Sgt. Aubrey Cosens V.C. Memorial Bridge over the Montreal River at Latchford

Investigation of Failure: Final Report

December 1, 2003

Prepared By: **Dino Bagnariol, P.Eng.**
Head Evaluation & Inspection Engineer
Bridge Office
Ministry of Transportation

Reviewed By: **Bala Tharmabala, P.Eng.**
Manager, Bridge Office
Ministry of Transportation

Brian Peltier, P.Eng.
Chief Engineer / Director
Engineering Standards Branch
Ministry of Transportation

Contents **Page No.**

Executive Summary 2
1) Introduction.....4
2) Scope of Investigation 5
3) Summary of the Cause of Failure 6
4) Failure Sequence.....7
5) Preliminary Visual Examination.....10
6) Metallurgical Analysis.....10
7) Steel Structure Condition Survey 12
8) Post-failure Inspections.....13
9) Structural Analysis.....16
10) Design and Rehabilitation History.....17
11) Inspection History.....21
12) Conclusions and Recommendations 23

Appendices

Appendix A – Preliminary Visual Examination Report
Appendix B – Metallurgical Report
Appendix C – Steel Structure Condition Survey Report (Part 1 and Part 2)
Appendix D - Structural Analysis Report
Appendix E – Interviews
Appendix F – Peer Review Panel

Executive Summary

A partial failure of the Sgt. Aubrey Cosens VC Memorial Bridge occurred on January 14, 2003 at approximately 3:00 p.m. This steel arch bridge is located on Highway 11 in Latchford and spans the Montreal River. As a Southbound tractor-trailer crossed the bridge, the concrete deck deflected approximately 2 metres at the Northwest corner due to the failure of 3 hanger rods. The bridge was immediately closed to traffic.

This report documents the Ministry of Transportation's (MTO) investigation into the cause of failure. The report describes the cause of failure and makes recommendations on how to prevent similar failures in the future.

In summary, the partial failure of the Sgt. Aubrey Cosens VC Memorial Bridge was caused by the fatigue-induced fracture of three steel hanger rods on the northwest side of the bridge. The failure of these rods can be attributed to a combination of factors:

1) The original design did not consider that the pins in the hangers could seize and cause bending fatigue stresses in the rods. The bending fatigue stresses led to the eventual fracture of the rods.
2) The threaded portion of the rods was damaged during construction 40 years ago.
3) The quality of the steel does not meet current standards for ductility in cold temperatures and chemical composition.
4) The critical parts of the hanger rods were hidden from inspection since they were inside the arch.

Recommendations stemming from this investigation can be grouped into the following three categories:

(a) Design Practices

It should be noted that the design of this bridge is unique and that it is the only one of its kind on the provincial highway network. In comparing design practices of the 1950's and those used today, many improvements have been made in this area. Current design codes recognize the problems associated with non-redundant structures and fatigue prone details. However, further improvements are recommended:

1) **The effects of connection rigidity and resulting bending fatigue stresses should be considered in the design and evaluation of steel arch and truss bridges.**
2) **For future designs and rehabilitations, the designer should identify critical details and ensure that the details can be easily inspected after construction.**

(b) Material and Construction Specifications

Similar to design specifications, material and construction specifications have improved greatly over the last 40 years. New and improved processes and materials, as well as more rigorous quality control and quality assurance requirements during the fabrication and construction processes, have improved the overall quality of steel and other materials. Material testing and performance in cold temperatures are integral parts of the steel fabrication process today.

As such, no new recommendations are being made under this category.

(c) Inspection Practices

The Ministry of Transportation has been improving bridge inspection practices over the last several years. The introduction of Ontario Regulation 104/97 in 1997 and the 2000 revision of the Ontario Structure Inspection Manual (OSIM) have provided better guidelines for inspectors to follow. However, further hands-on training for inspectors is warranted with particular attention to similar non-redundant, fatigue prone bridges. The following is recommended:

1) **Immediate inspection of MTO arch bridges with pin and hanger connections *(Completed).***
2) **Advise municipalities and other owners to inspect bridges with similar details *(Completed).***
3) **Expand sections of OSIM to emphasize the requirements for access and "close-up" visual inspection of each component during biennial inspections.**
4) **Provide additional OSIM training for inspectors to ensure that overall structure integrity is assessed by identifying unusual deflections, flexibility and other minor defects.**
5) **Detailed fatigue inspections and non-destructive testing of fatigue critical components should be carried out every 5 years.**
6) **For painting contracts, detailed inspection of all structural steel work should be carried out by a Professional Engineer for certain structures, after cleaning of steel and before painting.**
7) **Increase frequency of "walk-about inspections" by maintenance staff or area maintenance contractors for certain bridges.**
8) **Provide additional bridge inspection training for maintenance staff.**

(1) Introduction

A partial failure of the Sgt. Aubrey Cosens VC Memorial Bridge over the Montreal River at Latchford occurred on January 14, 2003 at approximately 3:00 p.m. As a Southbound tractor-trailer crossed the bridge, the concrete deck deflected approximately 2 metres at the Northwest corner due to the failure of 3 hanger rods (See Figure 1). The bridge was immediately closed to traffic.

The Sgt. Aubrey Cosens VC Memorial Bridge is located on Highway 11, between Temagami and New Liskeard. It is a 110 metre single span steel Through Arch bridge with vertical hangers. A concrete deck, supported by 12 vertical hangers on each side, is connected to 12 floor beams. The top portion of each hanger has a threaded rod that extends through the bottom plate of the steel arch and is held in place by two nuts. Each hanger also has an eyelet/pin system (top and bottom) to allow free rotation about a transverse axis. A series of steel stringers (6 per bay) tie the floor beams and the concrete deck together. The bridge was built in 1960, rehabilitated in 1992 and the structural steel painted in 1998.

This report documents the Ministry of Transportation's (MTO) investigation into the cause of failure of the Sgt. Aubrey Cosens VC Memorial Bridge. The main objectives of the investigation are:

- To determine the cause of failure and the sequence of events leading up to failure
- To make recommendations on how to prevent similar failures in the future

Three independent professional engineers have reviewed this report as part of a peer review process.

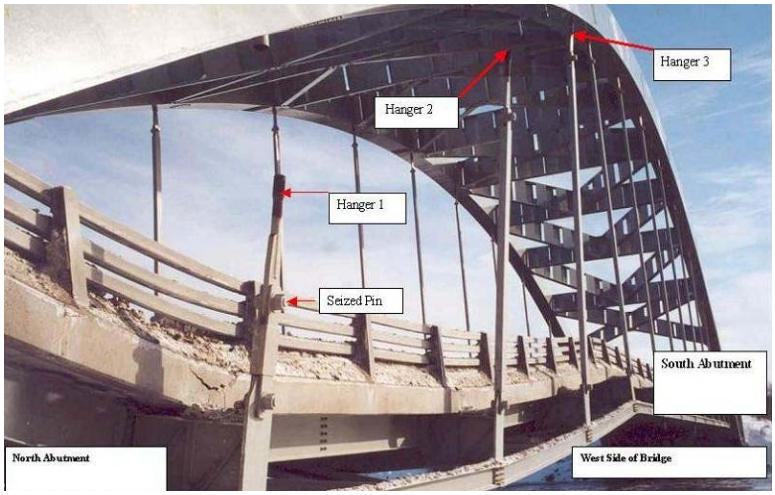

Figure 1: Northwest Corner

(2) Scope of Investigation

This report was compiled by reviewing information from several sources and then piecing the evidence together to form a scientific opinion as to what caused the partial failure of the bridge. The sources of information include the following:

(a) Site inspections after the failure
- As part of the investigation, four post-failure visual inspections of the bridge were conducted between January 15 and July 8, 2003. Observations noted during the inspections are summarized in Section 8.
- An MTO metallurgical Engineer also visited the site in January 2003 and recorded his observations (See report dated February 7, 2003 in Appendix A).
- A structural steel inspection company was retained to conduct a thorough inspection of all steel components on the bridge between March and July 2003. Non-destructive testing (Ultrasonic Testing and Magnetic Particle Testing) was also used for various components. (See reports dated April 11 and July 11, 2003 in Appendix C)

(b) Detailed metallurgical investigation of failed components
- An independent metallurgical expert was retained to conduct a visual examination, material testing and corrosion product analysis of the failed components (See report dated August 28, 2003 in Appendix B).

(c) Detailed Structural Analysis
- A detailed structural analysis was conducted using a 3-Dimensional computer model to further understand the structural behavior of the bridge and the possible failure sequence. (See report dated November 5, 2003 in Appendix D)

(d) Original design and rehabilitation drawings
- Original design drawings, shop fabrication drawings for various steel components, and 1992 rehabilitation drawings were used as references throughout the investigation
- The rehabilitation history of the bridge was also reviewed.

(e) Biennial Inspection and Maintenance Inspection records
- MTO inspection requirements and practices for the last 10 years were reviewed.
- Biennial Inspection reports over the last 10 years were reviewed
- Maintenance inspection reports over the last 4 years were reviewed

(f) Interview with contractor involved during temporary repair of damaged bridge
- Notes from this interview are in Appendix E

(g) Peer Review Process
- Three independent bridge experts have reviewed this report to certify that the findings are accurate and that appropriate steps and professional standards have been followed (See Appendix F).

(3) Summary of the Cause of Failure

The following is a summary of the conclusions made once all the information described in Section 2 was considered:

The main cause of failure was the fatigue-induced fracture of three vertical hanger rods that supported the deck (See Appendix B). The threaded rods fractured at the underside of the first nut, inside the arch section. Failure of the hangers was progressive over a period of several years. The failure of the hangers can be attributed to a combination of design, construction and material factors.

(i) Design Factors - Pin Seizure
In reviewing the design drawings, it appears that the hangers were originally designed to be free to rotate, about a transverse axis, at the top and bottom through pins. However, recent inspections during the investigation (See Sections 8 and 10) indicate that some of the pins had seized, thus preventing rotation. Once the pins had seized, the hanger rods were subjected to repetitive bending stresses as traffic moved over the bridge. Since the hangers were never designed to resist bending stresses, fatigue cracks subsequently developed in the hangers, and after a period of time, eventually fractured.

The seizing of the pins was confirmed by the contractor who removed some of the damaged components of the bridge in January 2003 (See Appendix E). The possible reasons for the pins seizing are discussed in Section 10. The seizing can be attributed to a combination of design detailing practices (no gap between hanger plate and brackets), corrosion and possible over-tightening of the nuts during construction.

(ii) Construction Factors – Hanger Installation
Another contributing factor to the failure relates to defects found on some of the hanger rods. As described in Appendix B, the metallurgist discovered defects in Hanger 1 (NW), when the nut was removed from the threaded rod. Close examination revealed that two-thirds of a thread may have been ground away (probably during construction in 1960), in an unsuccessful attempt to remove defects in the thread (See Figure 2). The defects were not visible once the nut had been placed on the threaded rod. The metallurgist confirmed that the fatigue cracks started at this location. These defects, combined with the seized pins, led to the propagation of the fatigue cracks in Hanger 1 (NW). Fatigue cracks also developed in other hanger rods as is discussed in Sections 6 and 7. However, these cracks propagated more slowly than in locations where defects were found in the threads.

(iii) Material Factors – Quality of Steel
The quality of the steel used for the hanger rods also played a role in the failure. Metallurgical tests performed on the hanger rods (See Appendix B) indicate that the steel that was used did not remain ductile in temperatures below –18 degrees Celsius, since the ductile to brittle transition temperature was between –18 and –29 degrees Celsius. Although the cold temperature contributed to the failure of Hanger 3 (NW) and the partial collapse of the deck on January 14, 2003, it was not the ultimate cause of failure. The hanger would have failed eventually due to fatigue and microscopic brittle cracking that occurred gradually over a period of several years.

However, as noted by the metallurgist (See Appendix B), the brittleness of the steel in cold temperatures accelerated the microscopic brittle cracking over the years in addition to the onset of final fracture.

Figure 2: Hanger 1 (NW) – Shiny Area - Ground down threads.

(4) Failure Sequence

Similar to Section 3, the following failure sequence was theorized once all the information described in Section 2 was considered:

(i) Hanger 1 (NW)
Metallurgical analysis (See Appendix B) of the corrosion product on the "fast fracture" face of Hanger 1 (NW) indicates that there is a high probability that it had fractured 5 to 7 years prior to January 14, 2003. "Fast fracture" is defined as the final fracture that resulted in the actual separation of the two sections of the cracked hanger rod. Prior to final fracture, a portion of the hanger rod would have been cracked as a result of bending fatigue stresses.

Once Hanger 1 (NW) had failed, the load was transferred to the neighboring hangers and to the abutment by the steel stringers, floor beams and concrete deck system. Continued loading resulted in fatigue cracks developing in Hangers 2 and 3 (NW) and in the floor beam to stringer connection plate.

There is additional evidence to suggest that Hanger 1 (NW) had been fractured for several years prior to January 2003. The threaded part of Hanger 1 (NW) shows signs of abrasion and wear caused by rubbing against the steel sleeve (up and down motion), as traffic crossed the bridge (See Figure 3). The wear pattern on the hanger indicates that the vertical deflection of the hanger with respect to the sleeve occurred in two stages. Wear on the upper part of Hanger 1 (NW) is more pronounced and occurred over a longer period of time (It started when Hanger 1 (NW) fractured). Wear on the upper part of the hanger was measured to be about 30 mm along its length. The wear on the lower part of the threads is not as pronounced and occurred over a shorter period of time. (It started when Hanger 2 (NW) fractured). Total wear on the hanger was measured to be about 100 mm along its length. These abrasion values are consistent with the theoretical Live Load deflection values determined using structural analysis (See Section 9 and Appendix D).

Figure 3: Hanger 1 (NW) – Threaded Area Showing Signs of Wear, Abrasion, and Impact Damage

(ii) Hanger 2 (NW) and Stringer to Floor Beam 1(NW) Connection Plate

Corrosion product analysis of the fast fracture face of Hanger 2 (NW) indicates that there is a high probability that Hanger 2 (NW) had failed 1 to 3 years prior to January 14, 2003. Fatigue cracks likely started in the connection plate between Stringer 1 and the first floor beam (NW) after Hanger 1 had failed (See Appendix B). Analysis of the corrosion product on the stiffener face indicates that the fatigue crack progressed through the bolt holes over a period of several years (See Figure 4). The fatigue crack extended through 85% of the stiffener cross-sectional area. The very light corrosion on the final fast fracture surface (about 15 % of the cross sectional area) indicates that it probably completely failed on January 14, 2003 (Time of partial collapse of deck).

Figure 4: Exterior stringer to floor beam #1 connection (NW corner)

(iii) Hanger 3 (NW)
Once the continuity between the abutment, Stringer 1 and the floor beam had been severed, Hanger 3 (NW), adjacent hangers and adjacent stringer to floor beam connections were forced to carry the entire additional load from failed Hangers 1 and 2. However, Hanger 3 (NW) was incapable of carrying this additional load and failed. This led to the partial collapse of the deck on January 14, 2003.

(iv) Fractured Face of Hangers
Examination of the fracture faces of Hangers 1, 2 and 3 (NW) revealed two distinct surfaces across each fractured face: a smoother surface where the fatigue crack propagated and a coarse/grainy surface where the fast fracture occurred (See Appendix B). Figure 5 shows the fractured face of Hanger 1 (NW). By examining the fractured faces of the 3 hangers, the metallurgist concluded that a fatigue crack covering 80% of the face for Hanger 1 (NW), 60% of the face for Hanger 2 (NW) and 30% for Hanger 3 (NW) existed prior to final fracture. The fact that the Hanger 1 did not fail until the crack had progressed through 80% of the section, shows that the hanger rods did have a significant factor of safety for carrying traffic loads. Hanger 2 (NW) failed with a smaller percentage of the section cracked since the load on this hanger increased when Hanger 1 (NW) failed. The same pattern holds true for the failure of Hanger 3 (NW).

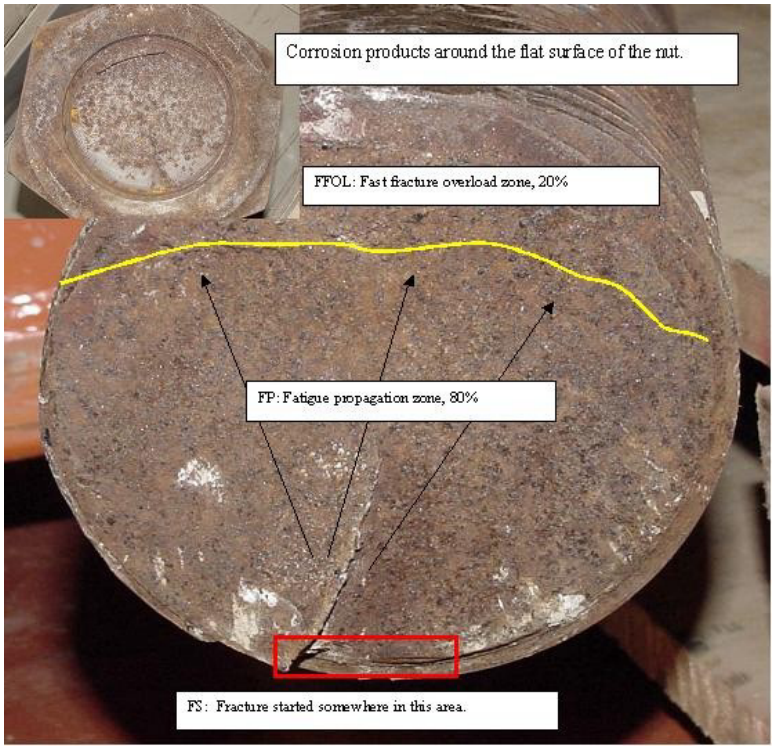

Figure 5: Hanger 1 (NW) – Fracture Face

As stated earlier, Hanger 3 (NW) failed on January 14, 2003, with Hangers 1 and 2 (NW) failing prior to this date. The extreme cold temperatures (-30 degrees Celsius, as reported in the newspaper) and the quality of the steel used in the hangers also played a role in reducing the ductility of the hangers and accelerating the onset of final fracture. Metallurgical tests confirmed that the hanger material did not remain ductile in temperatures much less than –18 degrees Celsius. With a ductile to brittle temperature between –18 and –29 degrees Celsius, it can be concluded that an over-loaded section would experience a low-ductility fracture at temperatures colder than –18 degrees Celsius.

(5) Preliminary Visual Examination

Shortly after January 14, 2003, a preliminary visual examination of the structure and some of the failed components was conducted by an MTO metallurgical engineer. These findings are summarized in Appendix A and are based solely on a visual examination of the bridge and its components. Subsequent to this inspection, a metallurgical expert did a more detailed investigation, including material testing and a corrosion product analysis of the failed components (See Appendix B).

(6) Metallurgical Analysis

An independent metallurgical expert was retained to analyze the failed components. The final report is contained in Appendix B. The following conclusions were made:

(i) Hanger Rods

(a) Chemical analysis of the corrosion product revealed the following:

Location	Age of Corrosion Product on Fast Fracture Face	Comments*
		(* See Appendix B for details)
Hanger 1 (Northwest)	5 to 7 years	Fatigue crack likely started several years earlier at the location where an attempt may have been made to grind a defect out of thread when the bridge was constructed in 1960
Hanger 2 (Northwest)	1 to 3 years	Fatigue crack likely started after Hanger 1 (NW) had failed
Hanger 3 (Northwest)	0 years	January 14, 2003 (Time of partial collapse of deck)
Hanger 1 (Southwest)**	5 to 7 years	Fatigue crack likely started several years earlier at the location where damage occurred on the threads when the bridge was constructed in 1960

***Subsequent to the partial collapse of the deck, it was discovered that an additional 6 Hangers were cracked and Hanger 1 (SW) was fractured completely. This was determined by the structural steel inspection described in Appendix C. It should be noted that once it was determined that Hanger 1 (SW) was fractured, it was removed from the bridge and sent to the metallurgist for analysis. The metallurgist states in his report that he believes Hanger 1 (SW) fractured 5-7 years ago and that the fracture started in a damaged portion of the threaded part of the rod (beneath the nut). He came to this conclusion by observing that the corrosion product on Hanger 1 (NW) and Hanger 1 (SW) were similar. (See Appendix B). The metallurgist also believes that the initial damage in the threads occurred at the time of installation of the hangers. The metallurgist also noted that grey paint is present on the fractured surface of Hanger 1 (SW). This is significant in that the bridge was last painted in 1998 and Hanger 1 (SW) must have been completely fractured at that time in order for paint to be present on fractured surface.*

(b) Material properties

The steel specified on the design drawings was American Society for Testing Materials (ASTM) A373-54T. This particular specification was temporary as denoted by the "T" at the end of the description. Due to the performance of this steel in very cold temperatures, the specification was dropped in favour of A36 in the early 1960's. The tested samples failed to meet all the chemical composition requirements for A373-54T steel. In addition, tensile tests showed that none of the 3 samples tested met the yield strength requirement for ASTM A373-54T steel. The yield strength was about 10% below the specified value of 32,000 psi (220 MPa). Ultimate tensile strength of two of the hangers was also slightly below the specified value of 58,000 psi (400 MPa).

(c) Notch toughness

The testing of the 3 hangers revealed that the ductile to brittle transition temperature was between -18 and –29 degrees Celsius. Based on these results, it can be concluded that this material would not remain ductile in temperatures colder than about – 18 degrees Celsius.

(d) Abrasion and wear

Hanger 1 in the NW corner showed signs of abrasion and wear (described in Section 3 above). Appendix B describes total wear length of 100 to 170 mm that includes some impact damage to the threads that was caused at the time of collapse. Appendix A only considers the length of the hanger subjected to wear (60 to 75 mm). This explains the reason for the length of the total abrasion and wear being different in Appendices A and B.

(ii) Floor beam to stringer connection plate (NW corner)

Corrosion product analysis of the fracture face of the stringer connection plate indicates that the fatigue crack progressed through the bolt holes over a period of several years. (See Appendix B). The fatigue crack extended through 85% of the stiffener cross-sectional area. The very light corrosion on the final fast fracture surface (15 % of the cross-sectional area) indicates that it probably completely failed on January 14, 2003 (Time of partial collapse of deck). It is suspected that the cracks in the plate started after Hanger 1 (NW) had failed (5-7 years ago).

In addition, the metallurgist notes that traces of grey paint were visible on the surface of the fractured face of the stiffener. Similar to the conclusions drawn for Hanger 1 (SW), the stiffener was at least partially cracked at the time of the painting contract (1998).

(7) Steel Structure Condition Survey

An independent firm conducted non-destructive testing of the remaining hanger rods. The ultrasonic testing revealed that an additional seven hanger rods were cracked. One of these (Hanger 1 in SW corner) was cracked all the way through and six others have cracks that have progressed through 10% to 15% of the total cross section. (See Appendix C – Part 1).

Additional non-destructive testing (See Appendix C – Part 2) has also been done on the remaining structural steel (floor beams, stringers, connections, bracing, etc). During this inspection, a crack was noted in the exterior stringer to first floor beam connection plate at the SW corner (See Figure 6 and Appendix C – Part 2) It is likely that the cracks started to form in this connection plate after Hanger 1 (SW) had failed (5-7 years ago as described in Section 6 and Appendix B). The inspection also revealed a few other minor defects as described in Appendix C.

Figure 6: Exterior stringer to floor beam 1 connection (SW corner)

(8) Post-failure Inspections

As part of the investigation, post-failure inspections were conducted on the following dates: January 15, January 29, February 11* and July 8, 2003. The following is a summary of the observations from those inspections. Note that some of the defects recorded were only visible once the snow had melted and the water level in the river had decreased. As such, the July 2003 inspection provided the best opportunity for observing some of the defects.

North End

- The first three hanger rods (NW) were fractured (See Figure 1)
- Connection plate between floor beam 1 (NW) and exterior stringer was fractured. Some corrosion was noted on fractured surface. (See Figure 4).
- Deck had deflected approximately 2 metres
- Extensive cracking in concrete deck at North end
- Many of the bolts connecting the stringers to the floor beams had failed as a result of the deck deflection
- Several stringers and wind brace members had twisted
- Some spalling of concrete at abutment wall, where stringer ties into abutment
- Some cracks in abutment wall

South End

- Transverse cracks throughout deck (0.5 to 1.0mm wide) spaced at 2.0 to 2.5 metres with two cracks at each floor beam location (Close to flange locations).
- Seven transverse cracks (0.5 mm wide, spaced at 200mm) on the western half of the deck, over floor beam 2 (SW corner). Also three transverse cracks in curb. (See Figures 7 and 8).
- A crack and rust staining in the connection plate between stringer 1 and floor beam 1 (SW corner). See Figure 6.
- Sag in handrail and exterior wind bracing at southwest corner (See Figures 9 and 10).
- Some corrosion at pin hanger/bracket interface at several locations (See Figure 11).
- Distortion of some of the washers at the pin/hanger bracket interface (See Figure 12).

* Subsequent to the February inspection, a survey of the floor beam and arch elevations measured the drop of floor beam 1(SW) to be about 80 mm. Note that the floor beams are attached to the exterior wind brace.

Figure 7: Transverse cracks in deck over Floor beam 2 (SW corner)

Figure 8: Transverse cracks in curb over Floor beam 2 (SW corner)

Figure 9: Sag in handrail (SW corner)

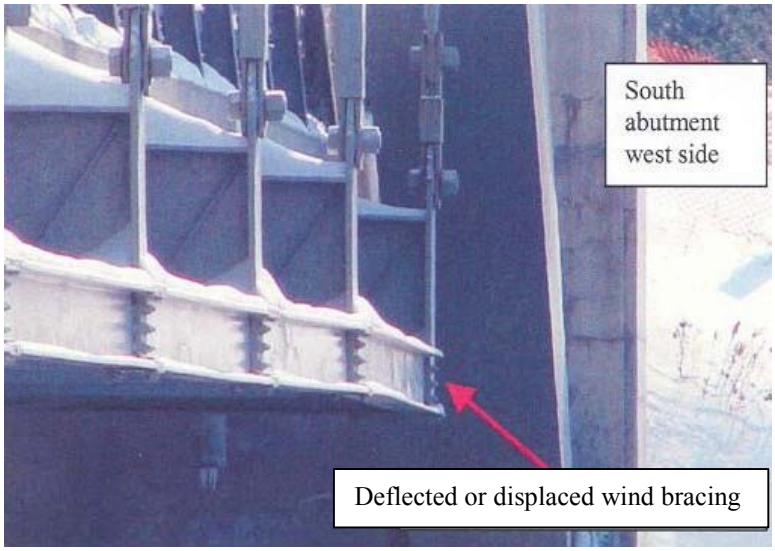

Figure 10: Deflected wind brace (SW corner)

(9) Structural Analysis

Detailed structural analysis was conducted to further understand the failure sequence of the bridge described in Section 3. The results are summarized in Appendix D. A 3-dimensional computer model was used to calculate forces and stresses and a check was made for the Serviceability (SLS) and Fatigue Limit States (FLS). The structure was first checked for the theoretical as-built condition assuming pins at the top and bottom of each hanger. The capacity of all members was adequate for this case using current Canadian Highway Bridge Design Code (CHDBC) loads.

The structure was then checked assuming the hanger pins had seized and the hanger rods were subjected to bending stresses. An iterative analytical procedure was then used where the failure of various components was assumed and the forces and stresses re-calculated for each iteration.

The following conclusions were made for the hangers in the Northwest corner:

- The analysis confirmed that a progressive failure as described in Section 4 is likely to have occurred.
- Bending stresses in the hangers (assuming seized pins) exceeded their theoretical bending capacity.
    - Maximum serviceability stress in Hanger 1 was 901 MPa (Allowable stress is 207 MPa)

- The fatigue stress range of the hangers was exceeded.
    - Maximum fatigue stress range in Hanger 1 was 437 MPa (Adjusted allowable fatigue stress range is 65 MPa – See Appendix D)

- Deflections calculated for Dead Load:
  - With Hanger 1 fractured, Dead Load deflection (Sag of deck) at the hanger location was about 69mm, increasing to 87mm as the connection to the first floor beam fractured.
  - With Hangers 1 and 2 fractured, Dead Load deflection was about 209 mm

- Deflections calculated for live load (truck traffic):
  - With Hanger 1 fractured, Live Load deflection at Hanger 1 was about 46mm, with about 29mm movement of the hanger within the arch sleeve.
  - With Hangers 1 and 2 fractured, Live Load deflection at Hanger 1 was about 108 mm, with about 90 mm movement of the hanger within the arch sleeve.

Although the Live Load deflections are theoretical, there is fairly good correlation with the length of the abrasion and wear marks found on Hanger 1 (NW) and Hanger 1(SW). (See Appendices A and B). Appendix B confirms that the abrasion marks are longer on Hanger 1 (NW) compared to Hanger 1 (SW), indicating that the northern part of the bridge was more flexible than the southern part of the bridge prior to the deck collapsing. This is consistent with the failure sequence described in Section 4 that describes Hanger 2 (NW) also having been fractured for a period of 1 to 3 years before January 2003. The fact that the Live Load deflections correlate with actual field observations validates the computer model used in the structural analysis.

The theoretical dead load deflections were also verified for the case with Hanger 1 (SW) fractured (See Section 8). The case for two hangers (NW corner) fractured has not been verified by field observations. Actual deflections may have been less than calculated due to the additional restraint provided by the catenary action of the stringers, wind bracing and deck. However, the theoretical Dead Load deflections above can be considered the possible upper bounds of the actual deflections.

(10) Design and Rehabilitation History

(i) Original Design

The bridge was designed in the late 1950's and built in 1960. The design itself is unique and is the only bridge of its kind owned by the Ministry of Transportation. As with most truss or arch structures, designers usually assume pin connections everywhere when they do their analysis. In this case the designer provided pins and as such, probably did not consider the adverse effects of introducing bending moments into the hangers or fatigue.

In many cases, ignoring rigidity at joints is a conservative assumption for the consideration of static strength, but not for the consideration of fatigue performance. In this case, once the pins seized, the governing load case became fatigue. A combination of several factors may have led to the pins seizing. The pin detail used contributed to the seizing of the pins since no gap existed between the hanger plate and the brackets (See Figure 11). As such, minimal corrosion, as shown in Figure 11, would lead to seizing of the pins. Figure 12 shows a typical washer that is distorted behind the nut. This distortion may have been caused by the over-tightening of the nuts during

installation (even though the required tightness was not specified on the original contract drawings). The washer distortion could also have been caused by crevice corrosion behind the washer. In either case, rotation of the pin probably would have been prevented. Figure 12 shows the distortion of a typical washer. A number of washers displayed this characteristic.

Figure 11: Typical seized pin

Figure 12: Possible over-tightened nut or crevice corrosion causing distortion of washer.

Other potential problems with the original design include:

- The threaded rod detail has a poor fatigue stress range rating
- The critical part of hanger assemblies (threaded rods and nuts) were hidden from inspection since the end of the threaded rods were inside the arch
- The dished washers used at the underside of the nuts (inside the arch) did not allow rotation as originally intended. The over-sized holes in the arch caused the washers to seat themselves on the perimeter of each hole, thus restricting rocking or rotation of the hanger rods. The variable vertical stiffness of the hanger support detail, that results largely from the presence of a diaphragm plate to one side of the hanger support, also contributed to eccentric/flexural loading of the critical threaded region of the hanger rod (See Figure 13).

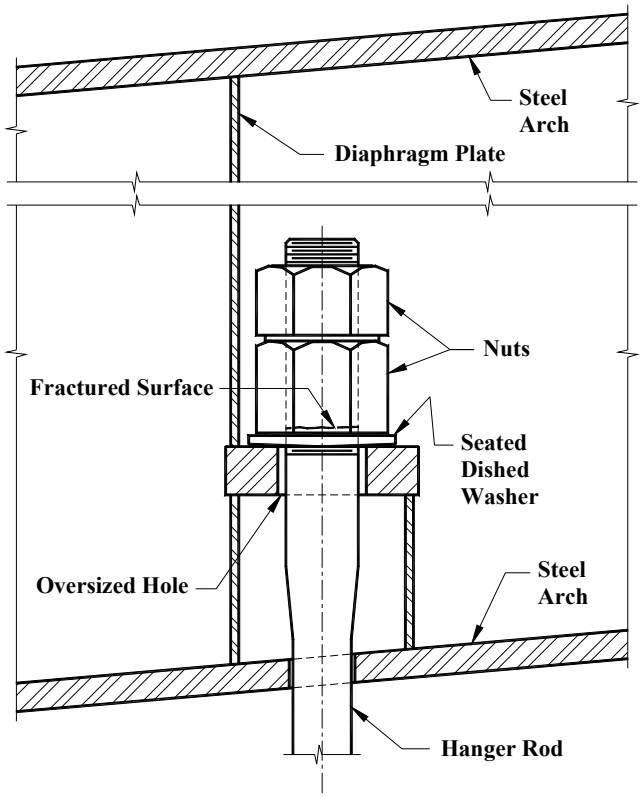

Figure 13: Dished washer used at top of hanger rod (Inside of steel arch)

(ii) Rehabilitation in 1992

The rehabilitation consisted of a complete deck replacement, repair work to the abutments and the replacement of the exterior line of stringers. The deck was made composite with the floor system and continuous over the floor beams. The impact of these changes in articulation is investigated in Appendix D. Overall, the resulting stresses and deflections did not change significantly.

Prior to the rehabilitation in 1992, the bridge was evaluated:

- The evaluator determined that the bridge was adequate to carry service loads
- The evaluator recommended that the "hangers should be inspected for any signs of defects" prior to rehabilitation
    - Ultrasonic testing was done on the pins, but not on the upper portion of the threaded rods
    - Ultrasonic testing would probably have been inconclusive if cracks were very small at that time.
- Fatigue analysis was not performed
    - Fatigue analysis is not usually required on this type of structure since all connections are assumed to be pinned
    - In many cases, ignoring rigidity at joints is a conservative assumption for the consideration of static strength.

(iii) Structural Steel Painting Contract in 1998

All steel components on the bridge were painted in 1998. As part of the painting contract, the contractor was required to evaluate the bridge for the highway loads along with the weight of the painting enclosure. A warranty inspection was performed in August 2000 by MTO staff to determine the condition of the coating system. The coating was deemed acceptable after this inspection.

There is evidence to suggest that Hanger 1 (NW corner) was already fractured at the time of the painting contract. Figure 14 shows that the part of the hanger actually painted varies between Hanger 1 (NW) and Hangers 2 and 3 (NW). The length of the round rod that is painted is longer on Hanger 1 (NW) compared to Hangers 2 and 3 (NW). The painted portion of Rod 1, above the shoulder, was measured to be about 150mm. The same length on Rods 2 and 3 was measured to be about 75 mm (See Appendix B). This indicates that Hanger 1 (NW) had already dropped about 75 mm at the time of painting, and is consistent with the theoretical calculation of the sag of the deck (See Appendix D). This is also consistent with the 5 to 7 year timeframe of failure of Hanger 1 based on the age of the corrosion product on the fractured face. Although it is apparent that an additional 75 mm of Hanger 1 was cleaned and painted, the defect associated with this (i.e. fractured hanger) would not have been easy to detect by the personnel normally involved in the inspection of coating contracts.

The metallurgist also notes in Appendix B that traces of paint were found on the fractured face of Hanger 1 (SW). This indicates it was fractured at that time of painting. Paint was also found on the surface of the fractured face of the stiffener to floor beam connection plate (NW). This indicates that the stiffener was at least partially cracked at the time of painting. However, these cracks were probably small and not easy to detect by the personnel normally involved in the inspection of coating contracts.

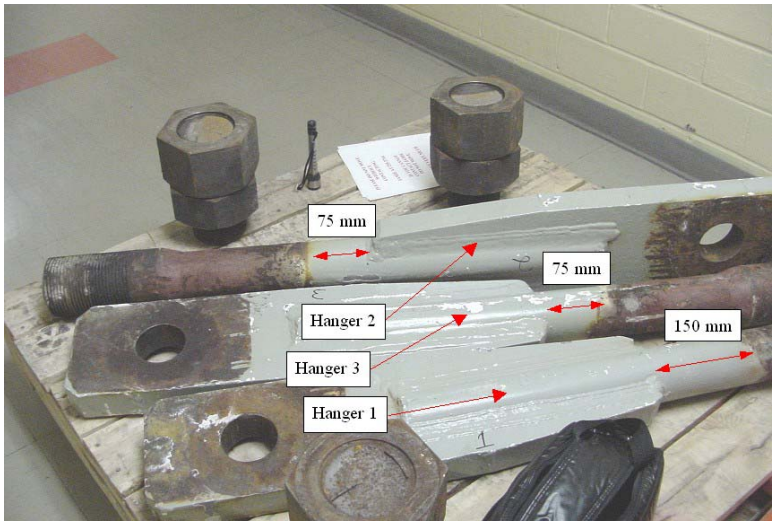

Figure 14: Paint lines on NW Hangers 1,2 and 3.

(11) Inspection History

(i) Inspection Requirements for Detailed Visual Inspections (Biennial Inspections)

Detailed visual inspection of all bridges in Ontario is required every 2 years in accordance with the Ontario Structure Inspection Manual (OSIM) as stated in Ont. Regulation 160/02 – (previously 104/97). The Regulation also states that all inspections must be done under the direction of a Professional Engineer.

A detailed visual inspection is an element-by-element "close-up" visual assessment of material defects, performance deficiencies and maintenance needs of a structure. "Close-up" is defined as "a distance close enough to determine the condition of the element". In some cases, the inspector would use special access equipment to facilitate the close-up visual inspection of various components.

Typically, detailed visual inspections are done by Structural Engineers (or by technicians under the direction of a structural engineer) with several years experience in bridge inspection. These engineers could work for the Ministry of Transportation or Consulting Engineering firms hired by the Ministry. To complete a visual assessment of all components, the inspector usually spends anywhere from 1 to 3 hours at an average bridge site, depending on the size, age and condition of the bridge. For large bridges, this time will increase.

OSIM was first released in 1985 and updated in 2000. The 2000 revision was quite extensive and resulted in a change in inspection philosophy. The original OSIM used a qualitative inspection procedure where the inspector would rate each component using a scale from 1 to 6. The new OSIM is more quantitative and requires that actual quantities of defects be recorded for each component. Several training sessions have been held for inspectors from 2000 to present to introduce the new inspection procedures.

Some common elements found in both versions of OSIM are:

- All bridge components must be inspected and material and performance defects noted.
- Some structures may require more frequent inspection (e.g. Structures with fatigue prone details, fracture critical components and pins in arch structures)
- More detailed investigations be performed when concerns are flagged during visual inspection

(ii) Inspection Requirements for Maintenance Inspections

In addition to the detailed visual inspections listed above, maintenance inspections are also required on a regular basis. A maintenance inspection is a visual inspection used to identify obvious defects. It is a less rigorous inspection than the OSIM inspection described above and the level of expertise required to perform the inspection is also less.

Prior to outsourcing highway maintenance work in October 1998, the ministry's Quality Maintenance Standards (Released in1981) required walk-about inspections every month and routine drive-by inspections by road patrols three times per week in summer and daily in winter. The walk-about inspections were informal and relatively short and did not involve the recording of information on a form. A routine inspection is defined as, "a visual overview of structures while travelling to and from work locations on the Contract, or inspections made during road patrols".

For the last several years, contractors hired by the Ministry using Area Maintenance Contracts (AMC) have been doing these inspections. The maintenance standards in effect at the time of the partial collapse required yearly maintenance inspections by a "qualified person" and routine drive-by inspections three times per week in summer and daily in winter. Monthly inspections of the deck surface were also required. The annual inspections are more detailed than the previous walk-about inspections and the results are recorded on a checklist type inspection form. If major defects are identified during these inspections, ministry staff are notified.

A "qualified person" is defined in the AMC as follows: "The person undertaking the inspection shall have knowledge of structure maintenance practices and aware of problems that arise from weathering, overloading and unusual behaviour of structure components. This knowledge is gained from a minimum of three years hands on experience related to maintenance of provincial or municipal structures in Canada".

(iii) Detailed Visual Inspection History

All inspection records, dating back to 1990, were reviewed. Inspections were performed by MTO staff on the following dates: October 1990, July 1992, November 1994, November 1996, September 1998, September 1999, August 2000, September 2001. Several different engineers and technicians from the MTO Regions and Head Office performed the inspections. All the inspections, except for the August 2000 inspection, were detailed visual inspections carried out in accordance with OSIM, without the use of special access equipment. The September 2001

inspection was done in accordance with the 2000 revision of OSIM. The reports dated after the 1992 rehabilitation contained the following information:

- Some cracks were identified in the concrete deck and curbs
- No defects were identified in any of the hangers
- No defects were identified in the structural steel
- No defects were identified in the wind bracing or handrail

(iv) Annual Maintenance Inspection History

Inspection records dating back to 1999 were reviewed. The Area Maintenance Contractor carried out a visual maintenance inspection every year from 1999 to 2002, with the last inspection taking place in April 2002. There were no reported problems. In 2002, the inspector did note that there was "light rust at seams" for the "Girders/Beams/Diaphragms" elements. The exact location of this rust was not noted.

12) Conclusions and Recommendations

In summary, the partial failure of the Sgt. Aubrey Cosens VC Memorial Bridge can be attributed to a combination of design, construction and material factors. The main contributors to the fatigue induced fracture of the hanger rods include: hanger pins that had seized, defects introduced in the hanger threads during construction and steel that did not remain ductile in very cold temperatures. The failure was progressive over a period of several years.

In reviewing inspection practices and records over the years, the bridge appeared to be well maintained and in good condition (evidenced by the regular inspections and recent rehabilitation/ painting contracts). Section 8 describes some defects that were identified during the inspections after January 14, 2003. It is likely that some of these defects were present once Hanger 1 (SW) and Hanger 1 (NW) had failed. However, the fact that many different inspectors and contractors (1998 painting contract) did not identify these defects, suggests that they were not easy to detect. In hindsight, the defects are consistent with problems identified in the failed hangers, however, taken in isolation, it would have been difficult to relate these defects to hanger failures at the time of inspection. If the defects were identified, the inspectors probably did not associate them with hanger failure, since they were minor in nature. In most structures, these types of minor defects (e.g. cracks in deck) would not be cause for alarm.

Recommendations stemming from this investigation can be grouped into three categories:

Design Practices, Material and Construction Specifications, and Inspection Practices.

(a) Design Practices

In comparing design practices of the 1950's and those used today, many improvements have been made in this area. Current design codes recognize the problems associated with non-redundant structures and fatigue prone details.

- The current Canadian Highway Bridge Design Code (CHBDC) and the previous Ontario Highway Bridge Design Code (OHBDC) discourages the use of "single load path" structures.
- CHBDC fatigue design requirements are stringent for new structures
- Details used for pin and hanger assemblies have been improved. These new details use cable assemblies with gaps between plates in eyelet and pin connections. Teflon washers are also used.

The design of new bridges has improved significantly since the 1950's, however, further improvements are recommended:

1) **The effects of connection rigidity and resulting bending fatigue stresses should be considered in the design and evaluation of steel arch and truss bridges.**

2) **For future designs and rehabilitations, the designer should identify critical details and ensure that the details can be easily inspected after construction.**

(d) Material and Construction Specifications

Similar to design specifications, material and construction specifications have improved greatly over the last 40 years. New and improved processes and materials, as well as more rigorous quality control and quality assurance requirements during the fabrication and construction processes, have improved the overall quality of steel and other materials. Material testing and performance in cold temperatures are integral parts of the steel fabrication process today.

As such, no new recommendations are being made under this category.

(e) Inspection Practices

The Ministry of Transportation has been improving bridge inspection practices over the last several years. The introduction of Ontario Regulation 104/97 in 1997 and the 2000 revision of the Ontario Structure Inspection Manual have provided better guidelines for inspectors to follow. However, further hands-on training for inspectors is warranted with particular attention to similar non-redundant, fatigue prone bridges. The following is recommended:

1) **Immediate inspection of MTO arch bridges with pin and hanger connections *(Completed).***
2) **Advise municipalities and other owners to inspect bridges with similar details *(Completed).***
3) **Expand sections of OSIM to emphasize the requirements for access and "close-up" visual inspection of each component during biennial inspections.**
4) **Provide additional OSIM training for inspectors to ensure that overall structure integrity is assessed by identifying unusual deflections, flexibility and other minor defects.**
5) **Detailed fatigue inspections and non-destructive testing of fatigue critical components should be carried out every 5 years.**
6) **For painting contracts, detailed inspection of all structural steel work should be carried out by a Professional Engineer for certain structures, after cleaning of steel and before painting.**
7) **Increase frequency of "walk-about inspections" by maintenance staff or area maintenance contractors for certain bridges.**
8) **Provide additional bridge inspection training for maintenance staff.**