing

In addition to the proposed eccentrically braced frames, a number of other bracing modifications and strengthening measures are required. In total, approximately 60 ton of additional steel is required for this alternative if the bridge is considered as an emergency route and approximately 140 ton if it is considered as lifeline structure.

The effect of the eccentrically braced frames is illustrated in **Figure 5.3**. **Figure 5.3(a)** shows the stress distribution in the bridge at the commencement of a large earthquake. It can be seen from the figure that the stiffness of the new bracing has attracted considerable stress and consequently will undergo deformation. This deformation creates a plastic hinge that dissipates energy and results in the redistribution of stress through the bridge as shown in **Figure 5.3(b)**. This redistribution results in an overall reduction in stresses throughout the bridge. The creation of the plastic hinge will not compromise the stability of the bridge.

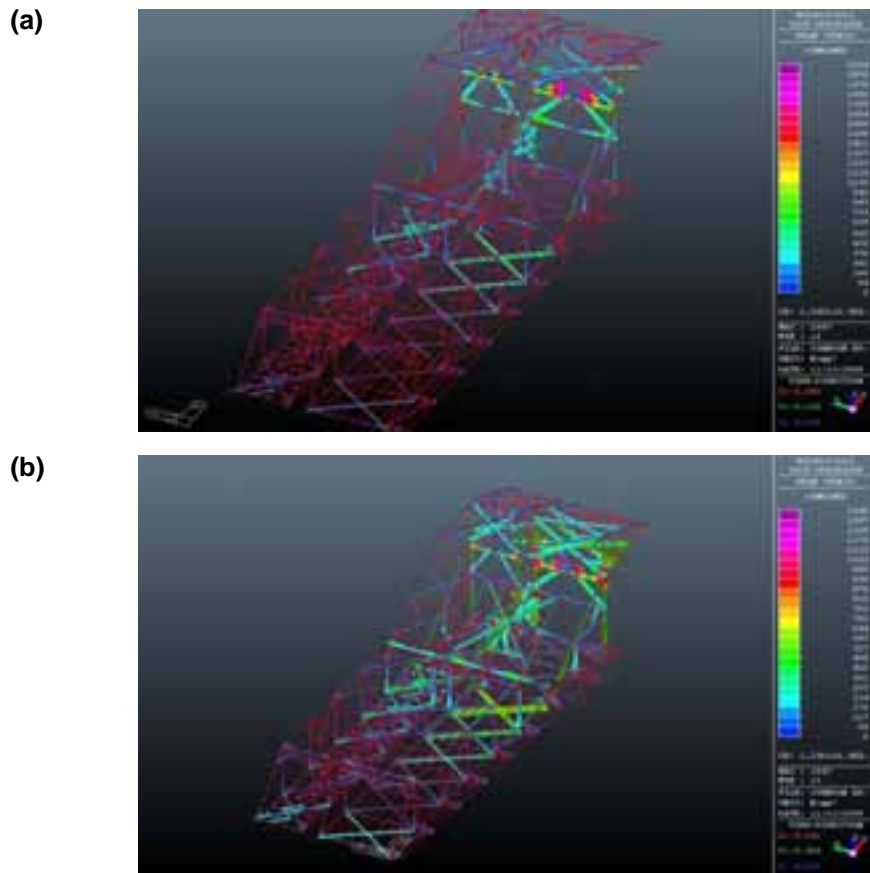


Figure 5.3 - Reduction and redistribution of the stresses after forming the plastic hinges in the horizontal member of the eccentrically braced frame (a) stresses before hinge formation; (b) stresses after hinge formation. [Note: stresses indicated in the legends of the figures should be divided by 3 to account for ductility.]

Option 4 - Seismic Isolation

Seismic isolation of the counterweight tower was investigated. To ensure that isolation is effective, isolation would also need to be provided at the rest pier. Different effective stiffnesses of isolation systems were considered in this regard. Considering that there are dynamic loads caused by the operation of the lift span, complete isolation cannot be provided and there must be sufficient friction in the isolating system to ensure stability during operation of the bridge under normal conditions.

Base isolation of the counterweight tower results in better engagement of the bridge as a whole with a subsequent lowering of stresses by redistribution. The maximum permissible displacements of the whole system played a significant role to choose the stiffness of the isolators, taking into account the displacement capacity of the isolator in each support. Illustrative

displacements and stresses in the bridge after the completion of this alternative are shown in **Figures 5.4** and **5.5**, respectively.

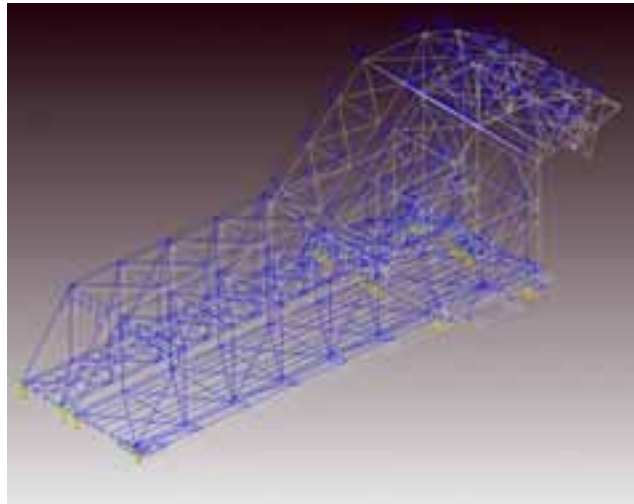
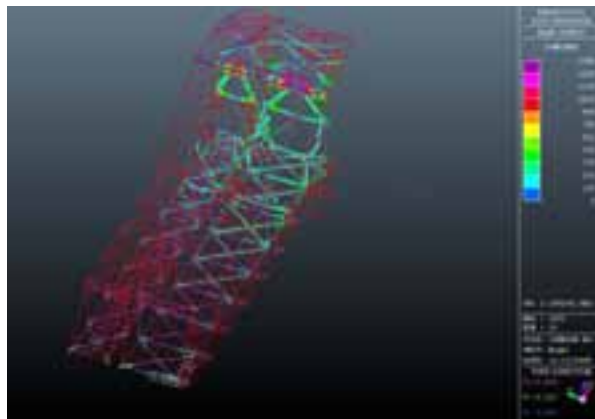


Figure 5.4 – Bridge displacements with seismic isolation bearings

(a)



(b)

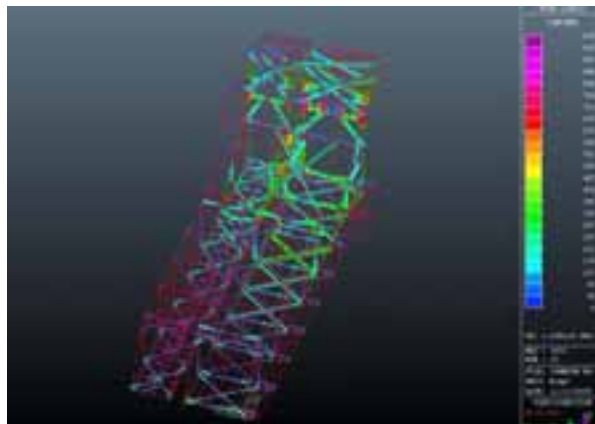
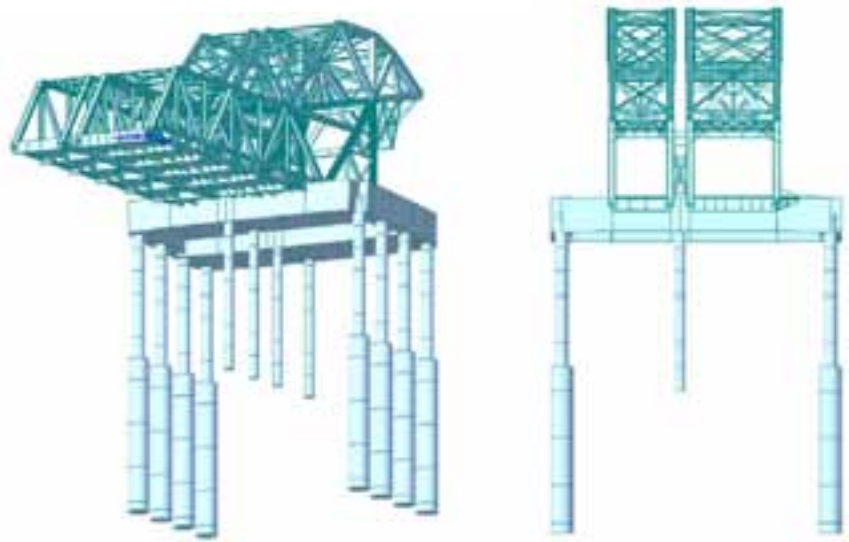


Figure 5.5 – Alternative 4 stresses at (a) commencement of earthquake; (b) after formation of plastic hinges. [Note: stresses indicated in the legends of the figures should be divided by 3 to account for ductility].

Option 5 – Substructure Upgrade

The exact configuration of the existing bridge foundations are not known accurately but are shown on the drawings as comprising timber piles driven to the top of the bedrock. Because of the unknowns surrounding the foundation configuration and its apparent weakness, consideration has been given to augmenting the existing foundations with a known pile arrangement that will provide predictable and adequate performance during an earthquake.

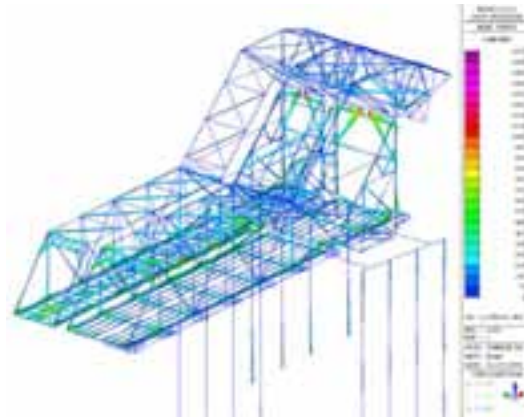


The installation of new shafts adjacent to the existing bridge substructure to carry all the service loads from the bridge is feasible. Two new rows beside the bridge and one row in the middle would carry the entire weight of the bridge. The new shafts would be dimensioned to shift the period of the structure sufficiently to allow a significant reduction in seismic loads. The location of plastic hinges in the shafts would be controlled by changing the diameter and reinforcement of the shaft near the mud line.

This option is shown in more detail on drawing SK-1 at the end of Section 5.

The stresses expected from this retrofit alternative are shown in **Figure 5.6**.

(a)



(b)

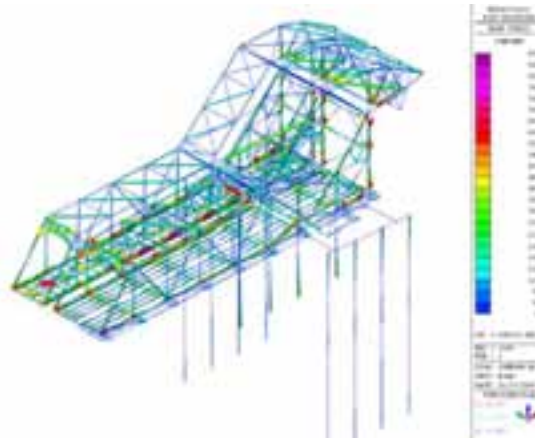


Figure 5.6 - Alternative 5 stresses (a) commencement of earthquake; (b) after formation of plastic hinges.

5.3 Comparison of Retrofit Strategies

The Do-Nothing and the five retrofit concepts are compared in **Table 5.5** on the basis of their effectiveness in reducing seismic risk, potential for traffic disruption, economic disruption, impact on the historic character of the site, and cost. Cost comparison has been in a qualitative manner. Quantitative costs have been determined for the preferred retrofit option and are presented in Section 8.

Table 5.5 – Comparison of Seismic Retrofit Alternatives

Strategy	Seismic Vulnerability	Impact on Traffic during Construction	Potential for Economic Disruption after moderate earthquake	Impact on Historic Character of the Site	Effort
Do Nothing	XXXXX	none	XXXX	none	\$
#1 Mass Reduction Only	XXXX	XX	XXX	none	\$\$\$
#2 Mass Relocation Only	XXX	XXX	X	XX	\$\$\$\$
#3 Strengthening Only	XXX	X	XX	X	\$\$
#4 Seismic Isolation Only	XX	X	X	none	\$\$
#5 Substructure Upgrade Only	X	X	X	X	\$\$\$\$
#1 & Underpinning	X	XXX	X	XX	\$\$\$\$\$
#2 & Underpinning	X	XXX	X	XXX	\$\$\$\$\$
#3 & Underpinning	X	XXX	X	XXX	\$\$\$\$\$
#4 & Underpinning	X	XXX	X	XXX	\$\$\$\$\$

Notes: X = negative impact; \$ indicates relative cost.

5.4 Recommended Seismic Retrofit Approach

The best strategy will provide a well-defined load path with predetermined plastic hinge locations and utilize redundancy. Based on the above discussions the following retrofit is suggested for further consideration and costing:

- Installation of energy dissipation bracing and strengthening of selected members (retrofit Option 3);
- Installation of new piles/substructure to relieve the existing foundations/substructure (Retrofit Option 5);
- Replacement of lacing with cover plates in built-up members;
- Installation of bracing to prevent pounding between the highway and roadway bridge;
- Extension of bearing seats or provision of restrainers for the approach spans and at the rest pier end of the bascule span;
- Improvement of shear capacity of cross beams at Piers P2 and P3;
- Modification of gusset plates to ensure ductile connections between truss members.

Given the potential for significant earthquakes in Victoria (the highest of any Canadian City) the Do Nothing option has considerable risk, particularly when considering the volume of traffic that uses the bridge daily, and as such is not recommended. The risk includes the potential for loss of life if the counterweight tower were to collapse and the potential for negative economic impacts if the bridge was closed after a seismic event. This risk is accentuated by the fact that to date, the bridge has experienced relatively low earthquakes as compared to the design earthquake recommended by the MoT supplement to the 2006 edition of the Canadian Highway Bridge Design Code.

Table 5.6 summarizes the effectiveness of various retrofit strategies. It can be seen from this table that substructure upgrading is essential. Retrofitting to emergency route or lifeline status, however, is probably not required given that emergency services, in accordance with discussions with the City, do not need to use the bridge. Similarly, lifeline status is typically required for extremely expensive infrastructure and this status is therefore difficult to justify for the Johnson Street Bridge.

Table 5.6 – Efficacy of Various Retrofit Strategies

Repair Strategies	Classifications of Bridge				
	Highest Risk	"Other"	Emergency Route	Lifeline	
Do Nothing	1				
Mass Reduction + Underpinning & Jacketing	2		1		
Mass Relocation + Underpinning & Jacketing	REJECTED STRATEGY (SEE NOTE)			1	
Structural Strengthening + Underpinning & Jacketing			2	1	
Seismic Isolation + Underpinning & Jacketing				2	
Substructure Upgrade Alone (Caissons)				2	
Probability of Exceedance Within 50 Years:	40%	10%	5%	2%	

Legend:

- 1 Significant Failure of Piles
- 2 Significant Superstructure Damage

NOTE: Reasons for rejection of this option include impracticality, high cost, and traffic disruption

The estimated cost for the seismic retrofit to meet CSA S6-06 as an “Other” bridge is given in **Table 5.7**.

Table 5.7 – Class “C” Estimate for Johnson Street Seismic Retrofit

Seismic Retrofit Costs	\$M
Energy Dissipating Bracing and cover plating	1.25
Substructure Upgrading	6.55
Pier Jacketing	1.0
Modification of Gusset Plates	0.25
Bearing seat extensions, restrainers, hold downs	0.25
Sub-total, Seismic Retrofit	9.30
Mobilization (5%)	0.47
Engineering (20%)	1.86
Contingency (40%)	3.72
Total, Seismic Retrofit	15.35

Notes to Cost Estimate:

1. Costs in 2008 Canadian dollars;
2. Costs based on preliminary studies and conceptual level work;
3. Costs assume a competitive bidding process with at least 3 bidders;
4. Geotechnical investigations were not carried out to determine costs;
5. Costs are indicative and not for budgeting.

6. REHABILITATION PROGRAM

Based on the results of **Sections 2, 3, 4** and **5** a rehabilitation program has been established with the intention of keeping the existing bridge in service for an additional 40 years. This program has been examined and detailed in order to compare with a replacement option.

The rehabilitation program would need to be implemented in the next 3 years to prevent further degradation of the bridge which would result in more expensive repairs and to mitigate seismic risk. As will be discussed in **Section 8** this rehabilitation program forms part of a more extensive maintenance program that would need to be implemented over the next 40 years to protect the integrity of the structure.

6.1 Structural Repairs

6.1.1 Re-Coating of Structural Steel

A full paint coating restoration should be undertaken. In advance of this, the details in the bridge that allow the accumulation of debris and moisture need to be modified.

Corroded areas should be sandblasted of all corrosion and mill scale and prepared to an appropriate SSPC finish. Horizontal gusset plates on both the railway bridge and the roadway bridge should be dealt with promptly. Heavily corroded areas once cleaned should be inspected again to document any deficiencies like section loss or potential fatigue cracking hidden by rust debris. Replaced rivets and gusset plates should also be inspected again.

Cleaning of structural steel within built-up sections will be difficult. In addition, areas where pack rust has started are impossible to clean and as such arresting corrosion will be difficult to achieve. This process can be simplified to some extent by combining it with replacement of the lacing with cover plates as recommended for the seismic retrofit.

6.1.2 Repair of Corrosion Damaged Members

Corrosion was observed in many members resulting in a net loss in available section to carry the applied loads. Areas where corrosion was found included:

- Top flange of floor-beams due to roadway leakage;
- Bottom flanges of members due to debris build-up;
- Lacing bars; and
- Around rivets.

The following approach for correcting corrosion damage is suggested:

- Do Nothing [Section loss < 15%];
- Repair Member [Section loss < 40%]; and
- Replace Member [Section loss > 40%].

6.1.3 Seismic Retrofit

Based on the retrofit alternatives discussed in Section 5, the following retrofit is suggested for further consideration and costing:

- Widening of bearing seats and provision of shear keys for the approach spans;
- Provision of lateral restraints for the free end of the bascule span;
- Installation of energy dissipation bracing and strengthening of selected members;
- Provision of bracing between the counterweight towers of the roadway and railway bridges; and
- Installation of new piles/substructure to relieve the existing foundations/substructure.

6.2 Mechanical Repairs

The following recommendations are separated into three groups. Group 1 contains those items that should be investigated or repaired on a priority basis. Group 2 contains those items that should be conducted in the near term (3 to 6 months) to keep the existing machinery operating reliably. Group 3 contains those items that should be considered to keep the bridge operating reliably in the long term and to increase efficiency, improve safety and reduce maintenance requirements. Group 1 repairs should be undertaken regardless of the chosen strategy. Group 2 repairs should be done if rehabilitation or replacement is more than 3 years out.

Group 1 – Priority Repairs

Highway Bridge

- Bearing B12 - Conduct further inspection to determine the cause of the bearing oscillation. Tighten the bearing base bolts after eliminating the oscillation.
- Bearing B13 - Tighten the bearing base and cap bolts. Verify that the bearing alignment is acceptable after tightening the bolts.
- North Emergency Brake - Remove lubricant and light corrosion from the brake wheel and clean all lubricant from the brake pads or replace the pads.
- North Motor Brake – Remove excess lubricant from the brake. Remove lubricant from the brake wheel and clean all lubricant from the brake

pads or replace the pads. Reduce the lubrication frequency at the brake pads.

- South Motor Brake – Replace the missing cotter pin.
- Gear G2 – Secure the existing key or replace the key.
- Increase the frequency of lubrication at the operating strut pin connection bearings.
- Auxiliary Drive - Install a warning placard to instruct maintenance personnel to turn off the span drive motor disconnect prior to engaging the auxiliary drive.
- Auxiliary Drive - Remove excess lubricant from the hand brake. Remove lubricant from the brake wheel and clean all lubricant from the brake pads or replace the pads.
- Span Lock Machinery – Tighten the west nut for the upper rod end connection.
- Span Lock Machinery – Position the thrust collar on the hand crank shaft so that the hand crank cannot be engaged without loosening and moving the thrust collar. Install a warning placard to instruct maintenance personnel to turn of the motor disconnect prior to engaging the hand crank.
- Air Buffers – Tighten the loose bolts at the bottom bearing.

Railroad Bridge

- Bearing B10 - Tighten the bearing base bolts. Verify that the bearing alignment is acceptable after tightening the bolts.
- North Emergency Brake – Tighten the solenoid housing bolts.
- South Emergency Brake – Adjust the brake assembly so that the brake is providing torque when set.
- South Motor Brake - Remove lubricant from the brake wheel and clean all lubricant from the brake pads or replace the pads.
- Increase the frequency of lubrication at the operating strut pin connection bearings.
- Motor M2 – Repair the north motor bearing.
- Auxiliary Drive - Install a warning placard to instruct maintenance personnel to turn off the span drive motor disconnect prior to engaging the auxiliary drive.
- Span Lock Machinery – Replace the corroded bolt at bearing B1N.
- Span Lock Machinery – Position the thrust collar on the hand crank shaft so that the hand crank cannot be engaged without loosening and moving the thrust collar. Install a warning placard to instruct maintenance personnel to turn of the motor disconnect prior to engaging the hand crank.

Group 2 – Near Term Mechanical Repairs

Highway Bridge

- Bearing B3 - Remove the bearing cap for internal inspection of the bearing.
- Bearing B4 - Remove the bearing cap for internal inspection of the bearing.
- Bearings B14 - Clean and inspect the bearing mounting bolts. Paint the existing bolts if they are determined suitable for continued service.
- Bearings B16 - Clean and inspect the bearing mounting bolts. Paint the existing bolts if they are determined suitable for continued service.
- Replace the span drive brakes.
- Differential – Conduct strain gage testing to determine if the differential is operating satisfactorily. Alternately the differential can be disassembled for inspection of the wearing components.
- Gearset G7/G8 – Remove the hardened lubrication deposits from the roots of the gear teeth.
- Gearset G10/G11 – Remove the hardened lubrication deposits from the roots of the gear teeth.
- Gear G10 – Monitor the key to determine whether or not key is backing out. Repair or replace the key as required.
- Clean and paint the operating strut.
- Motor M2 – Repair the oil leak at the motor bearing.
- Auxiliary Drive Engine – Repair the oil leak.
- Clean the trunnion and link pin grease grooves and purge the bearings with fresh lubricant. Implement a procedure that requires this to be done on an annual basis.
- Remove at least one main and one counterweight trunnion bearing cap for internal inspection and bearing clearance measurements.
- Conduct ultrasonic inspection of the trunnions and link pins.
- Remove debris from in and around the main and counterweight trunnion bearings and the 1st and 2nd link pin bearings and clean and paint as required.
- Span Lock Machinery – Replace the machinery enclosure.
- Air Buffers – Install pressure gages so that air pressure can be monitored during bridge seating.
- Air Buffers – Remove excess lubricant from the buffers.
- Incorporate operating procedures and or modify the electrical control system to allow for consistent seating of the movable span.
- Clean and paint the live load supports and centering devices as required.

Railroad Bridge

- Bearings B4, B6, B8, B11, B12, B13, and B14 – Adjust the clearance to within the limits of an RC6 fit or to the extent permitted by the existing bearing alignment.
- Replace the span drive brakes.
- Differential – Conduct strain gage testing to determine if the differential is operating satisfactorily. Alternately the differential can be disassembled for inspection of the wearing components.
- Gear G4 and Gear G2 – Replace the keys.
- Gearset G7/G8 – Remove the accumulated lubricant at this gearset.
- Gear G11 – Remove the hardened lubrication deposits from the roots of the gear teeth.
- Clean and paint the operating strut guide assembly
- Clean and paint the operating strut.
- Motor M1 – Replace the cover for the oil level check fitting. Repair the leak at the motor bearings.
- Auxiliary Drive Engine – Repair the oil leak.
- Clean the trunnion and link pin grease grooves and purge the bearings with fresh lubricant. Implement a procedure that requires this to be done on an annual basis.
- Remove at least one main and one counterweight trunnion bearing cap for internal inspection and bearing clearance measurements.
- Conduct ultrasonic inspection of the trunnions and link pins.
- Remove debris from in and around the main and counterweight trunnion bearings and the 1st and 2nd link pin bearings and clean and paint as required.
- Span Lock Machinery – Replace the machinery enclosure.
- Air Buffers – Install pressure gages so that air pressure can be monitored during bridge seating.
- Air Buffers – Remove excess lubricant from the buffers.
- Incorporate operating procedures and or modify the electrical control system to allow for consistent seating of the movable span.
- Tighten or replace the loose anchor bolts at the south live load strike plate.
- Clean and paint the live load supports and centering devices as required.

Group 3 – Mechanical Rehabilitation

Highway Bridge

- Rehabilitate the operating strut guide roller assemblies.
- Replace the span drive machinery that is located in the machinery house.
- Replace the span locks.

Railroad Bridge

- Replace the span drive machinery that is located in the machinery house.
- Replace the span locks.

6.3 Electrical Repairs

The following recommendations are separated into three groups. Group 1 contains those items that should be investigated or repaired on a priority basis. Group 2 contains those items that should be conducted in the near term (3 to 6 months) to keep the existing equipment operating reliably. Group 3 contains those items that should be considered to keep the bridge operating reliably in the long term, to improve safety, and reduce maintenance requirements. Group 1 repairs should be undertaken regardless of the chosen strategy. Group 2 repairs should be done if rehabilitation or replacement is more than 3 years out.

Group 1 – Priority Electrical Repairs

Highway Bridge

- Remove flammable materials from within the main control desk and wall-mounted control panel.
- Replace covers for conduit bodes as noted throughout the report.

Railroad Bridge

- Replace covers for conduit bodes as noted throughout the report.
- Isolate the north and south emergency brakes. Retest insulation resistance at each brake. If measurements continue to be below 5 meg Ohms then replace the solenoid.

Group 2 – Near Term Electrical Repairs

Highway Bridge

- Provide disconnecting means for in-coming main service at the bridge control house to facilitate maintenance operations.
- Provide proper color coding for drive motor leads.
- Provide proper color coding for all brake power leads.

- Install brake position limit switches for motor and emergency brakes. Interconnect limit switches with control system to prevent operation without releasing brake.
- Replace field-mounted position limit switches for locks and span position with limit switches suitable for the application.
- Provide a fender-mounted clearance gauge and light for the right side fender (as viewed while approaching the channel).

Railroad Bridge

- Retest motor insulation resistance for the north and south railway span drive motors. If measurements continue to produce low insulation readings, then rewind motors.
- Provide proper color coding for drive motor leads.
- Provide proper color coding for all brake power leads.
- Replace aging motor and emergency brake limit switches.
- Replace field-mounted position limit switches for locks and span position with limit switches suitable for the application.
- Provide a fender-mounted clearance gauge and light for the right side fender (as viewed while approaching the channel).

Group 3 – Electrical Rehabilitation*Highway Bridge*

- Provide emergency generator for bridge operation during the loss of main service. Provide automatic transfer switch for monitoring main and emergency service and for providing proper transfer between these two sources. This generator should be configured to provide power to both the highway and railway bridges.
- Replace existing power distribution equipment with a modern integrated system (Motor Control Center) meeting all current applicable codes. This equipment will contain all motor control equipment, lighting panels, and power panels.
- Provide integrated span drive system incorporating flux vector technology variable speed drives and matched, vector-duty induction motors. The system should utilize two motors and drives designed to provide redundancy in the event of failure of any single motor or drive. Provide dedicated disconnect switches for each span drive motor.
- Provide a corrosion resistant free standing control desk that would incorporate both existing control panels. The new control desk would be equipped with emergency stop pushbutton, interlocking relays, keyed bypass switches for traffic gates, span locks and brakes, LED status indicators, data line display, amp meters, selector switches, pushbuttons, and console lamp. The desk would provide control for

both the highway and railway bridges.

- Provide interlocking bridge control system. The system should utilize relay and PLC logic to fully interlock all bridge operations and to allow operation only in a pre-defined sequence. The system should utilize bypass systems and redundancy within the design to increase reliability.
- Provide new traffic, barrier, and pedestrian gates meeting current applicable codes.
- Install new submarine cables and dedicated junction boxes.
- Provide a new electrical installation for the complete bridge. The scope of this item would include new conduit, conductors, boxes, and supports.

Railroad Bridge

- Replace existing power distribution equipment with a modern integrated system (Motor Control Center) meeting all current applicable codes. This equipment will contain all motor control equipment, lighting panels, and power panels.
- Provide integrated span drive system incorporating flux vector technology variable speed drives and matched, vector-duty induction motors. The system should utilize two motors and drives designed to provide redundancy in the event of failure of any single motor or drive. Provide dedicated disconnect switches for each span drive motor.
- Provide interlocking bridge control system. The system should utilize relay and PLC logic to fully interlock all bridge operations and to allow operation only in a pre-defined sequence. The system should utilize bypass systems and redundancy within the design to increase reliability.
- Provide new pedestrian gates meeting current applicable codes.
- Provide a new electrical installation for the complete bridge. The scope of this item would include new conduit, conductors, boxes, and supports.

6.4 Estimated Costs of Bridge Rehabilitation Program

Rehabilitation Work	\$M
Structural	
Steel Repairs	0.50
Re-Coating	2.50
Seismic Retrofit	9.30
Electrical	1.60
Mechanical	0.40
Sub-total, Rehabilitation	14.30
Engineering (20%)	2.86
Mobilization (5%)	0.72
Contingency (40%)	5.72
Total Rehabilitation	23.60

Costs are exclusive of property, utilities, escalation, and only reflect capital costs.

7. REPLACEMENT CONSIDERATIONS

Replacement of the Johnson Street Bridge would need to consider a number of significant requirements in addition to strictly structural engineering parameters. These would include:

- The Bridge is considered an icon in Victoria and as such, consideration would need to be given to replacement with an equally remarkable structure;
- Staging of the replacement would likely require closure of the bridge to vehicles for a period of time. Such closure would likely not be acceptable to marine traffic;
- Vertical clearance under the bridge cannot be increased significantly and as such a new moveable bridge would be required to ensure continued access for marine traffic;
- The new bridge would need to carry 2 lanes of traffic, the commuter train and improved sidewalk capacity. As such the structure would be in the order of 20 m wide; and
- A total bridge length of about 120 m would be required.

A replacement bridge on the existing alignment would likely cost between \$18M and \$24M (\$2008) excluding engineering, property, utilities, permits, cost of traffic disruption, traffic management or contingencies.

To reduce replacement costs, the road/rail alignment of the bridge could be modified by relocating the existing train station to the west side of the crossing. This would eliminate the need to provide a crossing for the railway, allow for a better alignment on Johnson Street and free up land currently used by Johnson Street for other purposes or sale, see **Figure 7.1**. This alignment would reduce the bridge width from 20m to 15m and with proper staging allow construction of the new bridge with reduced disruption to Johnson Street traffic.

A replacement bridge on the alternate alignment would likely cost between \$15M and \$20M (\$2008) excluding property, utilities, engineering, permits, cost of traffic disruption, traffic management or contingencies.

The estimated cost of replacing the Johnson Street Bridge using the existing alignment is given in **Table 7.1**. The cost of the anticipated bridge construction reflects an average of the values noted above.



Figure 7.1 – Alternate alignment for Johnson Street Bridge Replacement
[red line indicates railway line, red circle indicates relocated train station, black line indicates Johnson Street, North is up].

Table 7.1 – Estimated cost of replacement of Johnson Street Bridge – Existing Alignment

Bridge Replacement Costs (Road and Rail)	\$M
Approach Roads	1.00
Traffic Management	0.50
New Moveable Bridge (120m x 20m x \$8,750/m ²)	21.00
Demolition and Removal of Existing Bridge	1.00
Sub-total, Bridge Replacement	23.50
Mobilization (5%)	1.18
Engineering (15%)	3.53
Contingency (30%)	7.05
Total, Bridge Replacement	35.26

Notes to Cost Estimate:

1. Costs in 2008 Canadian dollars;
2. Costs based on preliminary studies and conceptual level work;
3. Costs assume a competitive bidding process with at least 3 bidders;
4. Geotechnical investigations were not carried out to determine costs;
5. Unit cost of bridge replacement based on an average of the costs noted in the text;
6. Costs based on shutting the existing bridge and reconstructing on the existing alignment; and
7. Cost are indicative and not for budgeting.

8. LIFE CYCLE COST COMPARISON

In order to better appreciate the difference between the repair and replacement options, a life cycle comparison has been made between the two.

8.1 Repair Option

The repair option is based on the following initial scope of work:

- Structural rehabilitation to address corrosion;
- Replacement of the electrical system;
- Mechanical repairs; and
- Seismic Retrofit

In the life cycle costing presented in this section, it has been assumed that the rehabilitation/retrofit scope presented in **Sections 5** and **6** will take place in year 3. In addition, the 40 year maintenance program given in **Table 8.1** has been assumed.

8.2 Replacement Option

If the bridge is replaced, minimal work is anticipated over the next 10 years followed replacement. **Table 8.2** gives the life cycle costs associated with the replacement option.

8.3 Comparison of Life Cycle Costs for Repair and Replacement Strategies

Using the whole life costs given in **Tables 8.1** and **8.2**, the life cycle costs of the repair/retrofit option and the replacement option were compared using a discount rate of 2.1% as required by the City. Residual value was used to reflect the remaining value of the asset at 40 years to give a proper comparison of the two strategies.

Based on the discount rate of 2.1% it can be concluded that lower initial costs are anticipated with the repair option although over the 40 year horizon considered in the analysis, the difference in cost between the two approaches is less than 10% with replacement slightly better. When residual value is included, the 10% difference is in favour of repair. With higher discount rates, the repair option would become more advantageous.

8.4 Evaluation of Risks

Both the repair and replacement strategies effectively remove the seismic risk. For comparison purposes, **Table 8.3** outlines the life cycle costs for the do-nothing base case.

Although it may appear that this scenario is much cheaper, it does not address the seismic risk of operating a deficient asset. **Table 8.4**, although somewhat subjective, attempts to quantify this risk by assigning failure probabilities, capital costs, and user costs to the Do Nothing scenario. This table illustrates that doing nothing also carries its own risks. These risks amount to over \$800,000 per year.

Table 8.1 – Life Cycle Costs for Rehabilitation / Retrofit Strategy

Activity	Recurrence rate	Start year	End year	Cost in \$,000	Discount Factor*	Present Value of Cost \$,000
Initial Repair/Retrofit Cost	one time	3		\$19,600	0.94	\$18,415
Annual Maintenance	annual	5	40	\$75	22.18	\$1,664
Engineering Inspections	bi-annual	5	40	\$10	9.89	\$99
Minor repairs (paint touch-up, etc)	bi-annual	5	40	\$50	9.89	\$495
Deck Repairs, steel repairs, E/M replacements	one time	10		\$1,000	0.81	\$812
Major Repairs (deck replacement, E/M replacements)	one time	20		\$3,000	0.66	\$1,980
Repaint	one time	20		\$2,500	0.66	\$1,650
Replacement	one time	40		\$35,000	0.44	\$15,242
TOTAL						\$40,356

* Discount factor based on an effective discount rate of 2.1% per annum net of inflation.

Residual Value **\$15,242**

NET **\$25,114**

Table 8.2 – Life Cycle Costs for Replacement Option

Activity	Recurrence rate	Start year	End year	Cost in \$,000	Discount Factor*	Present Value of Cost \$,000
Replacement	one time	3		\$35,000	0.94	\$32,884
Annual Maintenance	annual	10	40	\$30	17.95	\$538
Engineering Inspections	bi-annual	5	40	\$10	9.89	\$99
Minor repairs (paint touch-up, etc)	bi-annual	10	40	\$25	7.21	\$180
Deck Repairs, steel repairs, E/M replacements	one time	25		\$1,000	0.59	\$595
Repaint	one time	25		\$2,500	0.59	\$1,487
Deck Replacement	one time	40		\$4,000	0.44	\$1,742
TOTAL						\$37,526

* Discount factor based on an effective discount rate of 2.1% per annum net of inflation.

Residual Value \$7,723

NET \$29,803

Table 8.3 –Life Cycle Costs for the Do Nothing Base Case

Activity	Recurrence rate	Start year	End year	Cost in \$,000	Discount Factor*	Present Value of Cost \$,000
Initial Repair/Retrofit Cost	one time	1		\$0	0.98	\$0
Annual Maintenance	annual	1	40	\$75	25.90	\$1,943
Engineering Inspections	bi-annual	1	40	\$10	12.55	\$126
Minor repairs (paint touch-up, etc)	bi-annual	1	40	\$50	12.55	\$628
Deck Repairs, steel repairs, E/M replacements	one time	5		\$2,000	0.90	\$1,803
Major Repairs (deck replacement, E/M replacements)	one time	20		\$8,000	0.66	\$5,279
Repaint	one time	10		\$2,500	0.81	\$2,031
Replacement	one time	40		\$35,000	0.44	\$15,242
TOTAL						\$27,051
Residual Value						\$15,242
NET						\$11,809

* Discount factor based on an effective discount rate of 2.1% per annum net of inflation.

Table 8.4 –Risk Costs for the Do Nothing Base Case

Event Threshold	Consequence	Return Period in years	Probability of Occurrence	Capital Cost	User Cost for detours	User Cost for delays	Loss of Life Cost	Risk Cost per annum	Notes
Noticeable seismic event	Inspection	20	0.05	\$5	\$0	\$56	\$0	\$3	1
Minor seismic event	Inspection	50	0.02	\$10	\$0	\$56	\$0	\$1	1
Timber Pile Damage	Repair piles, 4 week shutdown	119	0.00840	\$2,000	\$1,512	\$8,316	\$0	\$99	2,3,4
Major seismic event	Piles broken, bridge out of service for 6 months, superstructure damage	390	0.002564	\$12,000	\$9,720	\$53,460	\$0	\$193	5,3,4
Severe seismic event	Tower collapse, substructure collapse, closed for 18 months, replacement required, potential loss of life	600	0.001667	\$35,000	\$29,160	\$160,380	\$100,000	\$541	6,3,4,7
Total Risk Cost per Annum								\$834	

Notes

All costs are in \$,000

- 1 Delay costs based on 15 minutes per vehicle at \$15/hr (one half of employment time value)
- 2 Major pile damage requiring difficult underwater repair or strengthening
- 3 Detour cost based on 3 km detour at \$0.60/km
- 4 Delay costs based on 20 minutes per vehicle at \$15/hr and doubled for network congestion
- 5 Major structural damage, permanent deformation, unserviceable bridge
- 6 Collapse of bridge and replacement required
- 7 Loss of life cost based on \$14 million per occurrence, 20% fatality rate, and 3 minute exposure time

9. RECOMMENDATIONS AND CONCLUSIONS

The Johnson Street Bridge is more than 80 years old and, although maintained well over the years, is nearing the end of its design life. In addition it was designed at a time when earthquake engineering was not well understood and is therefore vulnerable to seismic loads. To address the question of repairing or replacing the bridge, scope and costs have been determined for a repair option and for a replacement option.

9.1 Repair Option

Based on a comprehensive inspection of the structural, electrical and mechanical elements of the Johnson Street Bridge as well as a preliminary review of the bridge's seismic vulnerability, a repair program has been developed to extend the life of the existing structure by approximately 40 years.

The repair program recommended in this regard consists of the following scope:

- Repair of corrosion-damaged steel;
- Complete re-coating of the bridge;
- Various repairs to the bridge's mechanical system;
- Replacement of the bridge's electrical system;
- Seismic retrofit.

As noted, the bridge is vulnerable to major damage that is likely not repairable and could be life threatening under earthquakes with a 35% chance of exceedance in 50 years. This represents an earthquake with accelerations that are less than any design earthquake. This happens to be greater than any earthquake the bridge has experienced in its lifetime so far, which would explain why the bridge has not sustained any seismic damage so far. A seismic retrofit of the bridge is recommended to allow the bridge to perform in any of the three recommended categories: 'lifeline', 'emergency route', or 'other' as defined in the CHBDC. This retrofit would include the following scope:

- Installation of energy dissipation bracing and strengthening of selected members;
- Installation of new piles/substructure to relieve the existing foundations/substructure; and
- Replacement of lacing with cover plates in built-up members;
- Installation of bracing to prevent pounding between the highway and roadway bridges;
- Extension of bearing seats or provision of restrainers for the

approach spans and at the rest pier end of the bascule span;

- Provision of lateral restraint at the rest pier for the bascule span;
- Provision of hold-down devices at the rest pier for the bascule span;
- Improvement of shear capacity of cross beams; and
- Modification of gusset plates to ensure ductile connections between truss members.

The estimated cost of the rehabilitation and seismic retrofit works is \$23.6M including engineering and contingencies as defined in **Sections 5** and **6**. It has been assumed that the repair option would be implemented within 3 years to minimize risks associated with the existing bridge's seismic vulnerability.

9.2 Replacement Option

Replacement of the Johnson Street Bridge would require consideration of the following:

- The Bridge is considered an icon in the Victoria area and as such, consideration would need to be given to replacement with an equally remarkable structure;
- Staging of the replacement would likely require closure of the bridge to vehicles for a period of time;
- Vertical clearance under the bridge cannot be increased significantly and as such a new movable bridge would be required to ensure continued access for marine traffic;
- The new bridge would need to carry 2 lanes of traffic, the commuter train and improved sidewalk capacity. As such, the structure would be in the order of 20 m wide; and
- A total bridge length of about 120 m would be required.

A replacement cost of \$35.26M was estimated including engineering and contingencies. If replacement was to be undertaken, it has been assumed that this would occur within 3 years in order to mitigate risks associated with the seismic vulnerability of the existing bridge.

9.3 Comparison of Repair and Replacement Options

A 40 year maintenance program with corresponding costs was established for both the repair and replacement options in order to allow a life cycle comparison. Using a discount rate of 2.1%, as specified by the City, lower initial costs are expected for the repair option while there is no significant difference between the total life cycle costs of the two options over the 40 year period considered in the analysis.

9.4 Recommended Approach

Based on the findings of this study either a repair or a replacement option could be justified from a cost perspective. There is, however, in our opinion a need to address the seismic vulnerability of the existing bridge given that it is heavily trafficked and located in the most seismically active city in Canada. In this report we have suggested that this vulnerability should be addressed within 2 to 3 years by implementing a seismic retrofit or by replacing the bridge.

In order to select either the repair or replacement approach, value needs to be placed on elements of the project that are beyond the scope of this study and are not associated with structural engineering. In particular the following could be considered and valued:

- Benefits derived from the improved access provided by a new bridge;
- Value associated with preserving the historical elements of the existing bridge; and
- Value of a new landmark bridge.

Appendix A

Photographic Records

Roadway Bridge Photographs (Span 1 – below deck)



Span 1 - Lateral Bracing Connection
Top Lateral Bracing
Lift Truss
Starting East Working West



Span 1 - South Side, North Face Built Up Panel
Top Lateral Bracing
Lift Truss



Span 1 - Lateral Bracing Connection
Top Lateral Bracing
Lift Truss
Starting East Working West



Span 1 - Efflorescence on Soffit
Top Lateral Bracing
Lift Truss
Starting East Working West



Span 1 - Corrosion on Bottom Flange and Rivets
Top Lateral Bracing
Lift Truss



Span 1 - Lateral Bracing Connection
Top Lateral Bracing
Lift Truss
Starting East Working West



Span 1 - Paint Loss and Corrosion on Bottom FlangeTop Lateral Bracing Lift Truss



Span 1 - Efflorescence on Concrete SoffitTop Lateral Bracing Lift Truss Starting East Working West



Span 1 - Paint Loss and Corrosion on Floor BeamTop Lateral Bracing Lift Truss



Span 1 - Corrosion on Lateral Bracing Connection PlateTop Lateral Bracing Lift Truss



Span 1 - Corrosion on Built-Up Web PanelsTop Lateral Bracing Lift Truss Starting East Working West



Span 1 - Corrosion on Web of Built-Up MembersTop Lateral Bracing Lift Truss



**Span 1 - Typical Paint Loss and Corrosion along Bottom Flange
of Beams Top Lateral Bracing
Lift Truss**



**Span 1 - Severe Paint Loss and Corrosion on Lateral Bracing Top
Lateral Bracing
Lift Truss**



**Span 1 - Debris Build Up at Lateral Bracing Connection, Paint
Loss and Corrosion Top Lateral Bracing
Lift Truss**

Roadway Bridge Photographs (Span 2 – below deck)



Span 2 - Open Grid Decking Top Lateral Bracing
Lift Truss
Starting East Working West



Span 2 - Edge of Side Walk Top Lateral Bracing
Lift Truss
Starting East Working West



Span 2 - Corrosion on Bottom Flange and Rivets Top Lateral
Bracing
Lift Truss



Span 2 - Corrosion on Rivets and Bottom Flange Top Lateral
Bracing
Lift Truss



Span 2 - Paint Loss on Web Top Lateral Bracing
Lift Truss
Starting East Working West



Span 2 - Pain Loss on Web Top Lateral Bracing
Lift Truss
Starting East Working West



Span 2 - Corrosion on Rivets and Bottom Flange Top Lateral Bracing Lift Truss



Span 2 - Corrosion on Rivets and Bottom Flange Top Lateral Bracing Lift Truss



Span 2 - Corrosion on Rivets and Bottom Flange Top Lateral Bracing Lift Truss



Span 2 - Paint Loss and Corrosion inside Built Up Members Top Lateral Bracing Lift Truss



Span 2 - Pain Loss and Corrosion inside Built Up Members Top Lateral Bracing Lift Truss



Span 2 - Corrosion on Rivets and Bottom Flange Top Lateral Bracing Lift Truss



Span 2 - Corrosion on Rivets and Bottom Flange Top Lateral Bracing Lift Truss



Span 2 - Corrosion on Rivets and Bottom Flange Top Lateral Bracing Lift Truss



Span 2 - Corrosion Along Sidewalk Top Lateral Bracing Lift Truss Starting East Working West



Span 2 - Corrosion Along Sidewalk Top Lateral Bracing Lift Truss Starting East Working West



Span 2 - Corrosion on Rivets and Bottom Flange Top Lateral Bracing Lift Truss



Span 2 - Corrosion on Rivets and Bottom Flange Top Lateral Bracing Lift Truss



**Span 2 - Corrosion on Rivets and Bottom FlangeTop Lateral
Bracing
Lift Truss**



**Top Lateral Bracing
Lift Truss
Starting East Working West**



**Top Lateral Bracing
Lift Truss
Starting East Working West**



**Span 2 - Corrosion on Rivets and Bottom FlangeTop Lateral
Bracing
Lift Truss**



**Top Lateral Bracing
Lift Truss
Starting East Working West**



**Top Lateral Bracing
Lift Truss
Starting East Working West**



**Span 2 - Exposed Rebar on Pier Top Lateral Bracing
Lift Truss
Starting East Working West**



**Span 2 - Corrosion inside Built Up Member Top Lateral Bracing
Lift Truss
Starting East Working West**



**Span 2 - Corrosion on Rivets and Bottom Flange Top Lateral
Bracing
Lift Truss**



**Span 2 - Corrosion on Rivets and Bottom Flange Top Lateral
Bracing
Lift Truss**



**Span 2 - Corrosion inside Built Up Member Top Lateral Bracing
Lift Truss
Starting East Working West**



**Span 2 - Corrosion Along Edge of Walkway Top Lateral Bracing
Lift Truss
Starting East Working West**



Span 2 - Corrosion on Rivets and Bottom Flange Top Lateral
Bracing
Lift Truss

Roadway Bridge Photographs (Span 2 – above deck)



North Side - North Face
Member to Member Connections
Starting East Working West



North Side - North Face
Member to Member Connections
Starting East Working West



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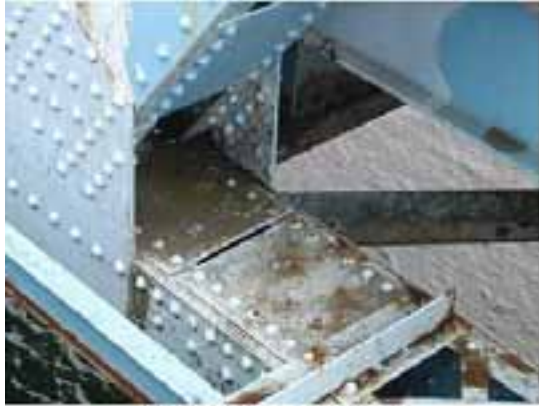
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Top Lateral Bracing
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Starting East Working West

Roadway Bridge Photographs (Span 3 – below deck)



Span 3 - Corrosion on Walkway Cantilever MemberTop Lateral
Bracing
Lift Truss



Span 3 - Corrosion on Walkway Cantilever MemberTop Lateral
Bracing
Lift Truss



Span 3 - Exposed Rebar and Spalling Concrete on Pier 2Top Lateral Bracing
Lift Truss



Span 3 - Exposed Rebar and Spalling Concrete on Pier 2Top Lateral Bracing
Lift Truss



Span 3 - Spalling Concrete on Pier 2Top Lateral Bracing
Lift Truss
Starting East Working West



Span 3 - Corrosion on Rivets and Bottom FlangeTop Lateral
Bracing
Lift Truss



Span 3 - Corrosion on Rivets and Bottom Flange
Top Lateral
Bracing
Lift Truss



Span 3 - Efflorescence Under Pedestrian Walkway
Top Lateral
Bracing
Lift Truss



Span 3 - Efflorescence Under Pedestrian Walkway
Top Lateral
Bracing
Lift Truss



Span 3 - Efflorescence Under Pedestrian Walkway
Top Lateral
Bracing
Lift Truss



Span 3 - Efflorescence Under Pedestrian Walkway
Top Lateral
Bracing
Lift Truss

Roadway Bridge Photographs (Span 4 – below deck)



Span 4 - Dirt Build Up on Connection Plate Top Lateral Bracing
Lift Truss
Starting East Working West



Span 4 - Dirt Build Up on Bottom Flange Top Lateral Bracing
Lift Truss
Starting East Working West



Span 4 - Efflorescence on Concrete Soffit Top Lateral Bracing
Lift Truss
Starting East Working West



Span 4 - Dirt and Corrosion on Lateral Bracing Connection
Plate Top Lateral Bracing
Lift Truss



Span 4 - Rivets Worn Away, Corrosion and Paint Loss Top Lateral
Bracing
Lift Truss



Span 4 - Rivets Worn Away, Corrosion and Paint Loss Top Lateral
Bracing
Lift Truss



**Span 4 - Corrosion at Lateral Bracing Top Lateral Bracing
Lift Truss
Starting East Working West**



**Span 4 - Corrosion at Lateral Bracing Top Lateral Bracing
Lift Truss
Starting East Working West**



**Span 4 - Rivets Worn Away, Corrosion and Paint Loss Top Lateral
Bracing
Lift Truss**



**Span 4 - Paint Loss at Lateral Bracing Plate Top Lateral Bracing
Lift Truss
Starting East Working West**



**Span 4 - Paint Loss and Corrosion on Web of Built Up
Member Top Lateral Bracing
Lift Truss**



**Span 4 - Paint Loss and Corrosion on Web Top Lateral Bracing
Lift Truss
Starting East Working West**



**Span 4 - Efflorescence on Concrete Soffit Top Lateral Bracing
Lift Truss
Starting East Working West**



**Span 4 - Efflorescence on Concrete Soffit Top Lateral Bracing
Lift Truss
Starting East Working West**



**Span 4 - Efflorescence on Concrete Soffit Top Lateral Bracing
Lift Truss
Starting East Working West**

Railway Bridge Photographs



HPIM3929.JPG
P001 North Side of Bridge



HPIM3961.JPG
P002 - West End of Bridge - Looking East



HPIM3875.JPG
P003 - Bridge From East



HPIM3957.JPG
P004 - West Abutment 1 - North Wing Wall - 2 mm Vertical Crack Full Height - Minor Cracking and Efflorescence - No Spalling



HPIM3958.jpg
P005 - West Abutment 1 - Minor Cracking and Efflorescence - No Spalling



HPIM3906.JPG
P006 - West Abutment 1 - North Fixed Bearing



HPIM3910.jpg

P007 - West Abutment 1 - Wide Diagonal Crack (4 mm) in Backwall Between Railway and Roadway Bridges Almost to Waterline



HPIM3884.JPG

P008 - East Abutment 2 - Minor Cracking, Efflorescence and Moisture Staining - Bearing Seat OK



HPIM3893.JPG

P009 - East Abutment 2 and Expansion Bearings



HPIM3956.JPG

P010 - Concrete Pier 1 - North Nosing and West Face



HPIM4011.JPG

P011 - Concrete Pier 1 - East Face



HPIM3908.JPG

P012 - Concrete Pier 1 - North Face - Minor Cracking and Efflorescence But No Spalling



HPIM3909.JPG

P013 - Concrete Pier 1 - North Face and Span 1 Expansion Bearings



HPIM3905.JPG

P014 - Concrete Pier 1 - Between Railway and Roadway Bridges



HPIM4012.JPG

P015 - Concrete Pier 2 - West Face



HPIM3990.JPG

P016 - Concrete Pier 2 - East Face - 2 mm Horizontal Crack Along Construction Joint - Moisture Staining and Efflorescence



HPIM3917.JPG

P017 - Concrete Pier 2 - West Face



HPIM3931.jpg

P018 - North Side of Span 3 - Piers 2 and 3 Connected with 2 Concrete Strong Beams Above Waterline



HPIM3991.JPG
P019 - North Strong Beam Between Piers 2 and 3 - 10 cm x 40 cm Spall Bottom of Beam



HPIM3989.JPG
P020 - Concrete Pier 3 - West Face - Minor Cracking and Efflorescence But No Spalling and Bearing Seats are Good



HPIM3933.JPG
P021 - Concrete Pier 3 - North Face - Piers Common for Both Railway and Roadway Bridges



HPIM3934.jpg
P022 - North Face of Pier 3 - 1 mm Vertical Crack Below Railway Bridge South Bearing



HPIM3895.JPG
P023 - Pier 3 - Two Concrete Strong Beams Connecting Pier 1 and 2 Above Waterline



HPIM3897.JPG
P024 - Concrete Pier 3 - North End



HPIM3886.JPG
P025 - Pier 3 - East Face



HPIM3882.JPG
P026 - Pier 3 - West Face



HPIM3959.jpg
P027 - West Approach Span 1 - Floor System - Stringers and Floorbeams Replaced Since Initial Construction



HPIM3960.JPG
P028 - West Approach Span 1 - Floor System - Bolted Connections - Bracing Replaced Since Initial Construction



HPIM3907.JPG
P029 - West Approach Span 1 - Floor Beam 1 - Bolted Connections



HPIM3902.JPG
P030 - West Approach Span 1 - South Main Girder



HPIM3903.JPG

P031 - West Approach Span 1 - Debris Buildup on South Main Girder



HPIM3904.JPG

P032 - West Approach Span 1 - Top Flange and Shelf Angle on South Main Girder



HPIM3901.JPG

P033 - West Abutment 1 - South Fixed Bearing for Span 1



HPIM3913.JPG

P034 - West Approach Span 1 - Floor Beams - Looking West



HPIM3914.JPG

P035 - Pier 1 - South Expansion Bearing for Span 1



HPIM3916.JPG

P036 - Span 1 - North Main Girder



HPIM3915.JPG
P037 - Pier 1 - North Expansion Bearing for Span 1



HPIM3911.JPG
P038 - West Approach Span 1 - 2 Stringers Above Floorbeams - Bolted Connections



HPIM3912.JPG
P039 - West Approach Span 1 - Floor Beam 1 - Bolted Connection to North Main Girder



HPIM3932.JPG
P040 - North Side of Through Truss Lift Span 2



HPIM3900.JPG
P041 - Lift Span 2 - North Bottom Chord



HPIM3918.JPG
P042 - Lift Span 2 - North Bottom Chord



HPIM3923.JPG

P043 - Lift Span 2 - Floor System is in Good Condition with Updated Stringers, Floor Beams and Bracing Since Initial Construction



HPIM3922.JPG

P044 - Lift Span 2 - South Main Stringer in Panel 1 - Bolted Connections



HPIM3927.JPG

P045 - Lift Span 2 - North Bottom Chord in Panel 1



HPIM3945.JPG

P046 - Lift Span 2 - North Bottom Chord in Panel 1 - 1 Holed Lacing Member



HPIM3946.jpg

P047 - Lift Span 2 - North Vertical Hanger - End of Panel 1 - 10 mm Pack Rust Between Angles



HPIM3947.jpg

P048 - Lift Span 2 - North Vertical Hanger - End of Panel 1 - 10 mm Pack Rust Between Angles



HPIM3948.JPG

P049 - Lift Span 2 - North Horizontal Gusset Plate Holed Along Bottom Chord at End of Panel 2



HPIM3949.JPG

P050 - Lift Span 2 - Debris Buildup Inside of Bottom Chord at Verticals - Typical



HPIM3950.JPG

P051 - Lift Span 2 - North Horizontal Gusset Plate Holed at Beginning of Panel 6



HPIM3954.jpg

P052 - Lift Span 2 - South Vertical Hanger - End of Panel 5 - 10 mm Pack Rust Between Angle and Vertical Gusset Plate



HPIM3955.JPG

P053 - Lift Span 2 - South Horizontal Gusset Plate at End of Panel 1 - Pitted up to 3 mm But Not Holed



HPIM3953.JPG

P054 - Lift Span 2 - South Bottom Chord of Panel 5 - Approx 5% of Lacing Members are 10% Reduced



HPIM3995.JPG

P055 - Lift Span 2 - Panel 6 - Lower Sway Braces are Welded and Bolted



HPIM3996.jpg

P056 - Lift Span 2 - Typical Main Stringer Floorbeam Connections are Bolted - Floor System Has Been Replaced Since Initial Construction in 1923



HPIM3997.jpg

P057 - Lift Span 2 - Typical Main Stringer Floorbeam Connections are Bolted - Floor System Has Been Replaced Since Initial Construction in 1923



HPIM3998.JPG

P058 - Lift Span 2 - Holed Gusset Plate - Southwest Corner of Panel 5



HPIM3999.JPG

P059 - Lift Span 2 - Vertical Cross Bracing Between Main Stringers - Middle of Panel 5 - Bolted and Welded - Typical



HPIM4000.JPG

P060 - Lift Span 2 - South Bottom Chord - Minor Bowing of Lacing Members



HPIM4001.JPG

P061 - Lift Span 2 - Horizontal Gusset Plate 4 mm Reduction Groove Along Bottom Chord - Southeast Corner of Panel 4



HPIM4002.JPG

P062 - Lift Span 2 - Welded Stiffeners in Stringers and Floorbeams - Typical



HPIM4006.JPG

P063 - Lift Span 2 - South Vertical at East End of Panel 3 - One Leg of Angle Has Reduction Groove with 50% Section Loss and Holed



HPIM4007.JPG

P064 - Lift Span 2 - Horizontal Gusset Plates - Reduction Grooves Along Bottom Chords are Typical



HPIM4008.JPG

p065 - Lift Span 2 - South Bottom Chord in Bay 2 - Horizontal Top Plate is Holed



HPIM4009.JPG

P066 - Lift Span 2 - Horizontal Gusset Plate Holed Along Bottom Chord - Northeast Corner of Panel 2



HPIM4010.JPG
P067 - Lift Span 2 - South Hanger at End of Panel 1 - Lower Vertical Plate is Pitted 10 mm



HPIM3919.JPG
P068 - South Rest Bearing on Pier 1 for Lift Span 2



HPIM3920.JPG
P069 - North Rest Bearing on Pier 1 for Lift Span 2



HPIM3944.JPG
P070 - Lift Span 2 - West Portal - No Collision Damage



HPIM3951.JPG
P071 - Lift Span 2 - North Lower Pin at Pier 2



HPIM3952.JPG
P072 - Lift Span 2 - South Lower Pin at Pier 2



HPIM3963.JPG
P073 - Lift Span 2 - East Portal



HPIM3970.JPG
P074 - Lift Span 2 - Overhead Bracing



HPIM3972.JPG
P075 - Lift Spans - Top Chords and Overhead Bracing



HPIM3981.JPG
P076 - Between Bridges - Lift Span 2 - Pins - Top of East Portals



HPIM3982.jpg
P077 - Lift Span 2 - South Top Chord



HPIM3984.jpg
P078 - Lift Span 2 - Top North Chord



HPIM3985.JPG

**P079 - Lift Span 2 - North Pin at Node 9 -
Top of East Portal**



HPIM3899.JPG

P080 - Spans 3 and 4 - North Side



HPIM3885.JPG

**P081 - Span 3 - 2 Large Stringers Below
Track, 2 Small Stringers at Cantilevers**



HPIM3889.JPG

**P082 - Span 3 - Shrinkage Cracks in
Paint System**



HPIM3887.jpg

**P083 - Span 3 - South Main Stringer
Connection to Floor Beam 0 - Bolted
Connections Typical**



HPIM3891.JPG

**P084 - Span 3 - South Small Stringer
Connection to Floorbeam 2 - Bolted
Connections Typical**



HPIM3890.JPG
P085 - Span 3 - North Bottom Chord



HPIM3888.JPG
P086 - Pier 3 - South Bearings - North Side



HPIM3896.JPG
P087 - Pier 3 - South Face of North Bearings



HPIM3898.JPG
P088 - Pier 2 - North Side of North Bearings



HPIM3880.JPG
P089 - Pier 3 - South Bearing for Span 3



HPIM3879.JPG
P090 - Pier 3 - North Bearings for Spans 3 and 4



HPIM3992.JPG

P091 - Pier 2 - Span 3 - Section Loss in Anchor Bolt Stiffeners



HPIM3993.JPG

P092 - Pier 2 - Span 3 - South Side of South Bearing Holed Anchor Bolt Stiffeners at Mortar Plate



HPIM3994.JPG

P093 - Pier 2 - Span 3 - North Side of South Bearing - Anchor Bolt Stiffeners at Mortar Plate are Not As Bad



HPIM3962.JPG

P094 - Concrete Floor Soffit - Elevated Mechanical Shed Over Railway Bridge - No Spalling



HPIM3964.JPG

P095 - Mechanical Shed Drive System



HPIM3965.JPG

P096 - Mechanical Shed Roof System - Corrugated Galvanized Roof Panels



HPIM3967.JPG

P097 - South Trunnion Pin at Node 19



HPIM3968.JPG

P098 - North Trunnion Pin at Node 19



HPIM3969.jpg

P099 - South Counterweight Support Truss



HPIM3971.JPG

P100 - Top of Concrete Counterweight



HPIM3973.JPG

P101 - Counterweight Support Truss Over Span 3



HPIM3974.JPG

P102 - Counterweight Support Truss Over Span 3



HPIM3975.JPG

P103 - Top Pins at Node 20 are Enclosed



HPIM3978.JPG

P104 - Bascule Drive Struts for Both Bridges



HPIM3979.JPG

P105 - Lift Span 2 - South Pin at Node 9 - Top of East Portal



HPIM3980.JPG

P106 - Roadway Bridge - North Pin - Top of East Portal



HPIM3986.JPG

P107 - Both Mechanical Sheds Above Span 3



HPIM3930.JPG

P108 - North Side of East Approach Span 4



HPIM3935.JPG
P109 - East Approach Span 4 - Floor Beams



HPIM3892.JPG
P110 - Span 4 - North Main Girder



HPIM3883.JPG
P111 - Span 4 - Looking East



HPIM3987.JPG
P112 - East Approach Span 4 - Sidewalk Supports Between Bridges - Steel Floorbeams with Timber Stringers and Timber Decking



HPIM3936.JPG
P113 - East Abutment 2 - North Bearing and Bearing Seat - 3 mm Pack Rust Between 2 Adjacent Bearing Stiffeners



HPIM3881.JPG
P114 - Span 4 - South (Fixed) Bearing on Pier 3



HPIM3876.JPG

P116 - Track Alignment is Curved in the East Approach Span



HPIM3878.JPG

P116 - North Steel Railing



HPIM3943.JPG

P117 - Span 1 - Timber Walkway on South Side



HPIM3942.JPG

P118 - 65 Span 1 Ties (10' x 12'') - No Rejects



HPIM3941.JPG

P119 - 147 Span 2 Ties (8' x 12'') - No Rejects



HPIM3940.JPG

P120 - 51 Span 3 Ties (8' x 12'') - No Rejects



HPIM3939.JPG
P121 - 99 Span 4 Ties - 10" x 14" - No Rejects



HPIM3926.JPG
P122 - Lift Span 2 - North Steel Grating Walkway Supported on Transverse Channels



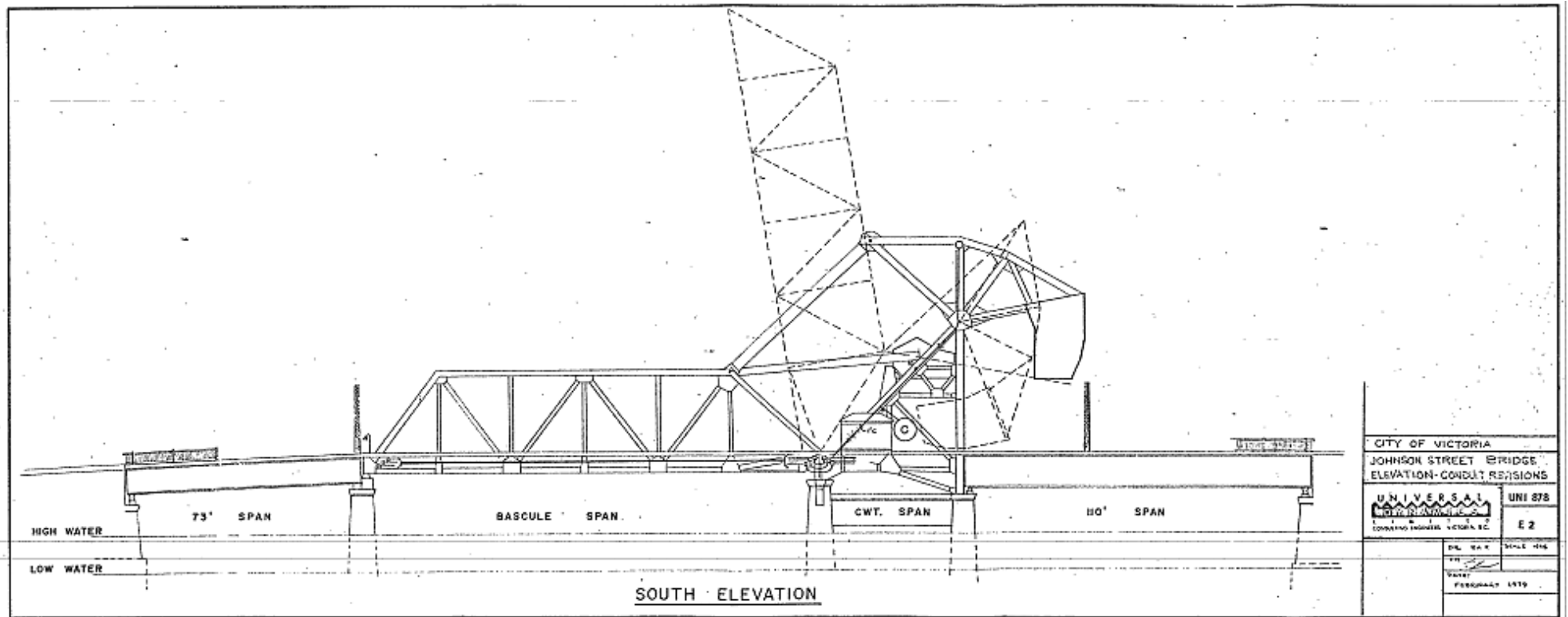
HPIM3877.JPG
P123 - Span 4 - Timber Walkway and Steel Railing on North Side

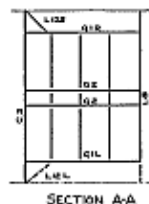
Appendix B

Inspection Forms

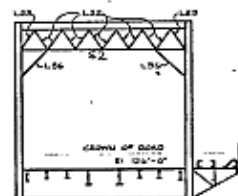
Appendix C

Reference Drawings

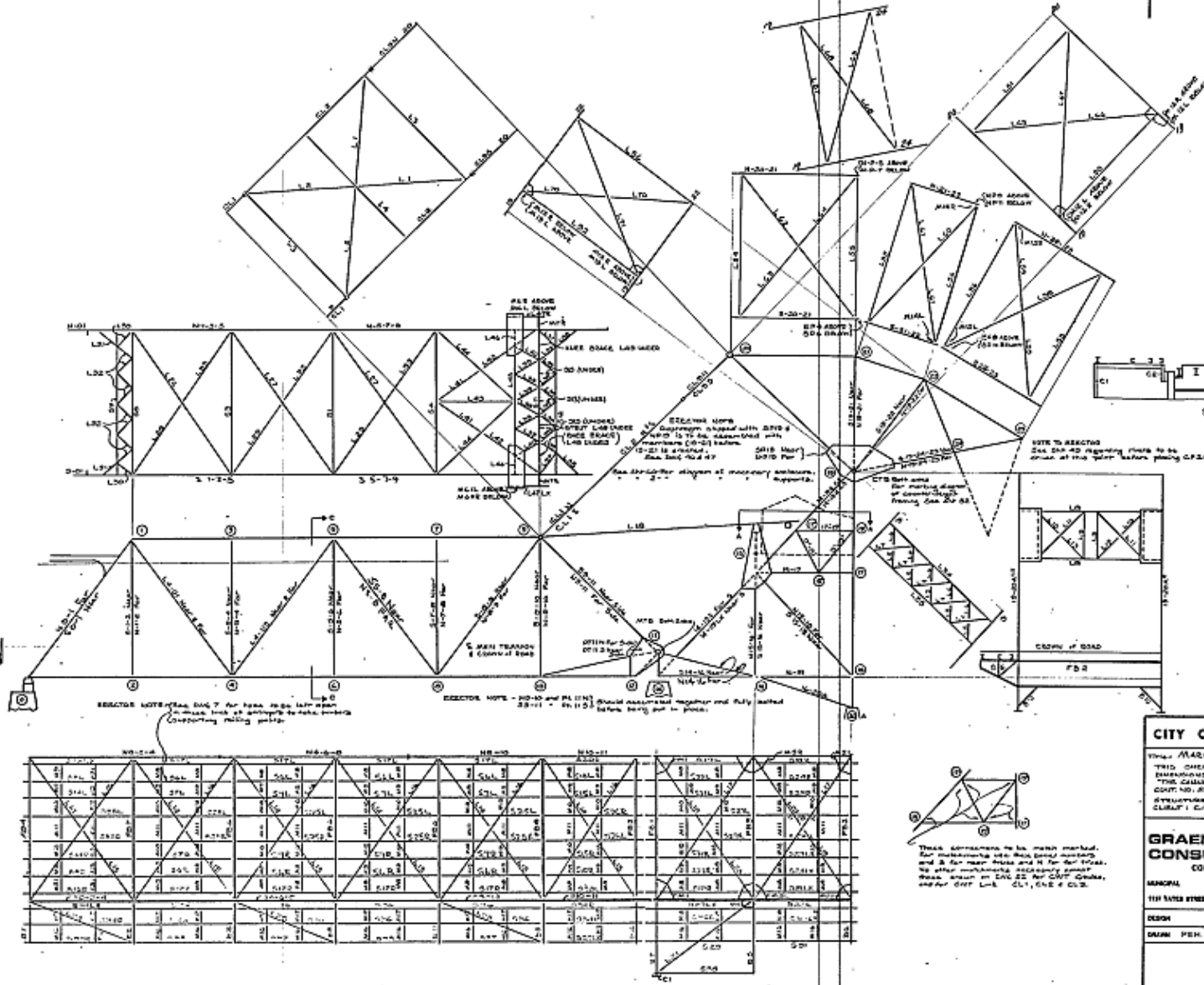


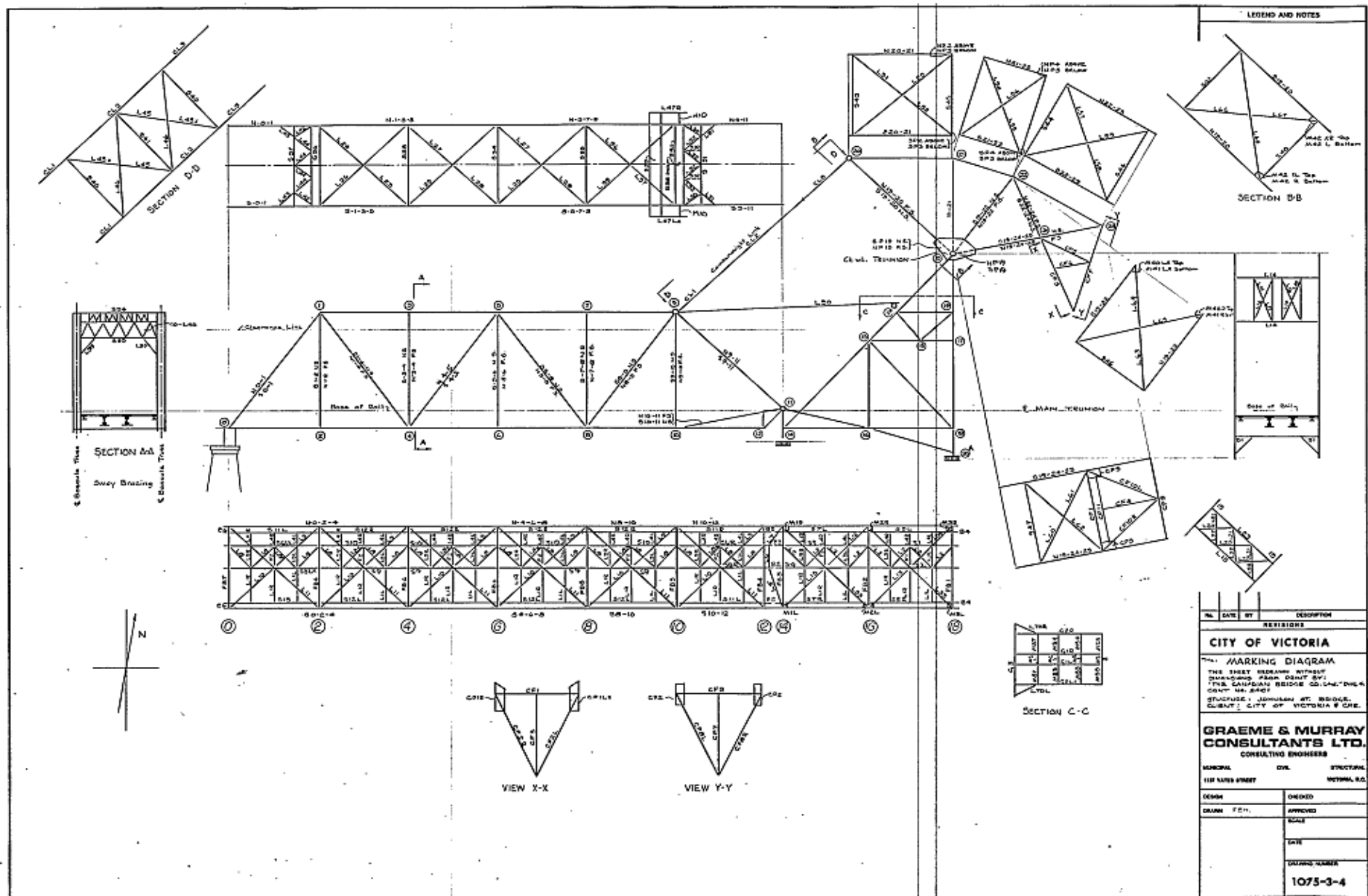


SECTION A-A



SECTION C-C





Appendix D

Seismic Analysis

Abbreviations:

Abbreviations:

T_y : the lateral fundamental period in seconds in Y direction

T_x : the longitudinal fundamental period in seconds in Y direction

I : Importance factor of the bridge.

D : displacement

F : reactions

 : Ductility factor

E_x: Axial modulus of Elasticity.

E_y; E_z: lateral modulus of Elasticity.

G= Shear Modulus.

R= Response modification factor.

Notes:

- All dimensions are in meters, periods in Seconds, forces in kN, displacement in mm, and stress in MPa.
- All the values that may be considered for comparisons between the alternatives were tabulated.

1 Modeling Assumptions

1.1 Boundary Conditions

In the computer model, boundary conditions were preset to best reflect the degree of freedom of bridge bearings. The bearings of the two approach spans, as indicated by available plans, are fixed at the west end and expansion at the east end. In the computer model, fixed bearings do not have translation flexibility in x, y, z-directions; however, expansion bearings have translation flexibility in the x-direction (longitudinal direction) but do not have translation flexibility movement in y or z directions.

With reference to Figure 2.1, the bearings on the counterweight span were modeled as fixed bearings at both Pier 2 and Pier 3. This is an assumption as exact bearing conditions were not provided on the available drawings. It is assumed that these bearings have no movement flexibility to ensure full anchorage. As well, the temperature expansion would not be significant due to the short span distance (14m).

With reference to Figure 4.1, the bearings on the bascule span were modeled as fixed at Pier 2 and compression only at Pier 1 where the bridge can be opened. This is also an assumption as exact bearing conditions were not provided on the available plan drawings.

To ensure that both the highway bridge and the railway bridge act as one body under external forces, rigid links were placed in the model at piers 1, 2, and 3 to connect the two bridges. The rigid links which ensure displacements in six degrees of freedom are equal at both ends of the link, which actually mimic the concrete pier at Piers 1, 2, and 3.

1.2 Dead Load

Dead load of the bridge is an important component especially in seismic analysis, because the distribution of mass would affect directly the dynamic behaviour of the bridge. Since the Johnson Street Bridge was built in 1924, it has undergone a few modifications that resulted in significant changes in the weight of the bridge. The main sources of dead load on the bridge include the truss structure, deck cover material and the concrete counter weight blocks.

- **Truss Weight:** The steel used in constructing the bridge truss was assumed to have a unit weight of 77 kN/m^3 in the model. Although the steel had a much lower strength capacity, it was assumed that the unit weight of steel remained the same as the current average unit weight.
- **Vehicle Bridge Deck Weight:** The deck on the highway bridge was originally covered with wood timbers on the bascule span. The timber deck became slippery in wet weather and absorbed water which became too heavy for the opening machinery. In 1966, the timbers were replaced by an open grid steel

decking, which is comprised of American standard channels 9" deep, weighing 13.4 pounds per foot[5] (0.64 kN/m²). This unit weight was used in the computer model. Assuming the original timbers weighted 6-9 kN/m³, this open grid steel decking would reduce the overall weight of the highway bridge significantly.

- Counterweight Span Deck Weight: The counterweight span, by inspection, was covered with concrete slabs of about 40cm. Similarly, the two approach spans were also covered with concrete slabs by inspection. The thickness was assumed to be 40 cm in the model. An average unit weight of 23.5 kN/m³ was used in the model for the concrete slabs.

- Railway Bridge Deck Weight: The deck on the railway bridge was covered with timbers, ballast and rail tracks throughout the main span and approach spans. The railway ties had an average thickness of approximately 15 cm. Assuming these railway ties are softwood, a unit weight of 6 kN/m³ was used for the railway bridge deck in the model. Similarly, a timber deck was also used for the pedestrian sidewalk on the bridges.

- Counterweight: A concrete block counterweight is placed at the east end of both the highway bridge and the railway bridge. When each counterweight is fully loaded, original Plans[1] showed that the weight of counterweight was 510 tons on the railway bridge and 785 tons on the highway span.

1.3 Other Assumptions

The following assumptions were made in the development of the computer model:

- The existing substructure is timber piles assumed to be Douglas Fir, fully saturated, and the properties are:

$$E_x = 6.10^3 \text{ Mpa}, E_y = E_z = 0.163 E_x = 978 \text{ mpa}$$

$$G = 0.086 E_x = 516 \text{ Mpa}, \text{Poisson ratio} = 0.4$$

The stresses for comparison:

Combined stress = 10 Mpa, short duration load factor = 2.8
and R = 1.25. as an Immediate Occupancy (IQ) case of the bridge

- All steel members were modeled with beam elements with six degrees of freedom at each joint.

- The beam members were placed along the centerlines of the actual members between joints.

- Truss members were rigidly connected to joints except where it was necessary to release certain forces to model support conditions.

- The connections between member ends were subjected to pure axial force that was intended in the original design. A k-value of 1.0 was assigned to calculate member capacities to reflect this end condition.
- Gusset plates and splice plates were not included in the model.
- Expansion Joints are open and the maximum gap displacement allowed is 70 mm. fixed expansion joint will be analyzed for the proper chosen alternative later.
- No verification for the seismic performance of the bridge was made for the open position.
- The counterweights are rigidly connected to the steel frame and no local concentrated strains could be encountered
- The rails 11-12 (links between moving and fixed part) are very stiff laterally due to the gaskets and plates and will transfer the lateral forces between moving parts and fixed parts of the bridge efficiently.

2 Retrofit Alternatives

2.1 Assessment of Existing Structure:

This alternative represents the base case where the bridge is assessed for the maximum earthquake that the bridge may survive. It is the PGA (peak ground acceleration) at which collapse may happen. The analysis was conducted for PGA of 0.336 g which represents the 10% probability of exceedance in 50 years, and return period 475 years.

Scaling factors were applied to this basic PGA in the analysis, and failure criteria were included in the bridge model. Maximum displacements (stability criterion) and maximum stresses (strength criterion). These failure criteria were for the steel superstructure, concrete substructure, and the timber piles. During the analysis the stresses in the shorter timber piles reached maximum values first (at one side of the foundation) before the concrete or steel superstructure, leading to excessive deformation as well as pile damage and breakage.

Different scaling factors were employed in the analysis and each case was repeated with the same model and the same basic earthquake but discarding the broken piles. It was found that the factor that limits the broken piles to an amount that it is acceptable do not affect the stability. The ratio was 0.54 from the basic PGA, which is 0.336 g, in other words the critical level of earthquake would be defined as an event with an intensity having a 35% probability of exceedance in 50 years, the peak ground acceleration associated with this event is about 0.18 g, means almost half of the peak rock acceleration of an event with intensity having 10% probability of exceedance in 50 years, and the failure started in the timber piles.

Thorough observation of the modal analysis results that has been carried out to determine the weak planes and the actual structural behavior during Earthquake events and the shape of displacement considerations, fig (A-D 2.1). It has been shown that longitudinal direction of ground motion (i.e. X axis) has less effects than the transverse direction (i.e. Y axis), in spite of the fact that the bridge is stiffer in the longitudinal direction and has less fundamental period means more spectral acceleration resulting in more seismic forces, but the structure is designed to carry such forces While in lateral (transverse i.e. Y axis) the fundamental period is higher but the structure very weak in this direction.

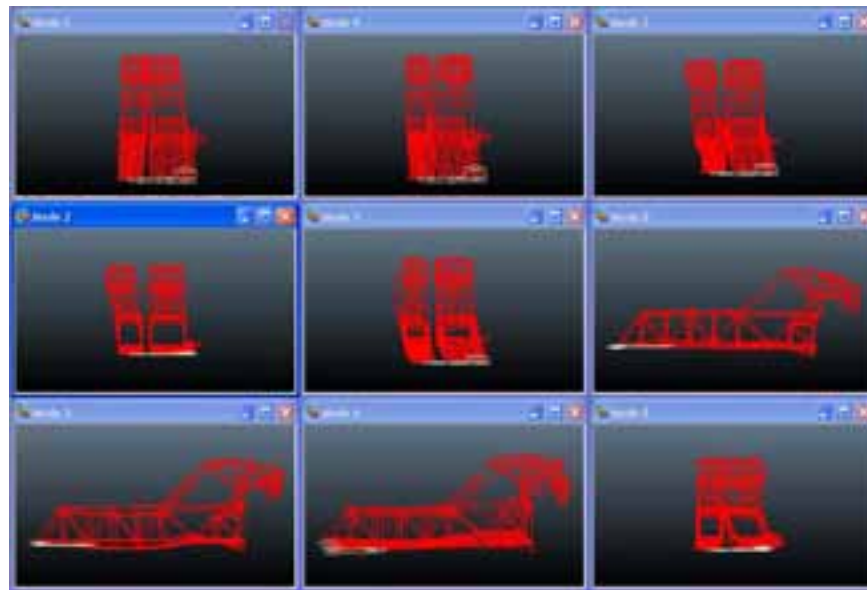


Fig (A-D 2.1) the first 9 mode shapes of the existing bridge

It could be shown on Fig (A-D 2.2) that some members in the superstructure still suffer excessive stresses and plastic deformation. These members are mainly the lateral bracing and the triangle chord.

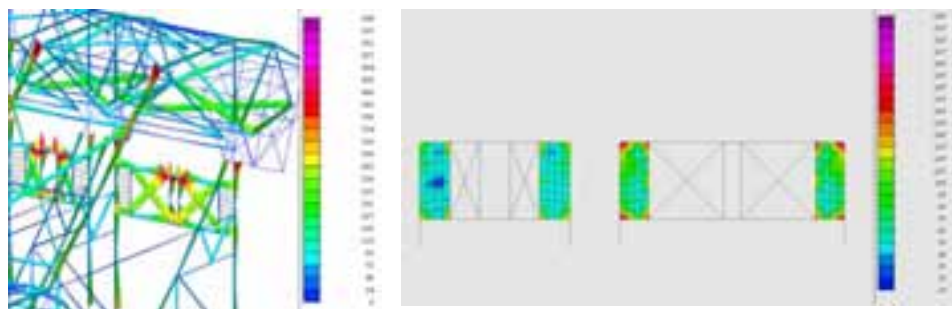


Fig (A-D 2.2) the stresses of the bracings of the existing bridge

2.1 Option 1 (Mass Adjustment):

The analysis shows that with this alternative that both stresses and displacements are reduced but that critical overstresses that could lead to the collapse of the counterweight tower. As such, this approach on its own is not recommended.

The behavior and mode shapes are the same as the existing bridge exactly, since no stiffness adjustment had taken place. Stresses dropped dramatically for two reasons: in the first hand, the contribution of the mass in seismic forces when it excited by the acceleration, and on the second hand, for the reduction of the dead load stresses in the members, so that may increase the safety margins in the members, which in turn will enable the member to carry more stresses than what it was designed to carry and as a result this procedure may minimize the seismic hazard on the bridge. It was shown in the dynamic analysis of this alternative 1 that the bridge has C/D ratio of less than 1 for the three proposed earthquakes, (i.e. 2%, 5%, and 10 % probability of exceedance in 50 years) and those ratios are 0.5, 0.67, and 0.91 respectively. The stresses in the existing bracing and the main verticals are higher than their ultimate capacity, even though the substructure has been adequately repaired at this point.

The ultimate capacity of the bridge that meet the requirements of the C/D ratio = 1 (i.e. the critical case) for this alternative was found to be 12% probability of exceedance in 50 years with return period 390 years, and the peak ground acceleration was $PGA = 0.3\text{ g}$. The displacements were acceptable

2.2 Option 2 (Relocation of Counterweight)

This alternative was ruled out early, therefore no discussion is made on this. The analysis procedure was very similar to Alternative 1.

2.3 Option 3 (Structural Strengthening):

This alternative comprised of adding elements to improve its structural behavior by achieving an improved load path. An iterative process was made to choose the proper cost-effective strengthening of the bridge. In order to achieve a practical retrofit methodology, some considerations were taken into account:

- Members that may be added should not affect the service condition of the bridge.
- Controlled behavior of the two bridges as they move laterally together. Pounding need to be prevented. A proper type of link between the two needed to be provided.
- Consideration of replacing or adding members to reduce the fundamental period of the bridges, or increase the mass.

- The load path after adding new members and the demand-capacity ratio for the members against the current new stress redistributions.
- Ideal locations to introduce a plastic hinge in the system that may dissipate the seismic energy.
- The importance factor of the bridge, as it has a great influence of the cost and the method of strengthening.

Seismic energy could be dissipated through hysteresis of short, replaceable link elements. These link elements can be designed to:

- Yield early, maximizing protection to main frame
- Yield in web shear rather than flexure
- Remain stable under large non-linear displacements

Fig (A-D 2.3) represents the new added members in this alternative.



Fig (A-D 2.3) the added members in Alternative 3

All link elements yielded and all main elements remained elastic under the design earthquake, inelastic behavior (damage) is therefore accepted if it does not cause collapse. This eccentric frame is ductile in nature and the damage occurs in designated components.

Cross bracing for joining the two bridges found to be better (in this case) as it will not create a concentration of stresses in the main vertical, and let the new proposed vertical bracing system to perform better, and does not heavily reduce the fundamental period of the bridge. In spite of the fact that the bridge after adding members became stiffer, but better stress redistribution has been gained as well. Plastic hinges have been located in the new proposed vertical bracing system and the result for this phenomenon has been added to the table.

The analysis indicates that the installation of the energy dissipating bracing results in an acceptable solution if the bridge is to function as an emergency route structure but that this alternative is not adequate to make the bridge a lifeline structure. It was shown in the dynamic analysis of this alternative 3 that the bridge has C/D ratios of less than 1 for the 2% probability of exceedance in

50 years (i.e. 0.87), The stresses in the replaceable link bracings and the main verticals exceed the ultimate capacity of these members, (the substructure assumed adequately repaired here). The stresses and those ratios are acceptable for the 5% and 10% probability of exceedance in 50 years (i.e. 1.2, 1.6) respectively. The stability of this alternative is guaranteed as displacements are less than the ultimate permissible displacement.

2.4 Option 4 (Seismic isolation):

The isolation will be applied to all directions, however the lateral direction (Y axis) will get little benefit for the isolation as the abutments limit it. In other words the only longitudinal movement of the bridge is 70 mm which is the gap of the expansion joints was allowed. And no period shift in that direction could be utilized.

The strengthened bridge after adding members was analyzed regarding the isolation, because no benefit would be acquired if the existing bridge is isolated. The strengthened one has a controlled displacement demands, mode shapes, and better internal force redistribution.

The bridge was analyzed assuming that the expansion joints allow small movement under the design earthquake.

Different effective stiffnesses of isolation systems have been modeled for each support in order to gain the best displacement demands that may not affect the stability criterion.

Isolation of pier 1 gave more displacements and little increase (6%) in the fundamental period (T_y).

The more Increase of the flexibility (less stiffness) of the isolation modeled, the more contribution of the main bascule trusses (moving parts), however not much less forces in the main carrying elements, so there should be a balance between the stiffness of the isolator that may adapted and the strength of the all members in the system. The policy is to figure out the best cost-effective solution for adding/replacing members regarding the contribution of specific members needed.

The isolation in Piers P2 and P3 attracts the horizontal bracing system in the whole bridge to work together to increase seismic performance.

Isolation gave less concentration of forces in the short verticals over the supports.

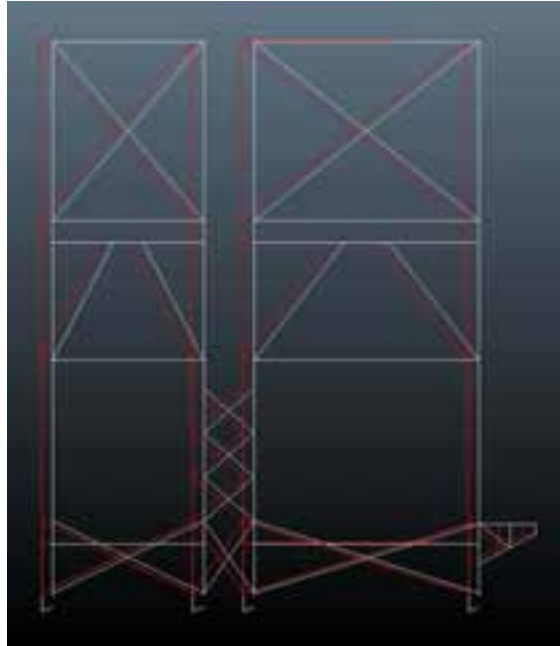


Fig (A-D 2.4) the elastic deflected shape of the main verticals of the isolated bridge, during the first mode of vibration (first shock)

The demand/capacity values given by the analysis indicate that the combination of seismic isolation bearings placed under the counterweight tower and the energy dissipating bracing described under Alternative 3 improve the response of the bridge.

It was shown in the dynamic analysis of this alternative 4 that the bridge has C/D ratio more than 1 for the three proposed earthquakes, (i.e. 2%, 5%, 10 % probability of exceedance in 50 years) and those ratio are 1.35, 1.84, 2.4 respectively. The stresses in the existing bracings and the main verticals are acceptable (the substructure assumed adequately repaired here).

The maximum displacement of the isolators controlled to be less than 250 mm, and the maximum displacement for the three proposed earthquakes, (i.e. 2%, 5%, 10 % probability of exceedance in 50 years) were 136, 180 and 246 mm respectively. The maximum displacement of the whole system (the bridge) during 10% probability of exceedance in 50 years event in three directions X=42 mm, Y= 180 mm, Z= 38 mm were acceptable.

The ultimate capacity of the bridge that meet the requirements of the $C/D = 1$ (i.e. the critical case) without any repair work of the existing substructure for this alternative was found to be 17% probability of exceedance in 50 years with return period 270 years, and the $PGA = 0.21$ g.

Further research for this alternative would also be required to ensure it is acceptable under the dynamic loads present in service conditions.

2.5 Option 5 (Drilled shafts / Substructure Upgrading):

The strengthened superstructure was modeled in this case as well.

Attention has been paid to the constructability, the limited work place, and the influence on the existing substructure. The dimensions of the shafts and the platform were selected to achieve the required stiffness in the assigned limited space available in the existing structure.

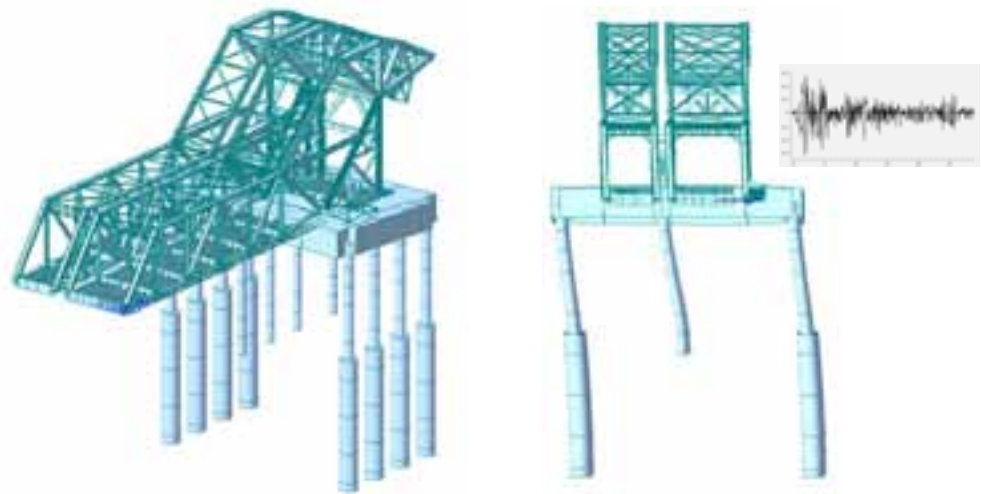


Fig (A-D 2.5) a; the proposed substructure upgrade, b; the deflected shape of the bridge during the design seismic event

The depth through the bedrock is a minimum of 5m and the sleeves extend in some places to 15 m. The platform is 4 rigid beams connect the shafts, the dimensions considered in the analysis were: 4 x10 m, 4 x 8.25 m and 4 x 2.5 m. 8 identical shafts were considered in the alternative, 4 in each row. And another row runs along the axis of the bridge. The number of shafts was selected to increase the redundancy of the substructure. The average length of the shafts is approximately 38 m. The diameter was taken as 1500 mm at the top 16 m and 2700 mm in the bottom 16 m. These lengths were considered in determining the optimum location of the plastic hinge.

The steel plastic hinge locations were specified in the new eccentric bracings and the shafts would be considered as strength protected, attention should be paid to provide detailing for these shafts to form the plastic hinge above the changing diameter of the shafts, and near the sand deposits line to get better performance with less cost of repair after an extreme seismic event.

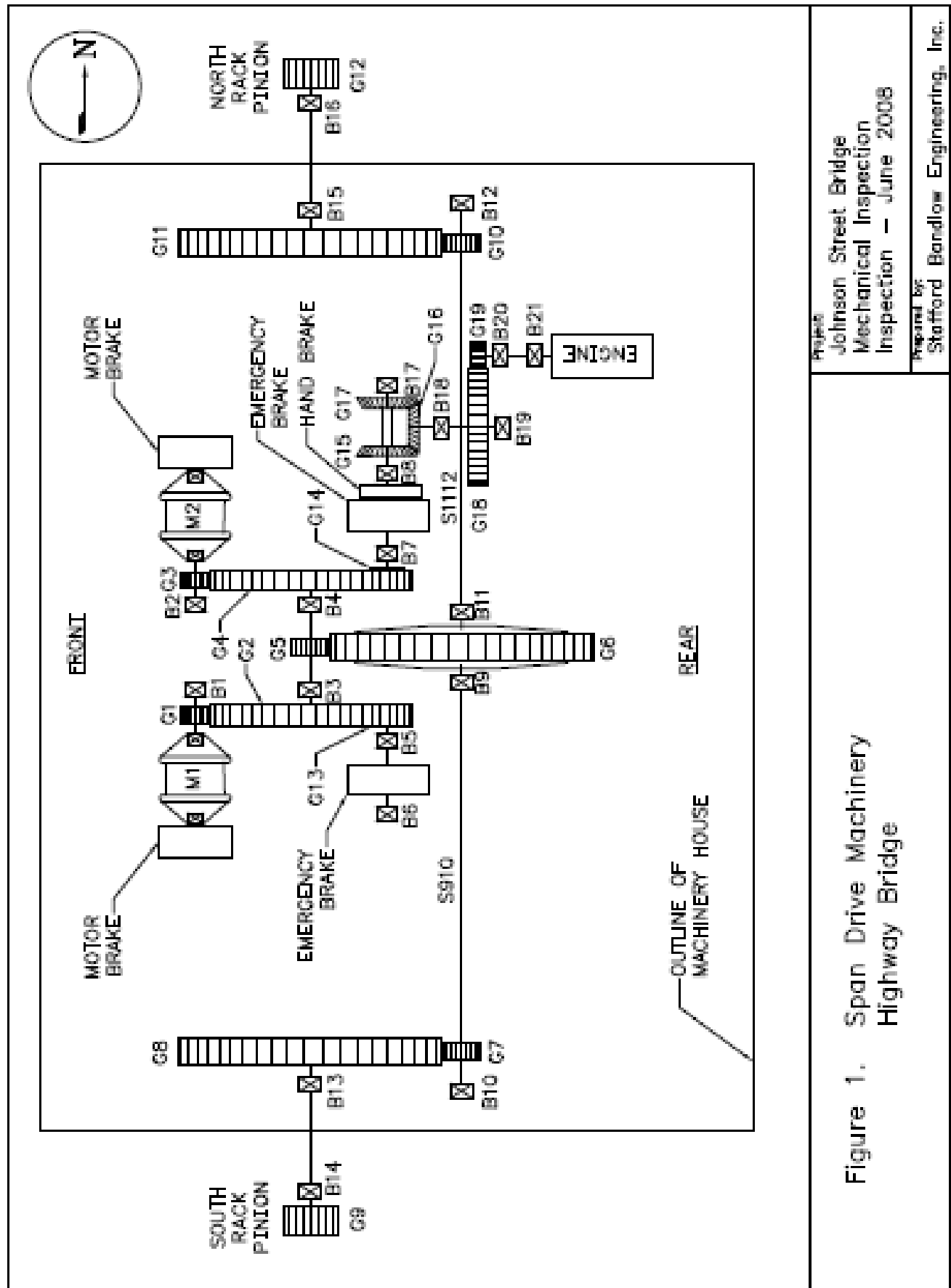
Vertical components for seismic forces has been considered in this case as the shafts are sensitive to the vertical forces and the combined interaction of horizontal and vertical forces. The combinations from CAN/CAS S6-06 was used.

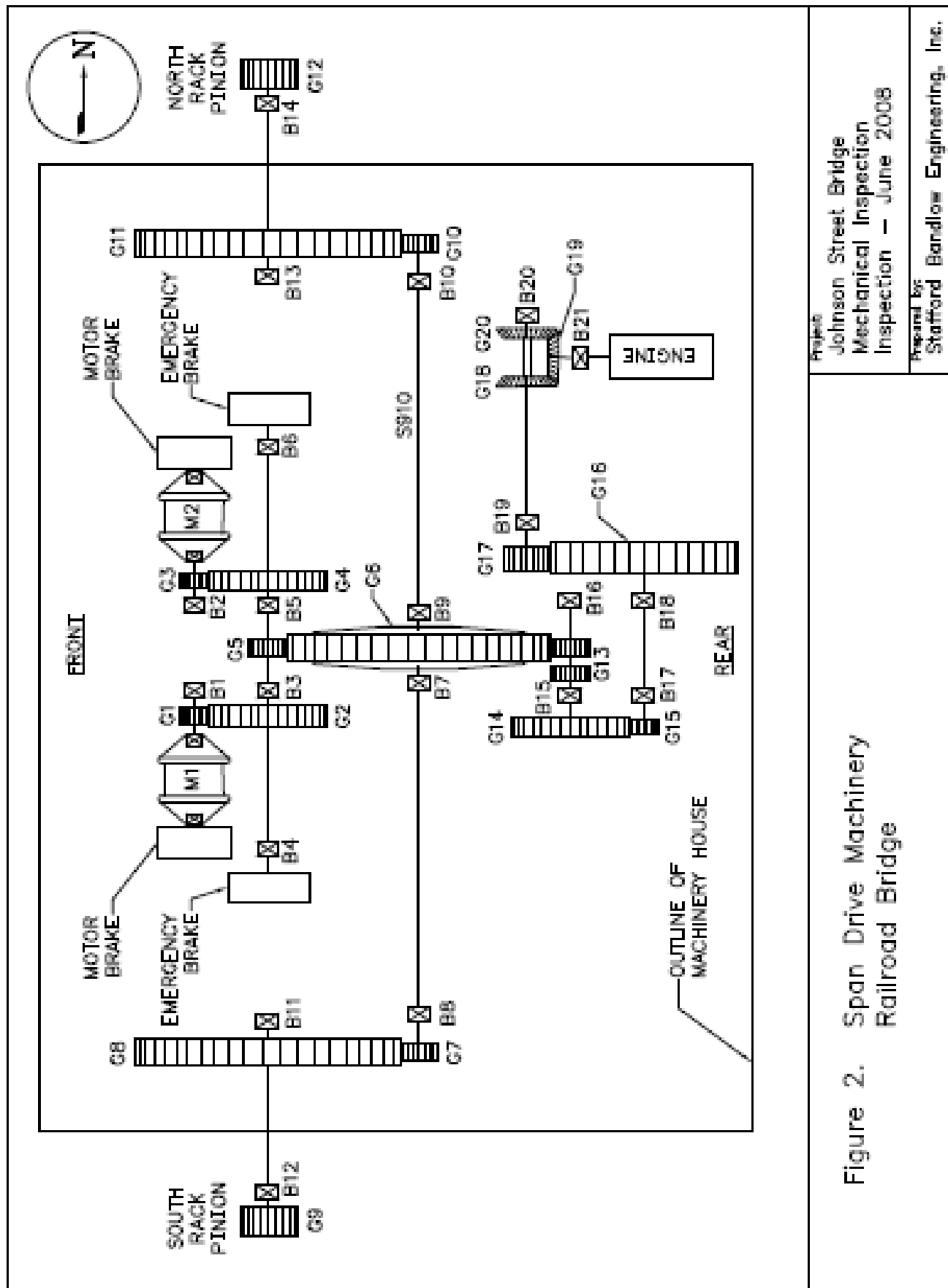
There is a great stress reduction in the substructure for this option. This reduction will improve if the strengthening policy of the bridge changed to match the overall required behavior, and this alternative could be considered as the best and only alternative as the whole elements in the bridge contribute, i.e. multi load paths which increase the degree of redundancy, which is the main concern in this kind of bridges.

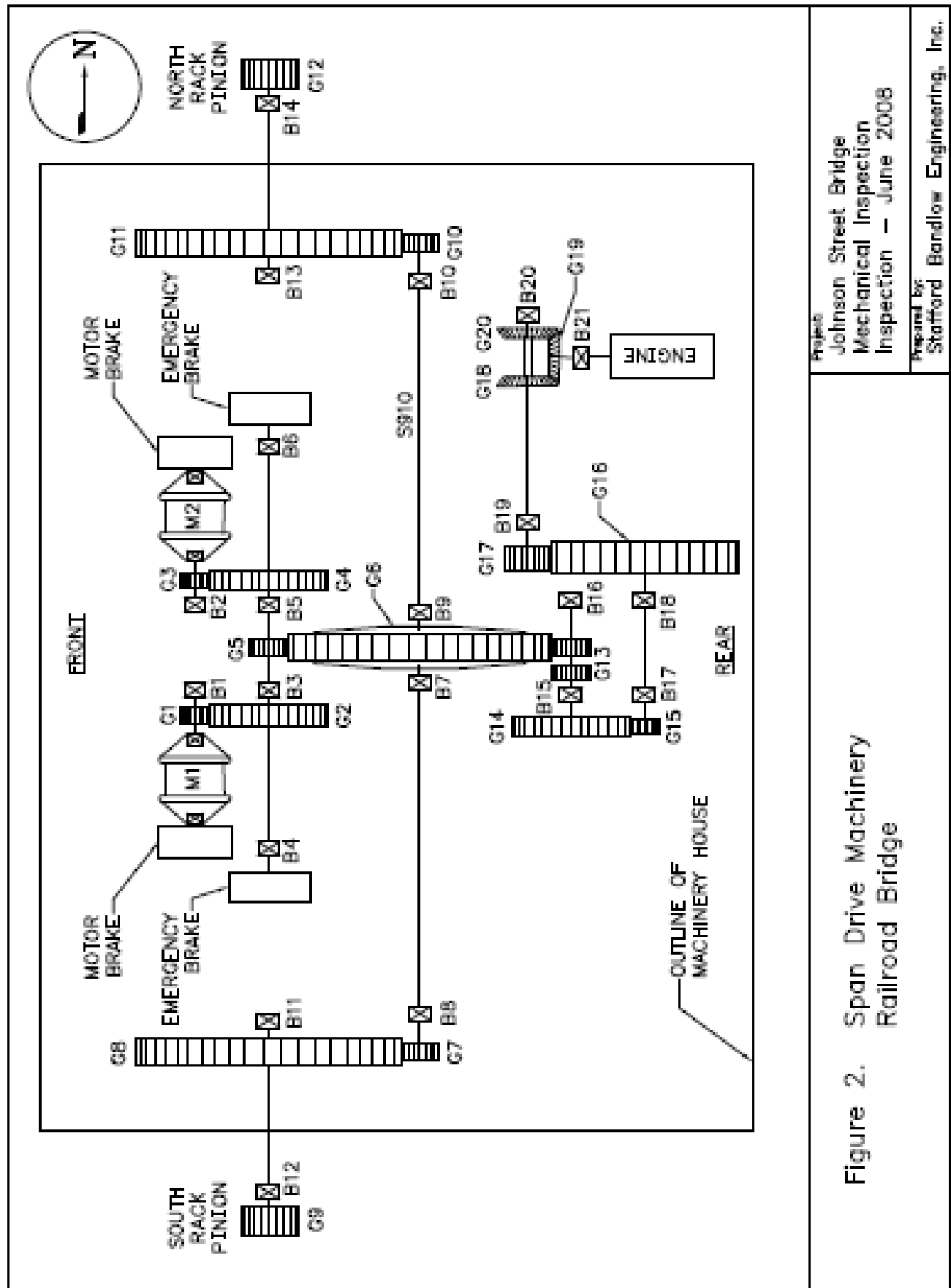
The maximum displacement of the whole system (the bridge) in during 10% probability of exceedance in 50 years event in three direction $X=39$ mm, $Y= 414$ mm, $Z= 37$ mm were acceptable

It was shown in the dynamic analysis of option 5 that the bridge has a C/D ratio of more than 1 for the three proposed earthquakes,(i.e. 2%, 5%, 10 % probability of exceedance in 50 years) and those ratio are 1.7, 2.3, 3 respectively. The stresses in the existing bracings and the main verticals are acceptable, and this alternative is the most reliable for major earthquakes.

Appendix E
Diagrams of Span
Drive Machinery

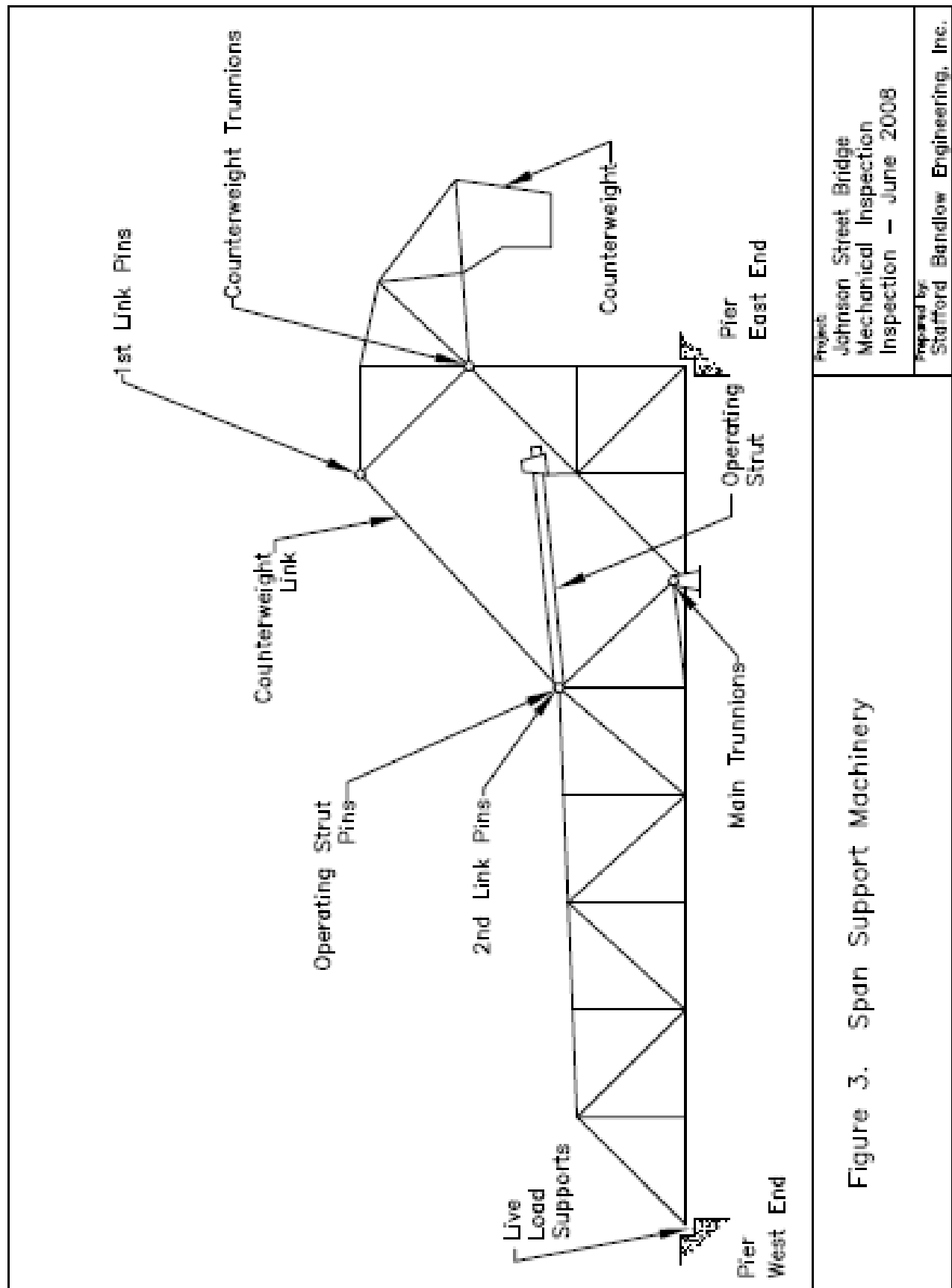






Project
Johnson Street Bridge
Mechanical Inspection
Inspection - June 2008

Prepared by
Stafford Bandlow Engineering, Inc.



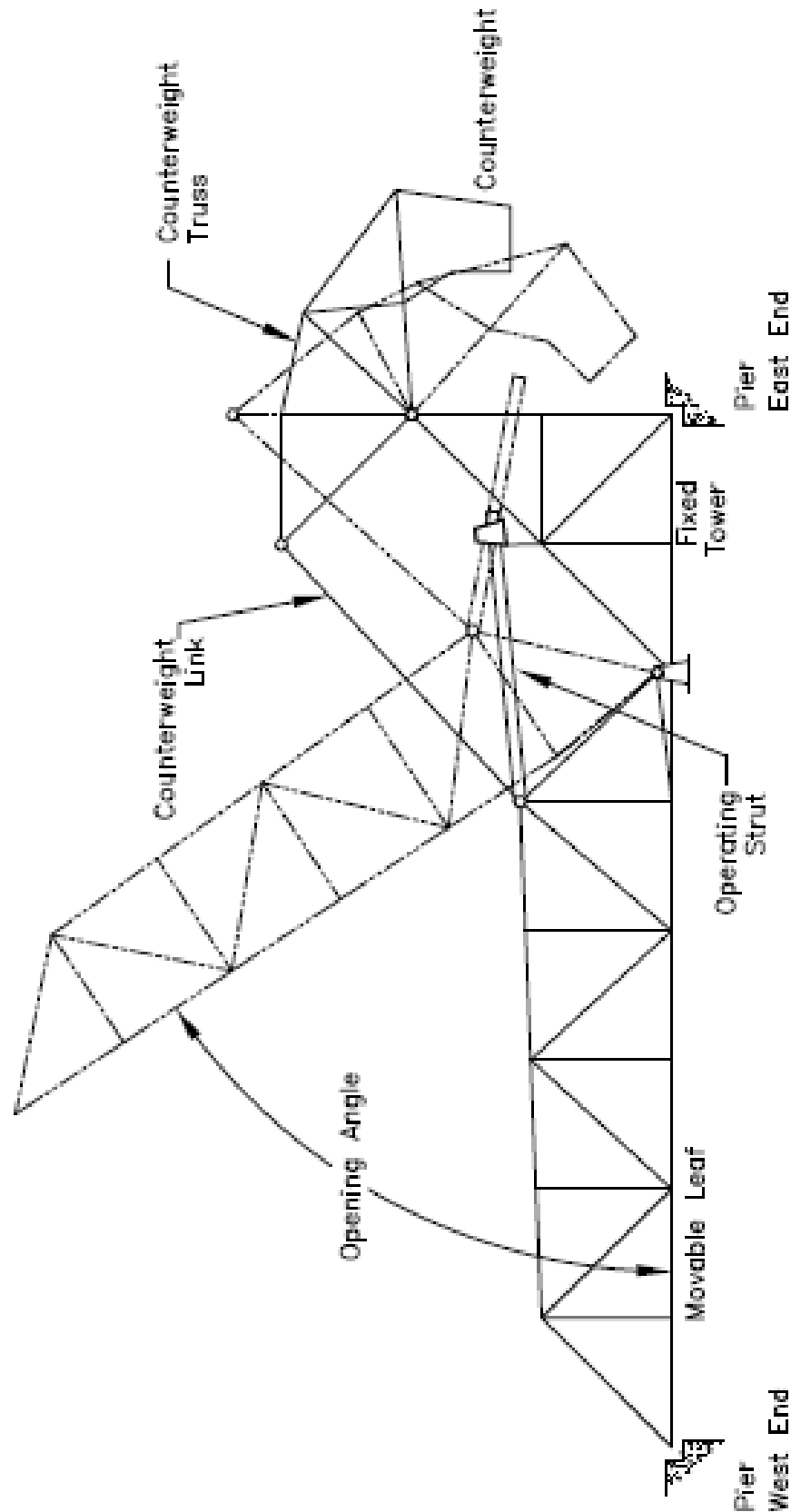


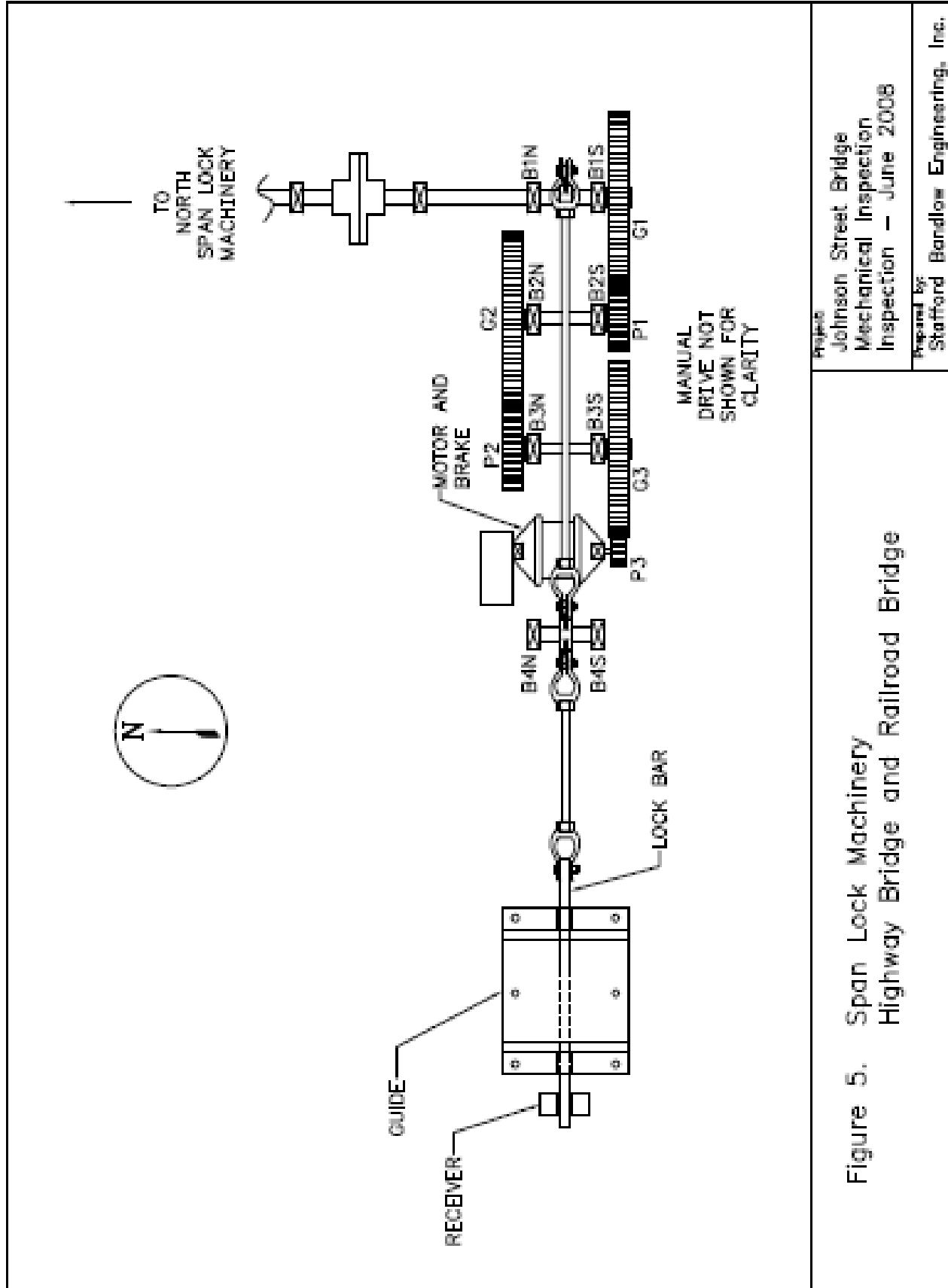
Figure 4. Schematic of Johnson Street Bridge
Shown in Closed Position – Solid Lines
Shown in Open Position – Phantom Lines

Project

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Appendix F

Mechanical Photographic Records



Photo M-1.1. Highway Bridge, Bearing B3. Note the deformation of the bearing cap and the degradation of the shaft.



Photo M-2. Highway Bridge, Bearing B12. This bearing oscillates during operation of the machinery. Note that the base bolts have been replaced.



Photo M-3. Highway Bridge, Bearing B13. Note the gap between the nuts and the support indicating that the bolts are loose.



Photo M-4. Highway Bridge, Bearing B14. The body of the upper inboard bearing mounting bolt is behind the heavily corroded stiffener. Note the accumulated debris.



Photo M-5. Highway Bridge, Bearing B16. The upper inboard bearing mounting bolt is located behind the stiffener. Note the accumulated debris and corrosion.



Photo M-6. Railroad Bridge, Bearing B14. Note the center lube fitting has been replaced with a pipe cap.



Photo M-7. Highway Bridge, North Motor Brake. The brake assembly is covered with lubricant and lubricant has contaminated the brake wheel.



Photo M-8. Highway Bridge, South Motor Brake. The brake has been retrofit with a custom actuator. A cotter pin is missing from one of the pins near the actuator.



Photo M-9. Railroad Bridge, North Emergency Brake. Note the loose bolts (arrow) at the solenoid.



Photo M-10. Railroad Bridge, South Emergency Brake. The taper gage is used to demonstrate clearance between the brake shoe and the brake wheel.



Photo M-11. Highway Bridge, Gear G2. The key for this gear is loose. The key was pulled out by hand to take this photo and then returned to its original position.



Photo M-12. Highway Bridge, Gearset G7/G8. Note the heavy accumulation of hardened lubrication deposits in the root of the teeth. Some of the lubricant was scraped away prior to taking the photo.



Photo M-13. Railroad Bridge, Gear G4. Note the addition of a second key secured by a hose clamp to restrain the primary key. Also note the fretting corrosion at the interface of the shaft and gear hub.



Photo M-14. Railroad Bridge, Gearset G5/G6. The abrasive wear and scoring seen on this gearset was the most severe surface degradation found at any of the gears.



Photo M-15. Railroad Bridge, Gearset G7/G8. Note the accumulated lubricant below Gear G7.

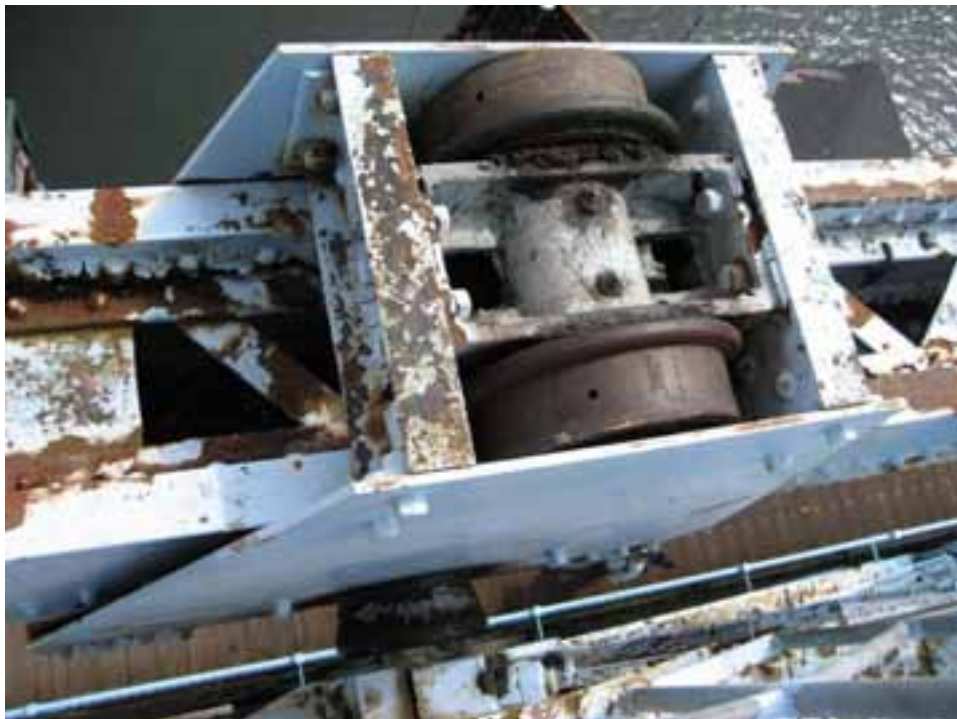


Photo M-16. Highway Bridge, South Operating Strut Guide. Note the paint deterioration and light to moderate corrosion.



Photo M-17. Highway Bridge, South Operating Strut. Note the paint deterioration and light to moderate corrosion.



Photo M-18. Highway Bridge, South Operating Strut. This is the area between the structural channels that form the operating strut. Note the accumulated debris.



Photo M-19. Highway Bridge, South Operating Strut Upper Outboard Guide Roller. The original drawings indicate a gap of 1/8" at this location. At the time of the inspection the gap measured between 0.350" and 0.475"



Photo M-20. Highway Bridge, Motor M2. Note the puddle of oil (arrow) between the brake and the motor.



Photo M-21. Railroad Bridge, Motor M1. The cover for the oil check fitting (arrow) is missing.



Photo M-22. Railroad Bridge, Motor M2. The opened cover provides access to the motor bearing. The bearing appears severely damaged but could not be photographed due to limited access. Also note the puddle of oil to the left of the motor bearing.



Photo M-23. Highway Bridge, Hand Brake. The brake assembly is coated with lubricant and there is oil on the brake wheel.



Photo M-24. Highway Bridge, Auxiliary Bridge Engine. The engine has a significant oil leak.



Photo M-25. Railroad Bridge, South Counterweight Trunnion. All of the lube piping is capped at this side (south) of the bearing and there is no indication that the caps have been removed recently. Proper maintenance requires removal of the caps to purge old lubricant.



Photo M-26. Railroad Bridge, North 2nd Link Pin. The accumulated lubricant (arrow) at the south side of the pin may indicate a loose or failed lube pipe connection.



Photo M-27. Railroad Bridge, South Counterweight Trunnion. The condition of this trunnion is typical of all the counterweight trunnions. Note the paint deterioration and corrosion.



Photo M-28. Highway Bridge, North Main Trunnion Bearing. The condition of this trunnion is typical of all the main trunnions. Note the accumulated debris.



Photo M-29. Highway Bridge and Railroad Bridge. The key for Gear G1 contacts the machinery enclosure. This photo is of the railroad bridge however the condition exists at both bridges.



Photo M-30. Highway Bridge, Span Lock Machinery. The nut for the connecting rod (white arrow) is loose, the connecting rod is bent, the thrust collar (yellow arrow) for the hand crank shaft is not properly position and the enclosure is in poor condition.



Photo M-31. Railroad Bridge, Span Lock Machinery Bearing B1. The nut for the bearing bolt has complete section loss due to corrosion.



Photo M-32. Railroad Bridge, Span Lock Machinery Enclosure. The enclosure is in poor condition.



Photo M-33. Highway Bridge, Air Buffer. The four mounting bolts for the bottom bearing are loose.



Photo M-34. Railroad Bridge, South Live Load Support. Three of the four anchor bolts for this support are loose. Note the paint deterioration and corrosion.

Appendix G

Electrical Photographic Records



Photo E1.1.1. Highway Machinery Room. Note difficult accessibility of drive motors.



Photo E1.1.2. Highway South Drive Contactor Panel. Note improper color coding of conductors.



Photo E3. Highway North Drive Motor Contactor. Note improper color coding of conductors.



Photo E4. Railway South Drive Motor Contactor. Note improper color coding of conductors.



Photo E5. Railway North Drive Motor Contactor. Note improper color coding of conductors.



Photo E6. Typical Motor Brake Enclosures. Note improper color coding of conductors.



Photo E7. Typical Emergency Brake Enclosures. Note improper color coding of conductors.



Photo E8. Typical Emergency Brakes. Note conductors not properly protected from accidental contact.



Photo E9. Typical Highway Brakes. Note brakes are not equipped with position limit switches.



Photo E10. Control Desk. Note age of equipment and indicators.



Photo E11. Wall Mounted Control Panel. Note indicators, buttons, and switches.

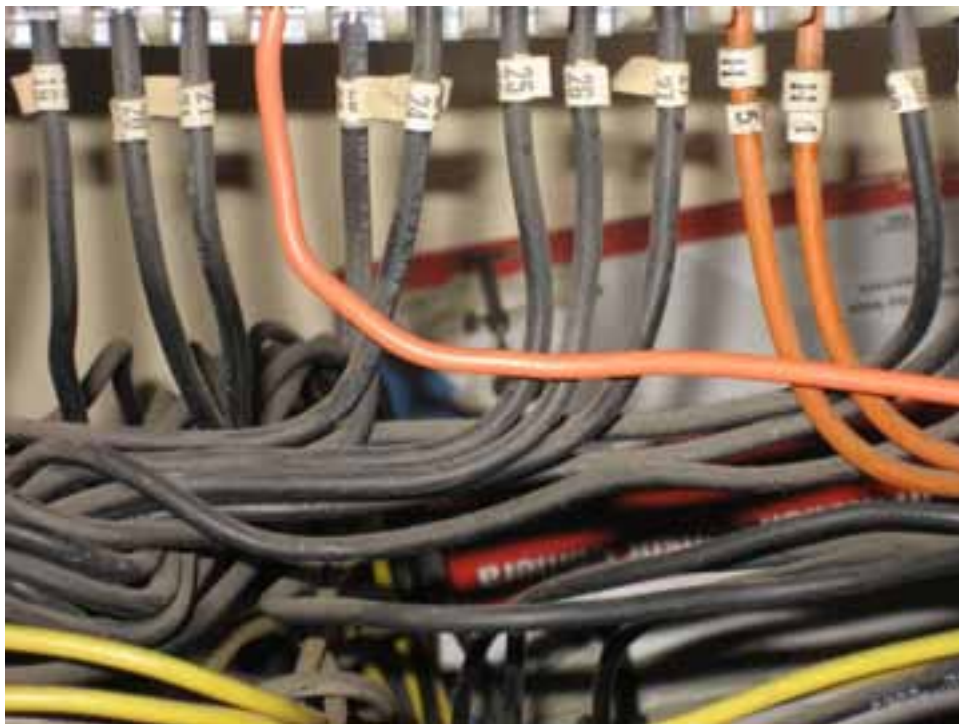


Photo E12. Control Desk. Note flammable materials stored in control desk housing.



Photo E13. Wall Mounted Control Panel. Note flammable materials being stored in and on equipment inside enclosure.



Photo E14. Control Desk. Note drum control switch used for span operation of bridge.



Photo E15. Typical Span Position Limit Switch. Note the fair to poor condition of the limit switch.



Photo E16. Control Desk. Note age and condition of span lock indicators.



Photo E17. Southeast Traffic Gate. Note traffic gate arm does not set at 90 degrees vertical.



Photo E18. Southeast Traffic/Pedestrian Gate. Note damage of both gate housing and arm.



Photo E19. Northeast Traffic Gate. Note traffic gate arm does not set 90 degrees vertical.



Photo E20. Conduit Box. Note the water-tight connection is damaged.



Photo E21. Northwest Traffic Gate. Note damaged traffic gate arm.



Photo E22. Northeast Railway Pedestrian Gate. Note the northeast railway pedestrian gate is over 90 degrees.



Photo E23. Northwest Railway Gate. Note the water-tight fitting for the SO cord is cracked.



Photo E24. Northeast Railway Gate. Note the water-tight seal for the conduit "LB" is damaged.



Photo E25. Northwest Railway Gate. Note the gate arm shows evidence of being damaged.



Photo E26. Northwest Railway Gate. Note minor surface corrosion on gate housing.



Photo E27. Southeast Traffic Gate. Note housing for the gate motor is not secured. Non-qualified personnel can access the electrical and mechanical equipment.



Photo E28. Far Side Submarine Cable Junction Box. Note the location of the junction box that the two submarine cables enter.



Photo E29. Typical Bascule Span SO Cables. Note the poor condition of both the SO cables and their associated junction boxes.



Photo E30. Highway Machinery Room. Note missing cover for the conduit tee body.



Photo E31. Highway Machinery Room. Note the use of solid conductors in the distribution equipment for power circuits.



Photo E32. Railway Machinery Room. Note the use of solid conductors in the distribution equipment for power circuits

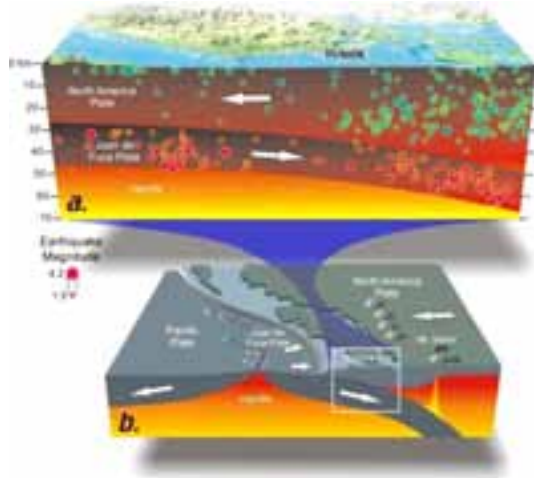


Photo E33. Control House. Note the use of solid conductors in the distribution equipment for power circuits.

Appendix H

Historical Seismic Records

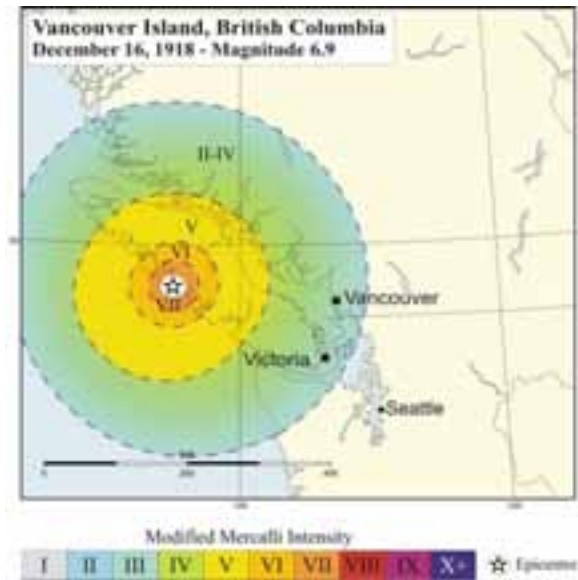
Victoria is located at the northern portion of the Cascadia Subduction Zone where active subduction of the oceanic plate beneath the continental North America plate has produced 36 moderately felt earthquakes (MMI \geq IV) within the past 139 years. Damaging earthquakes have included the 1949 MW 7.1 Puget Sound earthquake, the 1965 MW 6.5 Seattle earthquake, and the 2001 MW 6.8 Nisqually earthquake. In addition Two large plate earthquakes have occurred on Vancouver Island this century: a MS 6.9 event in 1918 and a MS 7.3 event in 1946.



As appoint of reference, the largest seismic event recorded in Victoria in 50 years was the 2001 MW 6.9 Nisqually event which caused peak horizontal ground accelerations (PGA) varying from .01g to .035 g. As such, it is clear that considerably higher earthquakes are possible in Victoria than the 2001 event and based on the above, the Johnson Street Bridge has only been exposed to moderately small earthquakes.

This section provides sources to obtain information on current and past seismic records across Canada and major earthquakes that caused significant structural damages, economical and life losses near the Victoria Region. It should be noted that information provided via telephone by the GSC Pacific in Sidney, B.C state that no earthquakes have occurred in the surrounding area of the Johnson Street Bridge and thus previous information on what the bridge can withstand is unavailable.

Past Seismic Events around Vancouver Island



1918 Earthquake

This large earthquake occurred just after midnight (12:41 am) on Friday December 6, 1918. The magnitude is about 7. Its exact location is uncertain, but it occurred near the west coast of Vancouver Island, and was felt very strongly at Estevan Point lighthouse and at Nootka lighthouse on the southern tip of Nootka Island. There was some damage to the Estevan Point lighthouse and to a wharf at Ucluelet. This earthquake awakened people all over Vancouver Island and in the greater Vancouver area. It was felt in northern Washington state and as far east as Kelowna, in the interior of British Columbia.



1946 Earthquake

Vancouver Island's largest historic earthquake (and Canada's largest historic onshore earthquake) was a magnitude 7.3 event that occurred at 10:15 a.m. on Sunday June 23, 1946. The epicenter was in the Forbidden Plateau area of central Vancouver Island, just to the west of the communities of Courtenay and Campbell River.

For the 1946 mid Vancouver Island earthquake, no measurements had been taken, so the uncertainty of what the ground motion was is a higher. We can approach the problem by using theoretical attenuation with distance curves for seismic waves and by looking at the intensity of shaking that individuals described in newspapers and personal reports and assigning an approximate acceleration. Both techniques give about the same answer. Peak acceleration on firm soil was likely in the range of 5%g to 10%g and up to a factor of 2 larger on pockets of soft soil.

This earthquake caused considerable damage on Vancouver Island (see photos below), and was felt as far away as Portland Oregon, and Prince Rupert B.C. The earthquake knocked down 75% of the chimneys in the closest communities, Cumberland, Union Bay, and Courtenay and did considerable damage in Comox, Port Alberni, and Powell River (on the eastern side of Georgia Strait). A number of chimneys were shaken down in Victoria and people in Victoria and Vancouver were frightened - many running into the streets. Two deaths resulted from this earthquake, one due to drowning when a small boat capsized in an earthquake-generated wave, and the other from a heart attack in Seattle.



Port Alberni B.C – Chimney Rotation



Comox B.C – House Foundation Failure



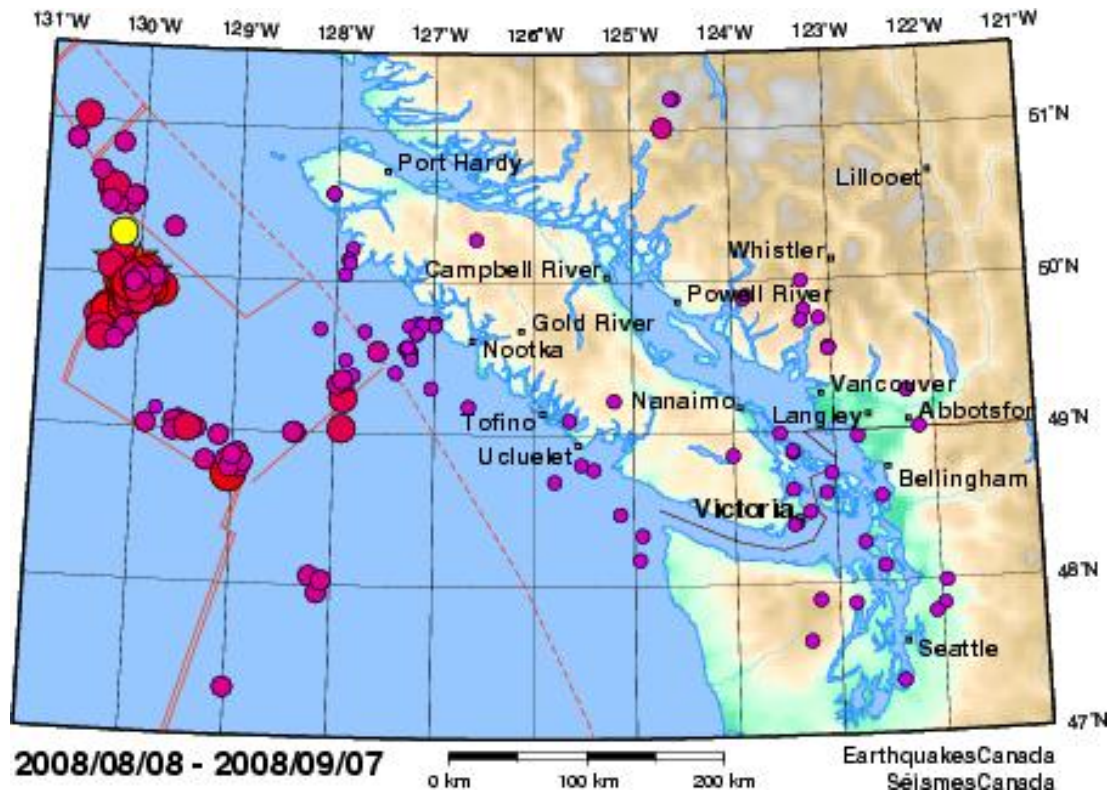
Courtenay B.C – Masonry Failure



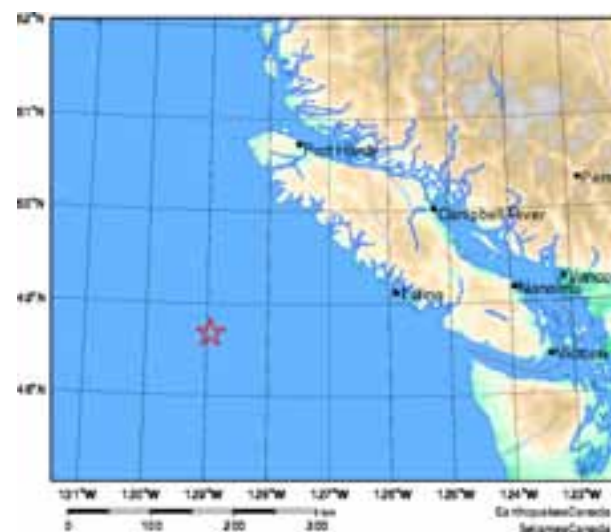
***North of Campbell River B.C
Soil Failure on the Kelsey Bay Highway***

Earthquakes Canada provides comprehensive information on recent and historic earthquakes, seismic hazard maps, general earthquake information, researches and publications. The reader can follow the link below to find relevant information on seismic activities across Canada:
http://earthquakescanada.nrcan.gc.ca/index_e.php.

Recent Seismic Events around Vancouver Island



Earthquake Information	
Sunday June 01, 2008	
Local Time:	07:24:12 PDT
Magnitude:	3.6 ML
Latitude:	48.68 North
Longitude:	128.94 West
UT Date and Time:	2008-06-01 14:24:12 UT



Earthquake Information

Friday May 30, 2008

Local Time: 05:38:29 PDT

Magnitude: 4.1 ML

Latitude: 50.46 North

Longitude: 130.41 West

**UT Date 2008-05-30
and Time: 12:38:29 UT**

