CONDITION ASSESSMENT
& LOAD EVALUATION
9^{TH} STREET BRIDGE OVER
SYDENHAM RIVER
CITY OF OWEN SOUND

GAMSBY AND MANNEROW LIMITED
CONSULTING PROFESSIONAL ENGINEERS
GUELPH – KITCHENER - LISTOWEL - OWEN SOUND

December 2007
Our File: B-1097

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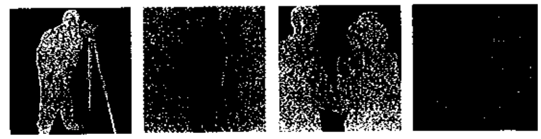

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1.0 BACKGROUND

The City of Owen Sound has retained the consulting firm of Gamsby and Mannerow Limited (herein after referred to as "G&M") to conduct a detailed condition assessment of the 9^{th} Street Bridge. This detailed assessment is a direct result of the inspection report conducted in 2006 by Henderson Paddon & Associates Limited. Following this inspection, they recommended that the structure be posted for "No Heavy Trucks" as a result of cracks in one of the ribs near midspan.

This condition assessment is to provide the City with an accurate and detailed description as to the condition of each bridge component and a recommended safe load posting for the overall structure.

Under the direction of Mr. Brent Willis, P. Eng., on May 2, 2007 Mr. Darryl Cowan, P.Eng and Mr. Derek Brewster of G&M along with Mr. Andrew Moad and his assistant of Vector Corrosion Technologies (hereinafter referred to as "Vector") attended the site to conduct the necessary testing. The testing was carried out over two full days, during which time the bridge was closed to vehicular traffic.

Based upon the information collected, the necessary calculations can be performed to provide an accurate load posting recommendation. As well, recommendations for repair are provided as an alternative, to extend the service life of the existing bridge.

2.0 BRIDGE DESCRIPTION

The 9^{th} Street Bridge was constructed in 1947 and is a single span concrete rigid frame beam bridge with counter balance. It measures 21.3 metres centre to centre of the rigid frame girder legs. The cantilever of each end span measures 4.9 metres, for an overall structure length of approximately 31.2 metres. The reinforced concrete deck is supported by four (4) rectangular reinforced concrete girder beams of varying depth. Plans for the original 1947 construction are available and they show the rigid frame legs supported on a pile cap, in turn supported by 300 mm diameter concrete filled steel pipe piles approximately 21.3 metres to 27.4 metres in length.

The deck width between curb-faced sidewalks is 9.76 metres. The 1.84 meter wide concrete sidewalks cantilever approximately 1.50 metres beyond the outer girders. Two lanes of traffic use the bridge.

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The overall width of the structure is 13.4 metres. Concrete railing posts and steel handrails are located on the outside edge of each walk.

A wood plank utility duct is hung from the north deck soffit, along the entire length of the structure.

3.0 HISTORY

Gamsby and Mannerow Limited has been involved with various projects regarding the 9th Street Bridge since about 1981, when a new structural slab was installed to replace the deteriorated deck surface. The bridge was reviewed again in 1996 at which time random cracks on the deck were noted and an asphalt overlay was recommended to maximize the service life of the deck. Several deficiencies on the underside of the deck were noted at that time, and a detailed inspection report was recommended.

In 1997, an inspection report was completed by the late Bill Mannerow, P.Eng., of Gamsby and Mannerow Limited. Recommendations included replacing the deteriorated concrete on the underside of the bridge deck.

In 1998, Polymer Modified Repair Mortar was used to restore the concrete on the underside of the bridge deck.

On April 4, 2004, the structure was inspected by Andrew Burgess, P.Eng., of Burgess Engineering Inc., for the purpose of identifying any deficiencies and recommending repairs and further detailed investigations. One of the recommendations was that a structural evaluation be performed to establish a suitable load posting. The other recommendations dealt with ongoing maintenance and repair.

Further to Mr. Burgess's recommendation, on January 21, 2005, Brent Willis, P. Eng, and David Grahlman, P. Eng., completed a structural Evaluation Report. This report concluded that there was no need to post the structure with load limits. It also recommended that crack gauges be placed across some cracks, and replacement of the structure within five to ten years.

In October 2006, Henderson Paddon & Associates Limited provided a bi-annual visual inspection of the structure. Based on the recommendations from this report, the bridge was posted for "No Heavy Trucks". This recommendation was due to a general worsening of the condition of structure, in particular the soffit of the interior north girder beam, and extensive concrete delamination on the bridge deck.

4.0 TESTING METHODOLOGY

Numerous tests were completed on the structure on May 2 and 3, 2007 in order to obtain the necessary data needed to make accurate recommendations. The following is a list of the tests completed in the field:

Visual Inspection – for purposes of completing an Ontario Structural Inspection Manual or "OSIM" standardized report. The OSIM report is attached in Appendix A.

Concrete Sounding – to determine areas where concrete has delaminated from the structure. A detailed drawing of the bridge showing specific locations and sizes of the delaminated concrete is attached in Appendix B.

Cover Survey – Utilized to determine the actual depth of concrete cover to the reinforcing steel provided in the structure. The full results are in the Vector report attached in Appendix C.
Corrosion Potential Survey – Indicates the probability and amount of active reinforcing steel corrosion. The full results are contained in the Vector report, in Appendix C.

Chloride Analysis – Determines the chloride content of the concrete at various locations. The full results are contained in the Vector report (see Appendix C).

Compression Testing – To determine the strength of the existing concrete utilized throughout the structure. The compressive testing data sheets are attached in Appendix D.

A barge with scaffolding was used to access the underside of the bridge, and a portable generator was employed to provide temporary power to the site.

5.0 TESTING RESULTS

5.1 VISUAL INSPECTION

The structure is beginning to show signs of deterioration due to its age. Complete results from the visual inspection are recorded in the OSIM report. Photographs of deficiencies are provided in Appendix E. The following is a general summary of the OSIM report:

a) There are numerous longitudinal and transverse narrow cracks on the west side of the bridge deck.
b) The deck soffit has spalled severely in localized areas exposing corroded reinforcing steel along the north and south edge of the soffit.
c) The concrete railing posts have hairline cracks and some spalling near the base.
d) The metal railings are in fair condition.
e) The girder beams and deck soffit have been repaired in the past. The shot-crete patches appear to have bonded well to the structure; however, most other patches are indicating some signs of delamination.
f) The vertical shear cracks at the ends of the girder beams appear to be widening.
g) Part of the armourage is missing at the mid-span on the west side of the bridge.
h) Abutment and ballast walls appear to be in fair condition.
i) Corrugated steel culverts in the south west corner of the bridge retaining wall have failed.

5.2 CONCRETE SOUNDING

The concrete sounding or "toning" technique involved dragging a steel chain across the deck wearing surface and interpreting the sounds to identify areas which have delaminated. The deck soffit and girder beams were accessed from a barge and were tested using a hammer to listen for areas which have delaminated. Identifying areas of concrete delamination is crucial as the delaminated areas are indicative of reinforcing steel corrosion.

In general, the girder beams, abutment walls and intermediate piers are in fair condition, with only small amounts of delaminated concrete located sporadically throughout these components.

The deck has numerous areas of delaminated concrete on the north half of the surface. Approximately 5% of the deck surface has delaminated. As a result of these delaminations, numerous cracks have developed and potholes are beginning to form on the surface. Once potholes develop, the rate of deterioration of the deck will increase exponentially.

The results of the deck toning must be interpreted with caution, as the deck was repaired in 1981. Therefore, areas that sound hollow may be indicative of the overlay debonding, rather than corrosion of the reinforcing steel. Neither condition is favourable, but the repair solutions differ depending on which mechanism has caused the debonding.

The potholes create a rough riding surface for motor vehicles, and repeated wheel load impact will creates larger holes. As well, dirt and debris remains trapped in the hole prolonging moisture exposure to the deck surface causing further delaminations and problems associated with freeze-thaw.

The toning also revealed that approximately 20% of the deck soffit has delaminated. The deck soffit is in poor condition and has a significant amount of patches. It appears that a majority of the patches are delaminating. The largest delamined areas are immediately surrounding the wood utility duct on the north side of the soffit. We suspect the wood utility duct is prolonging the exposure of moisture to the soffit. The chart below indicates the approximate areas of delamination in each bridge component.

Component	**Area of Delamination (m^2)**	**Total Area (m^2)**	**% Area Delaminated**
Exterior Girders/Piers	5.1	227.9	2.2
Interior Girders/Piers	4.0	230.7	2.1
Deck Soffit	59.5	324.8	18.3
Deck	14.4	295.2	4.9
Total	**83.0**	**1078.6**	**7.7**

5.3 COVER SURVEY

The depth of the concrete cover to the reinforcing steel was inspected by Vector in all components of the bridge. The depth of cover is an important measurement as shallow cover can lead to more rapid corrosion and subsequent deterioration of the structure. The depth of cover also affects the bending moment capacity of the structure. According to CAN/CSA S6-06, which is the current Canadian Highway Bridge Design Code (CHBDC), the following minimum covers are recommended.

Component	Concrete cover and tolerance (mm)
Soffit of slab < 300mm thick	50 ± 10
Soffit of Slab >= 300mm thick	60 ± 10
Vertical surface of girder beam	70 ± 10
All other locations	70 ± 20

The depth of cover in the deck ranged from 49mm in the north west corner to 80mm in the south east corner. The depth of cover in the soffit of the girder beams ranged from 25mm at the mid-span to 65mm near the piers.

In the years since this bridge was constructed, it became apparent that concrete cover has a direct relationship to the long-term durability of reinforced concrete structures. Obviously, the design does not meet the current standards.

From the above it can be seen that the mid-span of the girder beams may be an area of concern for corrosion of reinforcing steel. Minimum 50mm of cover is required under the CHBDC and only 25mm is provided. Fortunately, based on the visual inspection and hammer toning, the girder mid-span areas appear to be in good condition.

5.4 Corrosion Potential Survey

The corrosion potential survey is used to give the probability of corrosion activity of reinforcing steel in the concrete at the time of the survey. In this procedure, a reference electric current is connected to a voltmeter and also connected to the reinforcing steel. The difference in voltage between the reinforcing steel and the current source can be correlated to the amount of corrosion. The corrosion is measured as Low (<5% probability of corrosion), Uncertain (50% probability of corrosion) and High (>95% probability of corrosion).

The full results of the corrosion potential survey can be seen in the Vector report, but are summarized below.

Bridge Component	Corrosion Level	Percent Corrosion
Deck	Low	0.0%
	Uncertain	12.2%
	High	87.8%
Interior Girder Beams	Low	71.2%
	Uncertain	23.8%
	High	5.0%
Exterior Girder Beams	Low	3.4%
	Uncertain	17.4%
	High	79.2%

In light of the above, please note that the reinforcing steel in the deck and beams were not found to be continuous. As a result, completely accurate corrosion potentials cannot be found. Still, in the best case scenario, active corrosion is still taking place in at least 50% of the bridge deck and over 90% in the exterior beams. Without a doubt, these results indicate that active corrosion of the reinforcing steel is a problem throughout the entire structure.

5.5 CARBONATION ANALYSIS

Carbonation of concrete occurs when the pH of the concrete (normally 11 or 12) drops to a range between 9 and 10. When this occurs, the passivating film surrounding the reinforcing steel loses stability and starts to break down, exposing the steel to corrosion.

Four cores were taken from the bridge to test for depth of carbonation (two cores from the deck, one core from the north exterior girder and one core from the south exterior girder). The core from the north east corner of the deck had 5mm depth of carbonation. The core from the north face of the south exterior beam had 15mm depth of carbonation. The other two cores did not indicate any signs of carbonation. Complete results of the carbonation test can be seen in the Vector report.

When considered in combination with the cover survey, it appears that carbonation is not occurring at the depth of reinforcing steel. Therefore, at this time, carbonation of the concrete is not having an affect on the corrosion of the reinforcing steel.

5.6 CHLORIDE ANALYSIS

Chloride ions react with steel, causing deterioration of the structure. Road salt used in road maintenance operations is a primary source of chloride. Chloride ion concentration levels above threshold at the level of reinforcing steel indicate a strong likelihood that corrosion activity of the reinforcing steel is already taking place. Sixteen (16) samples were tested for chloride ion concentration from the same four (4) cores taken for the carbonation analysis.

The test revealed that 100% of the core samples had chloride ion concentrations above threshold at the level of reinforcing steel. This indicates that the presence of chlorides is a concern for the entire bridge, not just limited to specific areas. Since chlorides have already migrated to the level of reinforcing steel, corrosion will continue if mitigation methods are not employed. Refer to the Vector report for full test results.

5.7 COMPRESSION TESTING

The compression test will accurately verify the compressive strength of the concrete utilized in the structure after the concrete has fully cured. Four cores were taken from the structure, two from the deck, and one core from each exterior girder beam. The compressive strength of the concrete cores ranged from 50.6 MPa to 62.4 MPa. The full results of the compression test are attached in appendix D, but are summarized below:

Core	Location	Compressive Strength, MPa
C1	Deck, north east corner	62.4
C2	Deck, south west corner	50.6
C5	North exterior beam, south face	62.0
C6	South exterior beam, north face	52.3

The specified concrete compressive strength utilized for the initial design was not available on the construction drawings. Generally, a minimum 28 day compressive strength for cast-in-place concrete for a structure of this type is specified as 32 MPa. The concrete compressive strength is well in excess of what is normally specified.

6.0 LOAD EVALUATION

As stated in the Description, the bridge was constructed 60 years ago in 1947. The design criteria for highway bridges in Canada have changed significantly since this time. The bridge load carrying capacity has been evaluated using methods outlined in the CHBDC. The analysis assumed a concrete compressive strength of 50 MPa (based on our test results noted above), and deformed billet reinforcing steel bars with a yield strength of 230 MPa (based on the CHBDC for bridges constructed between 1933 and 1975). The area of the reinforcing steel was assumed to be reduced by 5% to account for the active corrosion currently taking place in the structure.

Our calculations indicate that under the CHBDC design loads, the structure is very close to 100% of its capacity. Therefore, there is no need to post the structure with a maximum load capacity at this time. Excerpts from our calculations are provided in Appendix H.

7.0 EVALUATION OF ALTERNATIVES

There are numerous technologies on the market that are intended to slow or stop the corrosion of reinforced concrete structures. Below is a summary of some of the technologies that we believe are viable for this bridge.

1) "Chip and Patch"
One of the traditional methods is the "Chip and Patch" process. Areas of delaminated concrete are delineated by toning the surface with a chain or hammer. Once identified, these areas are then chipped down to the reinforcing steel. Normally, corrosion is present in areas where concrete has delaminated. Therefore, prior to reinstating the concrete surface, the reinforcing steel is cleaned by sand blasting. Then new concrete or a patching compound can be applied to restore the concrete surface. We estimate that this structure will require a chip and patch treatment every 6 years.

   The chip and patch method may temporarily slow the corrosion process, but will not prevent future corrosion. It may even create new corrosion cells. Repetitive treatments will be required, at intervals ranging from 5 to 10 years.

2) Electrochemical Technologies
Electrochemical technologies interrupt the chemistry of the corrosion process, so that it is slowed or stopped completely. We have considered the following technologies for the 9th Street Bridge.

   a. *Discrete sacrificial anodes* may be included in any patch repairs performed in the chip and patch solution. The anodes work by providing galvanic protection to the area directly surrounding the patch. This solution is localized. They do not rid the concrete of chloride contamination or reverse the carbonation process, but will reduce the rate of corrosion in surrounding areas until they are consumed (approximately 10 to 20 years), at which time they could be replaced. The sacrificial method uses a different metal thereby not needing an external power supply.

   b. *Cathodic Protection (CP)* using the impressed current method is a permanent system that halts corrosion. Cathodic protection applies an electrical charge to the reinforcing steel thereby effectively stopping on-going corrosion on the entire structure. The impressed current method requires a permanent power supply and continual monitoring. It therefore has higher long term costs than the sacrificial anode method, but can provide 20 to 40 years of corrosion protection. Specialized contractors are required for installation and maintenance.

   c. *Electrochemical Chloride Extraction* also applies an electrical charge to the reinforcing steel. It is a temporary treatment that involves removing the chloride ions from the concrete and raising the pH level directly around the reinforcing steel. This treatment provides corrosion protection for approximately 15 to 25 years. It also does not involve any long term maintenance or monitoring costs. This option is most effectively used in combination with a coating.

7.1 ALTERNATIVE SOLUTIONS

Although a number of different options are available, we are proposing six alternatives, which are discussed below. Refer to Appendix F for detailed cost estimates.

7.1.1 Do Nothing

Obviously this is the least expensive option in the short term. However, the structure will continue to deteriorate over time and will lead to a much larger expenses in the near future. (i.e. replacement). It is estimated that the structure will last five to ten years under this alternative, and will probably have to be posted for a maximum load capacity within five years.

7.1.2 Replace Entire Structure

We estimate the cost of replacing the structure to be approximately $1.9 million. This alternative has the highest up front cost, but would last approximately 75 years. It also represents the least risky alternative both from a repair cost perspective and from a liability point of view (with respect to the potential for failure).

7.1.3 "Chip and Patch"

This option employs the "chip and patch" method to extend the life of the bridge as long as possible. It is estimated that repairing the concrete in this way could allow the bridge to be used for another 15 years. While the upfront costs are low at approximately $127,000, the cumulative cost of repairs is significant, and after 15 years the bridge will have to be replaced anyway. When compared to replacement, this alternative carries a greater risk of cost over-runs due to uncertainties that may arise during the repair work. It also carries a higher degree of liability with potential for failure to occur. It is also likely that the bridge will have to be posted with a maximum load capacity within ten years.

7.1.4 Deck Overlay with Electrochemical Treatment

This alternative involves patching all areas of delamination, and pouring a deck overlay. Electrochemical treatments would be employed on all surfaces.

It is expected that the initial repair will last 15 years, at which time the bridge will have to be replaced. The initial cost will be around $236,000.

Relative to replacement, this alternative carries a greater risk of failure due to the age of the original components, and the costs are less certain. It is also likely that a maximum load posting will be required within 10 years.

7.1.5 Patch Deck and Treat with Electrochemical Chloride Extraction

This method involves patching all of the concrete surfaces, and treating the deck with electrochemical chloride extraction technology. Prior to re-paving the deck, a water proofing membrane would be installed. All of the other surfaces would be treated with protection measures.

It is expected that the waterproofing membrane would need to be replaced after 10 years, and the entire bridge would have to be replaced in 15 years. We estimate the upfront costs to approximate $230,000.

Much like the previous two alternatives, this alternative carries an elevated risk of failure and the possibility of additional repairs. A maximum load posting will likely be required within 10 years.

7.1.6 Replace Entire Deck and Outer Beams

Since the outer beams have been identified as being worse than the interior ones, it may be possible to remove the two outer beams and the entire deck. New precast concrete girders would then be placed on the existing piers, prior to a new concrete deck being constructed. A considerable amount of work will be required to retrofit the existing structure to receive the new beams.

One major drawback to this alternative is that the existing piers and foundations must be relied upon. It is difficult to determine how much life is left in these components. At the very least, I predict that the inner beams will have to be repaired again in 6 to 10 years, and replaced within

15 years. I predict the initial cost of replacing the outer beams and deck to be around $1.1 million.

As this alternative is quite unique, it is difficult to accurately estimate its cost. When compared to the replacement options, this alternative comes with the potential for unexpected repairs to the original components, and a higher risk of failure of the original components. It is possible that a load posting will be necessary at some time before the original ribs are replaced.

8.0 CONSIDERATION OF ALTERNATIVES AND COST/BENEFIT ANALYSIS

We have attempted to forecast the costs associated with each option with the use of simplistic spreadsheets. Repair costs are forecast for each year under the four repair options. Future costs are increased by the assumed inflation rate of 3%. Total costs after 15 years, including interest costs, are calculated based on an assumed cost of capital of 6%, as if the City is borrowing the funds. Even if funds are not actually borrowed, the interest cost would represent the opportunity cost of the funds that could have been put to other uses. The timeline was ended after year 15 because it is unlikely that the bridge will have much useful life after this time, and will almost certainly have to be replaced. Details of this financial analysis are provided in Appendix G.

The following table summarizes the estimated cost associated with the various repair options.

Option		Present Value of Work	Future Value of Costs at Year 15[i]	Approximate remaining service life (years)
1	Replace entire structure	**$ 1,875,332**	$ 4,494,342	75 Years
2	Chip and Patch (repeat 3 times over next 15 years)	3 x $127,326 = **$381,978**	$ 3,586,813	5 – 10 years per repair
3	Deck overlay with Electrochemical Treatment on all surfaces (includes patching all areas of delamination)	**$ 236,266**	$ 3,374,823	Deck: 20-25 years Int. Beams: 10-15 years Ext. Beams: 15-20 years
4	Patch deck and treat with Electrochemical Chloride Extraction (includes patching all surfaces, a water proofing membrane, and replacement of asphalt/ water proofing membrane.	$230,321 + $14,500 = **$244,821**	$ 3,386,653	Deck: 10-15 years Int. Beams: 10-15 years Ext. Beams: 15-20 years
5	Replace entire deck and outer beams (includes patching of existing components)	**$ 1,113,751**	$ 4,571,551	Deck: 75 years Int. Beams: 10-15 years Ext. Beams: 75 years

[i] Includes 6% cost of capital, and 3% inflation rate. See Appendix G for details.

As the bridge appears to have some useful life left, replacing the entire structure immediately is less desirable from a financial point of view. Replacing the outer two girders involves significant costs, and uncertainties also, since there will be considerable effort required to retrofit the existing piers.

The financial analysis we have carried out relies on assumptions with respect to construction costs, inflation rates, and interest rates. Also, any federal or provincial funding programs will skew the results significantly. Furthermore, it is also difficult to incorporate intangible costs for the rehabilitation options, such as:

a) The inconvenience to truck traffic if the bridge is posted with a load restriction at some time in the future before it is replaced.
b) The risk of failure of the original structural components.
c) The risk of additional unforeseen repairs.

The most cost effective options appear to be ones that employ newer electrochemical technologies. The traditional "chip and patch" methods are only marginally higher in cost (about 6%) than the electrochemical or cathodic protection options.

It is worth nothing that the electrochemical technologies measures require specialized trades from larger centres, whereas the "chip and patch" method may be completed by local bridge contractors. Therefore, if the work is tendered, the "chip and patch" technique will probably yield more competitive bids.

9.0 SUMMARY OF FINDINGS AND RECOMMENDATIONS:

Based on our observations, the following is a summary of our findings and recommendations:

1. There is no need to post the structure for a maximum load capacity at this time.
2. There may be up to 15 years of useful life left in the bridge if it is rehabilitated in accordance with one of the options outlined in this report. Even if it is rehabilitated, it is likely that the bridge will require a maximum load posting within five to ten years.
3. In our opinion, the bridge will have to be replaced within 15 years.
4. Inspections should be carried out annually, including crack gauge monitoring (which has been on-going), until the bridge is replaced. As the bridge continues to deteriorate, more frequent monitoring will be necessary.

As the bridge will have to be replaced within 15 years, the City should begin budgeting for this work immediately. It would be prudent to take advantage of any funding programs that may be available at the federal or provincial levels.

All of which is respectively submitted.

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