Traffic Bridge
2010 - Detailed Visual Inspection and Assessment Report

Prepared for:
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Certifications and Limitations

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This report, entitled "Traffic Bridge", was produced by Stantec Consulting Ltd. and was written by the following individuals:

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Executive Summary

A detailed visual inspection and load rating was conducted on all components of the bridge during the week of August 23rd, 2010. Initial observations identified significant section loss in the components of the bridge below the deck. The section loss recorded appeared to have exceeded the projections established in 2006 when the strengthening that was precipitated from the 2005 inspection had been implemented.

The detailed load rating identified many elements that can no longer support the desired dead and live loads. As well, Span 4 has reached the point that it can no longer support its own weight safely.

Therefore, based on our analysis, we recommend following actions be implemented for the Traffic Bridge:

- All vehicle and pedestrians loads must remain off the structure until repairs are completed;
- Temporary shoring must be installed on the trusses over the Meewasin Valley Trail and the Saskatchewan Crescent East or the traffic accommodated by these facilities must be directed to alternate accesses; and
- If repairs are implemented, the extent of repair must be increased to address all components of the bridge. At this time, we believe that the only viable future option for this structure is to either replace the bridge with a new facility or to completely remove the lower portions of the truss and the entire deck structure system and replace with new.

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1.0 Introduction

1.1 STRUCTURE HISTORY

The Traffic Bridge was constructed in 1907 and spans the South Saskatchewan River. The structure has five main steel truss spans (2 @175 ft, 3 @ 200 ft) with short timber approach spans at each end of the bridge. The bridge originally carried streetcar traffic and currently carries two lanes of vehicle traffic with a sidewalk on the west side of the bridge for pedestrian use.

Past investigations of the Traffic Bridge were carried out in 1986, 1991, 1995, 2002, and 2005. Inspections were carried out previously to 1986, but no information was available from these reports. The 2005 detailed inspection revealed significant deterioration on the truss lower chords which precipitated a complete closure of the facility until repairs were implemented.

A summary of past repairs and maintenance is as follows:

- 1960 – Raised south spans, replaced original abutments and installed approach spans.
- 1978/79 – Recoated steel bottom chord and steel deck framing members.
- 1985 – Installed new exterior deck stringers to replace rotated elements and repaired select areas of the timber deck.
- 1992 – Installed new steel HSS traffic guardrails and pedestrian handrail on walkway.
- 1995 – Installed new concrete bearing pedestals, bearings, expansion joints, and repaired several damaged truss members. Installed new timber sidewalk.
- 1996 – Upgraded piles in timber approaches and new expansion joints.
- 2001 – Upgraded asphalt wearing surface.
- 2003 – Replaced and repaired impact damaged bottom chord members over Saskatchewan Crescent. Replaced damage rails on the north end of the pedestrian walkway. Replaced deteriorated timber stringers on the south approach.
- 2006 – Strengthening of the lower chords on Span 1, 2, and 3 plus isolated areas at discrete locations on the remaining elements.

After the conclusion of the 2006 repairs, the structure was re-opened to traffic with the same restriction a previous load rating completed in 1986 had identified of 5 tonnes.

1.2 CURRENT WORK

The City of Saskatoon (COS) commissioned Stantec Consulting Ltd. to perform a detailed inspection of the steel trusses, floor beams, stringers, and timber deck of the Traffic Bridge located in Saskatoon. This investigation was initiated as a follow-up to an earlier detailed inspection that was completed in 2005 which found significant section loss had compromised the safety of the structure. Subsequent to the 2005 inspection, strengthening was implemented on the lower chords of Span 1 to 3 and at isolated areas on the remaining elements to allow the structure to be returned to service.

Stantec personnel completed the inspection during the week of August 23rd, 2010 with a lift truck on the land spans and with climbers on the river spans. All critical components of the truss were inspected using visual means with rust buildup removed at discrete locations to permit measurement of section. All section loss measurements were compared to the results of 2005 and new F-factors were determined on each component.

Existing conditions and findings from the visual inspections are outlined in Section 2.0 of this report. Results of the load rating are presented in Section 3.0 with recommendations summarized in Section 4.0.

The detailed inspection report results are contained in Appendix A.

2.0 Visual Inspection

A detailed visual inspection was conducted during the week of August 23rd, 2010. Two separate crews were engaged, one who was responsible for the land spans while the second crew was responsible for climbing the bridge to access portions of the bridge over the river. Each inspector was experienced in the inspection and rating of steel structures including the climbing inspectors.

The inspection methodology used is based on the Ontario Structures Inspection Manual (OSIM), 2008 which has been adopted by the Saskatchewan Ministry of Highways and Infrastructure for the provinces infrastructure. This method employs a defect, severity, and extent approach to evaluating a bridge rather than the traditional 1 to 9 point or other similar point rating systems. The strength of the OSIM method is that discrepancies between different inspectors related to interpretation are minimized which ensures results obtained from each inspector are consistent. This consistency of reporting is very important on a structure the size of the Traffic Bridge where different specialists are employed to access the various elements.

However, due to the extent of defects observed and the challenge of physically removing extensive layers of coatings and corrosion by-products on nearly every surface inspected, not all defects could be easily documented and measured. As well, many of the connections are configured in such a way as to preclude a complete visual inspection or cleaning. In these instances, the inspectors used judgment in comparing inaccessible areas with areas that were accessible in order to obtain condition and section loss estimates for use in the load rating presented in Section 3.0.

The terminology used in an OSIM inspection is unique and consistent. In order to assist with your interpretation of the results, a field handbook has been included in Appendix B which contains the methodology used for interpreting the visual observations. If desired, a full OSIM manual can be obtained at: http://www.ogra.org/content_details.asp?itemcode=OGRA-NEWSINFO-MAIN&itemid=12648.

For reference purposes, abutments and piers are numbered from south (Nutana end) to north (Downtown end) and longitudinal members from west (upstream) to east (downstream).

In general terms, the visual inspection identified significant section loss in all elements below the deck surface. The corrosion observed had increased significantly in all areas over the observations recorded in 2005. New areas of corrosion were found which have jeopardized the structural capacity of the elements which had not been strengthened in 2006. As well, portions of the lower chord outside of the strengthening have experienced significant section loss which has also potentially caused the strengthening to be ineffective. When the extent of deterioration was found, the structure was immediately closed to all forms of transportation pending the load rating discussed in Section 3.0 was completed.

Detailed inspection forms and photos for this inspection can be seen in Appendix A. Listed below is a summary of the inspection for the major components of the bridge.

Lower Chord

- Coatings have completely failed;
- Very severe section loss typical on all elements with localized areas of complete section loss ranging from 10% to 60%;
- Locations at random panel points where rivets are missing or have corroded to the point where the rivet head is nearly missing.

Upper Chord

- Coatings are providing some protection; and
- Steel has not sustained any significant section loss due to corrosion, however, localized impact damage is evident on the end panels where the upper chords drop down to the bearing supports.

Verticals and Diagonals

- Above the deck, all verticals and diagonals are in similar condition to the upper chords with isolated areas of impact damage and corrosion;
- Below the deck, significant section loss has occurred on the verticals, diagonals, and gusset plates which connect each element; and
- At localized areas, rivets are missing and in some locations, corrosion has removed the rivet head.

Floor Beams

- Very severe section loss typically concentrated at the connection to each panel point.

Stringers

- Sidewalk stringers have very severe section loss at random locations on the top flange, bottom flange, and web; and
- Deck stringers have very severe section loss at connections and at random locations on the top flange.

Estimated section losses for those elements which impacted the load carrying capacity of the truss were established for each member based on physical measurements as shown below in Table 2.1 and 2.2. Included in Table 2.1 are section losses recorded during the 2005 inspection for the bottom chord. When establishing these section losses for 2010, the inspectors took the worst case identified for each element and applied that loss to the entire element. This is an acceptable approach given that elements presented are tension elements the capacity of which is based on the least section available within the entire element.

For other elements such as floor beams and stringers, section loss at critical locations such as the support or midspan was identified. Since the location of section loss can greatly influence the flexural capacity of a beam, additional information is provided in Table 2.2 which reflects where the section loss occurred. Due to the number of floor beams and stringers contained within this structure only the worst section loss is presented in Table 2.2.

Table 2.1
City of Saskatoon
Traffic Bridge
Summary of Section Loss – Bottom Chords

Upstream Bottom Chord										
Span	**1**		**2**		**3**		**4**		**5**	
Member	**2005**	**2010**	**2005**	**2010**	**2005**	**2010**	**2005**	**2010**	**2005**	**2010**
L0-L1	0%*	5%	32%	50%	13%	30%	24%	30%	12%	40%
L1-L2	0%*	5%	32%	40%	5%	30%	26%	30%	14%	40%
L2-L3	53%	60%	32%	40%	39%	40%	50%	50%	22%	40%
L3-L4	32%	40%	24%	30%	15%	30%	9%	30%	11%	40%
L4-L5	54%	60%	26%	30%	39%	40%	9%	30%	14%	40%
L5-L6	44%	50%	12%	30%	39%	40%	23%	30%	6%	40%
L6-L7	53%	55%	19%	30%	20%	30%	19%	40%	13%	40%
L7-L8	N/A	N/A	38%	40%	13%	30%	11%	50%	N/A	N/A
Downstream Bottom Chord										
Span	**1**		**2**		**3**		**4**		**5**	
Member	**2005**	**2010**	**2005**	**2010**	**2005**	**2010**	**2005**	**2010**	**2005**	**2010**
L0-L1	0%*	5%	49%	50%	19%	30%	19%	50%	21%	40%
L1-L2	0%*	5%	37%	40%	14%	30%	14%	60%	17%	40%
L2-L3	50%	60%	36%	40%	14%	30%	14%	50%	17%	40%
L3-L4	50%	60%	32%	40%	14%	30%	14%	50%	14%	40%
L4-L5	31%	40%	32%	40%	14%	30%	14%	50%	21%	40%
L5-L6	35%	60%	32%	40%	14%	30%	14%	50%	22%	40%
L6-L7	39%	60%	32%	40%	14%	30%	14%	50%	26%	40%
L7-L8	N/A	N/A	32%	40%	19%	30%	19%	50%	N/A	0%*

Table 2.2
City of Saskatoon
Traffic Bridge
Summary of Section Loss – Floor Beams and Stringers

Element	Flexure	Shear	Comments
Floor Beam	10%-30%	50%	Section loss most extreme near the end of the beam, thus primarily affecting shear capacity
Sidewalk Stringer	30%-80%	30%-80%	Section loss varied among the stringers but high losses did occur at regions of high flexural and shear stress
S20 Stringer	20%	20%	Section loss occurs at regions of both high flexural and shear stress
S12 Stringer	20%	20%	Section loss occurs at regions of both high flexural and shear stress

3.0 Detailed Load Rating

3.1 STRUCTURAL ANALYSIS

The Traffic Bridge consists of five steel truss spans 53.2/61/61/61/53.2 plus timber spans at the north and south approach to the structure. Substructure components are concrete for the north abutment and all river piers, steel for the south span south pier, and timber for the south and north timber span abutment.

Several load ratings have been conducted on this structure which has resulted in the current 5 tonne load limit. Prior to 2005, load ratings had identified deck stringers as the critical elements due to the lack of mechanical connection between the top flange and timber deck. Without this connection, under load, the stringers could twist out and cause the deck to fail. This failure mechanism had occurred in 1985 when a City street sweeper broke through the deck when an exterior stringer rotated out from under the deck.

After the 2005 visual inspection, extensive deterioration was observed on the lower chords of Spans 1, 2, and 3 and at isolated areas where verticals, diagonals, and bottom chords connected. The load rating completed as part of this visual inspection revealed the main trusses could no longer support dead and live loads due to the extensive section loss observed. Therefore, after the 2005 inspection, the critical component of the bridge became the bottom truss chord. Strengthening was completed in 2006, which consisted of the addition of post tensioned threaded bars to the bottom chords in the affected spans. These threaded bars were post-tensioned to reduce the tension in the bottom chord and anchored near the midpoint of each end panel and were not connected at any other point along the truss.

The current load rating covers all of the same elements completed in 2005 which includes the bottom chords, deck supports stringers, deck support beams, and sidewalk stringers. Top chord, diagonals and verticals have been added to this report although section loss in these elements is generally localized and concentrated at the connection points below the deck. Estimated corrosion section losses from the 2010 inspection have been incorporated into the analysis, as well as the effects of the 2006 rehabilitation. Of note is that not every element could be accurately measured either due to access issues or the difficulty in measuring the remaining section. Due to these challenges in obtaining inspection results on each element, there is a significant degree of uncertainty in the final calculations. As well, while all connections were inspected, many of them are beginning to show signs of significant pack rust which appears to be causing the rivets to pop off. This type of failure mechanism cannot be accurately quantified as the change in connection properties is abrupt when a rivet fails. Therefore, the results presented in this section must be considered as an approximation of the condition of the bridge. However, it cannot be stressed enough that even though every effort was made to identify the critical section losses which were used in the load rating, other issues could occur resulting in localized failures as the deterioration process continues.

The load rating was performed using the requirements of /CSA-S6-06 (Clause 14, Evaluations) with the design truck used for the 2005 analysis, the MS50 (GVW=5000kg)

3.2 TARGET RELIABILITY INDEX

In new bridge design the load factors employed are fixed for each type of load being considered. When evaluating an existing structure load factors are varied to match the expected reliability or the target reliability index β, for each component. This index is established on the basis of system behavior, element behavior, and inspection level, and is used to determine the dead and live load factors used in the analysis.

As an example, for this structure the trusses are categorized as an S1 system since failure of one portion of the truss such as the bottom chord, diagonals or verticals, would lead to total collapse. When considering individual elements under load, the bottom chords under tension can be characterized as a Category E1, where failure could occur suddenly at the net cross section. The inspection level of the bridge is Level INSP3, since an on-site inspection of critical and/or substandard components has been carried out and the evaluation calculations account for the information obtained by the inspection.

Using this analysis, the target reliability index's were obtained for each element evaluated in the analysis which are shown in Table 3.1.

3.3 MATERIALS

Yield strength of all existing steel elements was based on the coupon testing performed in 1986 which indicated a strength of 260MPa. The modulus of elasticity was taken to be 200GPa.

Table 3.1
City of Saskatoon
Traffic Bridge – 2010 Detailed Visual Assessment and Load Rating
Summary of Load Factors

Element	Bottom Chord	Diagonals	Verticals	Top Chord	Floor Beams	Traffic Stringers	Sidewalk Stringers	
System Behavior	S1	S1	S1	S1	S1	S3	S3	
Element Behavior	E1	E1	E1	E1	E3	E3	E3	
Insp. Level	INSP3	INSP3	INSP3	INSP3	INSP3	INSP3	INSP3	
β =	3.75	3.75	3.75	3.75	3.00	2.75	2.75	
D.L Factors								
D1 (α_D)	1.1	1.1	1.1	1.1	1.07	1.06	1.06	
D2 (α_D)	1.2	1.2	1.2	1.2	1.14	1.12	1.12	
D3 (α_D)	1.5	1.5	1.5	1.5	1.35	1.3	1.3	
L.L Factors								
MS50 (α_L)	1.7	1.7	1.7	1.7	1.49	1.42	1.42	

3.4 DEAD LOADS

CAN/CSA-S6-06 recommends the use of three types of dead loads, as shown below in table 3.2. Type D1 includes all factory produced components and cast-in-place concrete excluding decks. The calculated self-weight of the steel components was increased by 5% to allow for connection material. Type D2 is for cast-in-place concrete decks, wood, and asphalt measured in the field. For the bridge, the asphalt thickness was measured in 2001 and was found to vary between 140mm-230mm therefore an average thickness of 185mm was used for this analysis. Type D3 is used for asphalt if the asphalt thickness has not been measured and therefore was not used.

Table 3.2
City of Saskatoon
Traffic Bridge
Dead Loads

Type	Element	Dead Load
D1	Steel Self-weight	(Cross Sectional Area)x(Density)x(1.05) kN/m
D2	Wood Deck	1.50 kPa
D2	Sidewalk Deck	0.30 kPa
D2	Asphalt	4.48 kPa
D3	N/A	N/A

3.5 LIVE LOADS

The live loads considered are based on an MS style of vehicle which has been the standard vehicle used for assessment on this structure, refer to Figure 3.1 below.

The capacity of each element is assessed by calculating F-factors at critical locations of section change or regions of high stress using the following formula:

$$F = \frac{U\phi R - \sum \alpha_D D - \sum \alpha_A A}{\alpha_L L(1+I)}$$

Where U is the element resistance adjustment factor, Φ is a material reduction factor, R is the element resistance, α_D, α_A, and α_L are the dead load, additional loads, and live load factors respectively, D, A, and L are the element applied dead load, additional load, and live load respectively, and I is the impact value.

F-factors represent the percentage of load-carrying capacity remaining once dead loads have been accounted for. Therefore, an F-factor equal or greater than 1.0 implies that once dead loads are accounted for, the element still has the capacity required to resist live loads introduced by the worst case live loading condition. F-factors less than 0 indicate that the element cannot support the applied dead loads under the target reliability index selected for that element. F-factors that fall between 0 and 1 indicate that the element can support the dead loads but are unable to support all of the live loads.

Detailed Load Rating

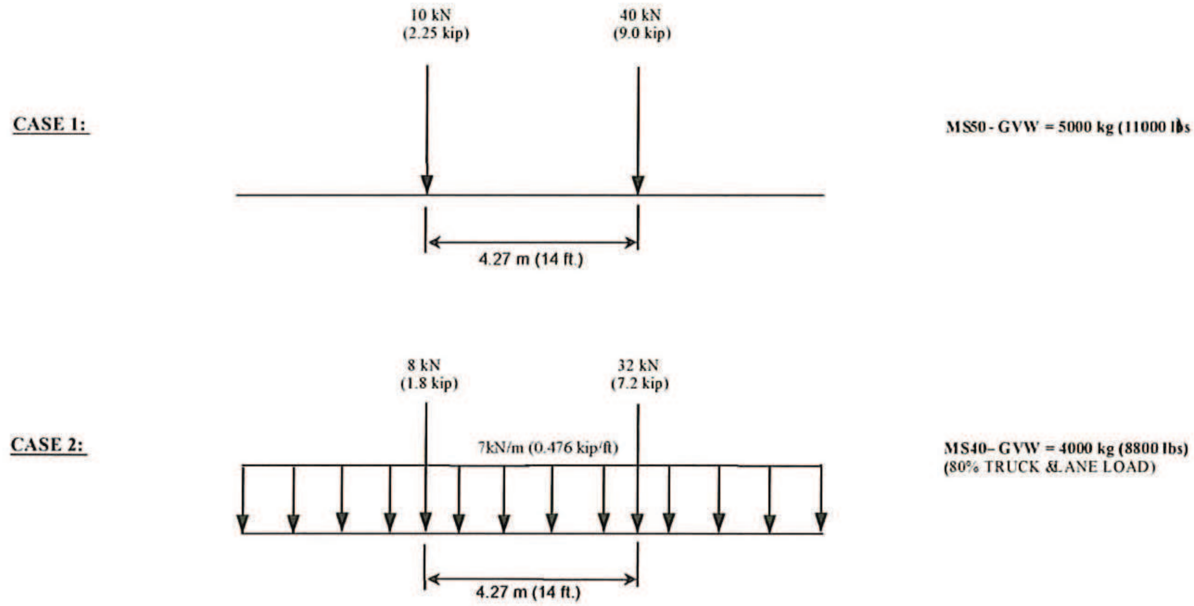

Figure 3.1. MS50 Truck

Two load cases of live loading were considered. The first load case was a full truck with a dynamic load factor of 1.30. The second load case consists of 80% of the design truck without a dynamic load allowance but with an additional lane load of 7 kN/m applied to each lane over the entire length of the bridge. A live load of 2.4 kPa on the pedestrian walkway was taken in combination with the vehicular traffic loading. This combination of loads causes heavier loading applied to the upstream truss which is reflected by the lower average F factors.

3.6 LOAD CARRYING CAPACITY

A summary of the load factor, F, for the MS50 rating is summarized in Tables 3.3 and 3.4. Table 3.3a includes load factors that had been calculated in 2005 for comparison purposes for the lower chord. Tables 3.3b through 3.3e include the worst F factors that have been calculated from the 2010 inspection for all other elements. Values for elements other than the lower chord were not calculated in 2005 as deterioration of these elements at that time had not progressed to the point that their strength was considered an issue.

The load rating indicates that strengthening completed in 2006 increased the capacity of those elements although in some cases, continued corrosion has lead to F factors less than 1.0 even for the strengthened elements. Areas that were not strengthened in 2006 have shown a drastic reduction in capacity which is directly related to the increased section loss observed. In particular, the downstream truss in Span 4 has F factors which are less than 0 which indicates that the truss cannot support its own weight using the target reliability values established previously. As well, end panels of the bottom chord have also changed significantly since the 2006 strengthening. At these locations, section loss has occurred outside of the area that was strengthened which has compromised the strengthening that had been installed.

Detailed Load Rating

For the stringers and floor beams, the section loss recorded in Section 2.0 was highly variable which presented challenges in determining the appropriate section properties required for determining the F-factor. Conservative section loss assumptions were made to account for this variability which is reflected in the values below.

Table 3.3a City of Saskatoon Traffic Bridge Summary of Truss Bottom Chord F-Factors										
Upstream Bottom Chord										
Span	**1**		**2**		**3**		**4**		**5**	
Member	**2005**	**2010**	**2005**	**2010**	**2005**	**2010**	**2005**	**2010**	**2005**	**2010**
L0-L1	3.14	2.38	1.29	0.06	2.13	0.92	1.66	0.92	2.55	0.74
L1-L2	3.14	3.49 *	1.30	1.60 *	2.46	2.03 *	1.55	0.92	2.43	0.74
L2-L3	0.43	0.71*	1.21	1.24 *	0.91	1.24 *	0.41	2.09 **	1.85	0.86
L3-L4	1.56	1.39 *	1.68	1.05 *	2.09	1.05 *	2.37	0.36	2.60	0.63
L4-L5	0.35	0.71*	1.61	1.05 *	1.01	0.70*	2.37	0.36	2.18	0.86
L5-L6	1.01	1.38 *	2.06	1.66 *	0.91	1.24 *	1.58	0.84	2.82	0.74
L6-L7	0.55	0.04	1.86	2.03 *	1.81	2.03 *	1.84	0.49	2.51	0.74
L7-L8	N/A	N/A	1.02	0.49	2.13	0.92	2.20	0.06	N/A	N/A
Downstream Bottom Chord										
Span	**1**		**2**		**3**		**4**		**5**	
Member	**2005**	**2010**	**2005**	**2010**	**2005**	**2010**	**2005**	**2010**	**2005**	**2010**
L0-L1	5.17	4.04	1.06	0.38	3.18	1.75	3.18	0.38	3.51	1.45
L1-L2	5.17	5.65 *	1.88	2.69 *	3.50	3.38 *	3.50	-0.30	3.85	1.45
L2-L3	1.08	1.31 *	1.82	2.16 *	3.36	2.82 *	3.36	0.28	3.52	1.65
L3-L4	1.28	0.96*	2.34	1.32 *	3.63	1.88 *	3.63	-0.27	4.08	1.28
L4-L5	2.50	2.88 *	2.34	1.32 *	3.63	1.88 *	3.63	-0.27	3.17	1.65
L5-L6	2.45	1.58 *	2.13	2.16 *	3.36	2.82 *	3.36	0.28	3.47	1.45
L6-L7	2.17	-0.03	2.24	2.69 *	3.50	3.38 *	3.50	0.38	3.15	4.41 **
L7-L8	N/A	N/A	2.24	1.06	3.18	1.75	3.18	0.38	N/A	N/A

* denotes members that were strengthened in 2006 with Post Tension system

** denotes members that were strengthened in 2006 with reinforcing plates

Table 3.3b City of Saskatoon Traffic Bridge Summary of Truss Top Chord F-Factors					
Summary of Truss F-Factors Upstream Top Chord (Compression Limit)					
Span / Member	**1**	**2**	**3**	**4**	**5**
L0-U1	1.48	1.33	1.33	1.33	1.48
U1-U2	1.87	1.23	1.23	1.23	1.87
U2-U3	1.65	1.34	1.34	1.34	1.65
U3-U4	1.56	1.22	1.22	1.22	1.56
U4-U5	1.65	1.22	1.22	1.22	1.65
U5-U6	1.87	1.34	1.34	1.34	1.87
U6-U(L)7	1.47	1.23	1.23	1.23	1.47
U7-L8	N/A	1.33	1.33	1.33	N/A
Summary of Truss F-Factors Downstream Top Chord (Compression Limit)					
Span / Member	**1**	**2**	**3**	**4**	**5**
L0-U1	1.65	2.40	2.40	2.40	1.65
U1-U2	2.05	2.25	2.25	2.25	2.05
U2-U3	1.83	2.42	2.42	2.42	1.83
U3-U4	1.74	2.25	2.25	2.25	1.74
U4-U5	1.83	2.25	2.25	2.25	1.83
U5-U6	2.05	2.42	2.42	2.42	2.05
U6-U(L)7	1.65	2.25	2.25	2.25	1.65
U7-L8	N/A	2.40	2.40	2.40	N/A

Table 3.3c City of Saskatoon Traffic Bridge Summary of Truss Vertical F-Factors					
Summary of Truss F-Factors Upstream Verticals (Tension or Compression Limit)					
Span / Member	**1**	**2**	**3**	**4**	**5**
L1-U1	4.37	4.38	4.38	4.38	4.37
L2-U2	18.23	11.18	11.18	11.18	18.23
L3-U3	14.33	20.93	20.93	20.93	14.33
L4-U4	14.34	17.42	17.42	17.42	14.34
L5-U5	18.23	20.93	20.93	20.93	18.23
L6-U6	4.37	12.12	12.12	12.12	4.37
L7-U7	N/A	5.93	5.93	5.93	N/A
Summary of Truss F-Factors Downstream Verticals (Tension or Compression Limit)					
Span / Member	**1**	**2**	**3**	**4**	**5**
L1-U1	4.51	6.44	6.44	6.44	4.51
L2-U2	18.28	13.96	13.96	13.96	18.28
L3-U3	14.47	24.60	24.60	24.60	14.47
L4-U4	14.47	23.87	23.87	23.87	14.47
L5-U5	18.28	24.60	24.60	24.60	18.28
L6-U6	4.51	13.96	13.96	13.96	4.51
L7-U7	N/A	8.44	8.44	8.44	N/A

Table 3.3d
City of Saskatoon
Traffic Bridge
Summary of Truss Diagonal F-Factors

Summary of Truss F-Factors
Upstream Diagonals (Tension Limit)

Span / Member	1	2	3	4	5
U1-L2	4.17	2.62	2.62	2.62	4.17
U2-L3	5.68	2.77	2.77	2.77	5.68
U3-L2	15.62	3.54	3.54	3.54	15.62
U3-L4	6.91	12.68	12.68	12.68	6.91
U4-L3	6.93	12.70	12.70	12.70	6.93
U4-L5	15.63	3.54	3.54	3.54	15.63
U5-L4	5.68	2.77	2.77	2.77	5.68
U6-L5	4.16	2.62	2.62	2.62	4.16

Summary of Truss F-Factors
Downstream Diagonals (Tension Limit)

Span / Member	1	2	3	4	5
U1-L2	4.32	4.15	4.15	4.15	4.32
U2-L3	5.81	4.08	4.08	4.08	5.81
U3-L2	15.52	5.26	5.26	5.26	15.52
U3-L4	6.95	10.39	10.39	10.39	6.95
U4-L3	6.95	10.39	10.39	10.39	6.95
U4-L5	15.57	5.27	5.27	5.27	15.57
U5-L4	5.81	4.08	4.08	4.08	5.81
U6-L5	4.32	4.15	4.15	4.15	4.32

Table 3.3e City of Saskatoon Traffic Bridge Summary of Truss F-Factors		
Deck Beams		
Member	**Limit**	**F Factor**
Transverse Beam	Moment	2.35
	Shear	3.41
Deck Stringer (S20x65)	Moment	5.08
	Shear	19.40
Deck Stringer (S12x31.5)	Moment	0.96
	Shear	7.69
Sidewalk Stringer	Moment	0.35
	Shear	8.84

4.0 Discussion and Recommendations

In the following sections, brief discussions are held regarding the visual observations and load calculations for each major component of the bridge excluding the substructure components and timber approach spans. Before final recommendations can be presented, an overview of how a truss performs and a presentation of system performance is required.

CSA S6 defines system performance of a structure in three broad categories as follows:

- Category S1, where element failure leads to total collapse. This includes failure of main members with no benefit from continuity or multiple-load paths;
- Category S2, where element failure probably will not lead to total collapse. This includes main load-carrying members in a multi-girder system or continuous main members in bending; and
- Category S3, where element failure leads to local failure only. This includes deck slabs, stringers, and bearings in compression.

For the truss components, almost every element in the truss, including bottom/top chords, verticals, diagonals, and connections fall into Category S1 as the truss by its very nature is non-redundant with failure of any one component resulting in failure of the entire truss. The floor deck and stringers are considered to meet Category S3 conditions, where only localized failures occur when one of these components fail.

As presented earlier, the configuration of the truss, extent of defects, and challenges in accessing all components create an environment where there is a high degree of uncertainty related to the current condition. As well, nearly all connections on the truss have some form of deterioration which is causing the rivets to fail which produces an environment where if one connection fails, the entire truss can collapse. The F-Factors that were produced in Section 3 have attempted to capture the uncertainty created by these features of the structure and the inspection methods employed (primarily visual). Therefore, the values identified can be considered realistic given the limitations imposed. However, there still remains the potential that a connection could abruptly fail as deterioration of heavily corroded connections generally do not follow a well defined path.

Based on the F-Factors calculated, it is apparent that the structure must remain closed to vehicles and pedestrians. Pedestrians cannot use the structure, either on the sidewalk or the roadway, as the individual sidewalk stringers have reached the point where they cannot support both live and dead loads as well, Span 4 cannot support its own dead load. As well, temporary shoring should be considered for those spans that cross over trails and streets as the potential exists that the truss above could abruptly fail. Therefore, shoring should be installed under spans 1, 2 and 5 as these spans cross over the Meewasin Valley trail system and the Saskatchewan Crescent East. Span 4 also presents a special challenge as the F-factors calculated are below 0 which indicates that the structure cannot support its own weight. Since shoring cannot be installed at this location easily, we recommend that all river traffic be directed to avoid this span.

Strengthening could be considered for this structure in a similar manner to what was proposed in 2006. However, this would only deal with the bottom chord deterioration and would not address the section loss and connection issues present on the floor beams, stringers, and panel point connections. Therefore, we do not recommend that any strengthening be implemented on this structure unless it addresses the entire structure. If localized strengthening is implemented in a similar manner to what was completed in 2006 the City runs the definite risk that future inspections will identify a major deficiency that will cause the structure to be closed unexpectedly.

Therefore, based on our analysis, we recommend the following actions be implemented for the Traffic Bridge:

- All vehicle and pedestrians loads must remain off the structure until repairs are completed;
- Temporary shoring must be installed on the trusses over the Meewasin Valley Trail and the Saskatchewan Crescent East or the traffic accommodated by these facilities must be directed to alternate accesses; and
- If repairs are implemented, the extent of repair must be increased to address all components of the bridge. At this time, we believe that the only viable future option for this structure is to either replace the bridge with a new facility or to completely remove the lower portions of the truss and the entire deck structure system and replace with new.