Long-term structural health monitoring of the Crowchild Trail Bridge

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ABSTRACT: Steel-free deck design is a relatively new approach to bridge deck construction. Several steel-free bridges constructed in Canada exhibited satisfactory behaviour immediately after construction, however, the long-term health of many of these structures has not been investigated. In many of these structures, significant cracking has minimized the potential construction of similar bridge deck structures. Long-term structural health monitoring of these structures is key in determining their long-term performance. Structural health monitoring data was obtained from the steel-free deck of the Crowchild Trail Bridge during its first seven years in service. Field monitoring included ambient vibration tests, static and dynamic load tests, and deck crack mapping. Despite significant amounts of cracks in the concrete deck and barriers, the overall behaviour of the Crowchild Trail Bridge has remained satisfactory and consistent. Future steel-free deck designs should consider improved fatigue resistance in order to meet serviceability requirements.

1 INTRODUCTION

1.1 *Background*

A large majority of Canadian bridge infrastructure consists of bridges with reinforced concrete decks. These bridge structures are subject to some of the harshest environmental and load conditions in the world. Freezing and thawing cycles may occur daily within the presence of moisture. In addition, deicing salts used to prevent the formation of ice on roadway surfaces are very harmful to the long-term health of internal reinforcement of concrete bridge decks. Corrosion of internal steel reinforcement leads to a variety of maintenance issues including the deterioration of the surrounding deck concrete. As a result, many of the reinforced concrete decks currently in service in Canada require extensive repairs or, in some cases, complete replacement.

In the last 10-15 years, alternatives to typical reinforced concrete deck construction have been researched (Bakht and Mufti, 1998; Hassan, et al., 2002; Yang, et al., 2004). Steel-free deck design incorporating innovative materials is one solution that eliminates the internal steel corrosion. Steel straps that resist tensile forces in the transverse direction are typically placed below the concrete deck. Several steel-free deck bridges were built in Canada during the mid-1990s and are still in service today. The Crowchild Trail Bridge in Calgary was the first continuous steel-free deck in the world when constructed in 1997 (Tadros, et al., 1998; Mufti and Newhook, 1999).

The satisfactory short-term behaviour of steel-free decks has been well established. However, bridges may show signs of deterioration after 10 or 20 years as a result of ageing materials, excessive use, overloading, and harsh climatic conditions. Significant research is required to establish the long-term performance of steel-free deck bridges. Structural Health Monitoring (SHM) tools have proven to be very useful in determining the short and long-term behaviour of steel-free bridge decks.

1.2 *Scope and Objectives*

The SHM program of the Crowchild Trail Bridge was first established in 1997 when the steel-free deck was constructed. Research objectives included the determination of the bridge deck performance under serviceability conditions, change of major bridge characteristics with time, and its long-term durability. Initial tests were used to determine the baseline characteristics. These tests included static and dynamic load tests, ambient vibration test, and extensive crack mapping. In order to examine the long-term durability of the bridge deck, further testing was completed most recently in 2004. Comparison of these test results with the baseline characteristics will be presented. Conclusions and recommendations will be drawn from these results in order to determine the current and projected structural health of the steel-free bridge deck.

2 MONITORING PROGRAMS OF STEEL-FREE BRIDGES IN CANADA

Traditionally, the most basic method of SHM has been periodical visual inspections. However, this type of inspection, used on its own, is very limited in effectiveness because many key bridge characteristics are not visible to the human eye. Especially for new steel-free deck construction, a more rigorous and thorough approach is required. A variety of SHM methods should be utilized to monitor the behaviour of these specialized structures accurately and efficiently. SHM techniques may also be used in assessing the performance and strength of bridges under static or dynamic vehicle loads, and in the verification of similar innovative designs. SHM ensures that these new materials function as intended and their structural behaviour is satisfactory. SHM is more beneficial when longer timeline performance is questioned. With the construction of steel-free bridge decks, SHM tools were required to determine the field performance of these structures under real-life service conditions (Mufti, 2002).

Between 1995 and 1999, five steel-free bridge decks were constructed in Canada. Immediately after the construction of these bridges, load tests and ambient vibration tests were completed to capture the base-line structural characteristics and validate design assumptions. Through these research efforts and constructed bridge decks, the short-term behaviour of steel-free bridge decks has been well established. Overall, the steel-free decks investigated in the 1990s performed very well within their first couple of years of service.

All of the steel-free bridge decks constructed during the 1990s have now been in service for more than five years. Few tests or observations have been completed since baseline characteristics were first established for these bridge decks. For example, the Salmon River Bridge was subject to load tests immediately after construction, however, only periodical visual inspections have been completed since then to monitor cracking patterns (Newhook and Mufti, 1996). Similarly, no additional tests have been investigated for the Kent County Road Bridge after static and dynamic tests were completed in November 1996. Furthermore, no additional data has been collected from the Waterloo Creek (Ventura and Cook, 1998) and Lindquist Creek Bridges since they were first constructed. Clearly, these bridge monitoring programs do not have clear objectives with respect to long-term health monitoring or the determination of long-term performance. There is very little published research currently available that focuses on the long-term performance of these structures.

Even though rigorous long-term SHM of steel-free deck bridges would be beneficial, many of the steel-free decks constructed in the 1990s have received minimal periodic monitoring since they were first constructed. The absence of long-term monitoring can be attributed to the lack of resources and funding available for these types of projects. Additional tests should be completed as the deck cracking characteristics in many of these bridges may influence their long-term behaviour and fatigue resistance. If the satisfactory long-term performance of steel-free decks were quantified, bridge owners would be more convinced of their benefits. With the information obtained through long-term SHM, they can accurately compare steel-free bridge deck construction to typical reinforced concrete deck construction from long-term structural and long-term cost points-of-view.

Compared to other steel-free decks, the Crowchild Trail Bridge in Calgary, Alberta has been subjected to a significant degree of load and ambient vibration testing since it was first constructed. For Crowchild Trail Bridge testing, SHM tools were utilized to complement visual observations. The Crowchild Trail Bridge long-term SHM program also shared goals highlighted in the previous paragraphs.

3 CROWCHILD TRAIL BRIDGE

3.1 *Location*

The steel-free deck of the Crowchild Trail Bridge, shown in Figure 1,was constructed in 1997 and is located in Calgary, Alberta (population: 1,000,000).

Figure 1. Crowchild Trail Bridge.

It is also important to note the location of the bridge: southbound Crowchild Trail is one of the main road arteries into Calgary's downtown business core. Therefore, the bridge design had to withstand repeated traffic loading of thousands of vehicles every day. Existing piers, abutments, and foundations constructed for the original bridge deck were reused without any modifications. The new bridge consists of three continuous spans over two pier supports measuring 29.830 m, 32.818 m, and 30.230 m for a combined total length of 92.878 m.

3.2 *Bridge Deck Construction*

The concrete deck measures 9030 mm wide and 185 mm thick as shown in Figure 2. At the interface between the concrete deck and the steel girders, the

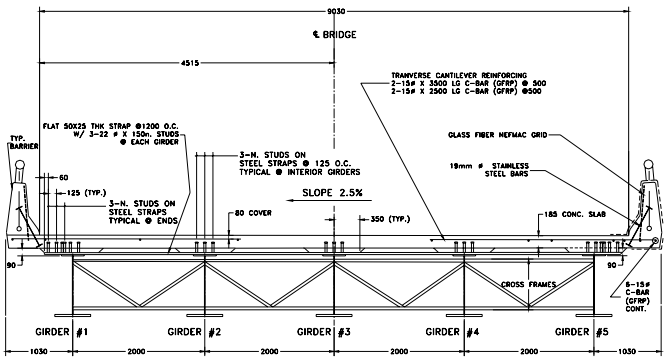

Figure 2. Crowchild Trail Bridge cross-section.

deck thickness increases to 265 mm with the addition of an 80 mm thick haunch. The concrete deck was constructed using 35 MPa concrete and consists of polypropylene fibres at a ratio of 4.5 kg/m^3. These additional fibres were utilized to minimize and control the effects of thermal cracking, shrinkage, and improve the overall durability of the bridge deck.

Glass fibre reinforced polymer (GFRP) reinforcement was placed in the longitudinal direction of the concrete deck over the intermediate pier locations and in the transverse direction of the cantilever deck regions. The south span deck is reinforced internally with GFRP bars and minimum temperature steel bars. However, the middle span is only partially reinforced with GFRP bars and does not contain any typical steel reinforcement. Finally, the north span only contains GFRP bars in the cantilevered portions of the bridge deck located outside of the steel girders.

Five, continuous steel plate girders, approximately 900 mm deep and spaced 2000 mm apart, are located below the concrete deck to carry the main structural loads. Double-headed shear studs were used to connect the steel girders to the concrete bridge deck. These studs are welded directly to the tops of the girder flanges. In addition to connecting the deck to the steel girders, they provide confinement in the longitudinal direction.

Instead of typical internal steel reinforcement, external steel tension straps were positioned directly under the concrete deck to resist tension stresses in the transverse direction. 50 x 25 mm steel bars were used as tension straps and were spaced 1.2 m apart. Double headed shear studs were used to provide anchorage for the steel straps.

3.3 *Instrumentation and Test Equipment*

Permanent and temporary instrumentation for the Crowchild Trail Bridge monitoring program was designed to capture static and dynamic behaviour. Table 1 summarizes the permanent instrumentation

Table 1. Location of Crowchild Trail Bridge instrumentation.

Description and quantity		Location
G-34		Steel plate girders
T-18		Steel tension straps
S-6		Shear studs
C-4	5 mm foil strain gauges	Cross frame
N-4		NEFMAC grid and barrier stainless steel rods
R-15		GFRP bars on deck reinforce.
RC-2		GFRP bars
RS-3		Smart bar
FC-2	Fibre optic sensors	GFRP bars
FS-3		Smart bars
E-17	Electrical resistance embedded strain gauges	Embedded inside precast concrete blocks within deck
TH-5	Thermistors	4 embedded in concrete deck 1 outside deck

installed within the Crowchild Trail Bridge. In total, the instrumentation consists of 108 strain gauges and five thermistors. 86 of the strain gauges are five mm foil strain gauges, two are glued-on fibre optic strain gauges, three are embedded fibre optic strain sensors in GFRP bars (smart bars), and 17 are electrical resistance embedded concrete strain gauges. The instrumentation was strategically placed to measure key aspects of the bridge behaviour. All of the instrumentation is located within the north span of the bridge where internal reinforcing steel is not present. Lead wire from all the gauges is routed to a central junction hub located between two of the steel girders. Main connectors were connected to a data acquisition unit on the ground. A desktop computer was used to record and store data (Alfami, et al., 1998; Cheng and Alfami, 1999).

Temporary cable transducers placed under the bridge deck were used to measure girder deflections. Accelerometers were also used temporarily on the bridge deck surface to measure bridge natural frequencies and mode shapes. Trucks loaded with heavy granular material were used.

4 TESTING PROGRAM

After the Crowchild Trail Bridge was reconstructed in 1997, several sets of tests were completed before the bridge was opened to traffic. These tests captured the baseline structural characteristics and behaviour of the bridge. Additional tests were completed in 1998 after the Crowchild Trail Bridge had been in service for a full year. Six years later, in 2004, another full set of field tests were performed

Table 2. Structural Health Monitoring Program.

Date	Tests / Measurements
August 1997	Ambient Vibration Tests (#1) - Identify initial natural frequencies and mode shapes Static Load Tests (#1) – Measure strains and deflections Crack Mapping (#1) – Record layout and size of cracks
September 1997	Temperature Tests (#1) – Bridge behaviour under known temperature changes
October 1997	Random Dynamic Load Tests (#1) – Measure strains under random traffic loading
June 1998	Random Dynamic Load Tests (#2) – Measure strains under random traffic loading
August 1998	Ambient Vibration Tests (#2) – Check natural frequencies and mode shapes Static Load Tests (#2) – Measure strains and deflections under known truck load Dynamic Load Tests (#1) – Measure strains, deflections, and accelerations under known truck load Crack Mapping (#2) – Document locations of new cracks and estimated sizes of existing cracks
June 1999	Crack Mapping (#3) – Document locations of new cracks and estimated sizes of new/existing cracks
July 2002	Crack Mapping (#4) – Document locations of new cracks and estimated sizes of new/existing cracks
June 2004	Random Dynamic Load Tests (#3) - Measure strains under random traffic loading Ambient Vibration Tests (#3) – Check natural frequencies and mode shapes Static Load Tests (#3) – Measure strains and deflections under known truck load Dynamic Load Tests (#2) – Measure strains and deflections under known truck loads and known speeds Crack Mapping (#5) – Update and document locations of new cracks and estimated sizes of existing cracks

to determine whether any significant structural changes had occurred. Field tests consisted of ambient vibration tests to determine the natural frequencies and mode shapes of the bridge. In addition, static and dynamic load tests were completed to determine bridge strains and behaviour under known truckloads. Finally, the underside of the bridge deck was visually inspected in order to map the progression of cracks. Table 2 lists all of the tests completed since the construction of the steel-free bridge deck (Taing, 2000; Ventura, et al., 1997).

5 TEST RESULTS AND DISCUSSION

5.1 *Ambient Vibration Tests*

Careful comparison of the ambient vibration test results can provide insight into the current status of the Crowchild Trail Bridge. Vibration characteristics of the bridge depend on its physical properties such as mass, stiffness, and boundary conditions. Variations of these properties over time will directly influence its modal parameters. Significant changes in modal parameters and natural frequencies can be used to locate and quantify changes in the overall behaviour and structural damage of the Crowchild Trail Bridge (Bagchi, et al., 2004; Ventura, et al., 2000).

Table 3. Crowchild Trail Bridge natural frequencies.

Mode Shape	1997 Hz	1998 Hz	2004 Hz
1st Vertical	2.78	2.60	2.80
1st Torsional	3.13	2.90	3.16
2nd Vertical	3.76	3.63	3.78
2nd Torsional	4.05	3.85	4.19
3rd Vertical	4.64	4.43	4.66
3rd Torsional	5.18	5.00	5.36
4th Torsional	7.13	6.85	6.89
4th Vertical	9.13	8.60	8.29

Table 3 lists the eight predominant natural frequencies below 10 Hz and their corresponding mode shapes for each test. In general, the natural frequencies measured in 1998 were approximately 0.20 Hz lower than the frequencies measured in 1997. To account for this change, it was speculated that the stiffness of the bridge reduced within the first year of operation. However, in 2004, the first six measured natural frequencies were closer to the results that were measured in 1997. Although the last two natural frequencies were difficult to quantify from the 2004 results, they are relatively close to the 1998 results.

There are several explanations for the observed shifts in measured natural frequencies. Changes in

bridge stiffness could be in part to member degradation from deterioration or damage. In addition, the dynamic behaviour of a bridge may be influenced by environmental factors. These factors include temperature, relative humidity, and wind. Changes in the natural frequencies may also be related to differences in the loading scenarios and computational inaccuracies.

One of the most notable damage detection tests was first completed by Farrar et al. (1999) on the I-40 Bridge in Albuquerque, New Mexico. For the I-40 Bridge tests, significant changes in natural frequencies and mode shapes were observed only after large amounts of structural damage had been induced; even then the natural frequency changes were less than 8%. It was noted that differences in obtained data are inevitable with test-to-test repeatability and changing test conditions. For future tests, they recommended that the sensitivity of modal test results to environmental conditions and test procedures be quantified. They also noted that a temperature differential of 25°C influenced the natural frequency of another bridge by as much as 0.4 Hz. Ward (1984) reported that under normal circumstances the stiffness of a bridge should decrease as it cracks or corrodes away, however, this tendency could be masked by changes in the support conditions.

Therefore, changes in the Crowchild boundary supports and ambient conditions may have contributed to the observed changes in natural frequencies. Clearly, variations in the natural frequencies and mode shapes of the Crowchild Trail Bridge over time are not strictly related to changes in stiffness or structural damage.

5.2 *Static Load Tests*

Table 4 compares the absolute maximum strain results from 1997, 1998, and 2004 caused by the static truck loadings. Overall, the results from the various tests indicate that all of the obtained strain data is reliable and the differences in strain range for most of the bridge components is relatively small.

Table 4. Comparison of 1997, 1998, and 2004 maximum static strains.

Gauge Type/Location	1997 Strains με	1998 Strains με	2004 Strains με
G3 – Girder 1 web	103.0	97.1	89.4
G33 – Girder 1 flange	94.0	100.5	92.9
G34 – Girder 1 flange	97.0	108.7	101.3
T3 – Steel Strap 8	57.0	89.2	79.2
RC1 – GFRP bar	120.0	39.8	58.1
E13 – Concrete deck	90.0	20.7	24.6
S4 – Steel stud	24.0	16.0	9.5
C4 – Cross frame	64.0	58.4	63.5
RS2 – Smart bar	34.0	31.5	13.5

The highest girder strains were recorded during the 1998 static loading tests. However, there is less than a 10% difference in measured girder strains between 1997, 1998, and 2004. The relatively small differences in measured girder strains may be attributed to the slight variability in truck weights and truck loading location, and very slight differences in bridge stiffness. Overall, this comparison indicates that the condition of the bridge girders has remained relatively consistent during the first seven years of service life.

Strains in the other monitored components have not changed significantly between 1998 and 2004. Strains in the other steel members, the cross-frame, steel strap, and steel studs, are nearly identical for the 1998 and 2004 results. For example, the gauges located on Steel Strap 8 indicated that it cycled between strains of –15 με and +85 με as the truck travelled across the entire length of the bridge in 1998. In 2004 the strains in the steel strap ranged between –6 με and +79 με. This relatively low strain cycle indicates that the steel straps behaved well within the elastic range and fatigue was not critical during service loading. However, these measured strains are approximately double the strain values produced under similar loading in 1997. Higher strains in the steel straps indicate that arching action within the deck is more prominent since 1997.

Figure 3 highlights the load sharing characteristics between girders based on the deflection measurements from 1997, 1998, and 2004. Load sharing

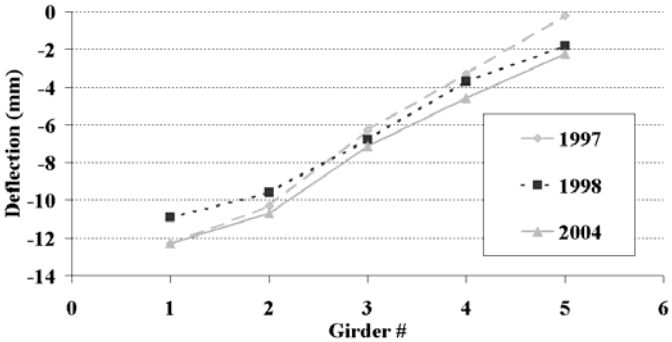

Figure 3. Girder deflection and load sharing.

is characterized by the contribution of girders not directly loaded by the truck. Load sharing characteristics among the girders has not changed significantly from 1997 to 2004. At a transverse section, the deflected shape of the girders is very close to a straight line. Adequate load sharing among the girders suggests a relatively stiff deck in the transverse direction from 1997 to 2004.

It is also interesting to compare the maximum deflections measured in Girder 1 during each of the load tests. The maximum downward deflections measured were 12.3 mm, 10.9 mm, and 12.3 mm for the 1997, 1998, and 2004 tests respectively. Differences between the 1997 and 1998 deflection results can be attributed to the position of the loading truck. Deflections in Girder 1 were the same for the 1997 and 2004 test results. However, due to the position of the loading truck and eccentricity of the load in 1997, the deflections measured in the other girders are lower compared to the 2004 results. Also, the average deflection of all the girders is the highest in the 2004 results because the heaviest truck was used during these tests.

One of the most important aspects that can be observed from the health monitoring data is the girder strain distribution. Figure 4 shows the strain distribution across Girder 1 in the positive moment region of the bridge span for 1998 and 2004 results.

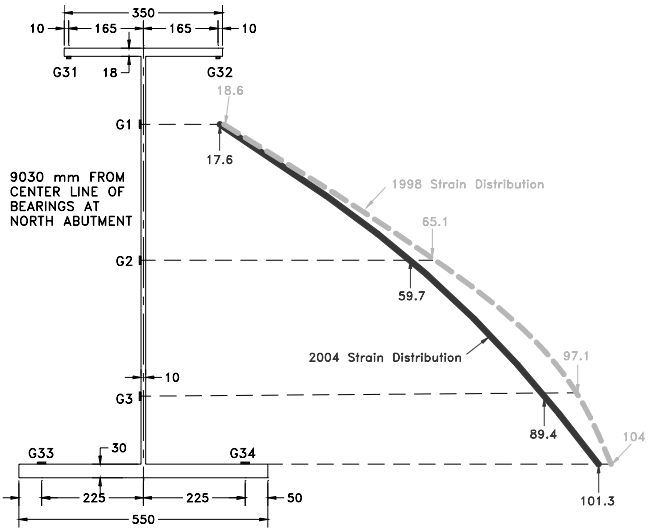

Figure 4. Positive moment region strain distribution.

Results from the 1997 data are not shown, as the strain values were nearly identical to the 1998 data. As seen in Figure 4 the strain values were relatively close between the 1998 and 2004 tests. Both sets of data show consistent linearity in strain distribution. From the 1998 data, the neutral axis is located approximately 55 mm below the bottom face of the flange. By looking at the 2004 data, the neutral axis was found to act at approximately the same girder cross-section location. These results suggest that there has been no change in composite action of the bridge in the positive moment region during the first seven years of service life.

Strain distributions were also analyzed in the negative moment region near the pier supports. Figure 5 shows the three separate strain distributions obtained from the 1997, 1998, and 2004 load tests. Again, all strain distributions are approximately linear across the girder cross-section. In the negative moment region, the neutral axis location has changed significantly over the last seven years. In

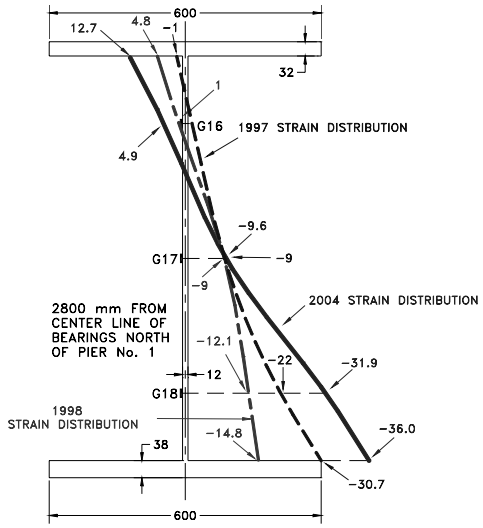

Figure 5. Negative moment region strain distribution

1997, the neutral axis was located approximately 120 mm below the bottom face of the top flange. In 1998 the neutral axis had shifted 70 mm further down to 190 mm below the bottom face of the top flange. In 2004 the neutral axis had shifted an additional 60 mm to approximately 250 mm below the bottom face of the top flange. These results suggest a steady loss in the composite action between the concrete deck and the steel girders in the negative moment region where the concrete is mostly in tension.

To estimate the approximate percentage of composite action lost between the concrete deck and the steel girders in the negative moment region overtime, two assumptions are made: 100 % composite action occurred in 1997 and 0 % composite action would result if the neutral axis was located at 480 mm below the bottom surface of the top flange (this is the neutral axis of just the steel girder without the presence of concrete). Therefore the composite action between the steel girders and the concrete deck was approximately 80% in 1998 and 64% in 2004. As the composite action has reduced over time, the girders have become the primary load-carrying components in the negative moment region. Increased girder web strains from the tests results and the changing location of the neutral axis support this conclusion. Strain results from 2004 indicate that the steel girder carries most of the longitudinal moment in the negative region.

5.3 *Crack Mapping*

Over the first seven years of service life, crack mapping of the underside of the Crowchild Trail Bridge deck has been inspected and recorded five times. The first complete crack mapping documentation

was completed in August 1997 immediately before the bridge was opened to traffic. Crack mapping was completed again in August 1998 after one year of operation. Crack mapping was completed three more times in June 1999, July 2002, and most recently in June 2004. As a result, the location, size, and rate of crack growth have been accurately documented throughout the service life of the bridge. Each of the bridge spans was constructed using different materials and, therefore, the crack behaviour is slightly different for each span.

To date the north span, which was constructed with the least amount of internal reinforcement (reinforced only in the cantilever edge with GFRP reinforcement), contains the largest number of longitudinal cracks. However, the north span contained the least amount of transverse cracks compared to the other two spans. The middle span, which was partially reinforced with GFRP bars, contains the most amounts of transverse cracks compared to the other two spans. It is interesting to note that the south span, which contained the minimum temperature internal steel reinforcement, had the least amount of cracks after seven years of service.

Figures 6 and 7 summarize the rate of longitudinal and transverse crack formation respectively in each of the spans. As shown in Figure 6, most of the longitudinal crack formation occurred within the first two years of service. During the last five years of service, relatively few new longitudinal cracks have formed. The north span contained significantly more longitudinal cracks compared to the other two spans. Approximately 25 meters of longitudinal cracks were observed in the middle and south spans by 2004. As shown in Figure 7, the amount of transverse cracking was relatively the same in all the spans. The north span contained the least amount of transverse cracks by 2004. However, both the middle and south spans contained approximately 100 meters of transverse cracks by 2004.

Deck leakage was clearly visible after significant amounts of precipitation were experienced during testing. In Figure 8 leakage is shown along a transverse crack at a construction joint. Figure 9 shows leakage along a longitudinal crack in the concrete deck haunch. Here the moisture has fully penetrated

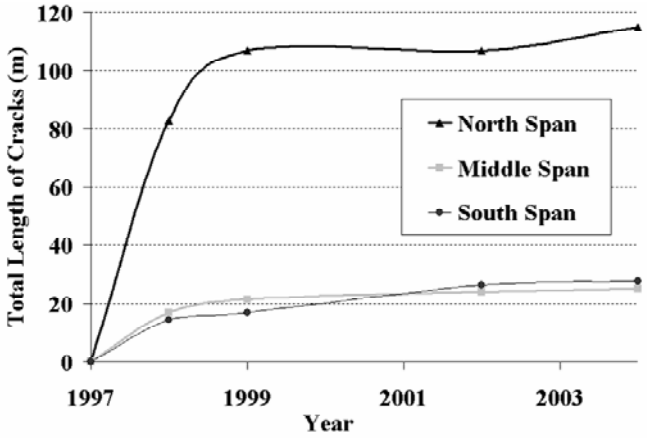

Figure 6. Rate of longitudinal crack formation.

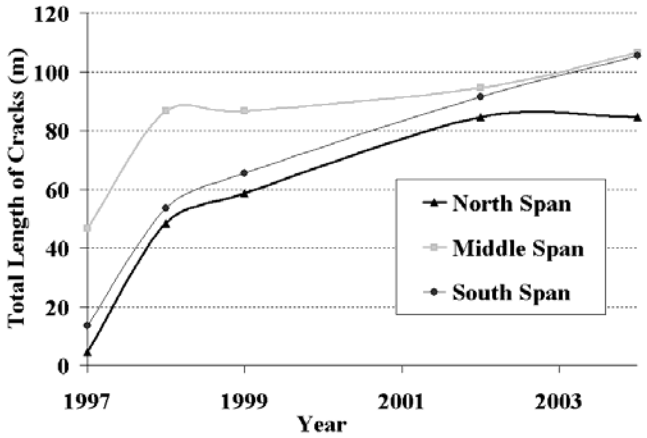

Figure 7. Rate of transverse crack formation.

Figure 8. Deck leakage along construction joint.

Figure 9. Deck leakage along deck haunch.

the crack and is migrating toward the steel plate girder where more potential damage could occur. The middle span, which was reinforced internally with only GFRP bars, contained the most amount of areas affected by water leakage. The south span, which was the only span to contain a small amount of internal steel reinforcement, contained the fewest affected areas. Most of the water leakage occurred at cracks orientated in the transverse direction. Water leakage was noted to occur at several cold joint locations.

Visible vertical cracks and stains were also located along both sides of the concrete bridge barrier. Figure 10 shows the typical cracking pattern and staining along the concrete side barriers viewed

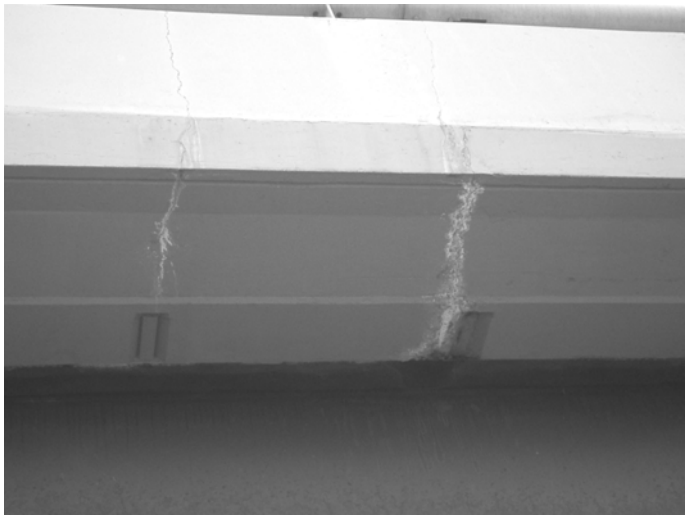

Figure 10. Bridge barrier cracks and water stains.

from both sides. As seen in Figure 10 cracks were spaced approximately 600 mm to 1000 mm apart along the barriers. Internal reinforcement used in the concrete barrier consisted of prefabricated NEFMAC GFRP grids. In addition, double headed stainless steel bars anchor the barriers to the bridge deck. Cracks in the barriers were the most visible areas of damage.

Overall, these inspections indicate that the rate of crack formation and crack growth had significantly decreased by 2004. Very few new cracks are expected to form within the entire bridge deck in the near future. Also, existing cracks are not expected to increase in size significantly. Transverse cracks that had formed before the first crack mapping stage were likely due to shrinkage of the concrete. Crack growth and the formation of new cracks were likely due to cyclic traffic loading and freeze-thaw cycles throughout the year. Temperatures typically fall to -30^{o}C in the winter and may be as high as $+30^{o}$C in the summer. However, the black asphalt surface of the deck tends to absorb the sun's radiation causing the deck surface temperature to be significantly higher. Cold joints were identified at two locations in the middle span and one location in the south span. These joints had formed between separate poured sections of the concrete deck. Cracks are noticeable because the adjacent concrete pours did not properly bond together.

However, if crack development continues indefinitely, the structural health of the bridge may be affected. If special provisions are not made in an attempt to control cracking, cracks are typically fewer and wider. On the other hand, if special provisions are implemented to control cracks, concrete-cracking widths will generally be within serviceability limits.

6 CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are based on the steel-free deck literature review and recent data obtained from Crowchild Trail Bridge:

(1) The five steel-free deck bridges constructed between 1995 and 1999 in Canada remain in service today. However, all of these slabs developed relatively wide, full-depth longitudinal cracks midway between the girders that exceed serviceability standards. With the exception of periodical crack inspections, minimal long-term structural health monitoring has been performed for these bridges to establish their long-term performance.

(2) Recent research has shown that GFRP reinforcement has the best fatigue resistance and prevents the formation of wide longitudinal cracks (Bakht, et al., 2003). In order to reduce deck cracking and improve fatigue resistance, steel-free deck designs require a nominal crack-control grid of GFRP reinforcement. This research has contributed to the development of second-generation steel-free decks with improved serviceability characteristics.

(3) Based on the overall static, dynamic, and ambient vibration tests in 2004, the Crowchild Trail Bridge behaviour has not changed significantly during the first seven years of service. Despite deck cracking and leakage serviceability concerns, the Crowchild Trail Bridge remains a safe and reliable part of Calgary's roadway infrastructure. With these considerations, the steel-free deck concept has proved to be a satisfactory alternative for bridge deck construction.

(4) The maximum girder deflection measured during the 2004 tests was -13.2 mm under dynamic loading due to a 415 kN truck load. This is well within the maximum serviceability requirements. Deflections measured in the other girders during static loading indicate excellent load

sharing characteristics in the transverse direction of the bridge deck. Therefore, transverse stiffness of the Crowchild Trail Bridge remains satisfactory since construction.

(5) Maximum measured stains during the 2004 tests for the girder, steel straps, and cross frames were 110 με, 79 με, and 64 με respectively. These results are well within the elastic range and capacity of these steel components. Strains in neighbouring girders also indicate satisfactory girder load sharing in the transverse direction. Measured strains in the other concrete and reinforcement bridge components were relatively small and are not a concern.

(6) Girder strain profiles obtained from the 2004 tests indicate that the location of the neutral axis has not changed in the positive moment region. However, the location of the neutral axis, relative to the top flange, has shifted downward in the negative moment region: 1997 - neutral axis located at 120 mm, 1998 - neutral axis located at 190 mm, 2004 - neutral axis located at 250 mm. These results suggest a steady loss in the composite action between the concrete deck and the steel girders in the negative moment region. The girders have become the primary load-carrying structural component in the longitudinal direction within this region. Overall, the Crowchild Trail Bridge static load-carrying characteristics are not expected to change significantly over the next ten years of service life.

(7) With the exception of a few higher modes, most of the mode shapes remain unchanged. In 1998, the first eight natural frequencies of the Crowchild Trail Bridge were approximately 0.20 Hz lower compared to 1997 results. This was attributed to a loss in concrete deck stiffness due to cracking. In 2004, the first eight natural frequencies were approximately 0.20 Hz higher compared to the 1998 results. Even with a slight decrease in deck stiffness, these results may be attributed to changes in the boundary conditions or ambient conditions. In general, however, the ambient vibration test results indicate that the dynamic characteristics of the Crowchild Trail Bridge have remained relatively the same over seven years. Based on the 2004 ambient vibration results, the vibration characteristics of the Crowchild Trail Bridge are not expected to change significantly within the next 5-10 years.

(8) Crack mapping has been thoroughly documented five times since the bridge was constructed. In the last couple of years, the number of new cracks has decreased significantly. Currently, the north span, which was constructed with the least amount of internal reinforcement (reinforced only in the cantilever edges and south end pier with GFRP reinforcement), contained the largest number of longitudinal cracks. However, the north span contained the least amount of transverse cracks compared to the other two spans. The middle span, which was partially reinforced with GFRP bars in the cantilever edges and both ends at pier locations, contained the most amount of transverse cracks compared to the other two spans. It is interesting to note that the south span, which contained the minimum temperature steel reinforcement, contained the least amount of cracks after seven years of service. Therefore a minimal amount of reinforcement is required for any deck design in order to control cracking. Water was observed to leak through the concrete bridge deck in several locations. Cracks in the bridge barriers are more visible compared to the deck cracks. Overall, very few new cracks are expected to form within the bridge concrete sections in the future and existing cracks are not expected to increase in size significantly.

(9) The behaviour of the Crowchild Trail Bridge is very similar to the behaviour of other steel-free bridge decks constructed in the same time period. Short-term measured deflections and strains of the steel-free decks are satisfactory and are well within elastic ranges of the respective construction materials. Longitudinal and transverse cracking observed in the Crowchild Trail Bridge was also evident in the other steel-free deck bridges. Most of the cracking was noted to occur primarily within the first year or two of service. After this initial period, further deck cracking in all steel-free deck bridges was minimal. The lack of long-term monitoring data of other steel-free deck bridges prevents further comparisons with long-term monitoring data from the Crowchild Trail Bridge.

(10) Thorough load and ambient vibration tests of other steel-free bridge decks constructed in the 1990s should be completed and compared to previous tests results. These results, in addition to deck cracking observations, would provide a more accurate analysis of the long-term performance of steel-free decks. This type of deck design will gain wider acceptance if satisfactory long-term structural performance is well established. Furthermore, long-term monitoring will lead to additional improvements in steel-free deck designs.

(11) Future field tests are recommended for the Crowchild Trail Bridge. Based on the results of the 2004 test results, it is recommended that similar load and ambient vibration tests be completed every five years. After future load tests, analysis of the concrete strains may contribute to a better understanding of the behaviour of the compression struts within the concrete deck. In addition, thorough visual inspections should be completed once a year to ensure that no signifi-

cant structural damage has occurred. Careful inspection of the top concrete deck surface is recommended to locate any through-thickness cracks. In order to increase the effectiveness of damage detection using VBDI techniques, the effects of temperature and boundary conditions on the dynamic behaviour of the Crowchild Trail Bridge should be isolated and examined in more detail. These steps taken during field tests would contribute to a better understanding of the long-term structural behaviour of steel-free decks.

(12) It is expected that the behaviour of the Crowchild Trail Bridge will remain relatively the same during the next five to ten years unless leakage through the concrete deck increases significantly. Deterioration of the concrete deck and structural steel components could result if deck leakage should increase. Maintenance should be performed to minimize the visual impact of the barrier cracks, staining, and leakage through deck cracks. For example, epoxy grout could be used to fill in these cracks so that they are not as noticeable. However, an investigation should be completed to determine the most efficient and cost-effective repair solution.

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