EXECUTIVE SUMMARY

Delcan Corporation (Delcan) has been retained by Public Works and Government Services Canada to conduct visual evaluations of 56 bridge structures and 6 culvert structures along the British Columbian portion of the Alaska Highway in both 2009 and 2011. Also, as part of this contract, Delcan was to complete live load capacity factor structural evaluations for 11 of the Alaska Highway bridge structures. In 2009, Delcan performed 6 of these live load capacity factor structural evaluations: Beatton River at km 232.8, Sikanni Chief River at km 256.1, Buckinghorse River at km 277.6, Bougie Creek at km 357.4, Adsett Creek at km 366.0, and Jackfish Creek at km 424.8. Enclosed in this submission are the live load capacity factor structural evaluations performed for the 5 other bridge structures: Kledo River at km 509.1, Steamboat Creek at km 515.3, Tetsa River 1 at km 584.6, Toad River at km 671.7, and Peterson Creek at km 678.6. Also enclosed are the 2009 condition inspection reports for these 5 bridges.

Delcan also visually inspected the 56 bridge structures and 6 culvert structures in 2001, 2005, and 2007, in 2005 performed structural demand-capacity and seismic evaluations of the Racing River and Muskwa River Bridges, at km 641.1 and km 451.8, respectively, and in 2007 performed a structural evaluation of the Lower Liard River Bridge at km 763.3 for dead, live, and wind load effects of a painting contractor's operations. Delcan has designed and replaced the Trout River bridge at km 732.6, has designed the replacement bridge for the Racing River at km 641.1, and has engineered and performed the complete structural rehabilitation of the Hyland River bridge at km 937.3. Therefore, Delcan has extensive knowledge on the conditions of the Alaska Highway structures. This information was extremely useful in completing the 11 load ratings.

The overall results of the enclosed 5 live load capacity factor structural evaluations are repeated here:

- **Kledo River Bridge**: There are no major load capacity issues with Kledo River Bridge and no posting is required. No LLCFs for Kledo River Bridge are less than 1 for the ultimate limit state. The existing end transverse bracing between the box girders is not as per shown on the original structural drawings (i.e. it is missing). It is important that this bracing be added in the field as soon as possible. Part of the 2009/2011 contract that Delcan has with PWGSC is to prepare sketches for the addition of these secondary members.
- **Steamboat Creek Bridge**: There are no major load capacity issues with Steamboat Creek Bridge and no posting is required. No LLCFs for Steamboat Creek Bridge are less than 1 for the ultimate limit state.
- **Tetsa River Bridge #1**: There are no major load capacity issues with Tetsa River Bridge #1 and no posting is required. No LLCFs for Tetsa River Bridge #1 are less than 1 for the ultimate limit state. The damaged portal and sway frame through-truss top chord lateral bracing should be repaired/replaced in the field as soon as possible. Part of the 2009/2011 contract that Delcan has with PWGSC is to prepare repair sketches for these damaged secondary members.
- **Toad River Bridge**: There are no major load capacity issues with Toad River Bridge and no posting is required. No LLCFs for Toad River Bridge are less than 1 for the ultimate limit state. The existing transverse bracing (end and intermediate) between the box girders is not as per shown on the original structural drawings. It is important that this bracing be updated/changed in the field as soon as possible. Part of the 2009/2011

contract that Delcan has with PWGSC is to prepare sketches for the addition/removal of these secondary members.

- **Petersen Creek Bridge**: There are no major load capacity issues with Petersen Creek Bridge and no posting is required. No LLCFs for Petersen Creek Bridge are less than 1 for the ultimate limit state.

PUBLIC WORKS AND GOVERNMENT SERVICES CANADA

KLEDO RIVER BRIDGE
ALASKA HIGHWAY, km 509.1

STRUCTURAL LIVE LOAD CAPACITY FACTOR EVALUATION

Prepared by:

TRANSPORTATION • INFORMATION TECHNOLOGY • WATER

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Peter Phillips, P.Eng., Intermediate Bridge Engineer

March 2010

TABLE OF CONTENTS

EXECUTIVE SUMMARY

A detailed visual condition assessment for the Kledo River Bridge, km 509.1 along the Alaska Highway, was performed in September 2009. This condition assessment and Chapter 14, “Evaluation”, of the “Canadian Highway Bridge Design Code (CHBDC)”, CAN/CSA-S6-06, were used to evaluate the load carrying capacity of this existing bridge.

Kledo River Bridge was evaluated for CL1-W truck/lane live loading (Evaluation Level 1) in accordance with the CHBDC. Load factors were determined from Clause 14.13 based on target reliability indices (beta-factors) as determined from Clause 14.12 of the CHBDC. Resistance adjustment factors as determined from Clause 14.14.2 of the CHBDC were also used. These target reliability indices and load factors were reviewed by Public Works and Government Services Canada (PWGSC).

Live load capacity factors (LLCFs) for each superstructure member, as per Clause 14.15, were calculated (if and when applicable) for ultimate limit states in bending, shear, and axial. Punching shear was checked for this bridge’s deck.

All members of the existing bridge are capable of carrying their combination of factored dead and imposed truck/lane live loads. No bridge strengthening or posting is currently required.

1.0 INTRODUCTION

To determine if the bridge structure requires strengthening or posting, a bridge inspection was first carried out. The bridge's original structural drawings were reviewed in order to familiarize oneself with the structure, prior to an assessment in the field of its member conditions and overall stability. A load rating was then undertaken to establish which members are under-strength, if any. Section 14 of the Canadian Highway Bridge Design Code (CHBDC) outlines this process: target reliability indices (beta-factors) for each member of the bridge were determined and then dead and live load factors for each member of the bridge were chosen.

A model of the bridge, based on the requirements of Section 5 of the CHBDC, was created. Largest combined dead and live factored loads were determined for each member. The resistance of each member was compared to its respective applied force, taking into consideration the deterioration of the members as well. This comparison was used to determine which members were under-strength, if any. If members are determined by analysis to be under-strength currently, it is due to changes in design code requirements, deteriorating member conditions in the field, lack of conservatism in the original designs, or poor original construction practices. The load rating in combination with the field inspection program was used to classify the capacity problem (i.e. problems in flexure, shear, compression, tension, torsion, serviceability, fatigue, maintainability, durability, etc.).

The following original structural drawings were provided by PWGSC to assist in the structural evaluation of this bridge. The drawings provided a reference for dimensions, methods of original construction, substructure details (i.e. details not visible in the field), and material strengths, etc. Relevant assumptions, however, also needed to be made and other pertinent dimensions, data, etc. were obtained in the field during the visual bridge inspections.

Drawings Obtained:
Kledo River Bridge: Drawings 1 to 14, dated 1978.

2.0 BRIDGE DESCRIPTION

An elevation photo of the Kledo River Bridge is shown in Figure 1, below.

Figure 1: Kledo River Bridge Photo.

Description of Kledo River Bridge:

a) Single span weathering steel box girders.
b) Reinforced concrete deck.
c) Reinforced concrete abutments.
d) Steel pipe pile foundations under abutments
e) Gabion wall slope protection.
f) Steel railing on reinforced concrete curb traffic barriers.

Relevant dimensions of the bridge's elevation and cross-section are shown in Figures 2 and 3, respectively.

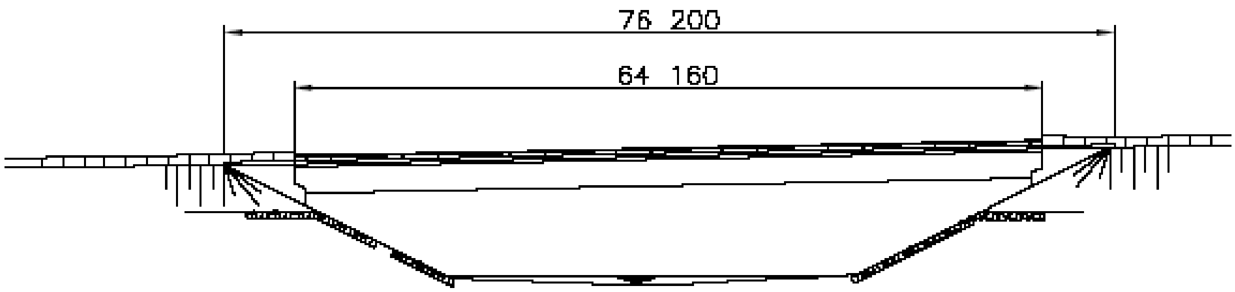

Figure 2: Elevation View of Kledo River Bridge.

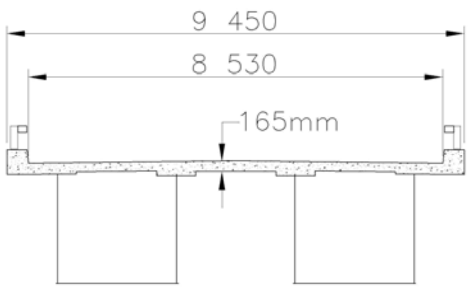

Figure 3: Cross-Section View of Kledo River Bridge.

3.0 CONDITION CONSIDERATION

The bridge is generally in good condition. Based on the site evaluation carried out in September 2009 of this structure, no significant loss of section or other bridge issues that could contribute negatively to the overall structural integrity of the bridge were found. For a detailed site condition rating of this bridge please refer to Delcan's 2009 bridge inspection report.

It is important to point out that Kledo River Bridge's end diaphragm cross-bracing between its two box girders at each end of this bridge has been improperly removed and should be replaced. Delcan modeled this bridge in its current (existing) condition with the end diaphragm cross-bracings removed.

See Figures 4 to 12, below, for condition photos taken during Delcan's recent 2009 inspections of the Alaska Highway structures.

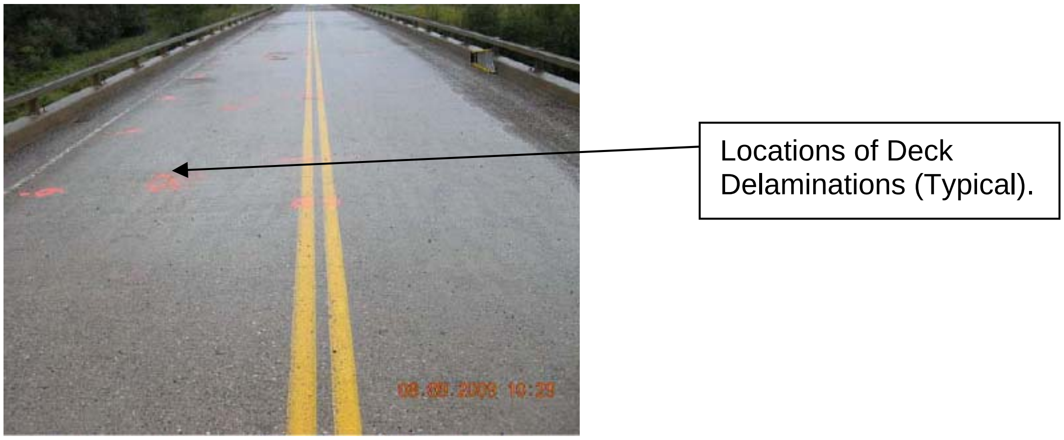

Figure 4: Deck.

Figure 5: Exterior of Box Girder.

Figure 6: Abutment and Wingwall.

Figure 7: Box Girder Bearing.

Figure 8: Between Box Girders.

Figure 9: End Bracing.

Figure 10: Typical Diaphragm Inside of Box Girder.

Figure 11: Water-Pooling Inside of Both Box Girders.

Figure 12: Efflorescence Staining Inside of Both Box Girders.

4.0 EVALUATION PARAMETERS AND METHODOLOGY

4.1 Evaluation Procedures

Kledo River Bridge was evaluated for CL1-W truck/lane live loading (Evaluation Level 1) in accordance with the CHBDC. Load factors were determined from Clause 14.13 based on target reliability indices (beta-factors) as determined from Clause 14.12. Resistance adjustment factors as determined from Clause 14.14.2 of the CHBDC were also used. These target reliability indices and load factors were reviewed by PWGSC.

Live load capacity factors for each superstructure member, as per Clause 14.15, were calculated (if and when applicable) for ultimate limit states in bending, shear, and axial. Punching shear was checked for this bridge's deck.

Delcan modeled the Kledo River Bridge using "Midas Civil" (Midas) software utilizing a grillage-type model. The structural model was developed using beam elements for the superstructure members and deck and pin or roller supports, as applicable, as substructure elements. As substructure elements were in good condition with respect to structural integrity and member stability based on the September 2009 visual bridge inspections, substructure elements were not considered further in this live load capacity factor evaluation of the Kledo River Bridge. It was assumed that the substructure elements provide full support to the superstructure members.

Also, member-to-member connections (all joints) were assumed to be fully effective (i.e. in providing full capacity to transfer loads between the connected elements). All connections of secondary members to primary members were assumed to be pinned-pinned connections.

See Figure 13, below, for a rendered view of the Midas model.

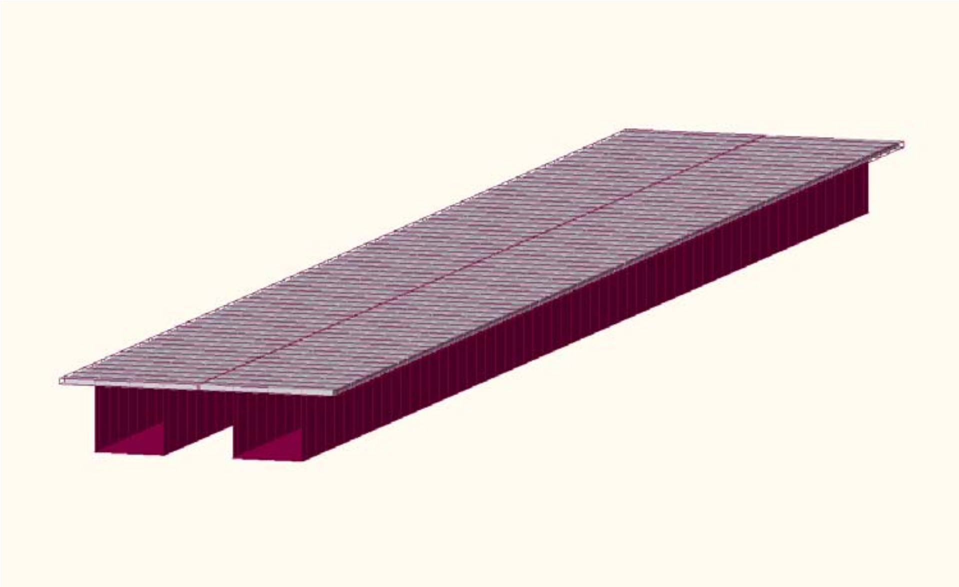

Figure 13: Kledo River Bridge Rendered Midas Civil Grillage-Type Model.

4.2 Reliability Indices and Load Factors

The following Table #1, below, provides:

a) System behaviour, element behaviour, and inspection level classifications.
b) Reliability indices as determined for “Normal Traffic”.
c) An adjustment to the reliability index of 0.25 based on Clause 14.12.5 and recognizing this bridge as an important structure.
d) Dead load factors based on Clause 14.13.2.1.
e) Live load factors for normal traffic based on Clause 14.13.3.1.
f) A multilane factor for normal traffic based on Clause 14.9.4.2.
g) Dynamic load allowances for normal traffic based on Clause 14.9.1.7.

It was assumed that a simply supported two-box girder structural system is not a redundant system, with respect to ‘System Behaviour’ – Clause 14.12.2 of the CHBDC. Also, all members would be subjected to ‘gradual failure with warning of probable failure’ regardless of material or load carrying direction / capacity, with respect to ‘Element Behaviour’ – Clause 14.12.3 of the CHBDC.

Project: Alaska Highway Load Rating		Design: P. Phillips, PEng		Date: 2010/01/27
Subject: Kledo: Reliability Index & Load Factors		Check: H. Hawk, MSc, PEng		Date: 2010/01/27
	Element			
	Box Girders	Cross-Bracing	Interior Bracing	Deck
Category				
System Behaviour	S1	S3	S3	S3
Element Behaviour	E3	E3	E3	E3
Inspection Level	I2	I2	I2	I2
Live Load Lateral Dist	Sophisticated Analysis	Sophisticated Analysis	Simplified Analysis	Simplified Analysis
Reliability Index, β: CL1-W	3.25	2.75	2.75	2.75
Important Structure: β Increased by	0.25			
CL1-W	3.50	3.00	3.00	3.00
Factors				
DL	D1	D1	D1	D2
α_D: CL1-W	1.09	1.07	1.07	1.14
DL	D2	D2	D2	D1
α_D: CL1-W	1.18	1.14	1.14	1.07
$\alpha_{L\,(CL1-W)}$: CL1-W	1.63	1.49	1.49	1.49
Multi-Lane Factor	0.9			
1+DLA	1.25	1.25	1.25	1.40

Table #1: Reliability Indices and Load Factors.

4.3 Resistance Adjustment Factors

Resistance adjustment factors, as follows, were determined from Clause 14.14.2 of the CHBDC. The factored resistance of an individual structural component under consideration was multiplied by the appropriate resistance adjustment factor.

Structural Steel

a) Shear: U = 1.02.

Composite – Slab on Steel Girder:

a) Bending: U = 0.96.
b) Shear connectors: U = 0.94.

Reinforced Concrete Deck:

a) Bending: U = 0.95.

4.4 Permanent Loads

The dead loads in the model include:

a) The full self-weight of the primary superstructure elements.
b) The weights of the steel box girders were adjusted upwards by 12% to account for splice plate weights, steel connections, gusset plates, longitudinal and transverse web stiffeners, bearing stiffeners, interior cross-frames within the box girders, cross-frames between the box girders, etc. as per Clause C14.8.2.1 of the CHBDC Commentary.
c) 8" concrete deck is still applicable as shown on original drawings as provided by PWGSC.
d) Weights of the stay-in-place steel deck formwork and the box girder flange concrete haunches.
e) No deck overlay included.
f) Bridge barriers in the field (curb and railing type barriers) are the same as the barriers shown on the drawings provided to Delcan.

4.5 Normal Traffic Live Loads

Kledo River Bridge was evaluated for Evaluation Level 1 CL1-W truck/lane live loading. Two lanes exist in the field (one Northbound lane and one Southbound lane) and therefore two traffic lanes were modeled, as specified in Clause 14.9.4.1. Appropriate multiple-lane load factors and dynamic load factors were applied to the truck and/or lane loading, when applicable.

4.6 Material Strengths

The material properties, as provided below, used in the resistance calculation processes were obtained from the structural drawings provided by PWGSC. Kledo River Bridge was originally built in 1978. Dates of any subsequent modifications made to this bridge are unknown to Delcan.

The following values were used in the evaluation of the Kledo River Bridge:

a) Concrete deck and curb compressive strength: 4000 psi (27.6 MPa), Drawing Number 1 of 14.
b) Reinforcing deck and curb steel yield strength: Grade 60 (400 MPa), Drawing Number 1 of 14.

c) Superstructure structural steel yield strength: Grade 50A (350 MPa), Drawing Number 1 of 14.
d) Superstructure structural steel ultimate strength: 480 MPa, Drawing Number 1 of 14.

5.0 RESULTS

5.1 Key-Plans/Elevations

The following key-plan diagram indicates the naming conventions of the individual structural members within this bridge that were adopted for this load rating. All members referenced in this 'Results' section will therefore be referred to by their key-plan names.

See Figure 14 for key-plan/elevation drawings for the Kledo River Bridge:

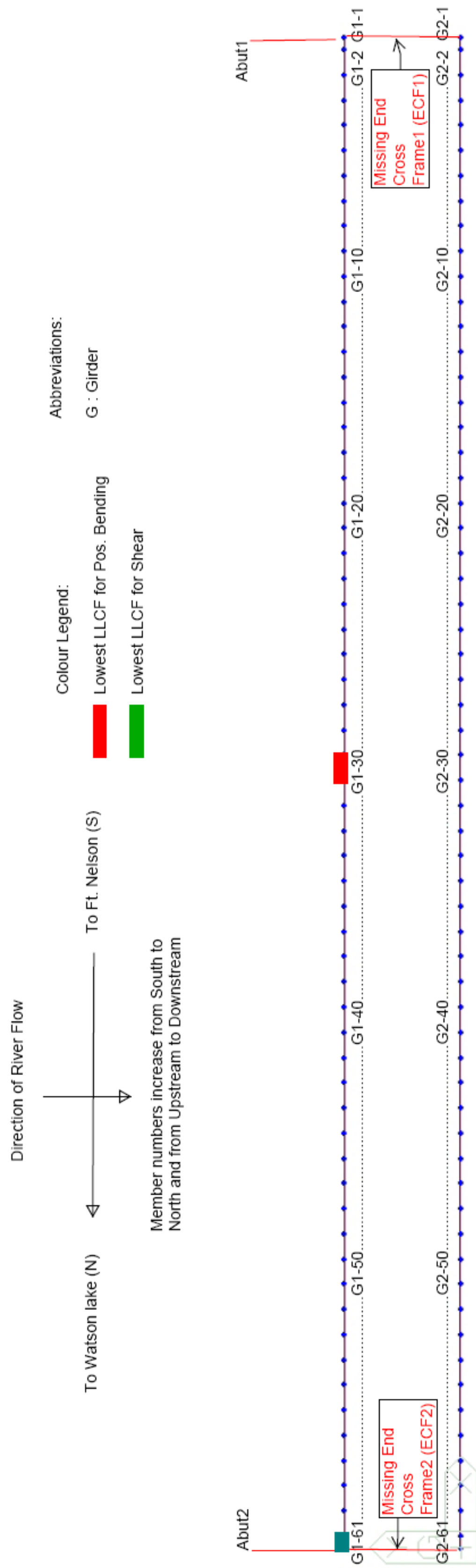

Figure 14: Kledo River Bridge Plan View Naming Conventions.

5.2 Live Load Capacity Factors

The following Figures 15 to 16 show the live load capacity factors (LLCFs) that have been calculated along Kledo River Bridge's members based on the requirements of Section 14 of the CHBDC S6-06. Specifically, LLCFs are calculated based on Clause 14.15.2, 'Ultimate Limit States'. LLCFs greater than 1 are deemed adequate for the prescribed live loading and LLCFs less than 1 generally require posting.

The location along a member which is considered to govern its design is the position where the member is most highly loaded relative to its resistance.

Due to overall bridge symmetry (i.e. no skew effects), symmetrical transient loading present, and simply supported bearing conditions, symmetrical force diagrams are produced within this bridge. In such cases, half-spans of the members need only to be shown.

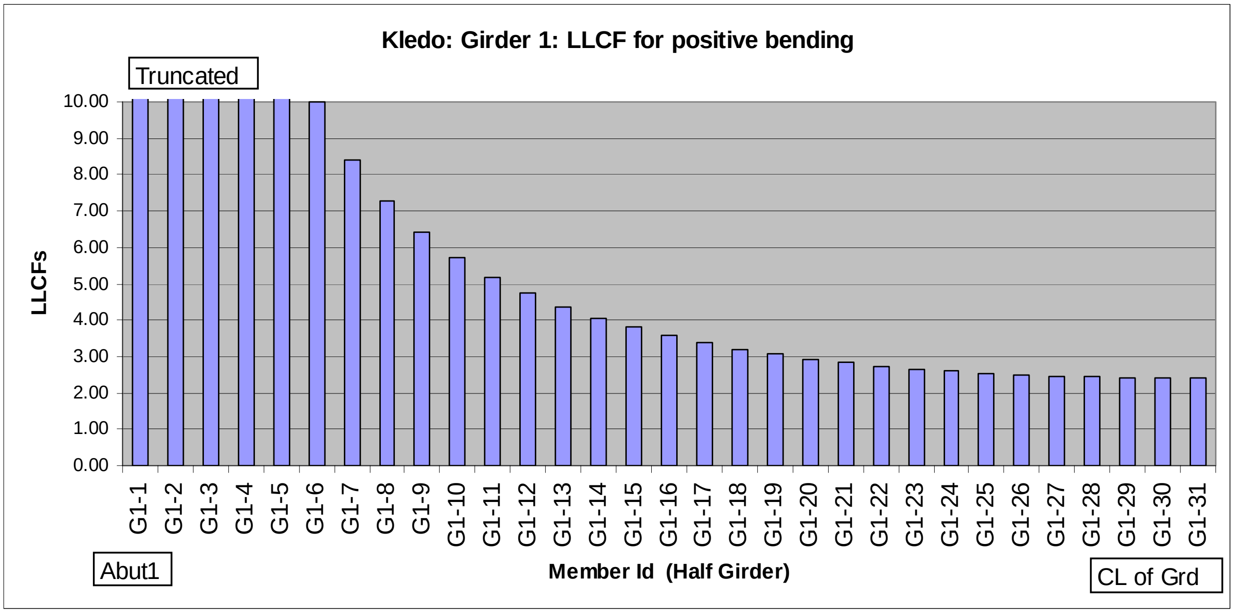

Figure 15: LLCF in Positive Flexure for a Half-Box Girder (Kledo River Bridge).

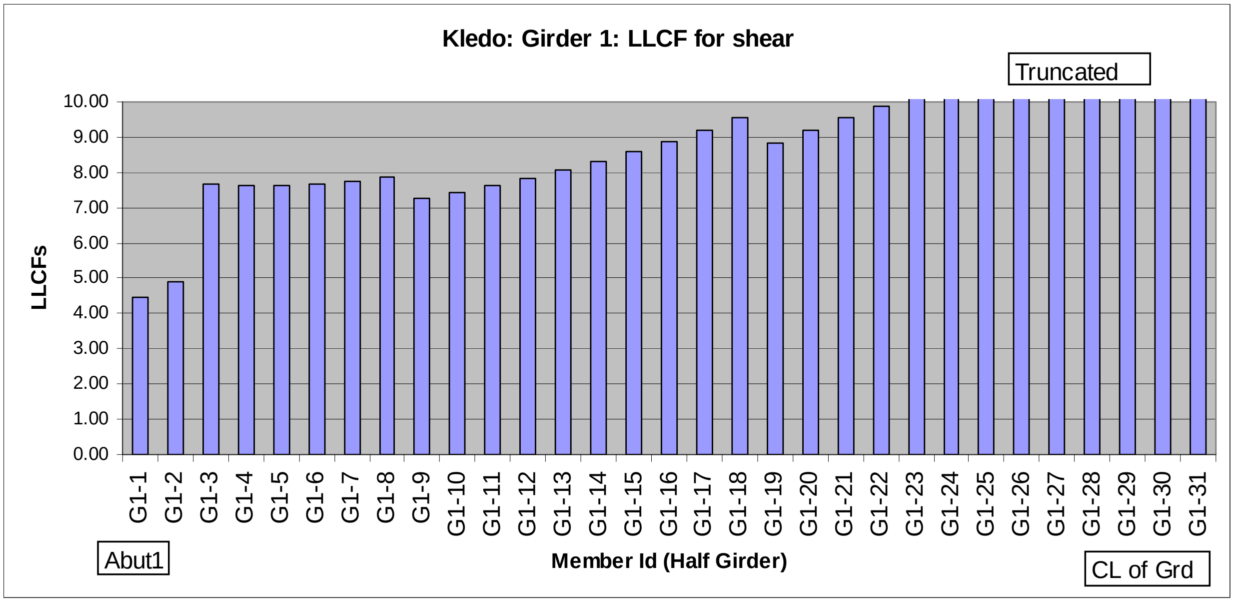

Figure 16: LLCF in Shear for a Half-Box Girder (Kledo River Bridge).

For each type of member in this bridge, Table #2 provides its lowest calculated LLCF value and the location of that LLCF. Also, refer to Figure 14 for the locations of the lowest LLCFs in plan.

Title:	Kledo River Bridge	Completed by:	A. Rafiquzzaman, Ph.D.	Date:	2/11/2010
Subject:	Lowest LLCFs	Checked by:	H. Hawk, M.Sc., P.Eng.	Date:	2/11/2010
				Revision:	3
Member ID	Location		LLCFp Bending(+)	LLCFs Shear	
G1-30 G1-61	At midspan of the girder At the end of the girder near abutment 2		2.42	 4.43	

Table #2: Lowest LLCF for Each Type of Member Within Kledo River Bridge.

5.3 Deck

The following Table #3 shows that the Kledo River Bridge satisfies the requirements for using the empirical deck design method of Clause 8.18 of the CHBDC S6-06. Clause 14.14.1.3.1 states that if a bridge meets the requirements for using the empirical deck design method then the deck shall be deemed to have adequate resistance to meet the loading requirements of an Evaluation Level 1 truck/lane, assuming that the physical condition of the deck is adequate as well of course. Therefore, no further calculations for the deck are required except for checking the deck's cantilever overhangs for wheel load induced bending effects. Kledo River Bridge's deck cantilevers, however, are not long enough to be of any concern.

Since, however, deck thickness deterioration (i.e. delaminations) has been noted for this bridge, punching shear calculations are also included in Table #3. No punching shear issues were determined through the below calculations. Delcan has conservatively assumed here that Kledo River Bridge's concrete deck is fully delaminated to below the level of the centroid of its top mat of reinforcement.

Title:	Kledo River Bridge	Completed by:	P. Phillips, P.Eng.	Date:	2010/02/09	CHBDC
Subject:	Deck Design	Checked by:	H. Hawk, M.Sc., P.Eng.	Date:	2010/02/09	S6-06
(Empirical Deck Design) General						8.18.4.1
Clause a)	Composite slab with parallel supporting beams				OK	
Clause b)	Actual ratio of the spacing of the girders to the thickness of the slab		12.8			
Clause b) (max)	Maximum ratio of the spacing of the girders to the thickness of the slab		18		OK	
Clause c)	Actual spacing of the girders		2.438	[m]		
Clause c) (max)	Maximum spacing of the girders		4	[m]	OK	
Clause d)	Longitudinal negative moment deck rebar for continuous spans				N/A	
(Empirical Deck Design) Cast-in-place deck slabs						8.18.4.2
Transverse rebar ratio (min)		ρ_{min} =	0.003			
Top transverse rebar ratio		ρ_{top} =	0.0090		OK	
Bottom transverse rebar ratio		ρ_{bottom} =	0.0090		OK	
Longitudinal rebar ratio (min)		ρ_{min} =	0.003			
Top longitudinal rebar ratio		ρ_{top} =	0.0060		OK	
Bottom longitudinal rebar ratio		ρ_{bottom} =	0.0044		OK	
(Rigorous Method) General						14.14.1.3.2
Clause a)	Actual spacing of the girders for a slab panel			[m]		
Clause a) (max)	Maximum spacing of the girders for a slab panel		4.5	[m]	N/A	
Clause a)	Slab extends sufficiently beyond the external beams				OK	
Clause b)	Actual ratio of the spacing of the girders to the thickness of the slab		12.8			
Clause b) (max)	Maximum ratio of the spacing of the girders to the thickness of the slab		20		OK	
Clause c)	Actual minimum thickness of the slab		190.5	[mm]		
Clause c) (min)	Minimum thickness of the slab		150	[mm]	OK	
Clause d)	Spacing of cross frames/intermediate diaphragms		9.144	[m]		8.18.5
Clause d) (max)	Maximum spacing of cross frames/intermediate diaphragms		8	[m]	NG	
Clause e)	Edge stiffening				OK	8.18.6
Cantilever factored resistance (negative moment)		Cantilever Bending is Not Applicable				
At centreline of exterior girder						
Live Load Capacity Factor		F =				
At edge of exterior girder flange						
Live Load Capacity Factor		F =				
Deck punching shear resistance						8.9.3.4
Factored resistance of deck slab		R_r =	209.278	[kN]		
Factored truck wheel load		R_f =	182.525	[kN]	OK	

Table #3: Deck Calculation Summary for Kledo River Bridge.

6.0 SEISMIC EVALUATION

Kledo River Bridge is a single span bridge. Seismic performance can, therefore, be assessed by examining the available lengths of the bearing seats on this bridge's abutments. This will ensure that the bridge span will not drop if exposed to seismic loading.

Clause 4.4.5.1 of the CHBDC states that: "Bridges in Seismic Performance Zone 1 need not be analyzed for seismic loads, regardless of their importance and geometry. However, the minimum requirements specified in Clauses 4.4.10.2 and 4.4.10.5 shall apply." Based on Table A3.1.1 of the CHBDC, Kledo River Bridge would be considered to be in Acceleration-Related Seismic Zone 0. Also, as a lifeline structure, Kledo River Bridge would be considered to be in Seismic Performance Zone 2. For single span bridges in Seismic Performance Zone 2, analysis is also not required, but the attachment of the superstructure to the substructure must be able to resist 10% of the weight of the bridge applied as a horizontal load just above the level of the bearings (Clause 4.4.10.2) and the bearing seat length as defined in Clause 4.4.10.5 must be available at each expansion bearing.

For Kledo River Bridge, the bearings consist of 4 base plates at each abutment bolted into the abutments, low rocker plates with shear pintles, and bolting of the upper bearing plate to the superstructure. The superstructure weight is approximately 5600 kN; therefore the system needs to resist a horizontal load of 560 kN. The anchor bolt capacity is approximately 200 kN per bolt and there are 16 bolts. Pintle resistance is approximately 200 kN per pintle and there are 8 pintles. There are 8 high strength bolts connecting the bearing upper plate to the superstructure with a capacity of approximately 130 kN per bolt and there are 32 bolts. All values are well in excess of the required resistance. At the expansion end of the bridge, bearing seat lengths need to be 305.7 mm long (Clause 4.4.10.5); the length provided at each expansion bearing is 550 mm.

7.0 SUMMARY OF RECOMMENDATIONS

There are no major load capacity issues with Kledo River Bridge and no posting is required. No LLCFs for Kledo River Bridge are less than 1 for the ultimate limit state. The existing end transverse bracing between the box girders is not as shown on the original structural drawings (i.e. it is missing). It is important that this bracing be added in the field as soon as possible. Part of the 2009/2011 contract that Delcan has with PWGSC is to prepare sketches for the addition of these secondary members.

APPENDIX A – ANALYSIS FILES (ON CD)

The CD accompanying this report includes the following documents:

1) .pdf file of this load rating report.
2) .txt printout of the Midas Civil model.
3) .mcb Midas Civil model.
4) .pdf printouts of all of the design spreadsheets.

PUBLIC WORKS AND GOVERNMENT SERVICES CANADA

STEAMBOAT CREEK BRIDGE
ALASKA HIGHWAY, km 515.3

STRUCTURAL LIVE LOAD CAPACITY FACTOR EVALUATION

Prepared by:

Delcan

TRANSPORTATION • INFORMATION TECHNOLOGY • WATER

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Hugh Hawk, M.Sc., P.Eng., Technical Director, Western Region
Stan Reimer, P.Eng., Senior Bridge Engineer
Abul Rafiquzzaman, Ph.D., Intermediate Bridge Engineer
Peter Phillips, P.Eng., Intermediate Bridge Engineer

March 2010

TABLE OF CONTENTS

EXECUTIVE SUMMARY

A detailed visual condition assessment for the Steamboat Creek Bridge, km 515.3 along the Alaska Highway, was performed in September 2009. This condition assessment and Chapter 14, “Evaluation”, of the “Canadian Highway Bridge Design Code (CHBDC)”, CAN/CSA-S6-06, were used to evaluate the load carrying capacity of this existing bridge.

Steamboat Creek Bridge was evaluated for CL1-W truck/lane live loading (Evaluation Level 1) in accordance with the CHBDC. Load factors were determined from Clause 14.13 based on target reliability indices (beta-factors) as determined from Clause 14.12 of the CHBDC. Resistance adjustment factors as determined from Clause 14.14.2 of the CHBDC were also used. These target reliability indices and load factors were reviewed by Public Works and Government Services Canada (PWGSC).

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1.0 INTRODUCTION

To determine if the bridge structure requires strengthening or posting, a bridge inspection was first carried out. The bridge's original structural drawings were reviewed in order to familiarize oneself with the structure, prior to an assessment in the field of its member conditions and overall stability. A load rating was then undertaken to establish which members are under-strength, if any. Section 14 of the Canadian Highway Bridge Design Code (CHBDC) outlines this process: target reliability indices (beta-factors) for each member of the bridge were determined and then dead and live load factors for each member of the bridge were chosen.

A model of the bridge, based on the requirements of Section 5 of the CHBDC, was created. Largest combined dead and live factored loads were determined for each member. The resistance of each member was compared to its respective applied force, taking into consideration the deterioration of the members as well. This comparison was used to determine which members were under-strength, if any. If members are determined by analysis to be under-strength currently, it is due to changes in design code requirements, deteriorating member conditions in the field, lack of conservatism in the original designs, or poor original construction practices. The load rating in combination with the field inspection program was used to classify the capacity problem (i.e. problems in flexure, shear, compression, tension, torsion, serviceability, fatigue, maintainability, durability, etc.).

The following original structural drawings were provided by PWGSC to assist in the structural evaluation of this bridge. The drawings provided a reference for dimensions, methods of original construction, substructure details (i.e. details not visible in the field), and material strengths, etc. Relevant assumptions, however, also needed to be made and other pertinent dimensions, data, etc. were obtained in the field during the visual bridge inspections.

Drawings Obtained:
Steamboat Creek Bridge: Drawings 1 to 12, dated 1978.

2.0 BRIDGE DESCRIPTION

An elevation photo of the Steamboat Creek Bridge is shown in Figure 1, below.

Figure 1: Steamboat Creek Bridge Photo.

Description of Steamboat Creek Bridge:

a) Single span weathering steel I-girders.
b) Reinforced concrete deck.
c) Reinforced concrete abutments.
d) Steel pipe pile foundations under abutments.
e) Gabion slope protection.
f) Steel railing on reinforced concrete curb traffic barriers.

Relevant dimensions of the bridge's elevation and cross-section are shown in Figures 2 and 3, respectively.

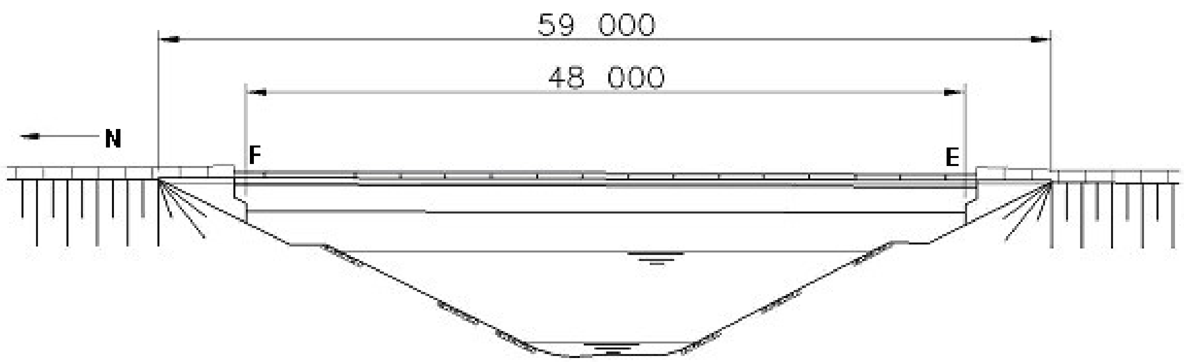

Figure 2: Elevation View of Steamboat Creek Bridge.

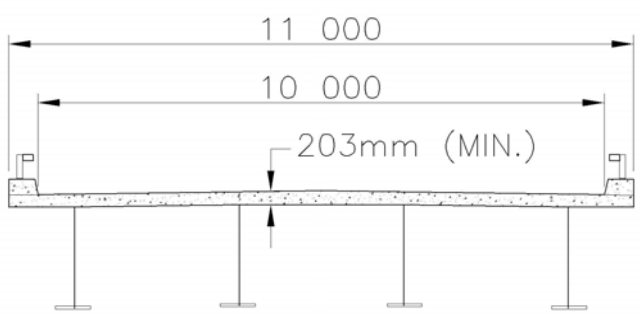

Figure 3: Cross-Section View of Steamboat Creek Bridge.

3.0 CONDITION CONSIDERATION

The bridge is generally in good condition. Based on the site evaluation carried out in September 2009 of this structure, no significant loss of section or other bridge issues that could contribute negatively to the overall structural integrity of the bridge were found. For a detailed site condition rating of this bridge please refer to Delcan's 2009 bridge inspection report.

See Figures 4 to 10, below, for condition photos taken during Delcan's recent 2009 inspections of the Alaska Highway structures.

Figure 4: Deck.

Figure 5: Exterior Girder.

Figure 6: Abutment and Wingwall.

Figure 7: Exterior Girder Bearing.

Figure 8: Girder Lines.

Figure 9: Intermediate Bracing.

End Diagonal Bracing Orientation is Shown Differently on Original Structural Drawings.

Figure 10: End Bracing.

4.0 EVALUATION PARAMETERS AND METHODOLOGY

4.1 Evaluation Procedures

Steamboat Creek Bridge was evaluated for CL1-W truck/lane live loading (Evaluation Level 1) in accordance with the CHBDC. Load factors were determined from Clause 14.13 based on target reliability indices (beta-factors) as determined from Clause 14.12. Resistance adjustment factors as determined from Clause 14.14.2 of the CHBDC were also used. These target reliability indices and load factors were reviewed by PWGSC.

Live load capacity factors for each superstructure member, as per Clause 14.15, were calculated (if and when applicable) for ultimate limit states in bending, shear, and axial. Punching shear was checked for this bridge’s deck.

Delcan modeled the Steamboat Creek Bridge using “Midas Civil” (Midas) software utilizing a grillage-type model. The structural model was developed using beam elements for the superstructure members and deck and pin or roller supports, as applicable, as substructure elements. As substructure elements were in good condition with respect to structural integrity and member stability based on the September 2009 visual bridge inspections, substructure elements were not considered further in this live load capacity factor evaluation of the

Steamboat Creek Bridge. It was assumed that the substructure elements provide full support to the superstructure members.

Also, member-to-member connections (all joints) were assumed to be fully effective (i.e. in providing full capacity to transfer loads between the connected elements). All connections of secondary members to primary members were assumed to be pinned-pinned connections.

See Figure 11, below, for a rendered view of the Midas model.

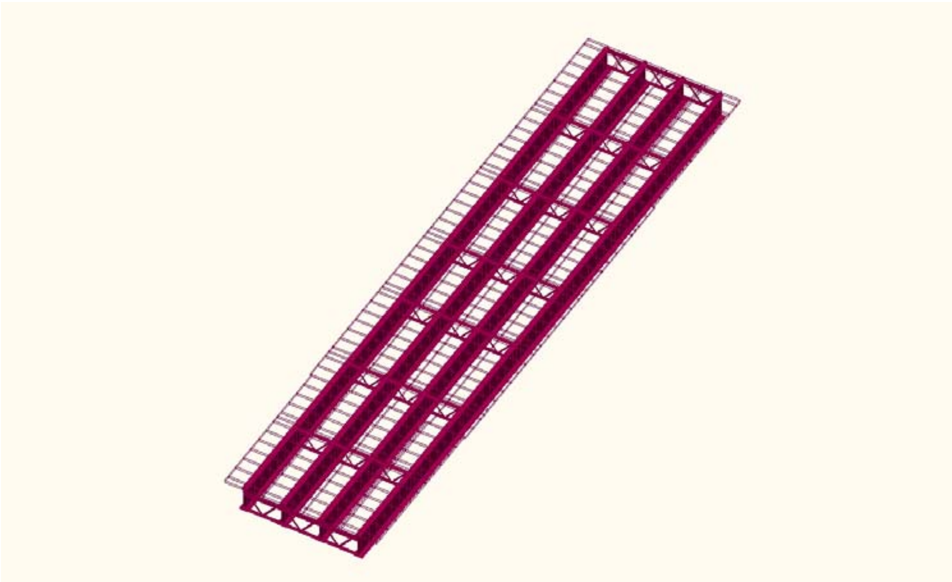

Figure 11: Steamboat Creek Bridge Rendered Midas Civil Grillage-Type Model.

4.2 Reliability Indices and Load Factors

The following Table #1, below, provides:

a) System behaviour, element behaviour, and inspection level classifications.
b) Reliability indices as determined for “Normal Traffic”.
c) An adjustment to the reliability index of 0.25 based on Clause 14.12.5 and recognizing this bridge as an important structure.
d) Dead load factors based on Clause 14.13.2.1.
e) Live load factors for normal traffic based on Clause 14.13.3.1.
f) A multilane factor for normal traffic based on Clause 14.9.4.2.
g) Dynamic load allowances for normal traffic based on Clause 14.9.1.7.

It was assumed that all members would be subjected to ‘gradual failure with warning of probable failure’ regardless of material or load carrying direction / capacity, with respect to ‘Element Behaviour’ – Clause 14.12.3 of the CHBDC.

Project: Alaska Highway Load Rating	Design: P. Phillips, PEng	Date: 2010/01/27	
Subject: Steamboat: Reliability Index & Load Factors	Check: H. Hawk, MSc, PEng	Date: 2010/01/27	
	Element		
	Girders	Cross-Bracing	Deck
Category			
System Behaviour	S2	S3	S3
Element Behaviour	E3	E3	E3
Inspection Level	I2	I2	I2
Live Load Lateral Dist.	Sophisticated Analysis	Sophisticated Analysis	Simplified Analysis
Reliability Index, β: CL1-W	3.00	2.75	2.75
Important Structure: β Increased by	0.25		
CL1-W	3.25	3.00	3.00
Factors			
DL	D1	D1	D2
α_D: CL1-W	1.08	1.07	1.14
DL	D2	D2	D1
α_D: CL1-W	1.16	1.14	1.07
$\alpha_{L\,(CL1\text{-}W)}$: CL1-W	1.56	1.49	1.49
Multi-Lane Factor	0.9		
1+DLA	1.25	1.25	1.40

Table #1: Reliability Indices and Load Factors.

4.3 Resistance Adjustment Factors

Resistance adjustment factors, as follows, were determined from Clause 14.14.2 of the CHBDC. The factored resistance of an individual structural component under consideration was multiplied by the appropriate resistance adjustment factor.

Structural Steel
- a) Compression or tension on gross section: U = 1.01.
- b) Shear: U = 1.02.

Composite – Slab on Steel Girder:
- a) Bending: U = 0.96.
- b) Shear connectors: U = 0.94.

Reinforced Concrete Deck:
- a) Bending: U = 0.95.

4.4 Permanent Loads

The dead loads in the model include:
- a) The full self-weight of the primary superstructure elements and secondary lateral bracing elements / diaphragms.

b) The weights of the steel plate girders were adjusted upwards by 12% to account for splice plate weights, steel connections, gusset plates, longitudinal and transverse web stiffeners, bearing stiffeners, etc. as per Clause C14.8.2.1 of the CHBDC Commentary.
c) A 200mm concrete deck is still applicable as shown on original drawings as provided by PWGSC.
d) No deck overlay included.
e) Bridge barriers in the field (curb and railing type barriers) are the same as the barriers shown on the drawings provided to Delcan.

4.5 Normal Traffic Live Loads

Steamboat Creek Bridge was evaluated for Evaluation Level 1 CL1-W truck/lane live loading. Two lanes exist in the field (one Northbound lane and one Southbound lane) and therefore two traffic lanes were modeled, as specified in Clause 14.9.4.1. Appropriate multiple-lane load factors and dynamic load factors were applied to the truck and/or lane loading, when applicable.

4.6 Material Strengths

The material properties, as provided below, used in the resistance calculation processes were obtained from the structural drawings provided by PWGSC. Steamboat Creek Bridge was originally built in 1978. Dates of any subsequent modifications made to this bridge are unknown to Delcan.

The following values were used in the evaluation of the Steamboat Creek Bridge:

a) Concrete deck and barrier compressive strength: 30 MPa, Drawing Number 1 of 12.
b) Reinforcing deck and curb steel yield strength: 400 MPa, Drawing Number 1 of 12.
c) Superstructure structural steel yield strength: Grade 50A (350 MPa), Drawing Number 1 of 12.
d) Superstructure structural steel ultimate strength: 480 MPa, Drawing Number 1 of 12.

5.0 RESULTS

5.1 Key-Plans/Elevations

The following key-plan/elevation diagrams indicate the naming conventions of the individual structural members within this bridge that were adopted for this load rating. All members referenced in this 'Results' section will therefore be referred to by their key-plan/elevation names.

See Figures 12 to 14 for key-plan/elevation drawings for the Steamboat Creek Bridge:

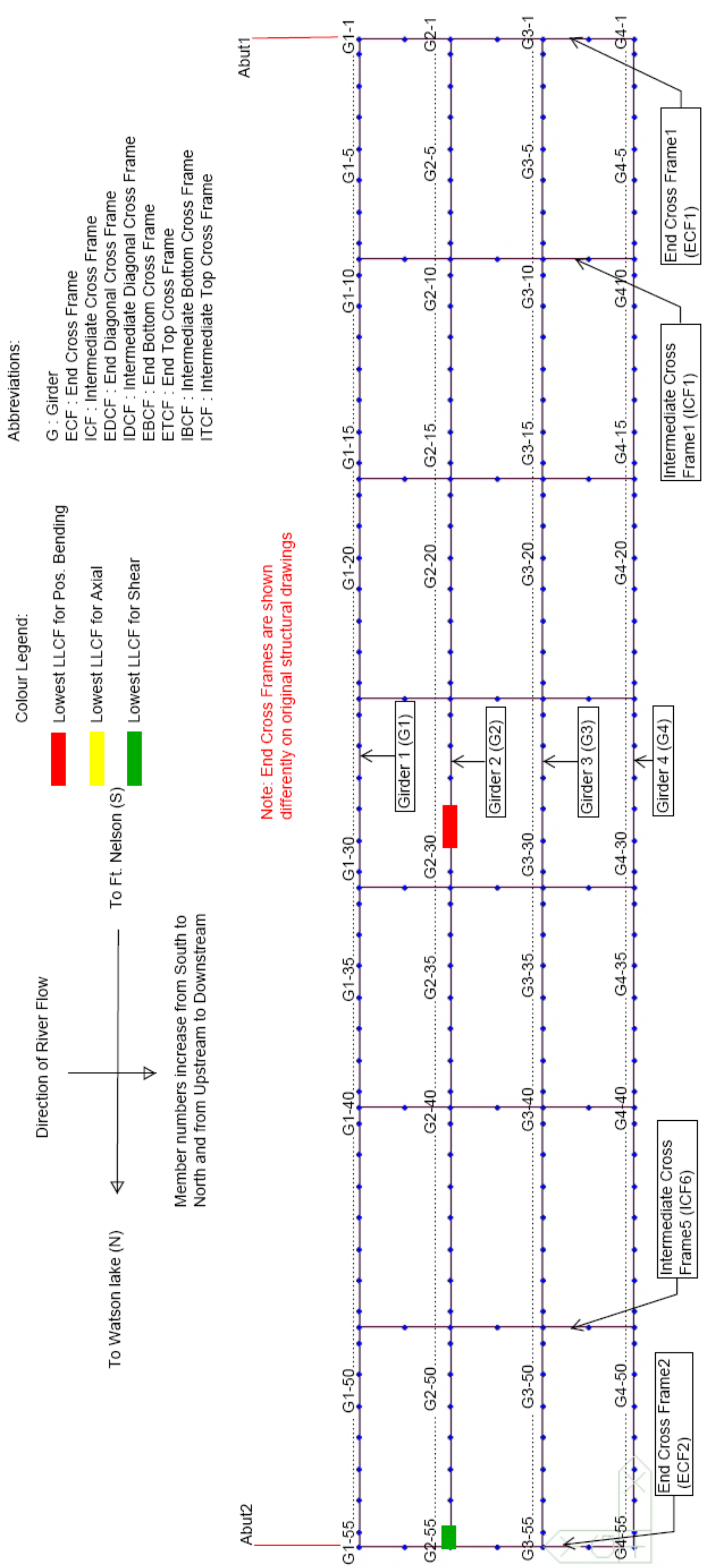

Figure 12: Steamboat Creek Bridge Plan View Naming Conventions.

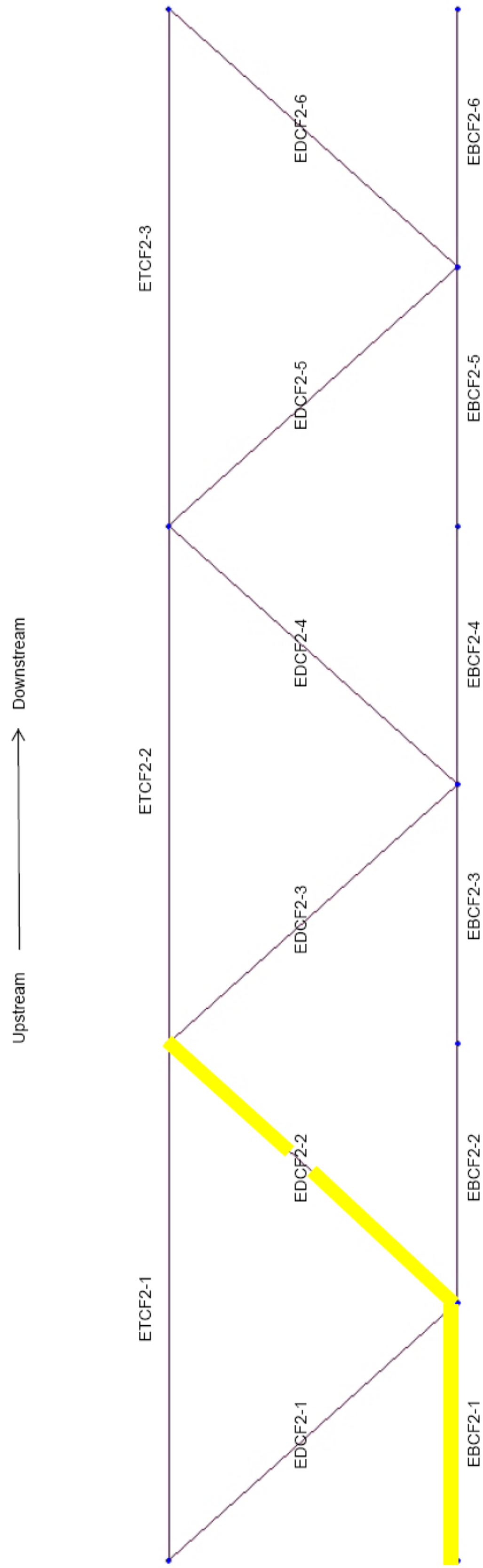

Figure 13: Steamboat Creek Bridge End Bracing View Naming Conventions.

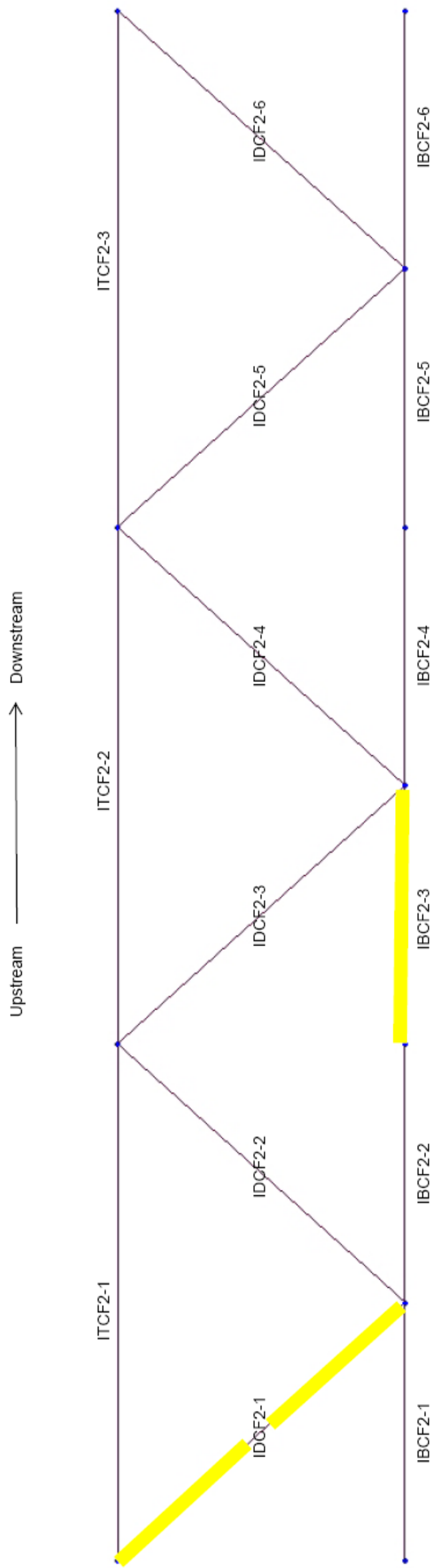

Figure 14: Steamboat Creek Bridge Intermediate Bracing View Naming Conventions.

5.2 Live Load Capacity Factors

The following Figures 15 to 20 show the live load capacity factors (LLCFs) that have been calculated along Steamboat Creek Bridge's members based on the requirements of Section 14 of the CHBDC S6-06. Specifically, LLCFs are calculated based on Clause 14.15.2, 'Ultimate Limit States'. LLCFs greater than 1 are deemed adequate for the prescribed live loading and LLCFs less than 1 generally require posting.

The location along a member which is considered to govern its design is the position where the member is most highly loaded relative to its resistance.

Due to overall bridge symmetry (i.e. no skew effects), symmetrical transient loading present, and simply supported bearing conditions, symmetrical force diagrams are produced within this bridge. In such cases, half-spans of the members need only to be shown.

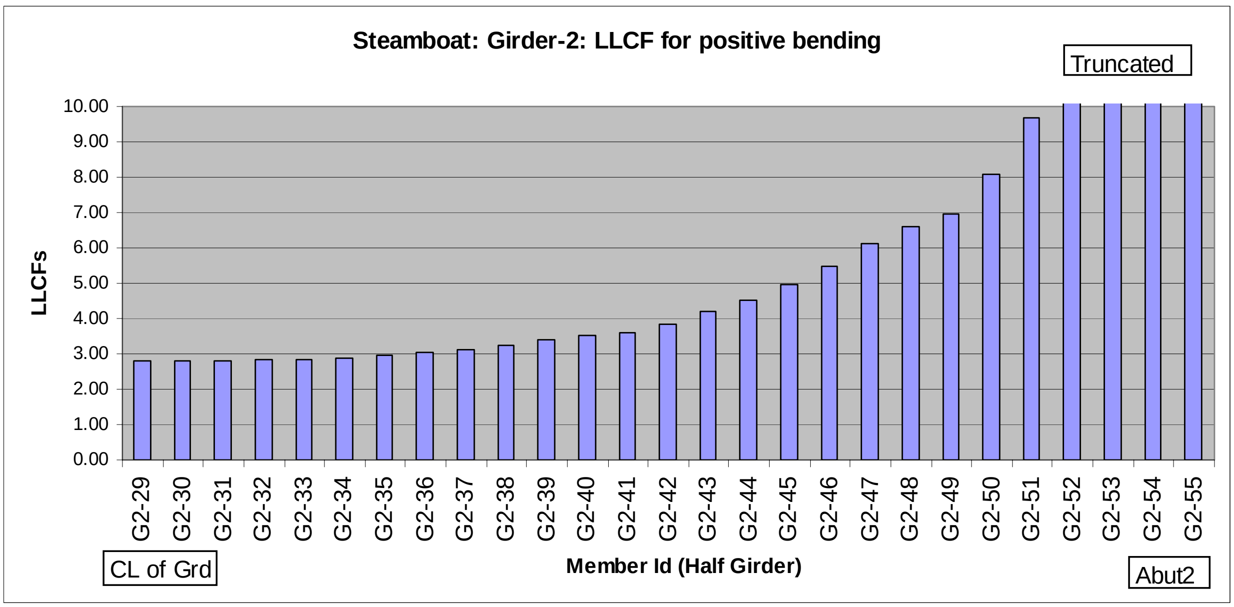

Figure 15: LLCF in Positive Flexure for a Half-Girder (Steamboat Creek Bridge).

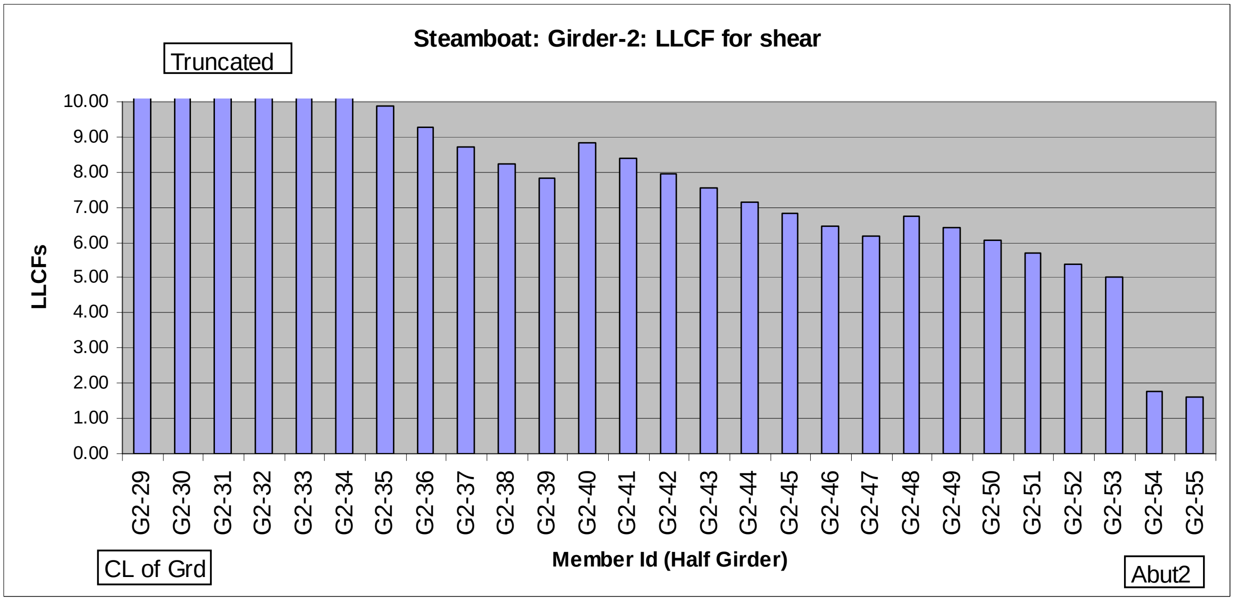

Figure 16: LLCF in Shear for a Half-Girder (Steamboat Creek Bridge).

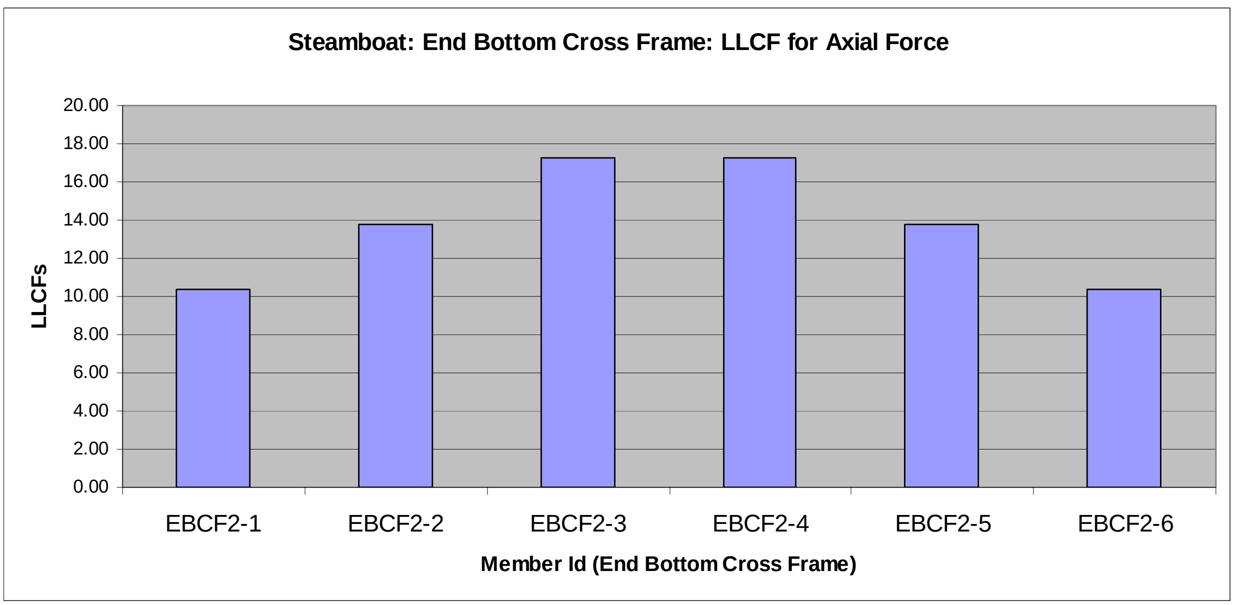

Figure 17: LLCF for Axial Force in End Bottom Horizontal Transverse Bracing (Steamboat Creek Bridge).

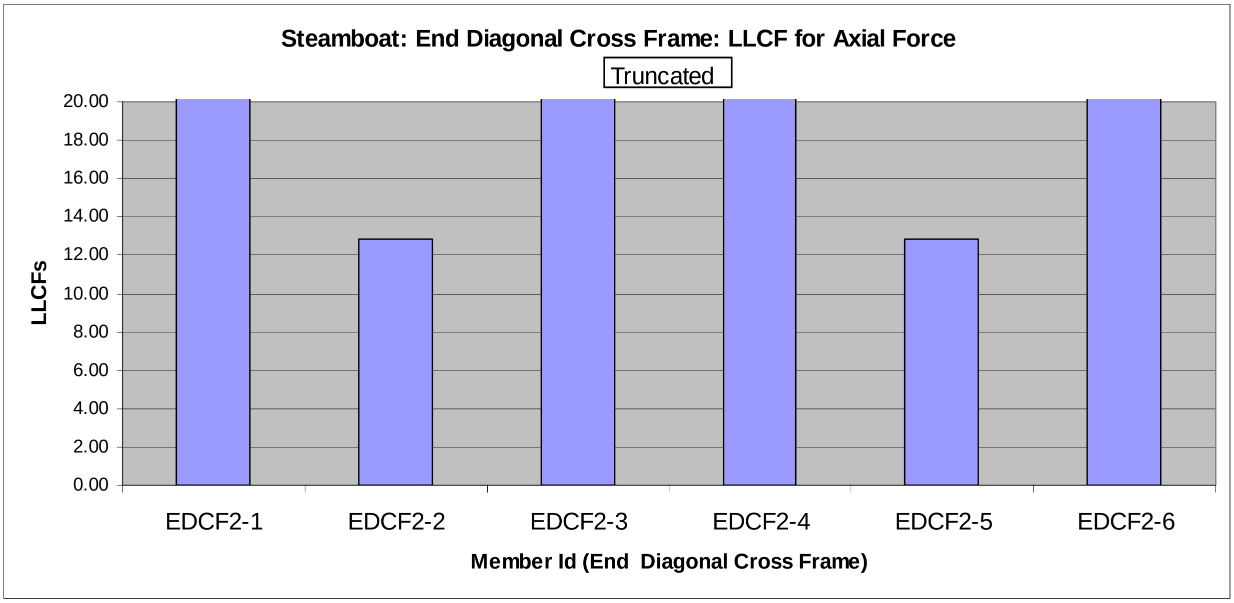

Figure 18: LLCF for Axial Force in End Diagonal Cross-Bracing (Steamboat Creek Bridge).

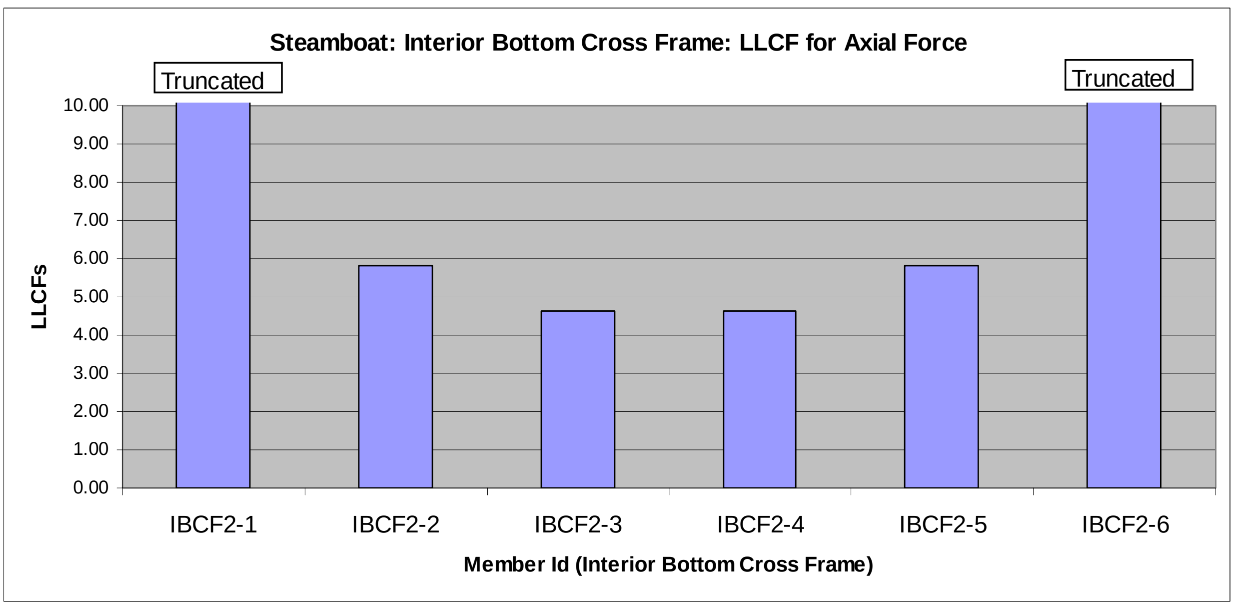

Figure 19: LLCF for Axial Force in Intermediate Bottom Horizontal Transverse Bracing (Steamboat Creek Bridge).

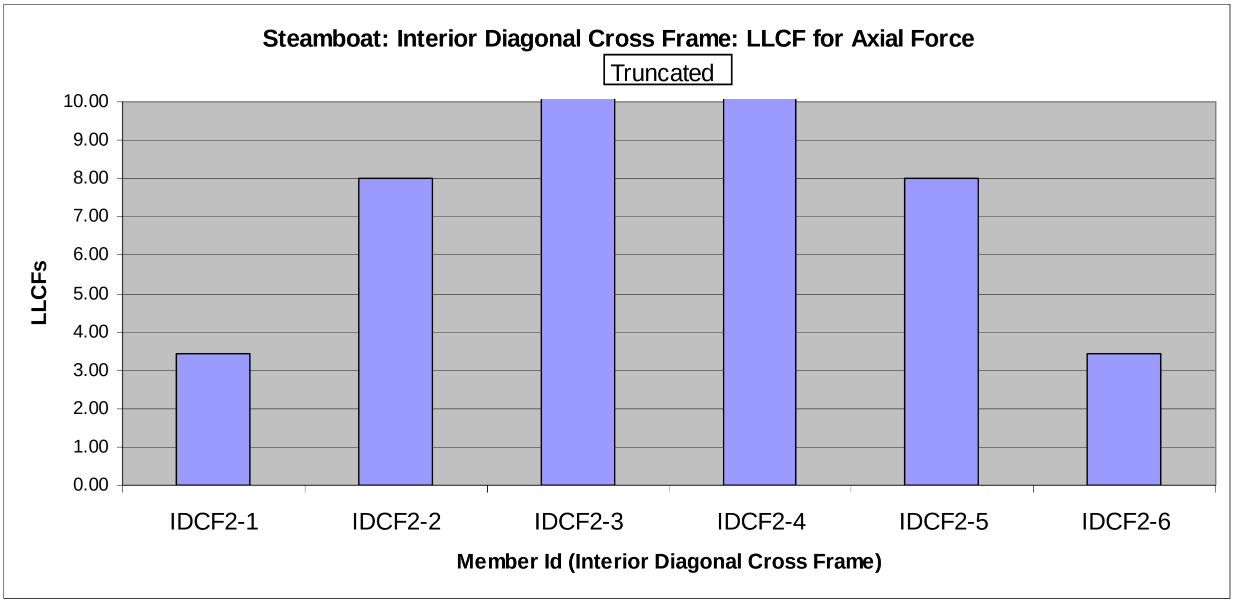

Figure 20: LLCF for Axial Force in Intermediate Diagonal Cross-Bracing (Steamboat Creek Bridge).

For each type of member in this bridge, Table #2 provides its lowest calculated LLCF value and the location of that LLCF. Also, refer to Figures 12 to 14 for the locations of the lowest LLCFs in plan/elevation.

Title:	Steamboat Creek Bridge	Completed by:	A. Rafiquzzaman, Ph.D.	Date:	2/11/2010
Subject:	Lowest LLCFs	Checked by:	H. Hawk, M.Sc., P.Eng.	Date:	2/11/2010
				Revision:	1
Member ID	Location	LLCFa Axial	LLCFp Bending(+)	LLCFs Shear	
G2-29	Interior girder midspan (ICF3-ICF4)		2.79		
G2-55	Interior girder at abutment (ICF6-Abut2)			1.61	
IBCF2-3	Interior bottom cross frame (ICF2)	4.64			
IDCF2-1	Interior diagonal cross frame (ICF2)	3.42			
EBCF2-1	End bottom cross frame (ECF2)	10.34			
EDCF2-2	End diagonal cross frame (ECF2)	12.85			

Table #2: Lowest LLCF for Each Type of Member Within Steamboat Creek Bridge.

5.3 Deck

The following Table #3 shows that the Steamboat Creek Bridge does not satisfy the requirements for using the empirical deck design method of Clause 8.18. Clause 14.14.1.3.1 states that if a bridge meets the requirements for using the empirical deck design method then the deck shall be deemed to have adequate resistance to meet the loading requirements of an Evaluation Level 1 truck/lane, assuming that the physical condition of the deck is adequate as well of course. Therefore, no further calculations for the deck are required except for checking the deck's cantilever overhangs for wheel load induced bending effects. Clause 14.14.1.3.1 then states that if a bridge's deck does not satisfy the requirements of Clause 8.18, then Clauses 14.14.1.3.2 a) to e) and 14.14.1.3.3 must be satisfied. Steamboat Creek Bridge's deck does satisfy these two latter requirements and therefore is not of any concern.

Since deck thickness deterioration (i.e. delaminations) has been noted for this bridge, punching shear calculations are also included in Table #3. No punching shear issues were determined through the below calculations. Delcan has conservatively assumed here that Steamboat Creek Bridge's concrete deck is fully delaminated to below the level of the centroid of its top mat of reinforcement.

Title:	Steamboat Creek Bridge	Completed by:	P. Phillips, P.Eng.	Date: 2010/02/10			CHBDC
Subject:	Deck Design	Checked by:	H. Hawk, M.Sc., P.Eng.	Date: 2010/02/10			S6-06
	Slab thickness as per drawing		t_{slab} =	167.5	[mm]		
	Slab thickness (min)		t_{slab} =	175	[mm]	NG	8.18.2
(Empirical Deck Design) General							8.18.4.1
	Clause a)	Composite slab with parallel supporting beams				OK	
	Clause b)	Actual ratio of the spacing of the girders to the thickness of the slab		17.313			
	Clause b) (max)	Maximum ratio of the spacing of the girders to the thickness of the slab		18		OK	
	Clause c)	Actual spacing of the girders		2.9	[m]		
	Clause c) (max)	Maximum spacing of the girders		4	[m]	OK	
	Clause d)	Longitudinal negative moment deck rebar for continuous spans				N/A	
(Empirical Deck Design) Cast-in-place deck slabs							8.18.4.2
	Clause b)	Transverse rebar is closest to the edges of the slab				NG	
	Transverse rebar ratio (min)		ρ_{min} =	0.003			
	Top transverse rebar ratio		ρ_{top} =	0.009		OK	
	Bottom transverse rebar ratio		ρ_{bottom} =	0.008		OK	
	Longitudinal rebar ratio (min)		ρ_{min} =	0.003			
	Top longitudinal rebar ratio		ρ_{top} =	0.006		OK	
	Bottom longitudinal rebar ratio		ρ_{bottom} =	0.004		OK	
(Rigorous Method) General							14.14.1.3.2
	Clause a)	Actual spacing of the girders for a slab panel			[m]		
	Clause a) (max)	Maximum spacing of the girders for a slab panel		4.5	[m]	N/A	
	Clause a)	Slab extends sufficiently beyond the external beams				OK	
	Clause b)	Actual ratio of the spacing of the girders to the thickness of the slab		17.313			
	Clause b) (max)	Maximum ratio of the spacing of the girders to the thickness of the slab		20		OK	
	Clause c)	Actual minimum thickness of the slab		167.5	[mm]		
	Clause c) (min)	Minimum thickness of the slab		150	[mm]	OK	
	Clause d)	Spacing of cross frames/intermediate diaphragms		7	[m]		8.18.5
	Clause d) (max)	Maximum spacing of cross frames/intermediate diaphragms		8	[m]	OK	
	Clause e)	Edge stiffening				OK	8.18.6
Cantilever factored resistance (negative moment)							
	At centreline of exterior girder						
	Live Load Capacity Factor		F =	1.115		OK	
	At edge of exterior girder flange						
	Live Load Capacity Factor		F =	1.154		OK	
Deck punching shear resistance							8.9.3.4
	Factored resistance of deck slab		R_r =	190.476	[kN]		
	Factored truck wheel load		R_f =	182.525	[kN]	OK	

Table #3: Deck Calculation Summary for Steamboat Creek Bridge.

6.0 SEISMIC EVALUATION

Steamboat Creek Bridge is a single span bridge. Seismic performance can, therefore, be assessed by examining the available lengths of the bearing seats on this bridge's abutments. This will ensure that the bridge span will not drop if exposed to seismic loading.

Clause 4.4.5.1 of the CHBDC states that: "Bridges in Seismic Performance Zone 1 need not be analyzed for seismic loads, regardless of their importance and geometry. However, the minimum requirements specified in Clauses 4.4.10.2 and 4.4.10.5 shall apply." Based on Table

A3.1.1 of the CHBDC, Steamboat Creek Bridge would be considered to be in Acceleration-Related Seismic Zone 0. Also, as a lifeline structure, Steamboat Creek Bridge would be considered to be in Seismic Performance Zone 2. For single span bridges in Seismic Performance Zone 2, analysis is also not required, but the attachment of the superstructure to the substructure must be able to resist 10% of the weight of the bridge applied as a horizontal load just above the level of the bearings (Clause 4.4.10.2) and the bearing seat length as defined in Clause 4.4.10.5 must be available at each expansion bearing.

For Steamboat Creek Bridge, the bearings consist of base plates bolted into the abutments, low rocker plates with shear pintles, and bolting of the upper bearing plate to the superstructure. The superstructure weight is approximately 4100 kN; therefore the system needs to resist a horizontal load of 410 kN. The anchor bolt capacity is approximately 200 kN per bolt and there are 16 bolts. Pintle resistance is approximately 200 kN per pintle and there are 8 pintles. Upper bolt resistance is approximately 520 kN per bearing and there are 4 bearings. All values are well in excess of the required resistance. At the expansion end of the bridge, bearing seat lengths need to be 280.0 mm long (Clause 4.4.10.5); the length provided at each expansion bearing is 425 mm.

7.0 SUMMARY OF RECOMMENDATIONS

There are no major load capacity issues with Steamboat Creek Bridge and no posting is required. No LLCFs for Steamboat Creek Bridge are less than 1 for the ultimate limit state.

APPENDIX A – ANALYSIS FILES (ON CD)

The CD accompanying this report includes the following documents:

1) .pdf file of this load rating report.
2) .txt printout of the Midas Civil model.
3) .mcb Midas Civil model.
4) .pdf printouts of all of the design spreadsheets.

PUBLIC WORKS AND GOVERNMENT SERVICES CANADA

TETSA RIVER BRIDGE #1
ALASKA HIGHWAY, km 584.6

STRUCTURAL LIVE LOAD CAPACITY FACTOR EVALUATION

Prepared by:

Delcan
TRANSPORTATION • INFORMATION TECHNOLOGY • WATER

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Peter Phillips, P.Eng., Intermediate Bridge Engineer

March 2010

TABLE OF CONTENTS

EXECUTIVE SUMMARY

A detailed visual condition assessment for the Tetsa River Bridge #1, km 584.6 along the Alaska Highway, was performed in September 2009. This condition assessment and Chapter 14, “Evaluation”, of the “Canadian Highway Bridge Design Code (CHBDC)”, CAN/CSA-S6-06, were used to evaluate the load carrying capacity of this existing bridge.

Tetsa River Bridge #1 was evaluated for CL1-W truck/lane live loading (Evaluation Level 1) in accordance with the CHBDC. Load factors were determined from Clause 14.13 based on target reliability indices (beta-factors) as determined from Clause 14.12 of the CHBDC. Resistance adjustment factors as determined from Clause 14.14.2 of the CHBDC were also used. These target reliability indices and load factors were reviewed by Public Works and Government Services Canada (PWGSC).

Live load capacity factors (LLCFs) for each superstructure member, as per Clause 14.15, were calculated (if and when applicable) for ultimate limit states in bending, shear, and axial.

All members of the existing bridge are capable of carrying their combination of factored dead and imposed truck/lane live loads. No bridge strengthening or posting is currently required.

1.0 INTRODUCTION

To determine if the bridge structure requires strengthening or posting, a bridge inspection was first carried out. The bridge's original structural drawings were reviewed in order to familiarize oneself with the structure, prior to an assessment in the field of its member conditions and overall stability. A load rating was then undertaken to establish which members are under-strength, if any. Section 14 of the Canadian Highway Bridge Design Code (CHBDC) outlines this process: target reliability indices (beta-factors) for each member of the bridge were determined and then dead and live load factors for each member of the bridge were chosen.

A model of the bridge, based on the requirements of Section 5 of the CHBDC, was created. Largest combined dead and live factored loads were determined for each member. The resistance of each member was compared to its respective applied force, taking into consideration the deterioration of the members as well. This comparison was used to determine which members were under-strength, if any. If members are determined by analysis to be under-strength currently, it is due to changes in design code requirements, deteriorating member conditions in the field, lack of conservatism in the original designs, or poor original construction practices. The load rating in combination with the field inspection program was used to classify the capacity problem (i.e. problems in flexure, shear, compression, tension, torsion, serviceability, fatigue, maintainability, durability, etc.).

The following original structural drawings were provided by PWGSC to assist in the structural evaluation of this bridge. The drawings provided a reference for dimensions, methods of original construction, substructure details (i.e. details not visible in the field), and material strengths, etc. However, the drawings, in general, are not complete. All original drawings required and other drawings from some of the rehabilitation works in the past were not included in the drawings package provided by PWGSC. Therefore, relevant assumptions needed to be made and other pertinent dimensions, data, etc. were obtained in the field during the visual bridge inspections.

Drawings Obtained:
Tetsa River Bridge #1:

a) Sheet 1 to 6, dated 1943.
b) Drawing 3400-34, dated 1954, Not Current.
c) Drawing 3400-46 to 49, dated 1955, Not Current.
d) Drawing 2119-5, dated 1955, Not Current.
e) Drawings 1 to 5, dated 1976, Portal and sway frame bracing rehabilitation drawings.
f) Drawings 1 to 4, dated 1989, Deck replacement and truss strengthening drawings.

Missing Drawings:
Tetsa River Bridge #1: All original structural steel drawings.

2.0 BRIDGE DESCRIPTION

An elevation photo of the Tetsa River Bridge #1 is shown in Figure 1, below.

Figure 1: Tetsa River Bridge #1 Photo.

Description of Tetsa River Bridge #1:

a) Simply supported painted steel I-girder jack-spans.
b) Simply supported painted steel through truss main spans.
c) Steel grating deck.
d) Reinforced concrete piers and abutments.
e) Reinforced concrete spread footings under piers and abutments.
f) Steel railing traffic barriers.

Relevant dimensions of the bridge's elevation and cross-section are shown in Figures 2 and 3, respectively.

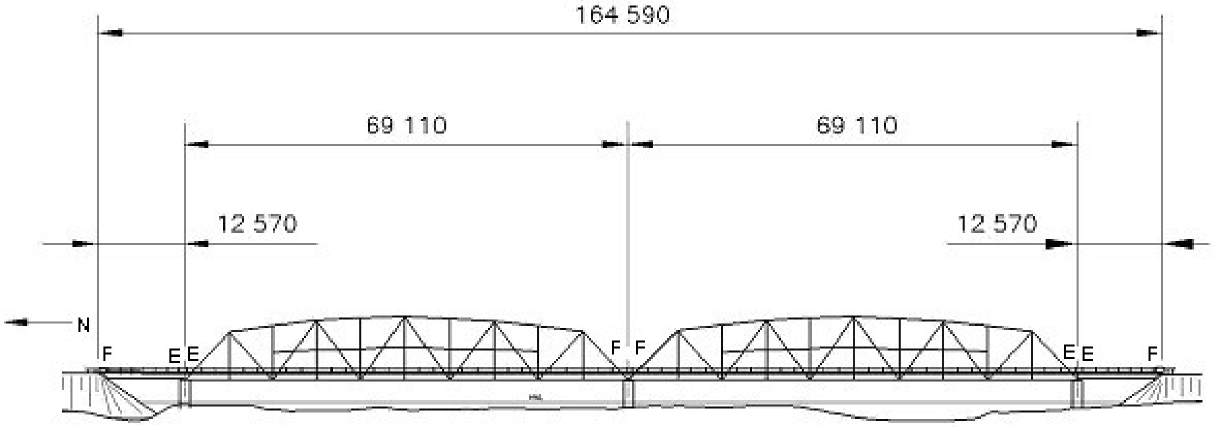

Figure 2: Elevation View of Tetsa River Bridge #1.

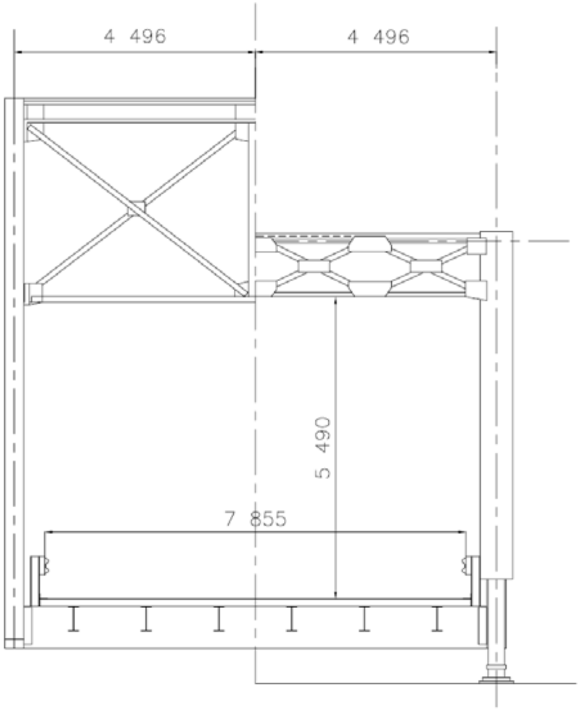

Figure 3: Cross-Section View of Tetsa River Bridge #1.

3.0 CONDITION CONSIDERATION

The bridge is generally in good condition. Based on the site evaluation carried out in September 2009 of this structure, no significant loss of section or other bridge issues that could contribute negatively to the overall structural integrity of the bridge were found. For a detailed site condition rating of this bridge please refer to Delcan's 2009 bridge inspection report.

Tetsa River Bridge #1 experienced some truck collision damage to all of its sway frames and portals (10 lateral bracing locations in total) sometime between the 2005 and 2007 inspection sessions of this bridge (see below for typical photos of the damage). Part of the 2009/2011 contract that Delcan has with PWGSC is to prepare repair sketches for these damaged secondary members. Therefore, Delcan has modeled this bridge in its future repaired condition with the sway frames and portals fully rehabilitated.

See Figures 4 to 24, below, for condition photos taken during Delcan's recent 2009 inspections of the Alaska Highway structures.

Figure 4: Deck.

Figure 5: Truss.

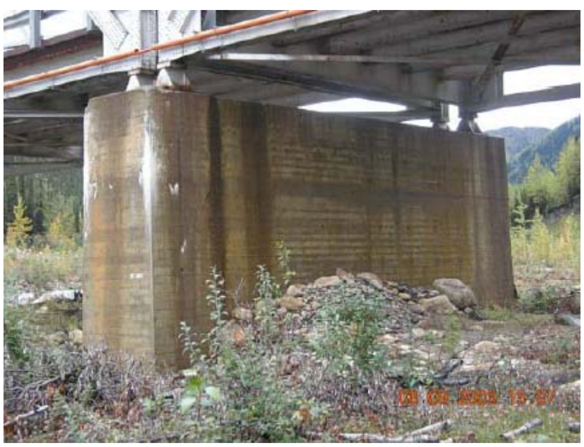

Figure 6: Truss-Span to Truss-Span Pier.

Figure 7: Truss Bearing.

Figure 8: Stringer Lines.

Figure 9: Stringer to Floor Beam Connections.

Figure 10: Floor Beam to Vertical Truss Member Connection and Diagonal Bracing to Floor Beam Connection.

Figure 11: Diagonal Truss Member to Vertical Truss Member and Bottom Chord Truss Member Connection.

Figure 12: Bottom Chord and Vertical Truss Members.

Figure 13: Top Chord Truss Member.

Figure 14: Previously Strengthened Diagonal Compression Truss Member.

Figure 15: Vertical and Diagonal Truss Member Horizontal Bracing.

Damaged Members Caused by Vehicle Collision.

Figure 16: Typical End Portal Lateral Bracing.

Damaged Members Caused by Vehicle Collision.

Figure 17: Typical Intermediate Sway Frame Lateral Bracing.


Figure 18: Example of Typical Vehicle Collision Damage to Truss Lateral Bracing.

Unused Steel Brackets Still Attached to Exterior Sides of Exterior Jackspan Girders.

Figure 19: Jackspan.


Figure 20: Jackspan Abutment.

Unused Steel Brackets Still Attached to Exterior Sides of Exterior Jackspan Girders.

Figure 21: Exterior Jackspan Girder Bearing.

Figure 22: Jackspan to Truss-Span Pier.

Figure 23: Jackspan Girder Lines and Intermediate Bracing.

Figure 24: Jackspan End Bracing.

4.0 EVALUATION PARAMETERS AND METHODOLOGY

4.1 Evaluation Procedures

Tetsa River Bridge #1 was evaluated for CL1-W truck/lane live loading (Evaluation Level 1) in accordance with the CHBDC. Load factors were determined from Clause 14.13 based on target reliability indices (beta-factors) as determined from Clause 14.12. Resistance adjustment factors as determined from Clause 14.14.2 of the CHBDC were also used. These target reliability indices and load factors were reviewed by PWGSC.

Live load capacity factors for each superstructure member, as per Clause 14.15, were calculated (if and when applicable) for ultimate limit states in bending, shear, and axial.

Delcan modeled the Tetsa River Bridge #1 using “Midas Civil” (Midas) software utilizing a grillage-type model. The structural model was developed using beam elements for the superstructure members and deck and pin or roller supports, as applicable, as substructure elements. As substructure elements were in good condition with respect to structural integrity and member stability based on the September 2009 visual bridge inspections, substructure elements were not considered further in this live load capacity factor evaluation of the Tetsa River Bridge #1. It was assumed that the substructure elements provide full support to the superstructure members.

Also, member-to-member connections (all joints) were assumed to be fully effective (i.e. in providing full capacity to transfer loads between the connected elements). All connections of secondary members to primary members were assumed to be pinned-pinned connections. Connections of the stringers to the floor beams were also assumed to be pinned-pinned connections.

See Figure 25, below, for a rendered view of the Midas model. Tetsa River Bridge #1 is geometrically symmetrical about its midspan. Therefore, only one of its through trusses and one of its jackspans were modeled.

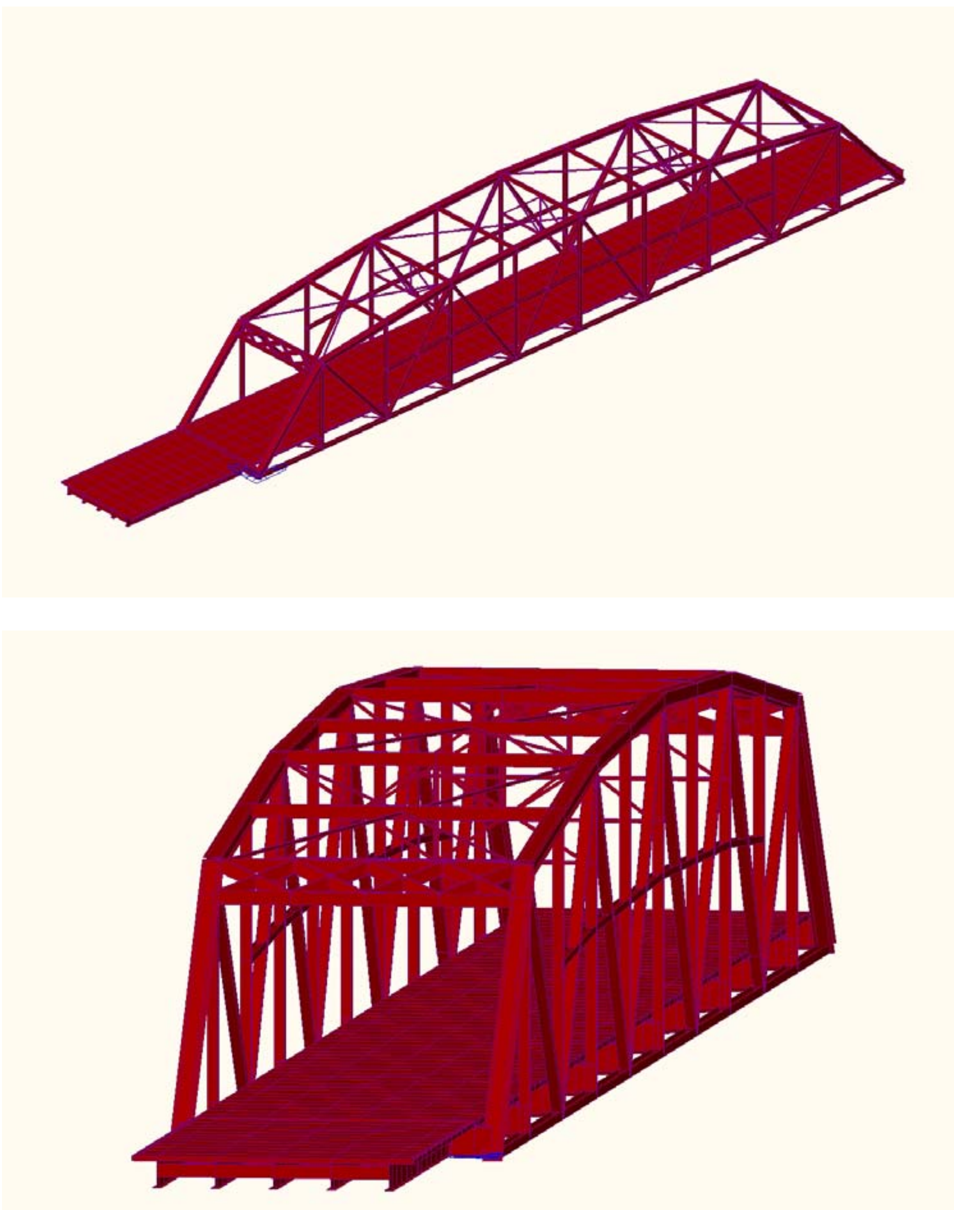

Figure 25: Tetsa River Bridge #1 Rendered Midas Civil Grillage-Type Model.

4.2 Reliability Indices and Load Factors

The following Table #1, below, provides:

a) System behaviour, element behaviour, and inspection level classifications.
b) Reliability indices as determined for "Normal Traffic".
c) An adjustment to the reliability index of 0.25 based on Clause 14.12.5 and recognizing this bridge as an important structure.
d) Dead load factors based on Clause 14.13.2.1.
e) Live load factors for normal traffic based on Clause 14.13.3.1.
f) A multilane factor for normal traffic based on Clause 14.9.4.2.
g) Dynamic load allowances for normal traffic based on Clause 14.9.1.7.

It was assumed that all members would be subjected to 'gradual failure with warning of probable failure' regardless of material or load carrying direction / capacity, with respect to 'Element Behaviour' – Clause 14.12.3 of the CHBDC, except for truss members in compression.

Project: Alaska Highway Load Rating			Design: P. Phillips, PEng	Date: 2010/01/27	
Subject: Tetsa1: Reliability Index & Load Factors			Check: H. Hawk, MSc, PEng	Date: 2010/01/27	
	Element				
	Floor Beams	Stringers	Diagonal Floor Bracing	Truss - Tension	Truss - Compression
Category					
System Behaviour	S2	S3	S3	S1	S1
Element Behaviour	E3	E3	E3	E3	E2
Inspection Level	I2	I2	I2	I2	I2
Live Load Lateral Dist.	Sophisticated Analysis	Sophisticated Analysis	Simplified Analysis	Sophisticated Analysis	Sophisticated Analysis
Reliability Index, β: CL1-W	3.00	2.75	2.75	3.25	3.50
Important Structure: β Increased by	0.25				
CL1-W	3.25	3.00	3.00	3.50	3.75
Factors					
DL	D1	D1	D1	D1	D1
α_D: CL1-W	1.08	1.08	1.07	1.09	1.10
$\alpha_{L\,(CL1\text{-}W)}$: CL1-W	1.56	1.49	1.49	1.63	1.70
Multi-Lane Factor	0.9				
1+DLA	1.40	1.40	1.40	1.40	1.40

Element (continued)				
Portals & Bays	Other Truss Bracing	Steel Deck	Jackspan Girders	Jackspan Cross-Bracing
S3	S3	S3	S2	S3
E3	E3	E3	E3	E3
I2	I2	I2	I2	I2
Sophisticated Analysis	Sophisticated Analysis	Simplified Analysis	Sophisticated Analysis	Sophisticated Analysis
2.75	2.75	2.75	3.00	2.75
3.00	3.00	3.00	3.25	3.00
D1	D1	D1	D1	D1
1.07	1.07	1.07	1.08	1.07
1.49	1.49	1.49	1.56	1.49
1.40	1.40	1.40	1.30	1.30

Table #1: Reliability Indices and Load Factors.

4.3 Resistance Adjustment Factors

Resistance adjustment factors, as follows, were determined from Clause 14.14.2 of the CHBDC. The factored resistance of an individual structural component under consideration was multiplied by the appropriate resistance adjustment factor.

Structural Steel

a) Compression or tension on gross section: U = 1.01.
b) Shear: U = 1.02.
c) Stringer Bending: U = 1.00.
d) All Other Steel Bending: U = 0.96.

Steel Grating Deck:

a) Bending: U = 1.00.

4.4 Permanent Loads

The dead loads in the model include:

a) The full self-weight of the primary superstructure elements and secondary lateral bracing elements / sway frames / portals / diaphragms.
b) All original structural steel drawings were not provided to Delcan. Therefore, all of the structural steel members' sizes needed to be measured onsite. When exact member sizes were unknown during design, conservative assumptions were made as to their properties.

c) The weights of the steel jackspan I-girders were adjusted upwards by 12% to account for steel connections, gusset plates, transverse web stiffeners, bearing stiffeners, etc. as per Clause C14.8.2.1 of the CHBDC Commentary.
d) The weights of the steel main-span truss members and floor beams were adjusted upwards by 20% to account for steel connections, gusset plates, transverse web stiffeners, bearing stiffeners, etc. as per Clause C14.8.2.1 of the CHBDC Commentary.
e) Weight and stiffness of the steel grating open deck estimated from the document entitled: "Fabricated Bridge Products" as published by "L.B. Foster".
f) Weights of the steel plates located between the tops of the top flanges of the stringers and the underside of the steel grating deck.
g) Bridge barriers in the field (railing type barriers) are the same as the barriers shown on the drawings provided to Delcan. The document entitled: "Guardrail" as published by "Canada Culvert" was used to estimate the weights of the guardrail components.

4.5 Normal Traffic Live Loads

Tetsa River Bridge #1 was evaluated for Evaluation Level 1 CL1-W truck/lane live loading. Two lanes exist in the field (one Northbound lane and one Southbound lane) and therefore two traffic lanes were modeled, as specified in Clause 14.9.4.1. Appropriate multiple-lane load factors and dynamic load factors were applied to the truck and/or lane loading, when applicable.

4.6 Material Strengths

The material properties used in the resistance calculation processes were obtained from the structural drawings provided by PWGSC. In some instances, the structural drawings provided did not show some of or any of the original material strengths. In these cases, Clause 14.7 of the CHBDC was followed or, for structural steel members, where the year of construction of the bridge was known, the document entitled "Obsolete Canadian Structural Steel Grades, 1935 – 1971" as published in 2005 by the "Canadian Institute of Steel Construction (CISC)" was used. Tetsa River Bridge #1 was originally built in 1943. Dates of some of the subsequent modifications made to this bridge are known to Delcan, while others are not.

The following values were used in the evaluation of the Tetsa River Bridge #1:

a) Steel deck yield strength: 350A MPa, Drawing Number 3 of 4, 1989.
b) Steel guiderail posts yield strength: 300 MPa, Drawing Number 3 of 4, 1989.
c) Superstructure structural steel yield strength: Table 14.1, Clause 14.7.4.2 of the CHBDC, 1943, 230 MPa.
d) Superstructure structural steel ultimate strength: Table 14.1, Clause 14.7.4.2 of the CHBDC, 1943, 420 MPa.
e) Portal and bay bracing reinforcing structural steel yield strength: Grade 44 (300 MPa), Drawing Number 2 of 5, 1976.
f) Portal and bay bracing reinforcing structural steel ultimate strength: Grade 65 (450 MPa), Drawing Number 2 of 5, 1976.
g) Truss member reinforcing structural steel yield strength: 300 MPa, Drawing Number 2 of 4, 1989.
h) Truss member reinforcing structural steel ultimate strength: 450 MPa, Drawing Number 2 of 4, 1989.

5.0 RESULTS

5.1 Key-Plans/Elevations

The following key-plan/elevation diagrams indicate the naming conventions of the individual structural members within this bridge that were adopted for this load rating. All members referenced in this 'Results' section will therefore be referred to by their key-plan/elevation names.

See Figures 26 to 30 for key-plan/elevation drawings for the Tetsa River Bridge #1:

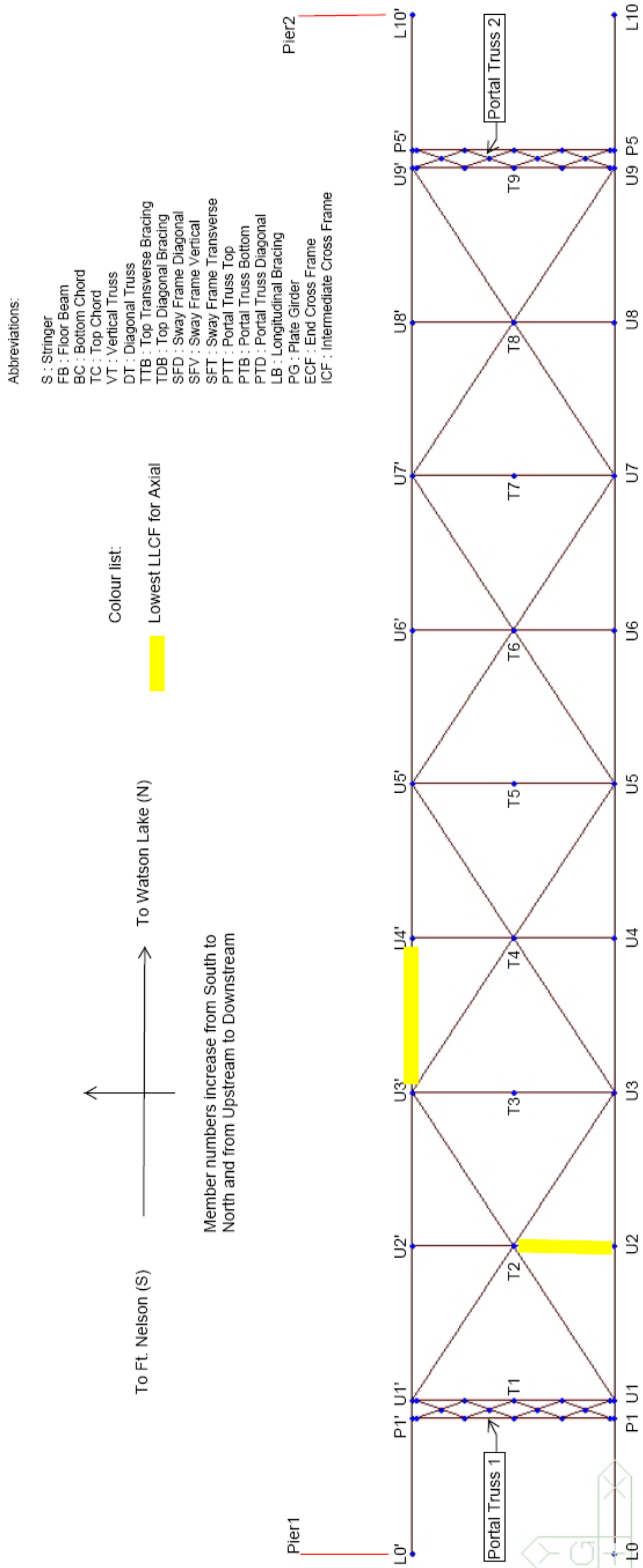

Figure 26: Tetsa River Bridge #1 Top Chord Plan View Naming Conventions.

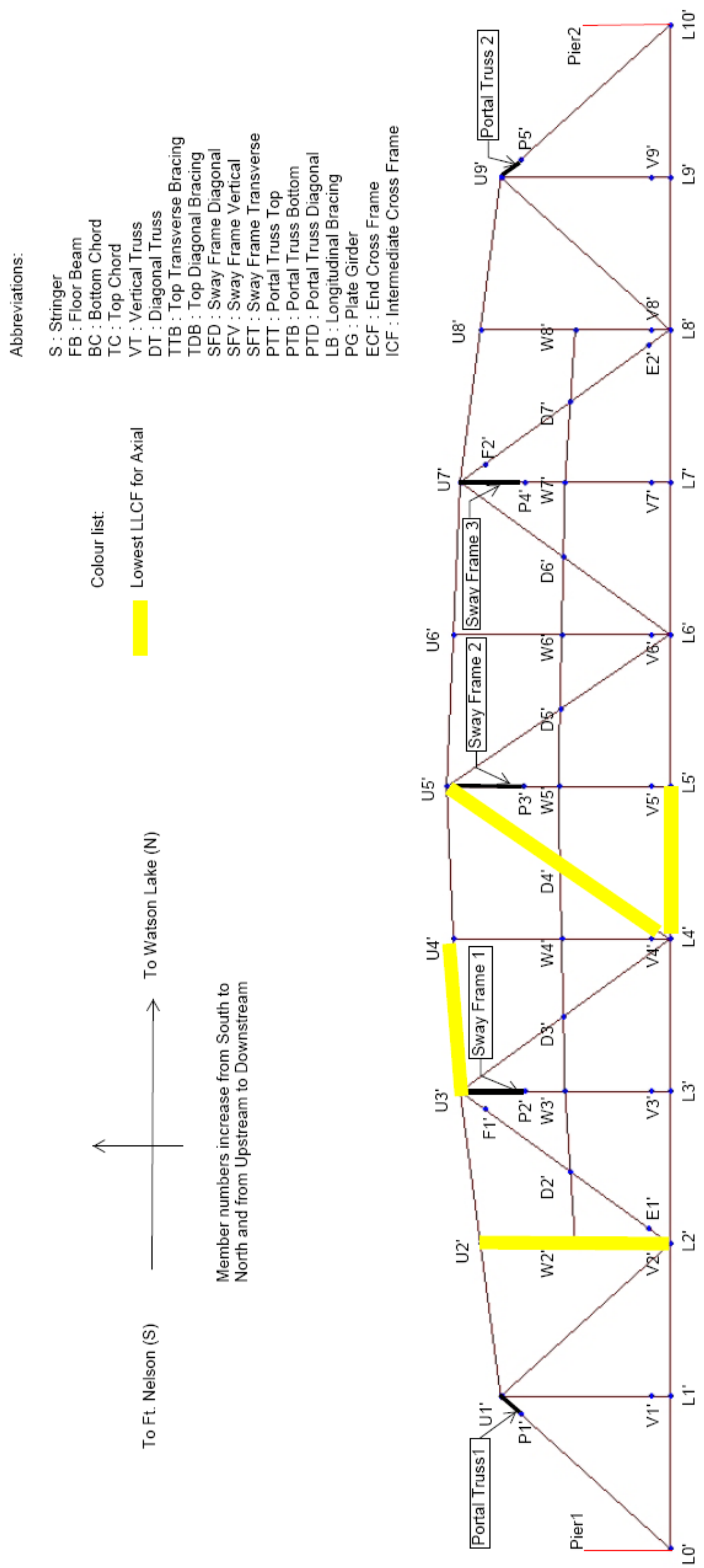

Figure 27: Tetsa River Bridge #1 Truss Elevation View Naming Conventions.

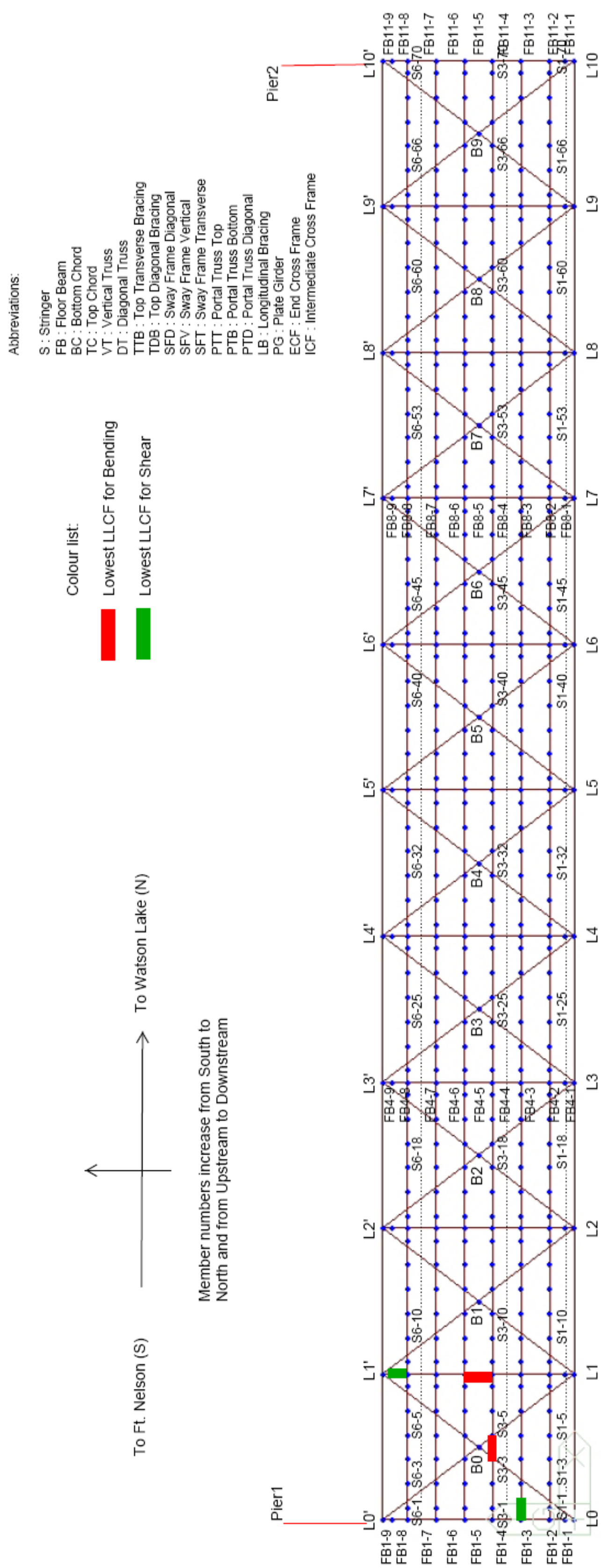

Figure 28: Tetsa River Bridge #1 Floor Beam and Stringer Plan View Naming Conventions.

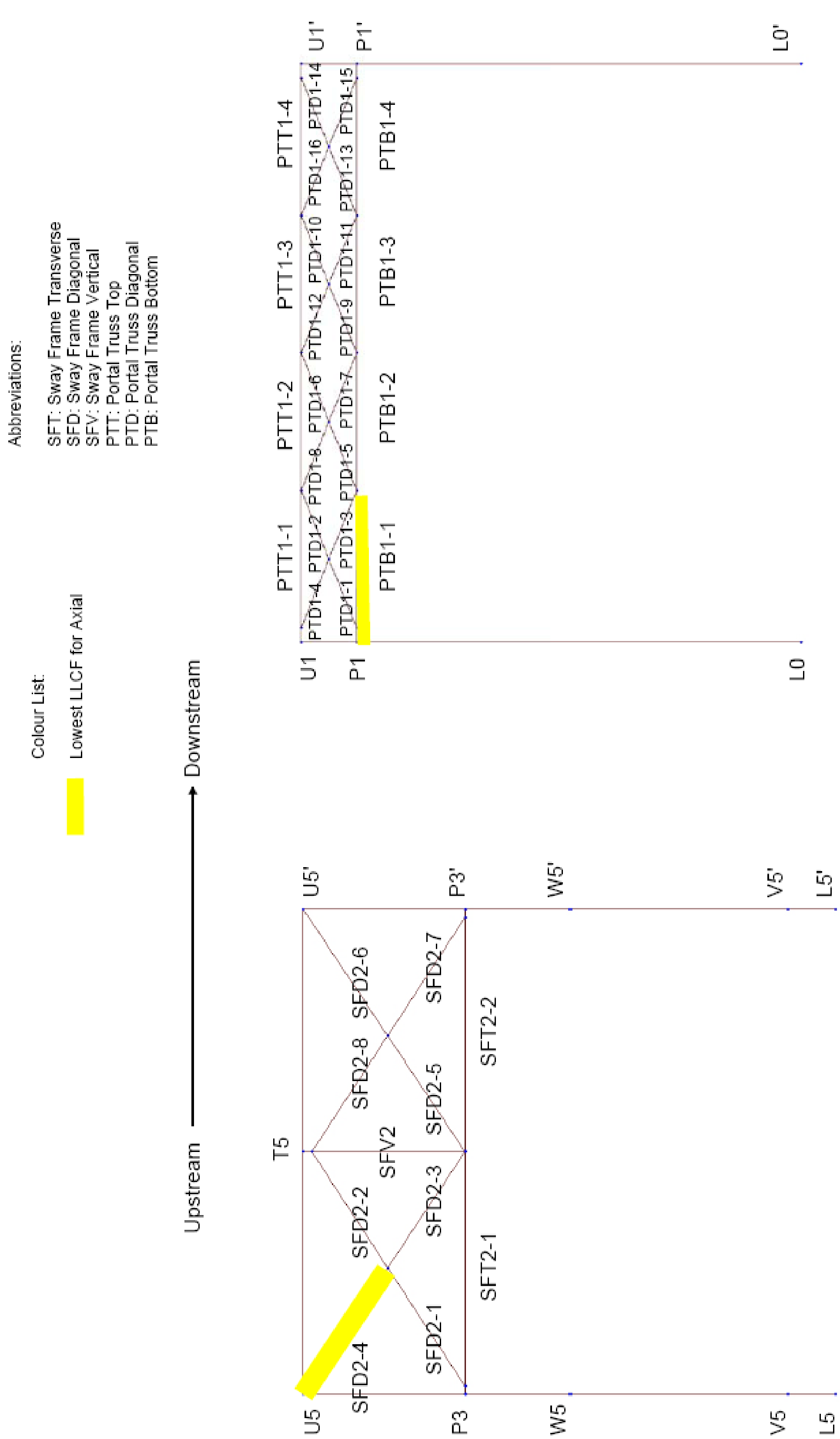

Figure 29: Tetsa River Bridge #1 Sway Frame and Portal Elevation Views Naming Conventions.

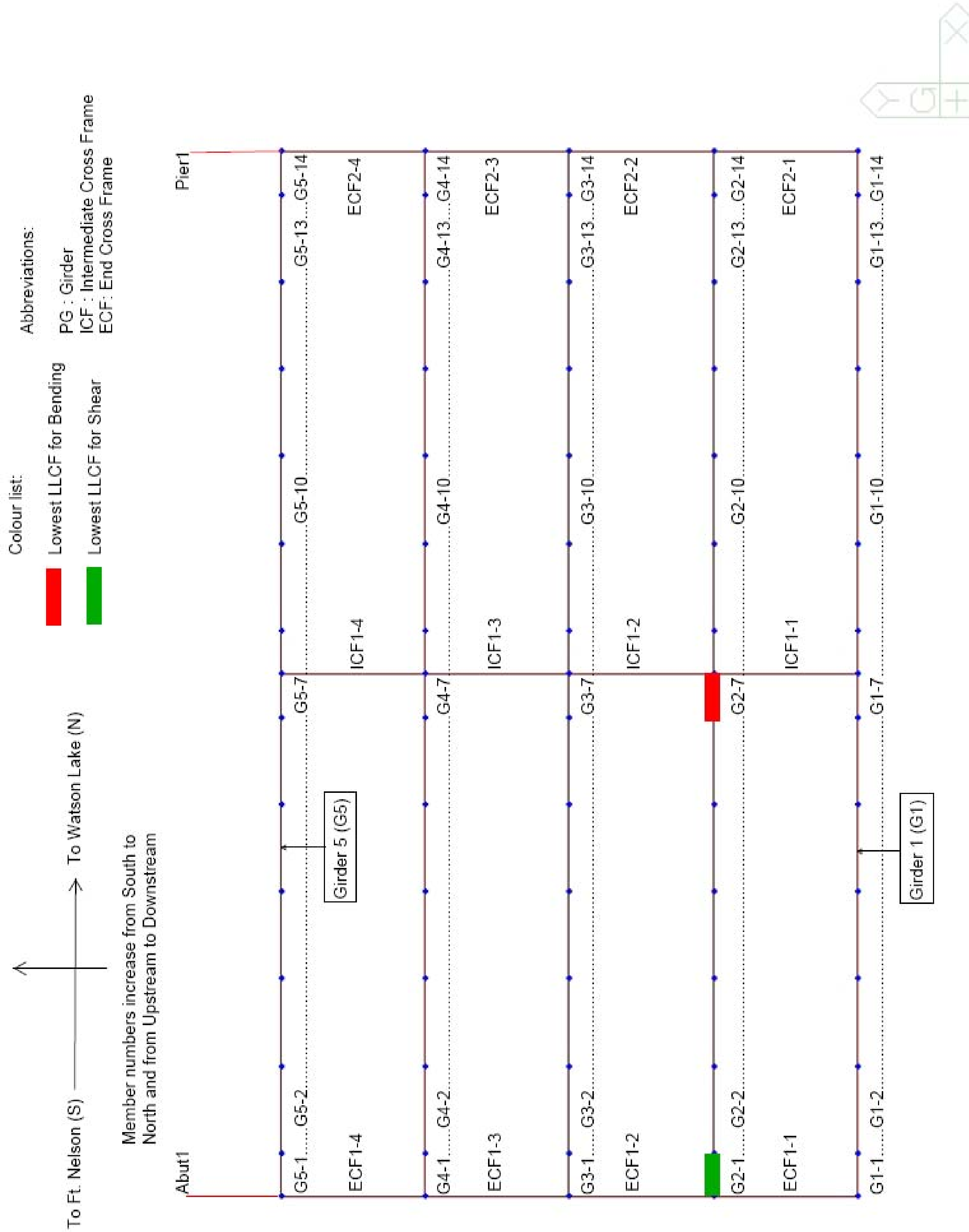

Figure 30: Tetsa River Bridge #1 Jackspan Plan View Naming Conventions.

5.2 Live Load Capacity Factors

The following Figures 31 to 41 show the live load capacity factors (LLCFs) that have been calculated along Tetsa River Bridge #1's members based on the requirements of Section 14 of the CHBDC S6-06. Specifically, LLCFs are calculated based on Clause 14.15.2, 'Ultimate Limit States'. LLCFs greater than 1 are deemed adequate for the prescribed live loading and LLCFs less than 1 generally require posting.

The location along a member which is considered to govern its design is the position where the member is most highly loaded relative to its resistance.

Due to overall bridge symmetry (i.e. no skew effects), symmetrical transient loading present, and simply supported bearing conditions, symmetrical force diagrams are produced within this bridge. In such cases, half-spans of the members need only to be shown.

No figure is shown below for the stringers in positive flexure, however. Delcan calculated the LLCF(s) for the stringers in positive flexure based upon two different methods: 1) the live load distribution as determined by the Midas Civil grillage type model and 2) the live load distribution determined by the simplified method of Clause 5.7.1.2.1.2 of the CHBDC. The method of Clause 5.7.1.2.1.2 was slightly more conservative than the Midas Civil grillage type model method and therefore was adopted by Delcan for this case. See the attached CD (Appendix A) for the calculations based upon Clause 5.7.1.2.1.2. The LLCF determined by the method of Clause 5.7.1.2.1.2 is 1.09.

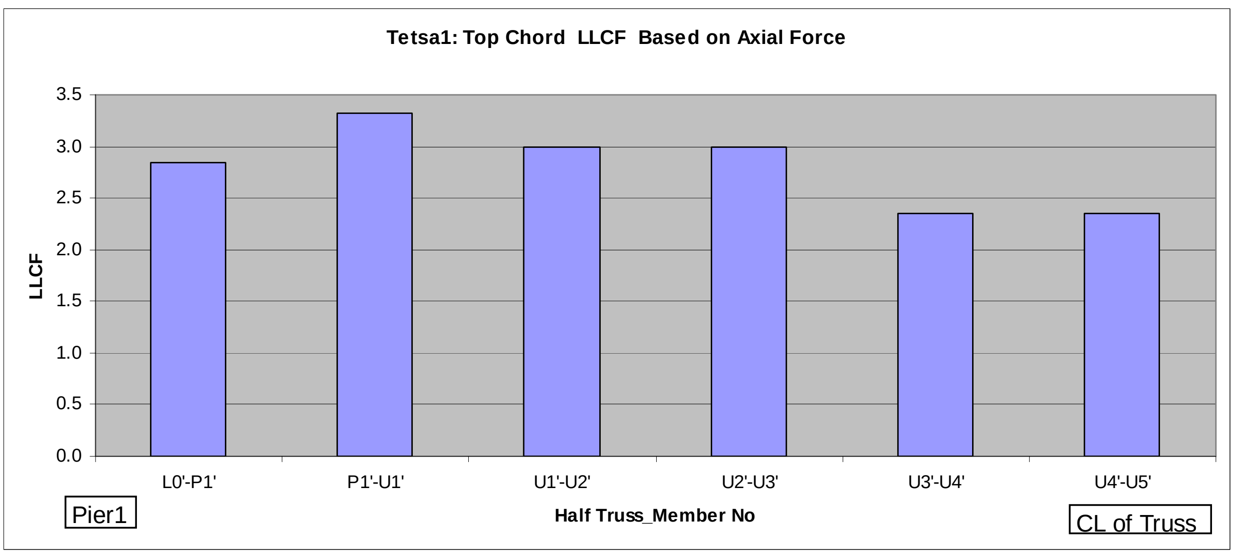

Figure 31: LLCF for Axial Force in the Top Chord of a Half-Through Truss (Tetsa River Bridge #1).

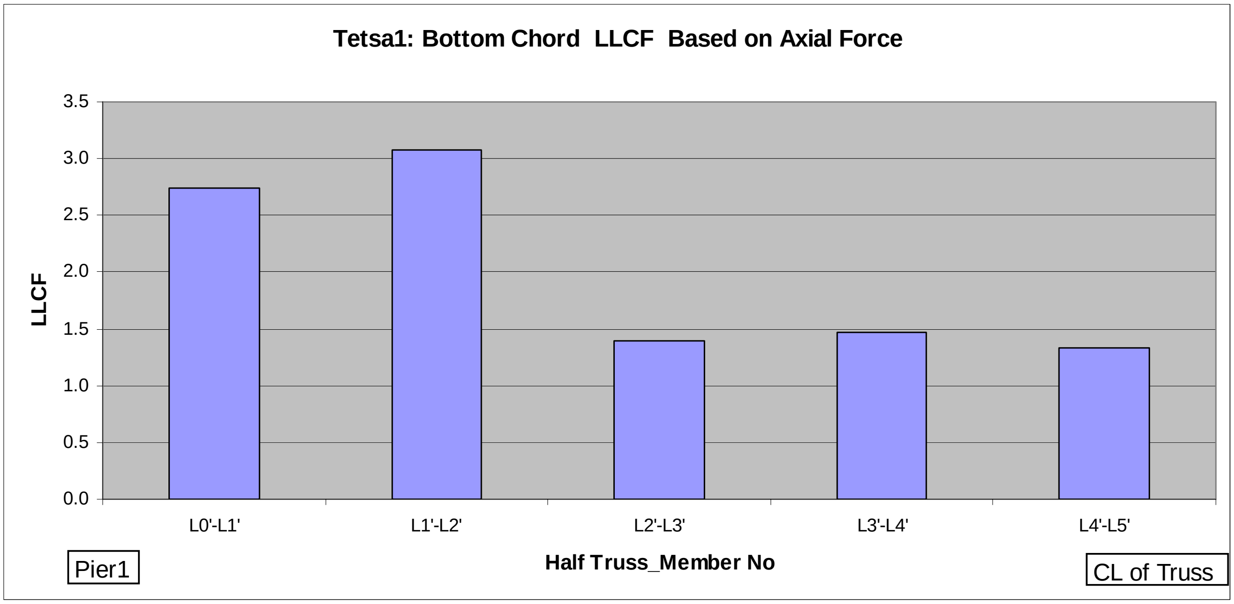

Figure 32: LLCF for Axial Force in the Bottom Chord of a Half-Through Truss (Tetsa River Bridge #1).

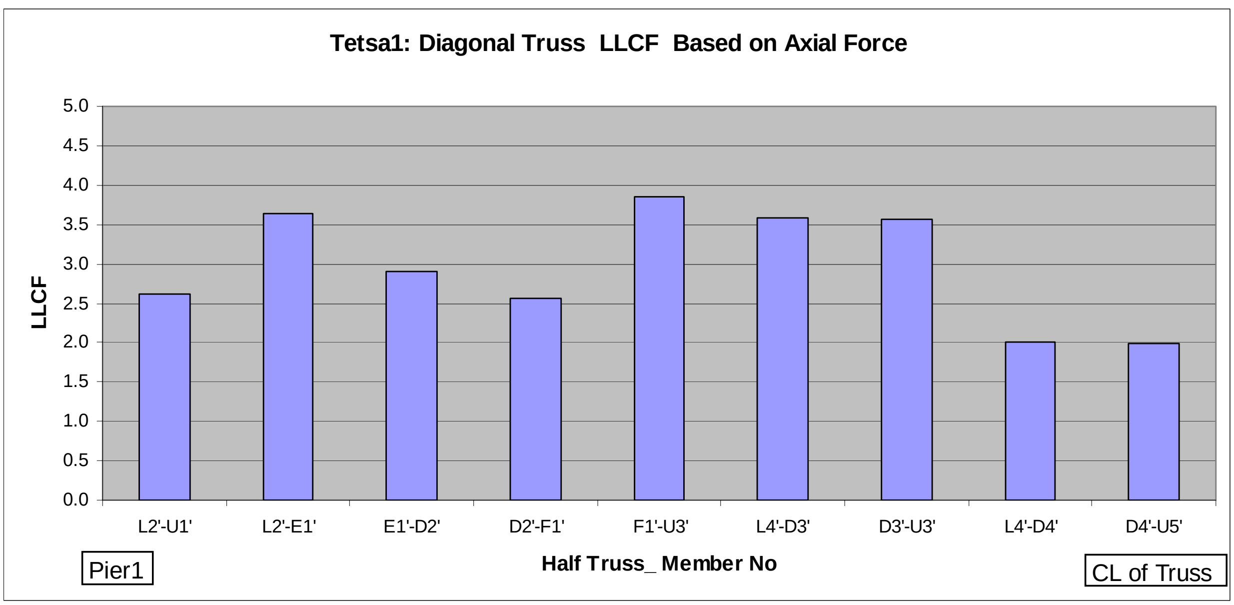

Figure 33: LLCF for Axial Force in the Diagonal Members of a Half-Through Truss (Tetsa River Bridge #1).

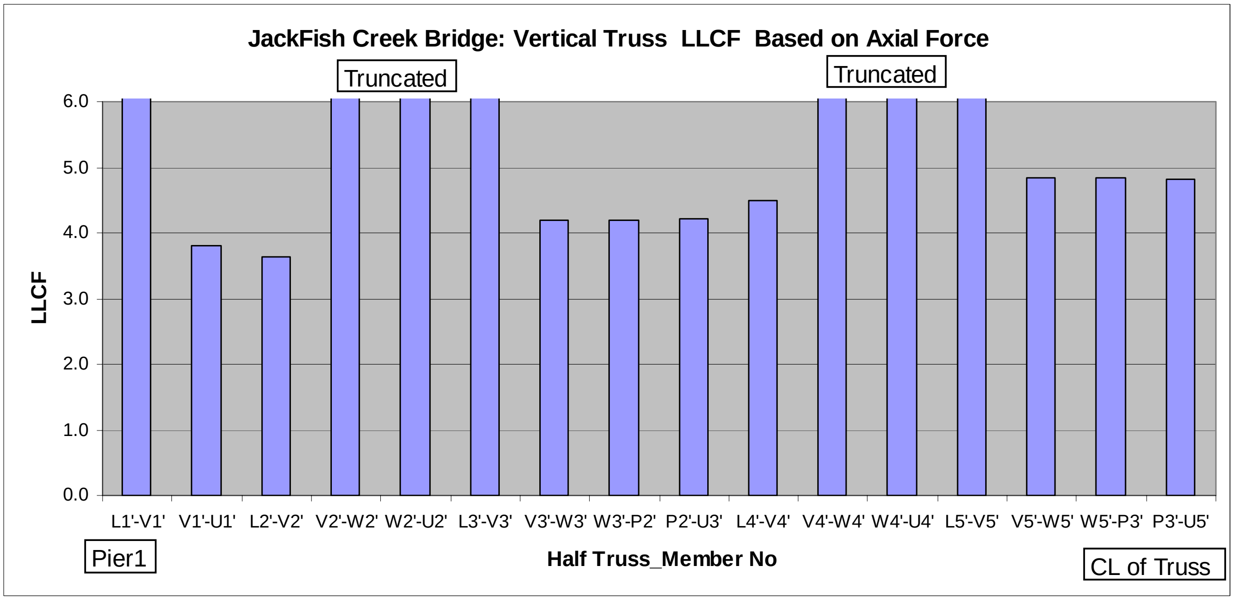

Figure 34: LLCF for Axial Force in the Vertical Members of a Half-Through Truss (Tetsa River Bridge #1).

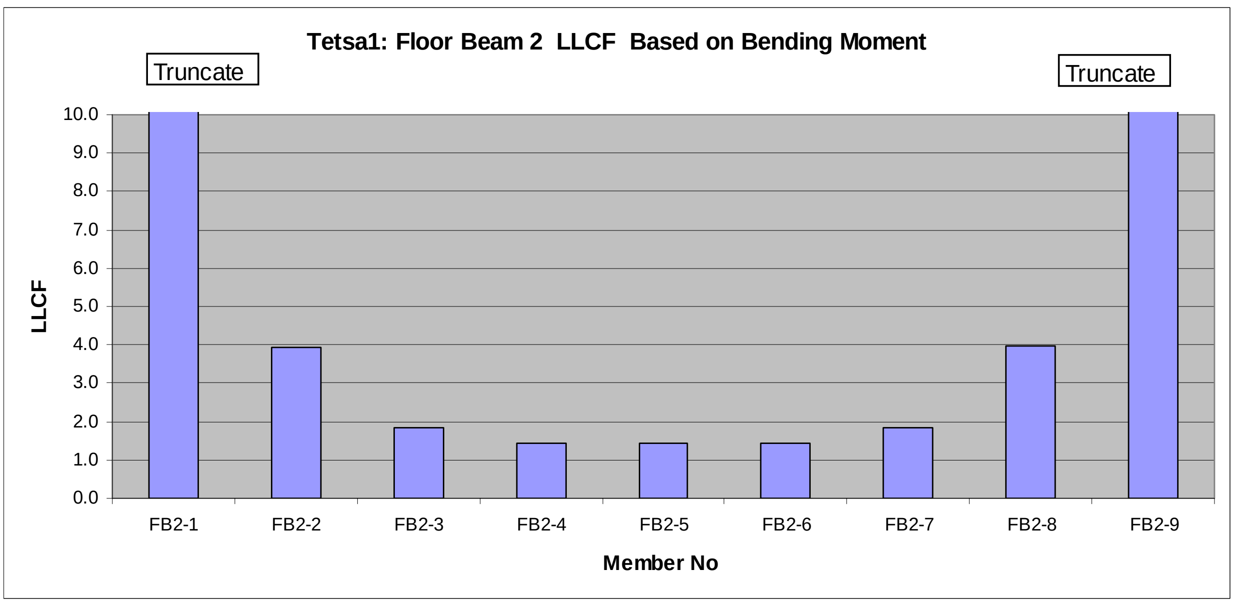

Figure 35: LLCF in Positive Flexure for a Floor Beam (Tetsa River Bridge #1).

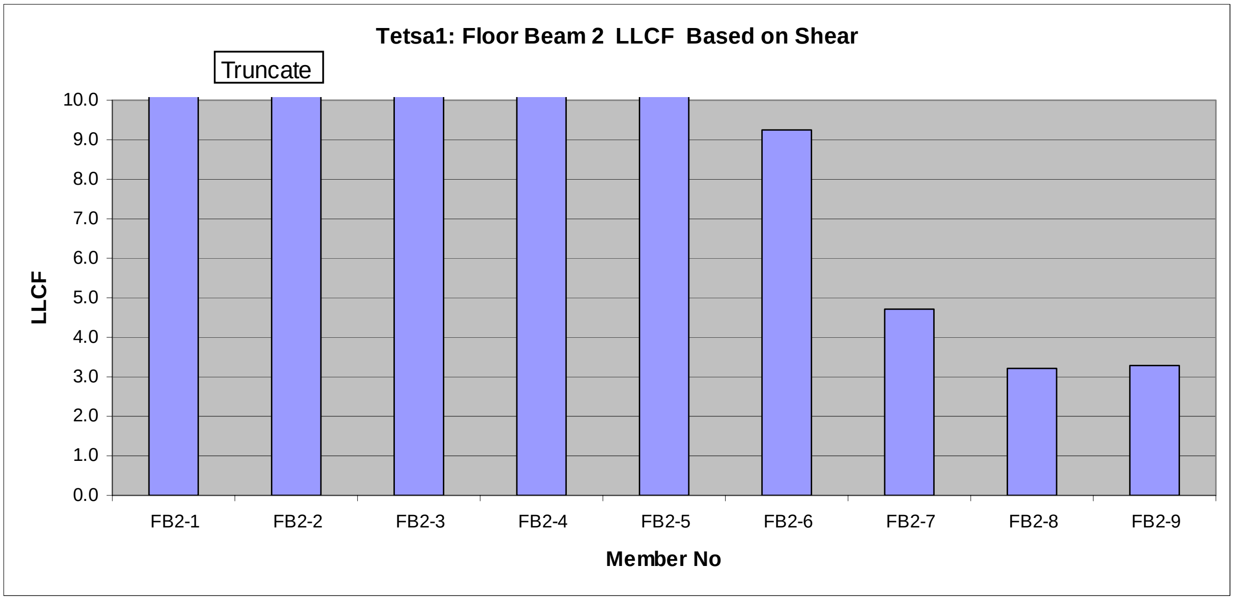

Figure 36: LLCF in Shear for a Floor Beam (Tetsa River Bridge #1).

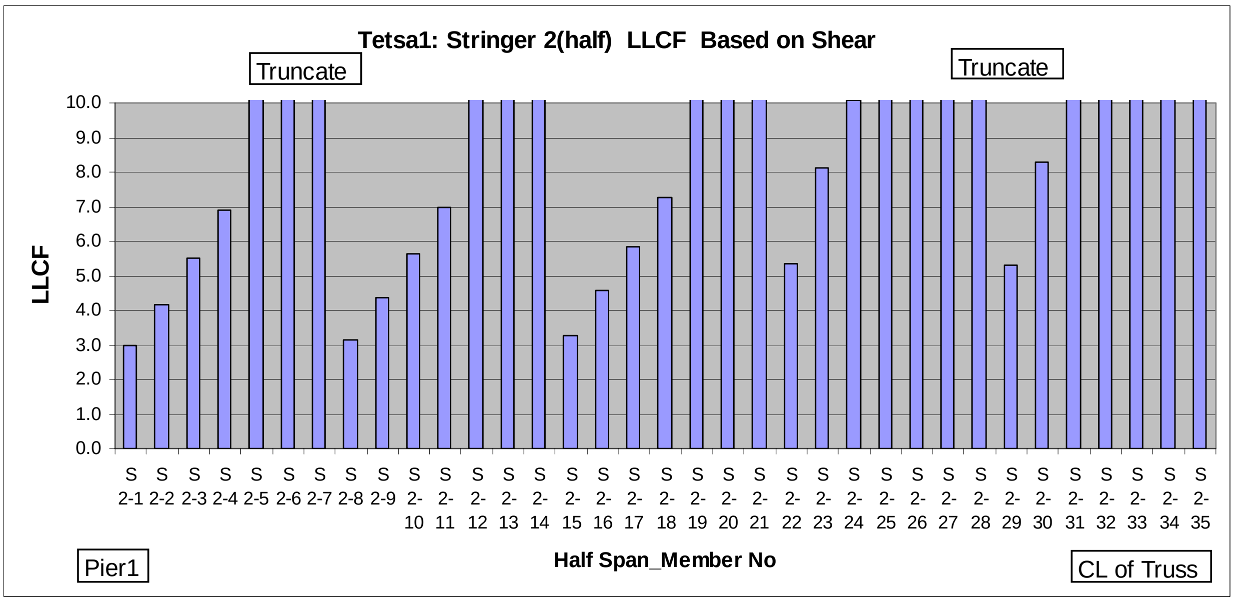

Figure 37: LLCF in Shear for a Line of Stringers of a Half-Through Truss (Tetsa River Bridge #1).

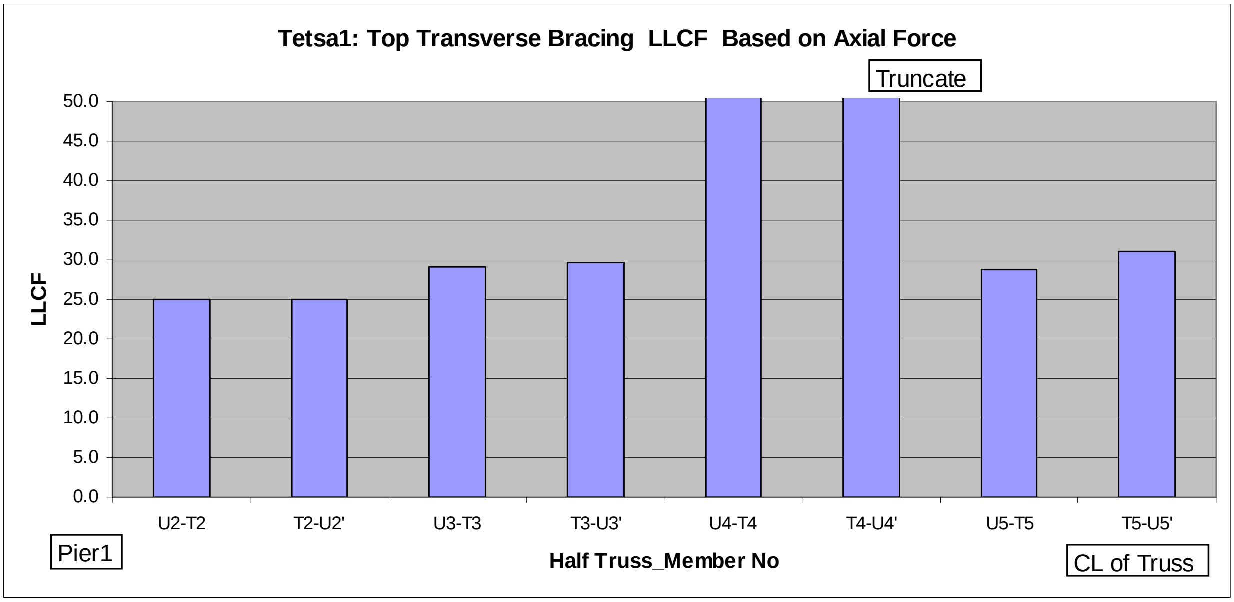

Figure 38: LLCF for Axial Force in Top Horizontal Transverse Truss Top Chord Bracing of a Half-Through Truss (Tetsa River Bridge #1).

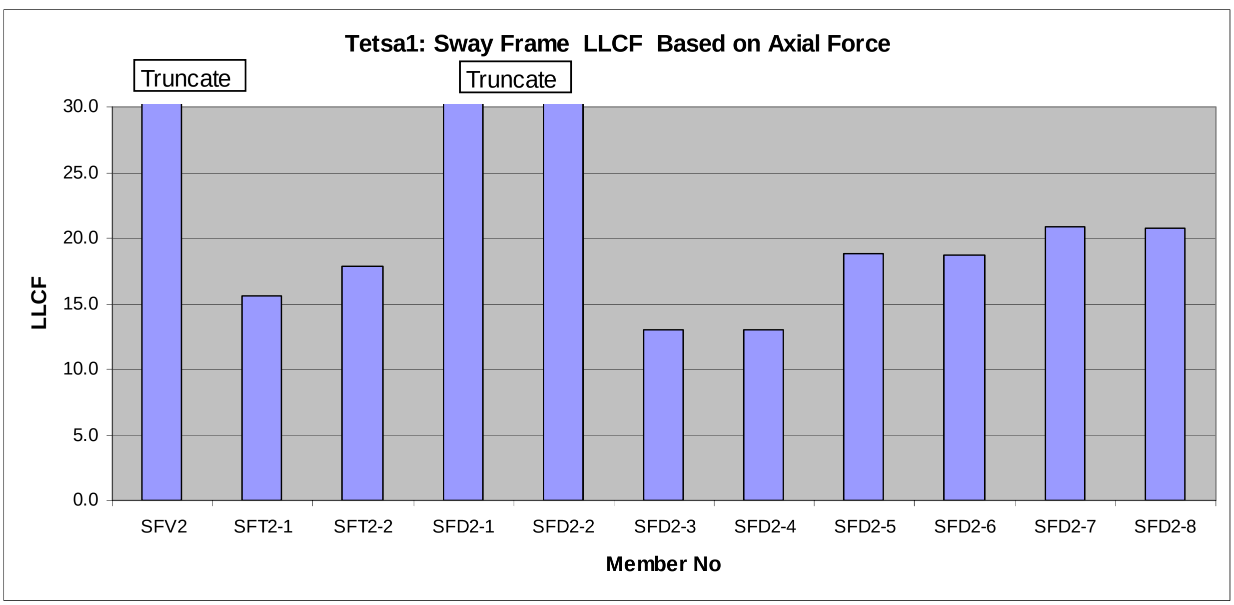

Figure 39: LLCF for Axial Force in a Sway Frame (Tetsa River Bridge #1).

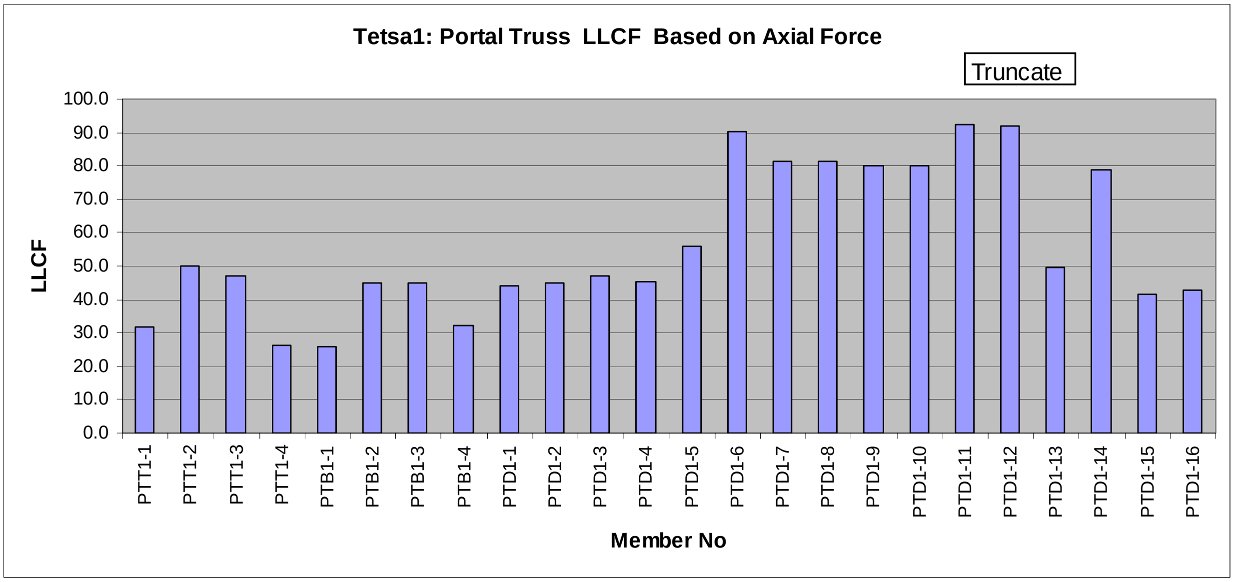

Figure 40: LLCF for Axial Force in a Portal Truss (Tetsa River Bridge #1).

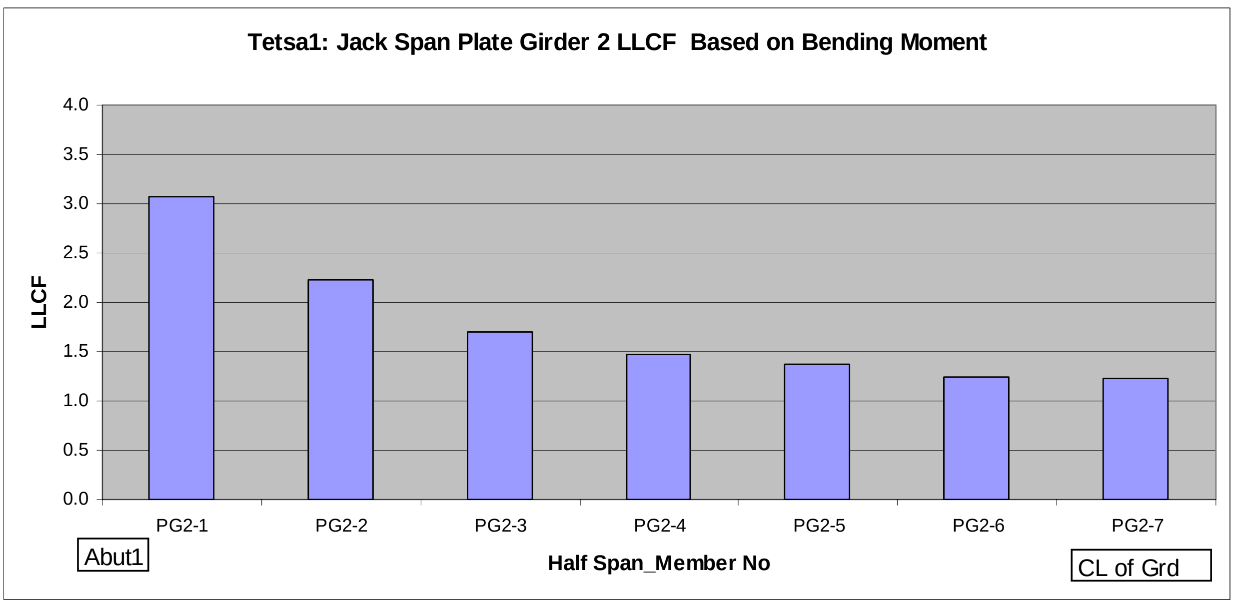

Figure 41: LLCF in Positive Flexure for Half of a Jackspan Girder (Tetsa River Bridge #1).

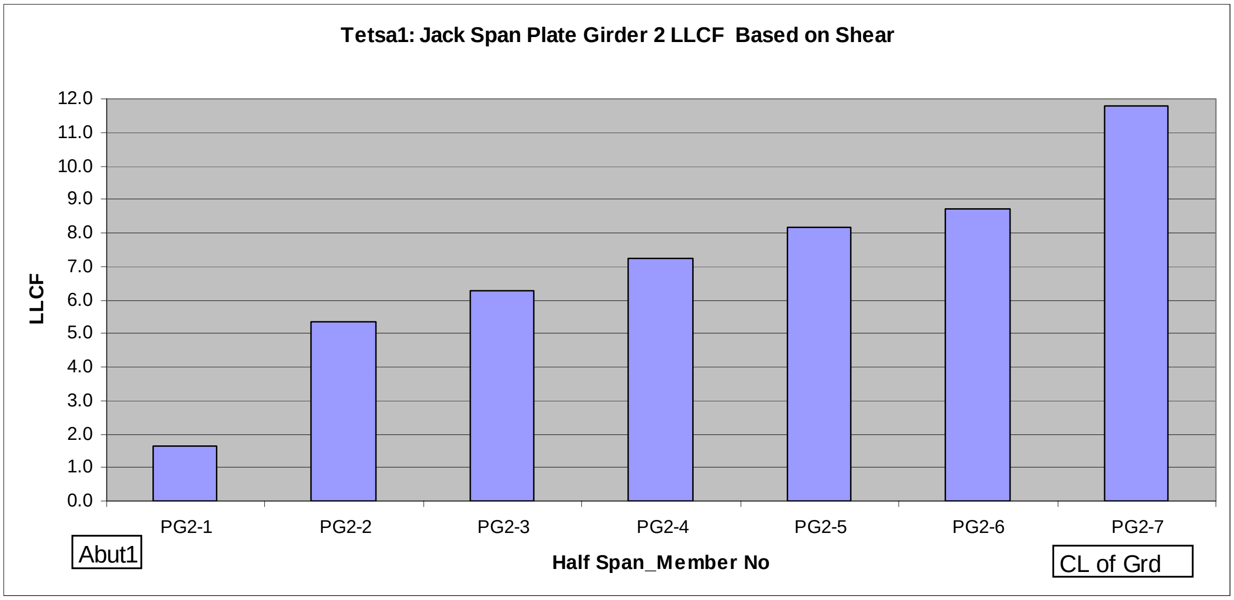

Figure 42: LLCF in Shear for Half of a Jackspan Girder (Tetsa River Bridge #1).

For each type of member in this bridge, Table #2 provides its lowest calculated LLCF value and the location of that LLCF. Also, refer to Figures 26 to 30 for the locations of the lowest LLCFs in plan/elevation.

Title:	Tetsa River Bridge #1	Completed by:	A. Rafiquzzaman, Ph.D.	Date:	2/11/2010
Subject:	Lowest LLCFs	Checked by:	H. Hawk, M.Sc., P.Eng.	Date:	2/11/2010
				Revision:	4
Member ID	Location		LLCFa Axial	LLCFp Bending(+)	LLCFs Shear
U3'-U4'	Top Chord		2.35		
L4'-L5'	Bottom Chord		1.34		
D4'-U5'	Diagonal Truss		1.99		
L2'-V2'	Vertical Truss		3.63		
U2-T2	Top Transverse Bracing		24.93		
SFD2-4	Sway Frame_Diagonal Strut		12.96		
PTB1-1	Portal Frame_Bottom Strut		25.92		
FB2-4	Floor Beam 2			1.45	
PG2-7	Jack Span Plate Girder2			1.23	
FB2-8	Floor Beam2				3.21
S2-1	Stringer 2				3.00
PG2-1	Jack Span Plate Girder2				1.65

Table #2: Lowest LLCF for Each Type of Member Within Tetsa River Bridge #1.

5.3 Deck

The following Table #3 shows that Tetsa River Bridge #1's deck satisfies the requirements of Clause 5.7.1.7.2 of the CHBDC, 'Steel Grid Decks'. The deck's cantilever overhangs were also checked for wheel load induced bending effects.

Title:	Tetsa River Bridge #1	Completed by:	P. Phillips, P.Eng.	Date:	2010/02/10	CHBDC
Subject:	Steel Grid Deck Design	Checked by:	H. Hawk, M.Sc., P.Eng.	Date:	2010/02/10	S6-06
				Revision:	-	

Diaphragms					8.18.5
Spacing of floor beams		6.858			
Spacing of floor beams (max)		8	m	OK	
Moment of inertia method					5.7.1.7.2.3
Thickness of the deck	t_{deck} =	131.763	mm		
Elastic section modulus about top	S_{top} =	2.302E-04	m^3/m		
Elastic section modulus about bottom	S_{bottom} =	2.828E-04	m^3/m		
Neutral axis location from top	y_{top} =	72.641	mm		
Neutral axis location from bottom	y_{bottom} =	59.121	mm		
Moment of inertia	I_y =	1.672E-05	m^4/m		
Steel yield strength	f_y =	350	MPa		
Steel resistance factor	ϕ_c =	0.9			
Resistance adjustment factor	U =	1.00			
Resisting moment	M_r =	89.080	kNm/m		
Wheel load (axle 4)	P =	87.5	kN		
Dynamic load allowance	DLA =	1.40			
Live load factor	α_{LL} =	1.49			
Live load factored moment	M_{fLL} =	54.758	kNm/m		
Span	L =	1.675	m		
Weight of slab	w_{deck} =	0.920	kPa		
Live load factor	α_{DL} =	1.07			
Dead load factored moment	M_{fDL} =	0.345	kNm/m		
Total factored moment	M_f =	55.103	kNm/m	OK	
Cantilever bending					
Width of base of traffic barrier	w =	184.6	mm		
Cantilever length	S_c =	695	mm		
Distance of wheel past centreline of exterior girder		210.4	mm		
Weight of traffic barrier		0.670	kN/m		
Weight of continuous deck end plates		0.220	kN/m		
Resisting moment	M_r =	72.501	kNm/m		
Live load factored moment	M_{fLL} =	38.403	kNm/m		
Dead load factored moment	M_{fDL} =	0.840	kNm/m		
Total factored moment	M_f =	39.244	kNm/m	OK	

Table #3: Deck Calculation Summary for Tetsa River Bridge #1.

6.0 SEISMIC EVALUATION

For Tetsa River Bridge #1, the bearings consist of base plates bolted into the abutments, high fixed rocker plates, and full attachment of the upper bearing plate to the superstructure. The superstructure weight is approximately 2200 kN for each truss span; therefore the system needs to resist a horizontal load of about 220 kN. The anchor bolts are pulled in tension at a force approximately equal to twice the horizontal load and their tensile capacity is approximately 200 kN per bolt and there are 4 bolts. Upper bolt/rivet resistance is unknown but likely adequate. All values are assumed to be in excess of the required resistance. At the expansion ends of the main through-truss spans, bearing seat lengths need to be about 600 mm long. This is a ballpark figure based on Clause 4.4.10.5 and recognizing that there may be incoherency in the movement of the two spans (i.e.: the movement requirement for each span may be additive at the pier); the length provided at each expansion bearing is approximately 700 mm. However, at a movement of about 150 mm to 200 mm the rockers would probably overturn and the truss would drop several hundred millimeters. This is not desirable but can be considered acceptable for an extreme event. At the expansion ends of the jackspans, bearing seat lengths need to be 220.2 mm long (Clause 4.4.10.5); the length provided at each expansion bearing is more than 300 mm.

7.0 SUMMARY OF RECOMMENDATIONS

There are no major load capacity issues with Tetsa River Bridge #1 and no posting is required. No LLCFs for Tetsa River Bridge #1 are less than 1 for the ultimate limit state. The damaged portal and sway frame through-truss top chord lateral bracing should be repaired/replaced in the field as soon as possible. Part of the 2009/2011 contract that Delcan has with PWGSC is to prepare repair sketches for these damaged secondary members.

APPENDIX A – ANALYSIS FILES (ON CD)

The CD accompanying this report includes the following documents:

1) .pdf file of this load rating report.
2) .txt printout of the Midas Civil model.
3) .mcb Midas Civil model.
4) .pdf printouts of all of the design spreadsheets.

PUBLIC WORKS AND GOVERNMENT SERVICES CANADA

TOAD RIVER BRIDGE
ALASKA HIGHWAY, km 671.7

STRUCTURAL LIVE LOAD CAPACITY FACTOR EVALUATION

Prepared by:

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Calgary, Alberta, Canada
T2P 3E8
403-228-9450

Hugh Hawk, M.Sc., P.Eng., Technical Director, Western Region
Stan Reimer, P.Eng., Senior Bridge Engineer
Abul Rafiquzzaman, Ph.D., Intermediate Bridge Engineer
Peter Phillips, P.Eng., Intermediate Bridge Engineer

March 2010

TABLE OF CONTENTS

EXECUTIVE SUMMARY

A detailed visual condition assessment for the Toad River Bridge, km 671.7 along the Alaska Highway, was performed in September 2009. This condition assessment and Chapter 14, "Evaluation", of the "Canadian Highway Bridge Design Code (CHBDC)", CAN/CSA-S6-06, were used to evaluate the load carrying capacity of this existing bridge.

Toad River Bridge was evaluated for CL1-W truck/lane live loading (Evaluation Level 1) in accordance with the CHBDC. Load factors were determined from Clause 14.13 based on target reliability indices (beta-factors) as determined from Clause 14.12 of the CHBDC. Resistance adjustment factors as determined from Clause 14.14.2 of the CHBDC were also used. These target reliability indices and load factors were reviewed by Public Works and Government Services Canada (PWGSC).

Live load capacity factors (LLCFs) for each superstructure member, as per Clause 14.15, were calculated (if and when applicable) for ultimate limit states in bending, shear, and axial. Punching shear was checked for this bridge's deck.

All members of the existing bridge are capable of carrying their combination of factored dead and imposed truck/lane live loads. No bridge strengthening or posting is currently required.

1.0 INTRODUCTION

To determine if the bridge structure requires strengthening or posting, a bridge inspection was first carried out. The bridge's original structural drawings were reviewed in order to familiarize oneself with the structure, prior to an assessment in the field of its member conditions and overall stability. A load rating was then undertaken to establish which members are under-strength, if any. Section 14 of the Canadian Highway Bridge Design Code (CHBDC) outlines this process: target reliability indices (beta-factors) for each member of the bridge were determined and then dead and live load factors for each member of the bridge were chosen.

A model of the bridge, based on the requirements of Section 5 of the CHBDC, was created. Largest combined dead and live factored loads were determined for each member. The resistance of each member was compared to its respective applied force, taking into consideration the deterioration of the members as well. This comparison was used to determine which members were under-strength, if any. If members are determined by analysis to be under-strength currently, it is due to changes in design code requirements, deteriorating member conditions in the field, lack of conservatism in the original designs, or poor original construction practices. The load rating in combination with the field inspection program was used to classify the capacity problem (i.e. problems in flexure, shear, compression, tension, torsion, serviceability, fatigue, maintainability, durability, etc.).

The following original structural drawings were provided by PWGSC to assist in the structural evaluation of this bridge. The drawings provided a reference for dimensions, methods of original construction, substructure details (i.e. details not visible in the field), and material strengths, etc. Relevant assumptions, however, also needed to be made and other pertinent dimensions, data, etc. were obtained in the field during the visual bridge inspections.

Drawings Obtained:
Toad River Bridge: Drawings 1 to 18, dated 1979.

2.0 BRIDGE DESCRIPTION

An elevation photo of the Toad River Bridge is shown in Figure 1, below.

Figure 1: Toad River Bridge Photo.

Description of Toad River Bridge:

a) Single span weathering steel box girders.
b) Reinforced concrete deck.
c) Reinforced concrete abutments.
d) Concrete caisson piles under East abutment.
e) Rock foundation under West abutment.
f) Gabion slope protection around East abutment.
g) Steel railing on reinforced concrete curb traffic barriers.

Relevant dimensions of the bridge's elevation and cross-section are shown in Figures 2 and 3, respectively.

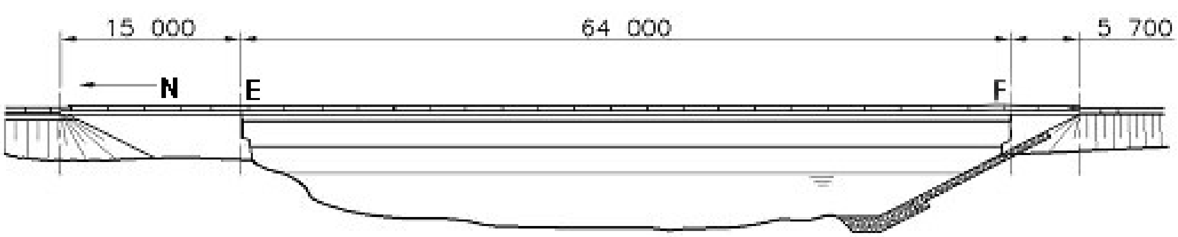

Figure 2: Elevation View of Toad River Bridge.

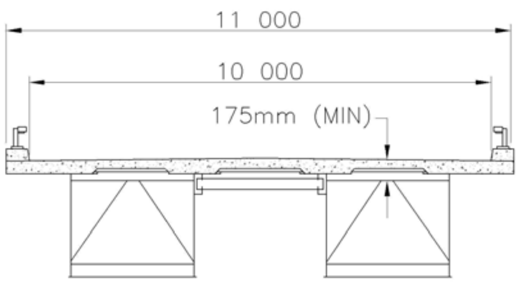

Figure 3: Cross-Section View of Toad River Bridge.

3.0 CONDITION CONSIDERATION

The bridge is generally in good condition. Based on the site evaluation carried out in September 2009 of this structure, no significant loss of section or other bridge issues that could contribute negatively to the overall structural integrity of the bridge were found. For a detailed site condition rating of this bridge please refer to Delcan's 2009 bridge inspection report.

It is important to point out that Toad River Bridge's South end diaphragm cross-bracing between its two box girders has been improperly removed and should be replaced. However, its North end diaphragm cross-bracing has been properly left in place. Also, all of this bridge's intermediate cross-bracing between the two box girders should have been removed as instructed by the original structural drawings for this bridge. Overall, Delcan modeled this bridge in its current (existing) condition.

See Figures 4 to 14, below, for condition photos taken during Delcan's recent 2009 inspections of the Alaska Highway structures.

Locations of Deck Delaminations (Typical).

Figure 4: Deck.

Figure 5: Exterior of Box Girder.

Figure 6: Box Girder Bearing.

Figure 7: Wingwall.

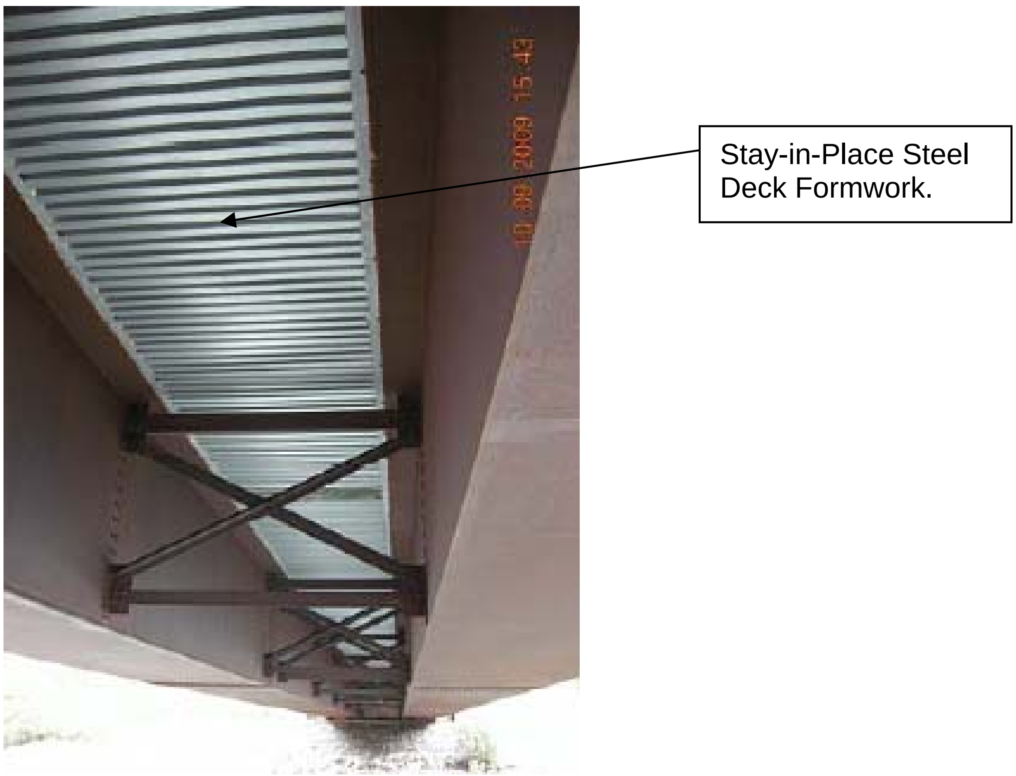

Figure 8: Between Box Girders.

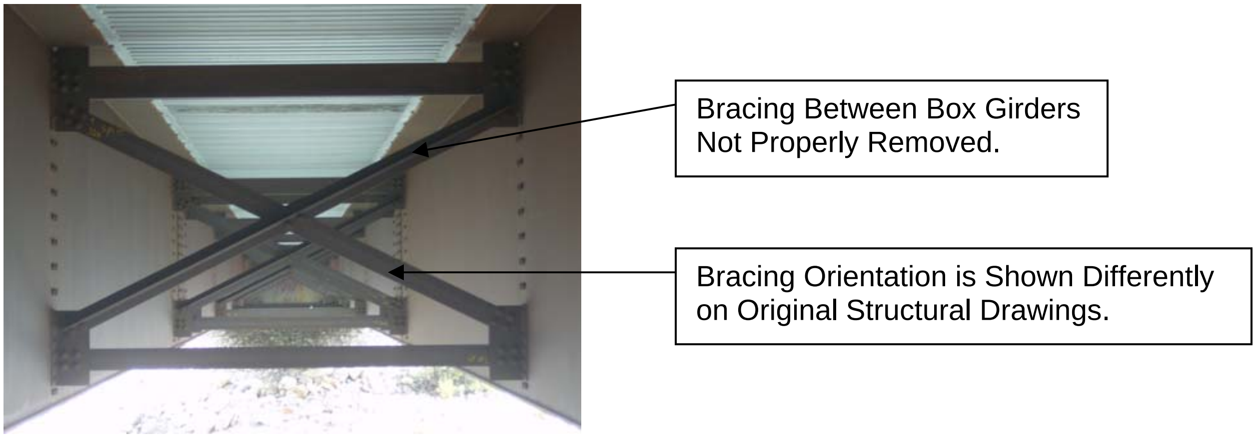

Figure 9: Intermediate Bracing.

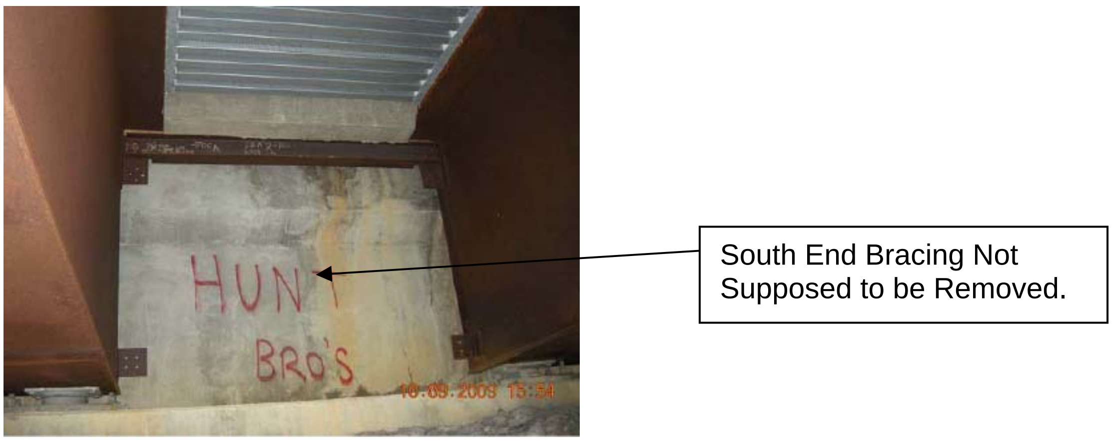

Figure 10: South End Bracing.

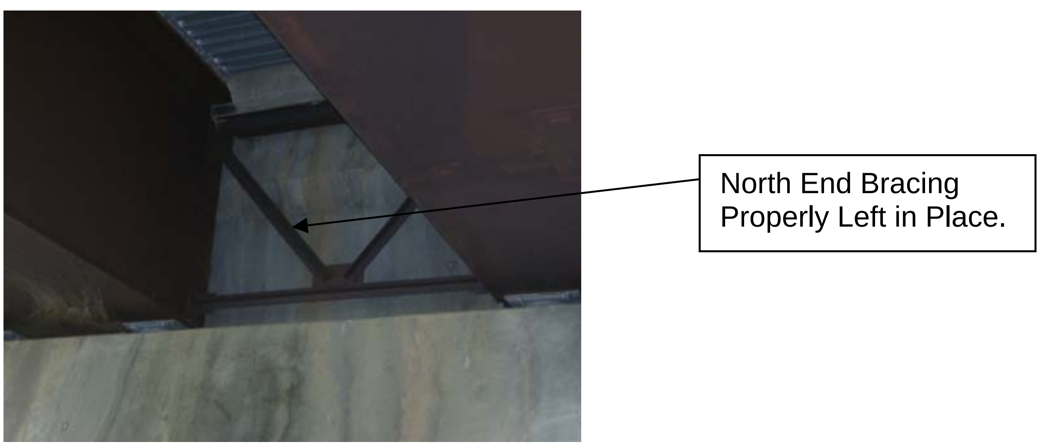

Figure 11: North End Bracing.

Figure 12: Typical Diaphragm Inside of Box Girder.

Figure 13: Water-Pooling Inside of West Box Girder.

Figure 14: Typical Spalling of Bearing Seats.

4.0 EVALUATION PARAMETERS AND METHODOLOGY

4.1 Evaluation Procedures

Toad River Bridge was evaluated for CL1-W truck/lane live loading (Evaluation Level 1) in accordance with the CHBDC. Load factors were determined from Clause 14.13 based on target reliability indices (beta-factors) as determined from Clause 14.12. Resistance adjustment factors as determined from Clause 14.14.2 of the CHBDC were also used. These target reliability indices and load factors were reviewed by PWGSC.

Live load capacity factors for each superstructure member, as per Clause 14.15, were calculated (if and when applicable) for ultimate limit states in bending, shear, and axial. Punching shear was checked for this bridge's deck.

Delcan modeled the Toad River Bridge using "Midas Civil" (Midas) software utilizing a grillage-type model. The structural model was developed using beam elements for the superstructure members and deck and pin or roller supports, as applicable, as substructure elements. As substructure elements were in good condition with respect to structural integrity and member stability based on the September 2009 visual bridge inspections, substructure elements were not considered further in this live load capacity factor evaluation of the Toad River Bridge. It was assumed that the substructure elements provide full support to the superstructure members.

Also, member-to-member connections (all joints) were assumed to be fully effective (i.e. in providing full capacity to transfer loads between the connected elements). All connections of secondary members to primary members were assumed to be pinned-pinned connections.

See Figure 15, below, for a rendered view of the Midas model.

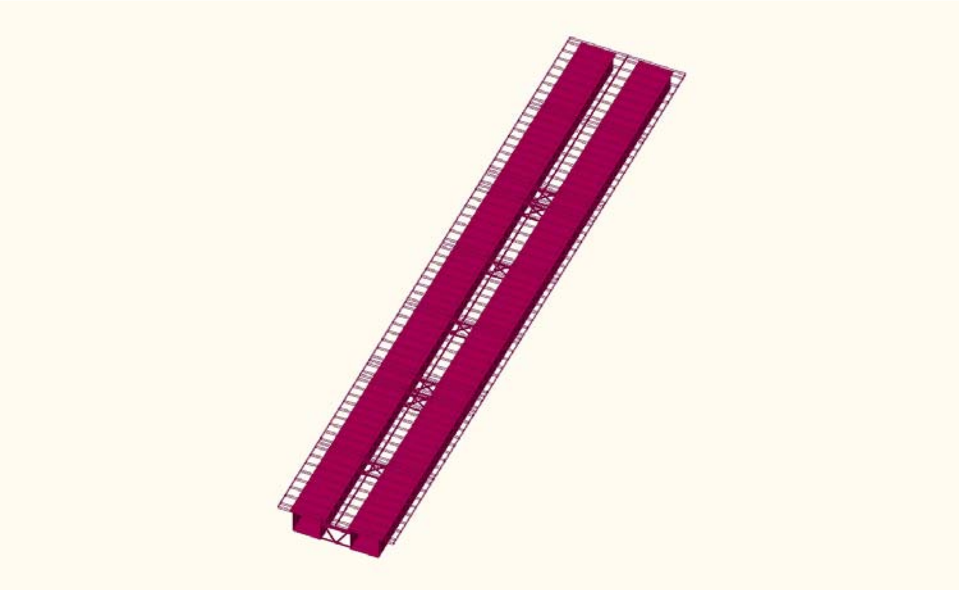

Figure 15: Toad River Bridge Rendered Midas Civil Grillage-Type Model.

4.2 Reliability Indices and Load Factors

The following Table #1, below, provides:

a) System behaviour, element behaviour, and inspection level classifications.
b) Reliability indices as determined for "Normal Traffic".
c) An adjustment to the reliability index of 0.25 based on Clause 14.12.5 and recognizing this bridge as an important structure.
d) Dead load factors based on Clause 14.13.2.1.
e) Live load factors for normal traffic based on Clause 14.13.3.1.
f) A multilane factor for normal traffic based on Clause 14.9.4.2.
g) Dynamic load allowances for normal traffic based on Clause 14.9.1.7.

It was assumed that a simply supported two-box girder structural system is not a redundant system, with respect to 'System Behaviour' – Clause 14.12.2 of the CHBDC. Also, all members would be subjected to 'gradual failure with warning of probable failure' regardless of material or load carrying direction / capacity, with respect to 'Element Behaviour' – Clause 14.12.3 of the CHBDC.

Project: Alaska Highway Load Rating		Design: P. Phillips, PEng		Date: 2010/01/27
Subject: Toad: Reliability Index & Load Factors		Check: H. Hawk, MSc, PEng		Date: 2010/01/27
	Element			
	Box Girders	Cross-Bracing	Interior Bracing	Deck
Category				
System Behaviour	S1	S3	S3	S3
Element Behaviour	E3	E3	E3	E3
Inspection Level	I2	I2	I2	I2
Live Load Lateral Dist.	Sophisticated Analysis	Sophisticated Analysis	Simplified Analysis	Simplified Analysis
Reliability Index, β: CL1-W	3.25	2.75	2.75	2.75
Important Structure: β Increased by	0.25			
CL1-W	3.50	3.00	3.00	3.00
Factors				
DL	D1	D1	D1	D2
α_D: CL1-W	1.09	1.07	1.07	1.14
DL	D2	D2	D1	D1
α_D: CL1-W	1.18	1.14	1.14	1.07
$\alpha_{L\,(CL1\text{-}W)}$: CL1-W	1.63	1.49	1.49	1.49
Multi-Lane Factor	0.9			
1+DLA	1.25	1.25	1.25	1.40

Table #1: Reliability Indices and Load Factors.

4.3 Resistance Adjustment Factors

Resistance adjustment factors, as follows, were determined from Clause 14.14.2 of the CHBDC. The factored resistance of an individual structural component under consideration was multiplied by the appropriate resistance adjustment factor.

Structural Steel
a) Compression or tension on gross section: U = 1.01.
b) Shear: U = 1.02.

Composite – Slab on Steel Girder:
a) Bending: U = 0.96.
b) Shear connectors: U = 0.94.

Reinforced Concrete Deck:
a) Bending: U = 0.95.

4.4 Permanent Loads

The dead loads in the model include:
a) The full self-weight of the primary superstructure elements and cross-frames between the box girders.
b) The weights of the steel box girders were adjusted upwards by 12% to account for splice plate weights, steel connections, gusset plates, longitudinal and transverse web stiffeners, bearing stiffeners, interior cross-frames within the box girders, etc. as per Clause C14.8.2.1 of the CHBDC Commentary.
c) 225mm concrete deck is still applicable as shown on original drawings as provided by PWGSC.
d) Weights of the stay-in-place steel deck formwork and the box girder flange concrete haunches.
e) No deck overlay included.
f) Bridge barriers in the field (curb and railing type barriers) are the same as the barriers shown on the drawings provided to Delcan.

4.5 Normal Traffic Live Loads

Toad River Bridge was evaluated for Evaluation Level 1 CL1-W truck/lane live loading. Two lanes exist in the field (one Northbound lane and one Southbound lane) and therefore two traffic lanes were modeled, as specified in Clause 14.9.4.1. Appropriate multiple-lane load factors and dynamic load factors were applied to the truck and/or lane loading, when applicable.

4.6 Material Strengths

The material properties, as provided below, used in the resistance calculation processes were obtained from the structural drawings provided by PWGSC. Toad River Bridge was originally built in 1979. Dates of any subsequent modifications made to this bridge are unknown to Delcan.

The following values were used in the evaluation of the Toad River Bridge:
a) Concrete deck and curb compressive strength: 30 MPa, Drawing Number 1 of 18.
b) Reinforcing deck and curb steel yield strength: 400 MPa, Drawing Number 1 of 18.

c) Superstructure structural steel yield strength: Grade 50A (350 MPa), Drawing Number 1 of 18.
d) Superstructure structural steel ultimate strength: 480 MPa, Drawing Number 1 of 18.

5.0 RESULTS

5.1 Key-Plans/Elevations

The following key-plan/elevation diagrams indicate the naming conventions of the individual structural members within this bridge that were adopted for this load rating. All members referenced in this 'Results' section will therefore be referred to by their key-plan/elevation names.

See Figures 16 to 18 for key-plan/elevation drawings for the Toad River Bridge:

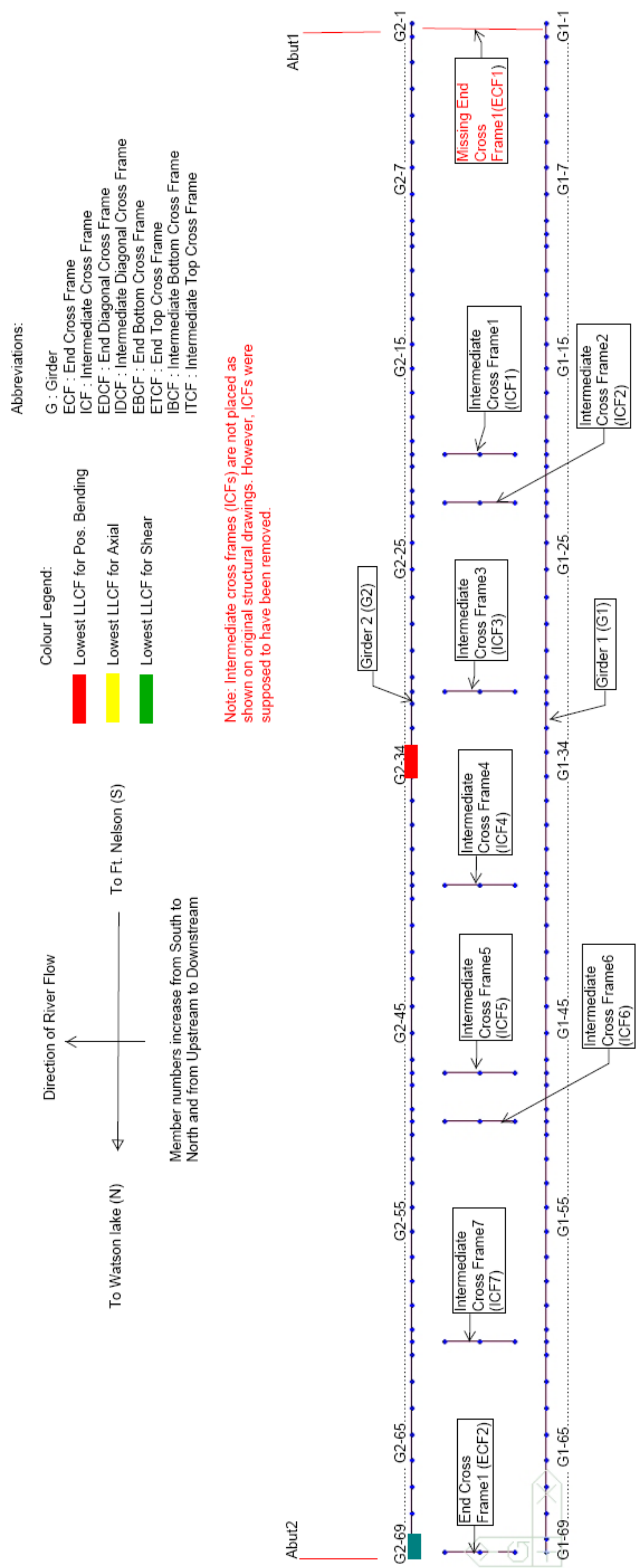

Figure 16: Toad River Bridge Plan View Naming Conventions.

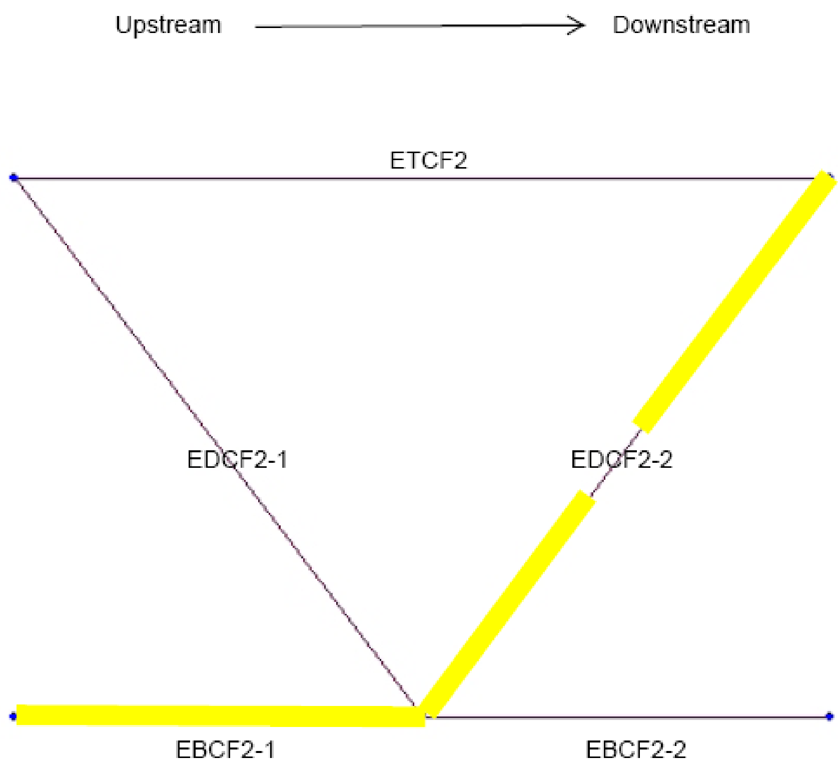

Figure 17: Toad River Bridge End Bracing View Naming Conventions.

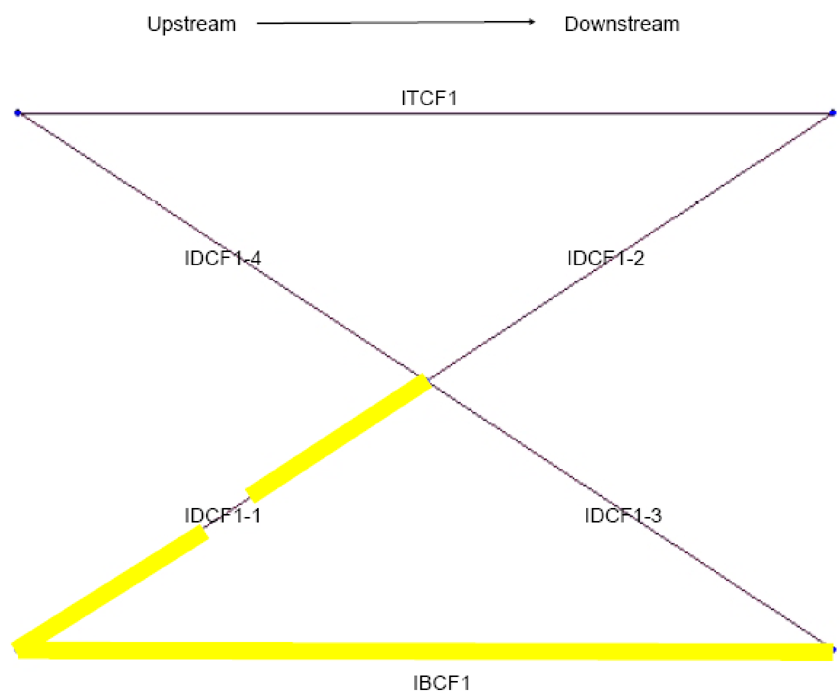

Figure 18: Toad River Bridge Intermediate Bracing View Naming Conventions.

5.2 Live Load Capacity Factors

The following Figures 19 to 24 show the live load capacity factors (LLCFs) that have been calculated along Toad River Bridge's members based on the requirements of Section 14 of the CHBDC S6-06. Specifically, LLCFs are calculated based on Clause 14.15.2, 'Ultimate Limit States'. LLCFs greater than 1 are deemed adequate for the prescribed live loading and LLCFs less than 1 generally require posting.

The location along a member which is considered to govern its design is the position where the member is most highly loaded relative to its resistance.

Due to overall bridge symmetry (i.e. no skew effects), symmetrical transient loading present, and simply supported bearing conditions, symmetrical force diagrams are produced within this bridge. In such cases, half-spans of the members need only to be shown.

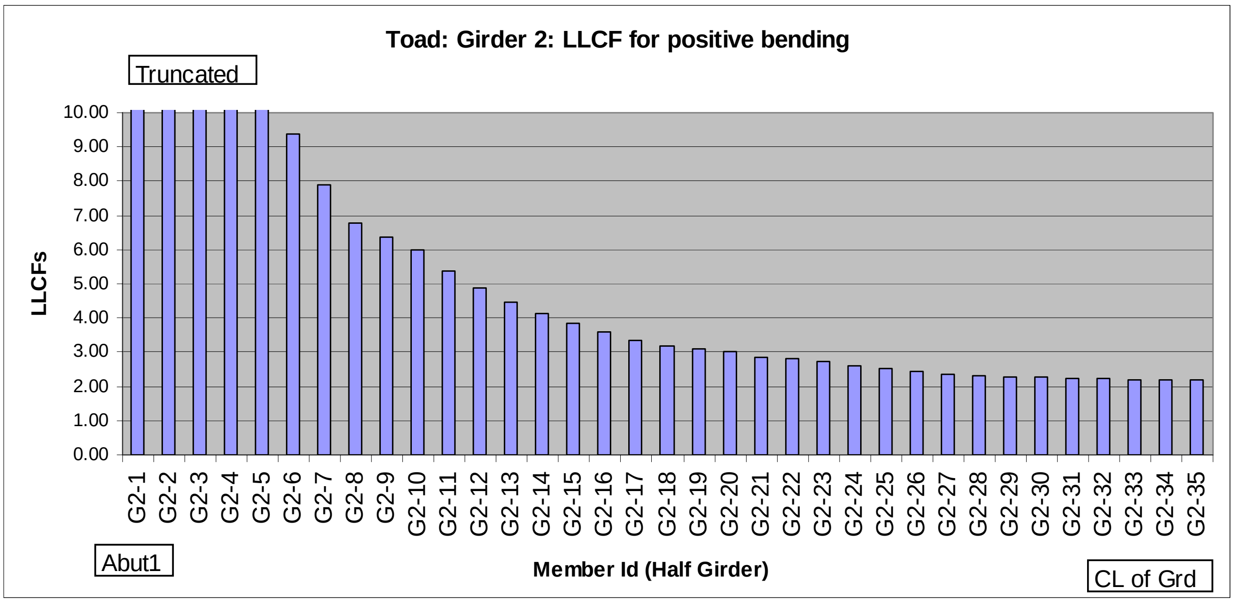

Figure 19: LLCF in Positive Flexure for a Half-Box Girder (Toad River Bridge).

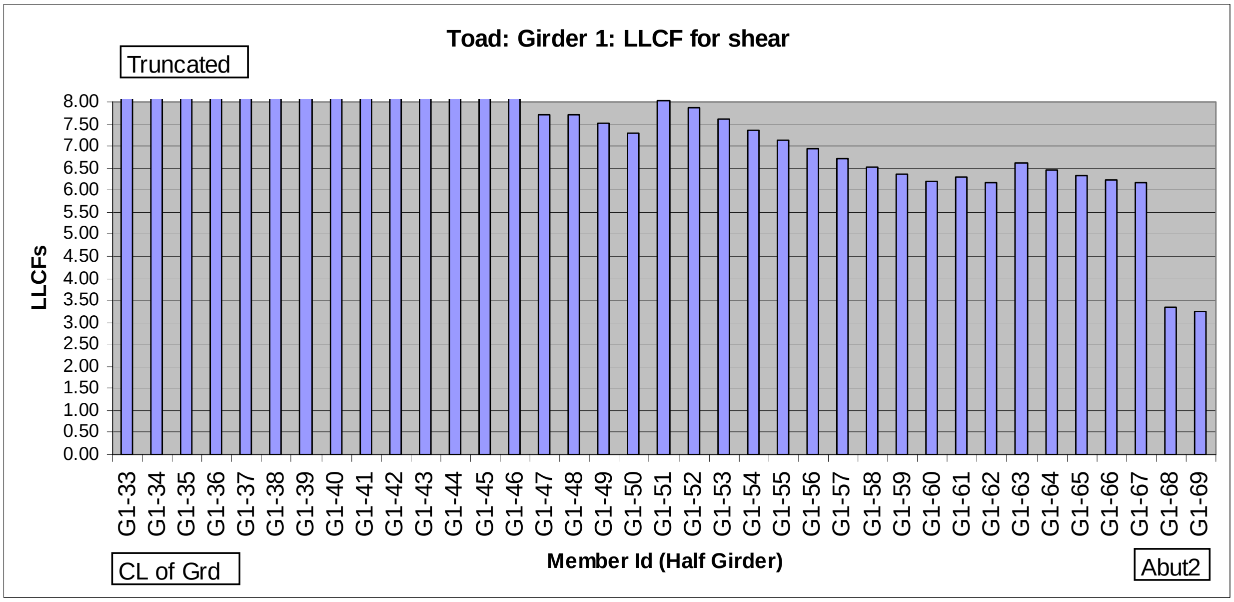

Figure 20: LLCF in Shear for a Half-Box Girder (Toad River Bridge).

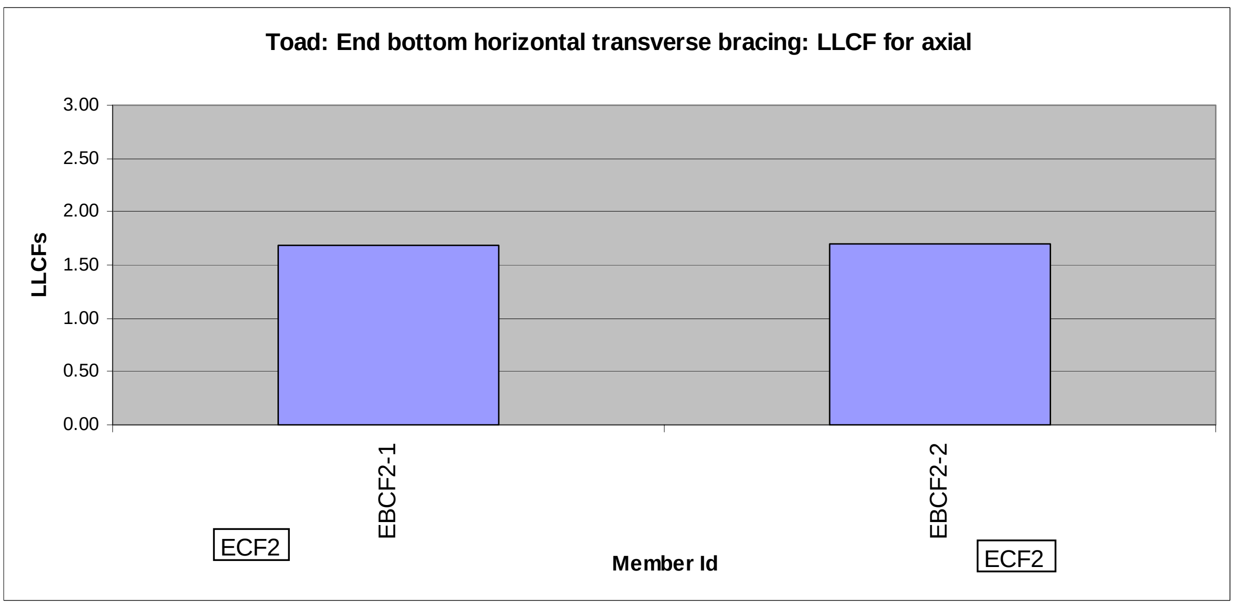

Figure 21: LLCF for Axial Force in End Bottom Horizontal Transverse Bracing (Toad River Bridge).

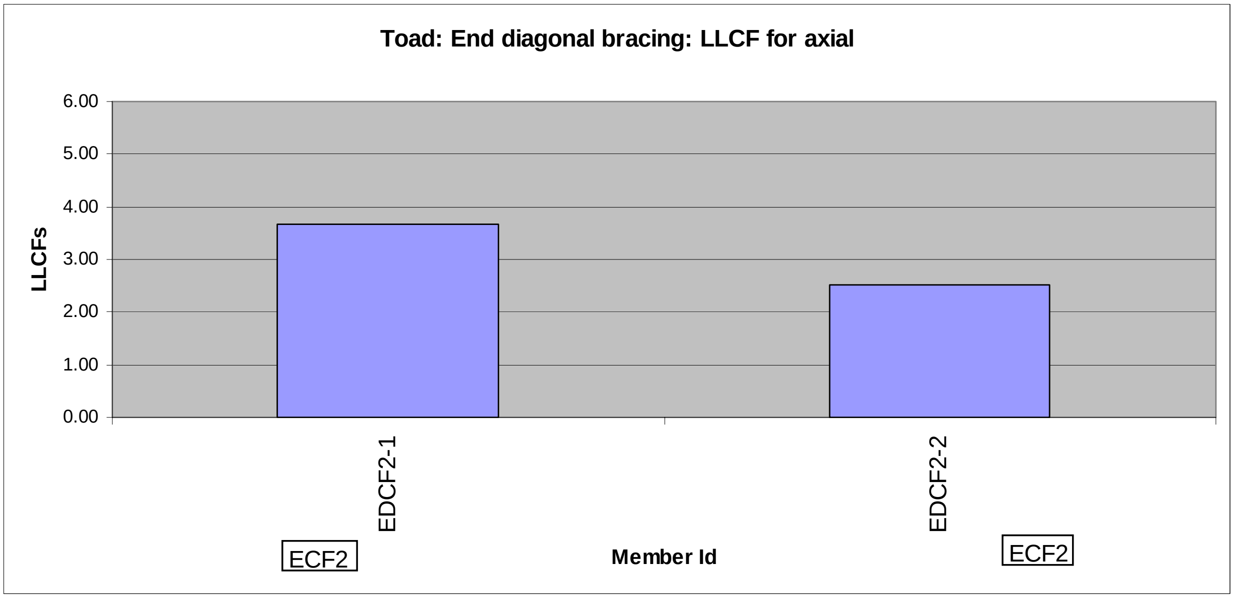

Figure 22: LLCF for Axial Force in End Diagonal Cross-Bracing (Toad River Bridge).

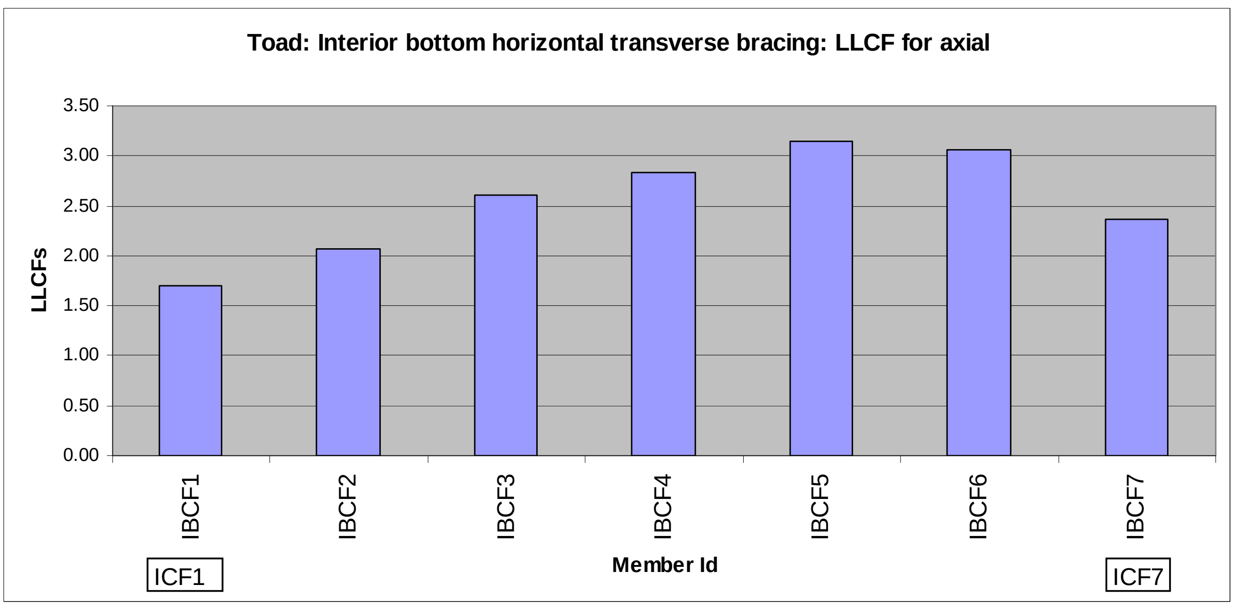

Figure 23: LLCF for Axial Force in All Intermediate Bottom Horizontal Transverse Bracing (Toad River Bridge).

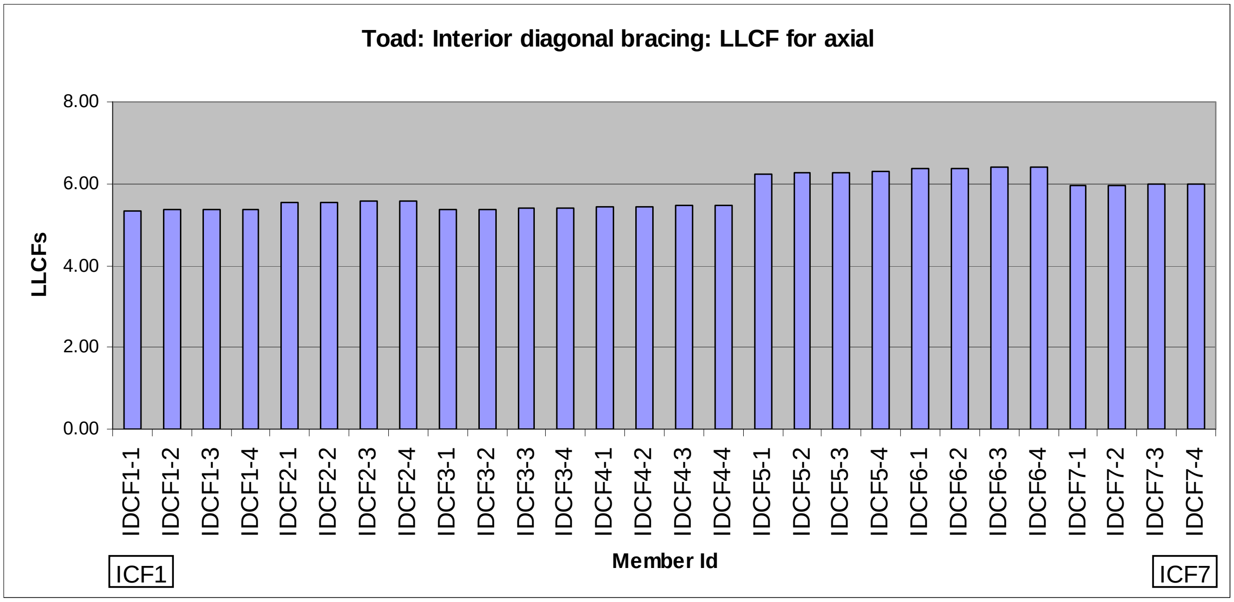

Figure 24: LLCF for Axial Force in All Intermediate Diagonal Cross-Bracing (Toad River Bridge).

For each type of member in this bridge, Table #2 provides its lowest calculated LLCF value and the location of that LLCF. Also, refer to Figures 16 to 18 for the locations of the lowest LLCFs in plan/elevation.

Title:	Toad River Bridge	Completed by:	A. Rafiquzzaman, Ph.D.	Date:	2/11/2010
Subject:	Lowest LLCFs	Checked by:	H. Hawk, M.Sc., P.Eng.	Date:	2/11/2010
				Revision:	3
Member ID	**Location**	**LLCFa Axial**	**LLCFp Bending(+)**	**LLCFs Shear**	
G2-34	Girder midspan		2.20		
G1-69	Girder end			3.25	
IBCF1	Interior cross frame 1	1.70			
IDCF1-1	Interior cross frame 1	5.34			
EBCF2-1	End cross frame 2	1.68			
EDCF2-2	End cross frame 2	2.51			

Table #2: Lowest LLCF for Each Type of Member Within Toad River Bridge.

5.3 Deck

The following Table #3 shows that the Toad River Bridge satisfies the requirements for using the empirical deck design method of Clause 8.18 of the CHBDC S6-06. Clause 14.14.1.3.1 states that if a bridge meets the requirements for using the empirical deck design method then the deck shall be deemed to have adequate resistance to meet the loading requirements of an Evaluation Level 1 truck/lane, assuming that the physical condition of the deck is adequate as well of course. Therefore, no further calculations for the deck are required except for checking the deck's cantilever overhangs for wheel load induced bending effects.

Since, however, deck thickness deterioration (i.e. delaminations) has been noted for this bridge, punching shear calculations are also included in Table #3. No punching shear issues were determined through the below calculations. Delcan has conservatively assumed here that Toad River Bridge's concrete deck is fully delaminated to below the level of the centroid of its top mat of reinforcement.

Title:	Toad River Bridge	Completed by:	P. Phillips, P.Eng.	Date:	2010/02/10	CHBDC
Subject:	Deck Design	Checked by:	H. Hawk, M.Sc., P.Eng.	Date:	2010/02/10	S6-06

(Empirical Deck Design) General							8.18.4.1
Clause a)	Composite slab with parallel supporting beams				OK		
Clause b)	Actual ratio of the spacing of the girders to the thickness of the slab		13.920				
Clause b) (max)	Maximum ratio of the spacing of the girders to the thickness of the slab		18		OK		
Clause c)	Actual spacing of the girders		2.9	[m]			
Clause c) (max)	Maximum spacing of the girders		4	[m]	OK		
Clause d)	Longitudinal negative moment deck rebar for continuous spans				N/A		
(Empirical Deck Design) Cast-in-place deck slabs							8.18.4.2
Transverse rebar ratio (min)		ρ_{min} =	0.003				
Top transverse rebar ratio		ρ_{top} =	0.014		OK		
Bottom transverse rebar ratio		ρ_{bottom} =	0.009		OK		
Longitudinal rebar ratio (min)		ρ_{min} =	0.003				
Top longitudinal rebar ratio		ρ_{top} =	0.006		OK		
Bottom longitudinal rebar ratio		ρ_{bottom} =	0.004		OK		
(Rigorous Method) General							14.14.1.3.2
Clause a)	Actual spacing of the girders for a slab panel			[m]			
Clause a) (max)	Maximum spacing of the girders for a slab panel		4.5	[m]	N/A		
Clause a)	Slab extends sufficiently beyond the external beams				OK		
Clause b)	Actual ratio of the spacing of the girders to the thickness of the slab		13.920				
Clause b) (max)	Maximum ratio of the spacing of the girders to the thickness of the slab		20		OK		
Clause c)	Actual minimum thickness of the slab		208.333	[mm]			
Clause c) (min)	Minimum thickness of the slab		150	[mm]	OK		
Clause d)	Spacing of cross frames/intermediate diaphragms		9.1	[m]			8.18.5
Clause d) (max)	Maximum spacing of cross frames/intermediate diaphragms		8	[m]	NG		
Clause e)	Edge stiffening				OK		8.18.6
Cantilever factored resistance (negative moment)							
At centreline of exterior girder							
Live Load Capacity Factor		F =	1.315		OK		
At edge of exterior girder flange							
Live Load Capacity Factor		F =	1.320		OK		
Deck punching shear resistance							8.9.3.4
Factored resistance of deck slab		R_r =	191.941	[kN]			
Factored truck wheel load		R_f =	182.525	[kN]	OK		

Table #3: Deck Calculation Summary for Toad River Bridge.

6.0 SEISMIC EVALUATION

Toad River Bridge is a single span bridge. Seismic performance can, therefore, be assessed by examining the available lengths of the bearing seats on this bridge's abutments. This will ensure that the bridge span will not drop if exposed to seismic loading.

Clause 4.4.5.1 of the CHBDC states that: "Bridges in Seismic Performance Zone 1 need not be analyzed for seismic loads, regardless of their importance and geometry. However, the minimum requirements specified in Clauses 4.4.10.2 and 4.4.10.5 shall apply." Based on Table A3.1.1 of the CHBDC, Toad River Bridge would be considered to be in Acceleration-Related Seismic Zone 0. Also, as a lifeline structure, Toad River Bridge would be considered to be in Seismic Performance Zone 2. For single span bridges in Seismic Performance Zone 2, analysis is also not required, but the attachment of the superstructure to the substructure must be able to resist 10% of the weight of the bridge applied as a horizontal load just above the level of the bearings (Clause 4.4.10.2) and the bearing seat length as defined in Clause 4.4.10.5 must be available at each expansion bearing.

For Toad River Bridge, the bearings consist of base plates bolted into the abutments, standard pot bearings, and bolting of the upper bearing plate to the superstructure. The superstructure weight is approximately 6600 kN; therefore the system needs to resist a horizontal load of 660 kN. The anchor bolt capacity is approximately 200 kN per bolt and there are 16 bolts. Pot bearings, on the drawings were to be designed for a longitudinal load of 20 kN and a vertical load of 1120 kN. The 20 kN value per bearing would be grossly inadequate for seismic; however, standard practice for manufacturers was to design for a minimum of 10% of vertical (i.e.: 112 kN per bearing). This gives a total horizontal resistance of 448 kN unfactored or about 700 kN factored, which is adequate. Upper bolt resistance is approximately 520 kN per bearing and there are 4 bearings. All values except for the bearing assemblies themselves are well in excess of the required resistance. At the expansion end of the bridge, bearing seat lengths need to be 305.3 mm long (Clause 4.4.10.5); the length provided at each expansion bearing is 625 mm.

7.0 SUMMARY OF RECOMMENDATIONS

There are no major load capacity issues with Toad River Bridge and no posting is required. No LLCFs for Toad River Bridge are less than 1 for the ultimate limit state. The existing transverse bracing (end and intermediate) between the box girders is not as per shown on the original structural drawings. It is important that this bracing be updated/changed in the field as soon as possible. Part of the 2009/2011 contract that Delcan has with PWGSC is to prepare sketches for the addition/removal of these secondary members.

APPENDIX A – ANALYSIS FILES (ON CD)

The CD accompanying this report includes the following documents:

1) .pdf file of this load rating report.
2) .txt printout of the Midas Civil model.
3) .mcb Midas Civil model.
4) .pdf printouts of all of the design spreadsheets.

PUBLIC WORKS AND GOVERNMENT SERVICES CANADA

PETERSEN CREEK BRIDGE
ALASKA HIGHWAY, km 678.6

STRUCTURAL LIVE LOAD CAPACITY FACTOR EVALUATION

Prepared by:

Delcan
TRANSPORTATION • INFORMATION TECHNOLOGY • WATER
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Hugh Hawk, M.Sc., P.Eng., Technical Director, Western Region
Stan Reimer, P.Eng., Senior Bridge Engineer
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Peter Phillips, P.Eng., Intermediate Bridge Engineer

March 2010

TABLE OF CONTENTS

EXECUTIVE SUMMARY

A detailed visual condition assessment for the Petersen Creek Bridge, km 678.6 along the Alaska Highway, was performed in September 2009. This condition assessment and Chapter 14, “Evaluation”, of the “Canadian Highway Bridge Design Code (CHBDC)”, CAN/CSA-S6-06, were used to evaluate the load carrying capacity of this existing bridge.

Petersen Creek Bridge was evaluated for CL1-W truck/lane live loading (Evaluation Level 1) in accordance with the CHBDC. Load factors were determined from Clause 14.13 based on target reliability indices (beta-factors) as determined from Clause 14.12 of the CHBDC. Resistance adjustment factors as determined from Clause 14.14.2 of the CHBDC were also used. These target reliability indices and load factors were reviewed by Public Works and Government Services Canada (PWGSC).

Live load capacity factors (LLCFs) for each superstructure member, as per Clause 14.15, were calculated (if and when applicable) for ultimate limit states in bending, shear, and axial. Punching shear was checked for this bridge’s deck.

All members of the existing bridge are capable of carrying their combination of factored dead and imposed truck/lane live loads. No bridge strengthening or posting is currently required.

1.0 INTRODUCTION

To determine if the bridge structure requires strengthening or posting, a bridge inspection was first carried out. The bridge's original structural drawings were reviewed in order to familiarize oneself with the structure, prior to an assessment in the field of its member conditions and overall stability. A load rating was then undertaken to establish which members are under-strength, if any. Section 14 of the Canadian Highway Bridge Design Code (CHBDC) outlines this process: target reliability indices (beta-factors) for each member of the bridge were determined and then dead and live load factors for each member of the bridge were chosen.

A model of the bridge, based on the requirements of Section 5 of the CHBDC, was created. Largest combined dead and live factored loads were determined for each member. The resistance of each member was compared to its respective applied force, taking into consideration the deterioration of the members as well. This comparison was used to determine which members were under-strength, if any. If members are determined by analysis to be under-strength currently, it is due to changes in design code requirements, deteriorating member conditions in the field, lack of conservatism in the original designs, or poor original construction practices. The load rating in combination with the field inspection program was used to classify the capacity problem (i.e. problems in flexure, shear, compression, tension, torsion, serviceability, fatigue, maintainability, durability, etc.).

The following original structural drawings were provided by PWGSC to assist in the structural evaluation of this bridge. The drawings provided a reference for dimensions, methods of original construction, substructure details (i.e. details not visible in the field), and material strengths, etc. However, the drawings, in general, are not complete. Drawings from rehabilitation works in the past were not included in the drawings package provided by PWGSC. Therefore, relevant assumptions needed to be made and other pertinent dimensions, data, etc. were obtained in the field during the visual bridge inspections.

Drawings Obtained:
Petersen Creek Bridge: Drawings 2135-11 to 18, dated 1963.

Missing Drawings:
Petersen Creek Bridge: New deck and bridge barrier drawings.

2.0 BRIDGE DESCRIPTION

An elevation photo of the Petersen Creek Bridge is shown in Figure 1, below.

Figure 1: Petersen Creek Bridge Photo.

Description of Petersen Creek Bridge:

a) Single span painted steel I-girders.
b) Reinforced concrete deck.
c) Reinforced concrete abutments.
d) Reinforced concrete spread footings under abutments.
e) Reinforced concrete traffic barriers.

Relevant dimensions of the bridge's elevation and cross-section are shown in Figures 2 and 3, respectively.

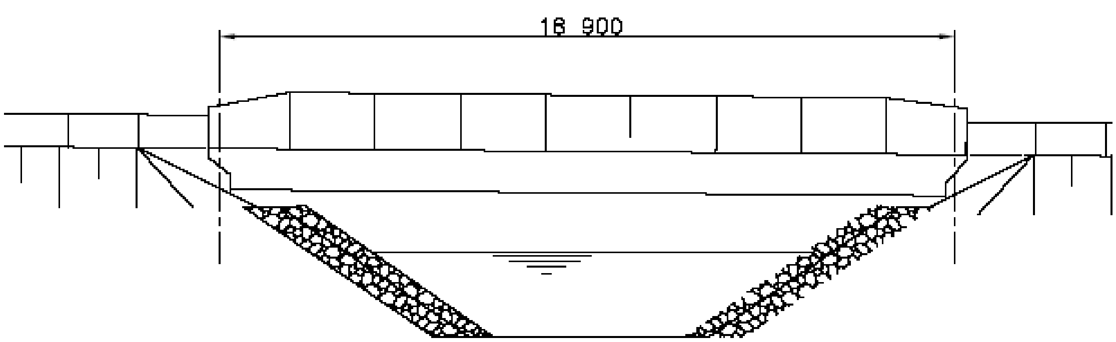

Figure 2: Elevation View of Petersen Creek Bridge.

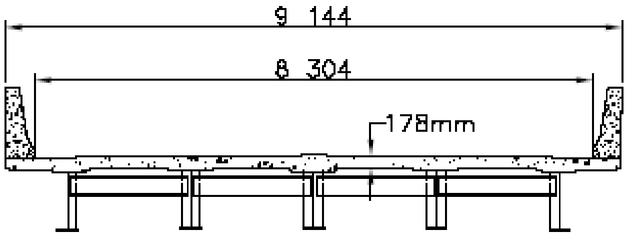

Figure 3: Cross-Section View of Petersen Creek Bridge.

3.0 CONDITION CONSIDERATION

The bridge is generally in good condition. Based on the site evaluation carried out in September 2009 of this structure, no significant loss of section or other bridge issues that could contribute negatively to the overall structural integrity of the bridge were found. For a detailed site condition rating of this bridge please refer to Delcan's 2009 bridge inspection report.

See Figures 4 to 11, below, for condition photos taken during Delcan's recent 2009 inspections of the Alaska Highway structures.

Locations of Deck Delaminations (Typical).

Figure 4: Deck.

Stiffening Plates Welded Along Centre Portions of Girders' Bottom Flanges (from the Time of Original Construction).

Figure 5: Exterior Girder.

Stiffening Plates Welded Along Centre Portions of Girders' Bottom Flanges. Bad Fatigue Detail. However, Not Enough Loading Cycles, Due to Low Traffic Volumes on the Alaska Highway, to Have Caused Any Damaged Currently. However, Fatigue Cracking Here Needs to be Continually Checked for During Bridge Inspection Sessions.

Figure 6: Plates Welded to Girder Bottom Flanges.

Figure 7: Abutment and Wingwall.

Figure 8: Exterior Girder Bearing.

Figure 9: Girder Lines.

Figure 10: Intermediate Bracing.

Figure 11: End Bracing.

4.0 EVALUATION PARAMETERS AND METHODOLOGY

4.1 Evaluation Procedures

Petersen Creek Bridge was evaluated for CL1-W truck/lane live loading (Evaluation Level 1) in accordance with the CHBDC. Load factors were determined from Clause 14.13 based on target reliability indices (beta-factors) as determined from Clause 14.12. Resistance adjustment factors as determined from Clause 14.14.2 of the CHBDC were also used. These target reliability indices and load factors were reviewed by PWGSC.

Live load capacity factors for each superstructure member, as per Clause 14.15, were calculated (if and when applicable) for ultimate limit states in bending, shear, and axial. Punching shear was checked for this bridge's deck.

Delcan modeled the Petersen Creek Bridge using "Midas Civil" (Midas) software utilizing a grillage-type model. The structural model was developed using beam elements for the superstructure members and deck and pin or roller supports, as applicable, as substructure elements. As substructure elements were in good condition with respect to structural integrity and member stability based on the September 2009 visual bridge inspections, substructure elements were not considered further in this live load capacity factor evaluation of the Petersen Creek Bridge. It was assumed that the substructure elements provide full support to the superstructure members.

Also, member-to-member connections (all joints) were assumed to be fully effective (i.e. in providing full capacity to transfer loads between the connected elements). All connections of secondary members to primary members were assumed to be pinned-pinned connections.

See Figure 12, below, for a rendered view of the Midas model.

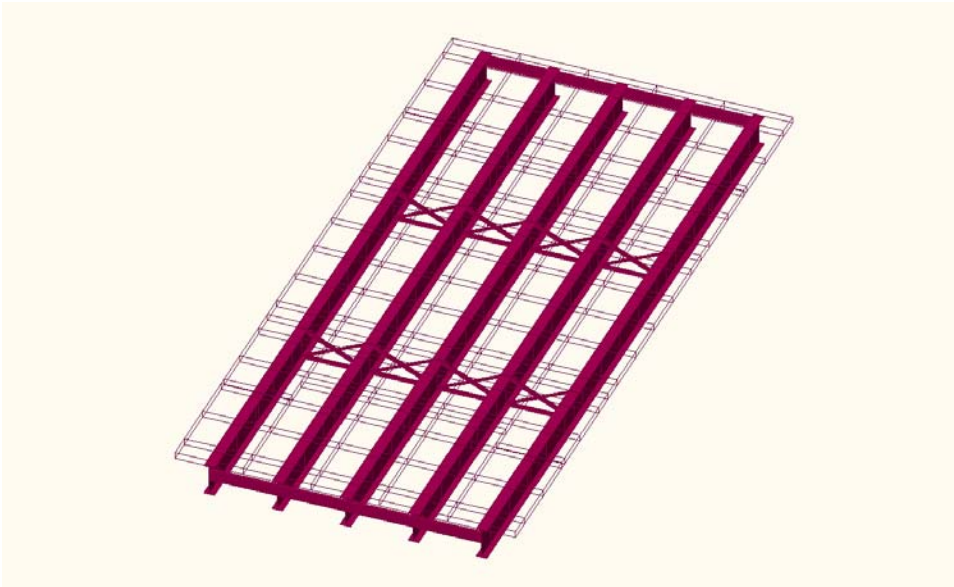

Figure 12: Petersen Creek Rendered Midas Civil Grillage-Type Model.

4.2 Reliability Indices and Load Factors

The following Table #1, below, provides:

a) System behaviour, element behaviour, and inspection level classifications.
b) Reliability indices as determined for "Normal Traffic".
c) An adjustment to the reliability index of 0.25 based on Clause 14.12.5 and recognizing this bridge as an important structure.
d) Dead load factors based on Clause 14.13.2.1.
e) Live load factors for normal traffic based on Clause 14.13.3.1.
f) A multilane factor for normal traffic based on Clause 14.9.4.2.
g) Dynamic load allowances for normal traffic based on Clause 14.9.1.7.

It was assumed that all members would be subjected to 'gradual failure with warning of probable failure' regardless of material or load carrying direction / capacity, with respect to 'Element Behaviour' – Clause 14.12.3 of the CHBDC.

Project: Alaska Highway Load Rating		Design: P. Phillips, PEng	Date: 2010/01/27
Subject: Petersen: Reliability Index & Load Factors		Check: H. Hawk, MSc, PEng	Date: 2010/01/27
	Element		
	Girders	Cross-Bracing	Deck
Category			
System Behaviour	S2	S3	S3
Element Behaviour	E3	E3	E3
Inspection Level	I2	I2	I2
Live Load Lateral Dist	Sophisticated Analysis	Sophisticated Analysis	Simplified Analysis
Reliability Index, β: CL1-W	3.00	2.75	2.75
Important Structure: β Increased by	0.25		
CL1-W	3.25	3.00	3.00
Factors			
DL	D1	D1	D2
α_D: CL1-W	1.08	1.07	1.14
DL	D2	D2	D1
α_D: CL1-W	1.16	1.14	1.07
$\alpha_{L\,(CL1\text{-}W)}$: CL1-W	1.56	1.49	1.49
Multi-Lane Factor	0.9		
1+DLA	1.25	1.25	1.40

Table #1: Reliability Indices and Load Factors.

4.3 Resistance Adjustment Factors

Resistance adjustment factors, as follows, were determined from Clause 14.14.2 of the CHBDC. The factored resistance of an individual structural component under consideration was multiplied by the appropriate resistance adjustment factor.

Structural Steel

a) Compression or tension on gross section: U = 1.01.
b) Shear: U = 1.02.

Composite – Slab on Steel Girder:

a) Bending: U = 0.96.
b) Shear connectors: U = 0.94.

Reinforced Concrete Deck:

a) Bending: U = 0.95.

4.4 Permanent Loads

The dead loads in the model include:

a) The full self-weight of the primary superstructure elements and secondary lateral bracing elements / diaphragms.
b) The weights of the steel I-girders were adjusted upwards by 12% to account for steel connections, gusset plates, transverse web stiffeners, bearing stiffeners, etc. as per Clause C14.8.2.1 of the CHBDC Commentary.
c) A 7.5" deck thickness was measured in the field. Therefore, the 7" concrete deck as shown on original drawings as provided by PWGSC was not used in the design.
d) Weight due to I-girder flange concrete haunches.
e) No deck overlay included.
f) Barriers in the field are not as shown on the drawings provided to Delcan. Actual field dimensions of barriers used. Field dimensions show that slightly smaller barriers than the typical 1116mm high by 420mm wide (at base) precast barriers are installed on this bridge.

4.5 Normal Traffic Live Loads

Petersen Creek Bridge was evaluated for Evaluation Level 1 CL1-W truck/lane live loading. Two lanes exist in the field (one Northbound lane and one Southbound lane) and therefore two traffic lanes were modeled, as specified in Clause 14.9.4.1. Appropriate multiple-lane load factors and dynamic load factors were applied to the truck and/or lane loading, when applicable.

4.6 Material Strengths

The material properties used in the resistance calculation processes were obtained from the structural drawings provided by PWGSC. In some instances, the structural drawings provided did not show some of or any of the original material strengths. In these cases, Clause 14.7 of the CHBDC was followed or, for structural steel members, where the year of construction of the bridge was known, the document entitled "Obsolete Canadian Structural Steel Grades, 1935 – 1971" as published in 2005 by the "Canadian Institute of Steel Construction (CISC)" was used. Petersen Creek Bridge was originally built in 1963. Dates of any subsequent modifications made to this bridge are unknown to Delcan.

The following values were used in the evaluation of the Petersen Creek Bridge:

a) Concrete deck and barrier compressive strength: 3000 psi (20.7 MPa), Drawing Number 2135-11.
b) Reinforcing deck and barrier steel yield strength: Table 14.2, Clause 14.7.4.4 of the CHBDC, "Medium or Intermediate" grade steel, 1963, 275 MPa.
c) Superstructure structural steel yield strength: ASTM A36, Drawing Number 2135-14, CISC, 1963, 36 ksi (248.2 MPa).
d) Superstructure structural steel ultimate strength: ASTM A36, Drawing Number 2135-14, CISC, 1963, 60 ksi (413.9 MPa).

5.0 RESULTS

5.1 Key-Plans/Elevations

The following key-plan/elevation diagrams indicate the naming conventions of the individual structural members within this bridge that were adopted for this load rating. All members referenced in this 'Results' section will therefore be referred to by their key-plan/elevation names.

See Figures 13 to 14 for key-plan/elevation drawings for the Petersen Creek Bridge:

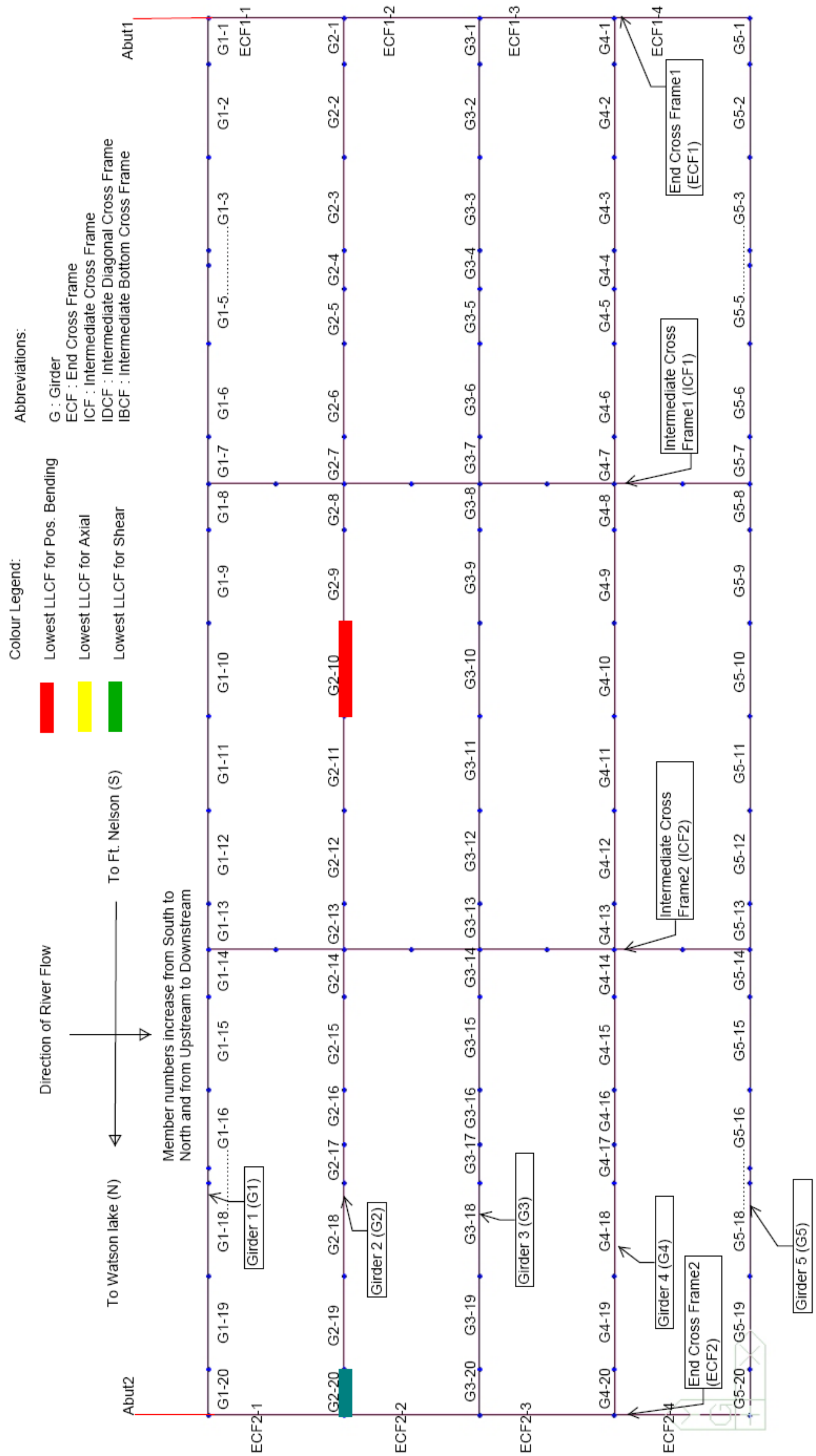

Figure 13: Petersen Creek Bridge Plan View Naming Conventions.

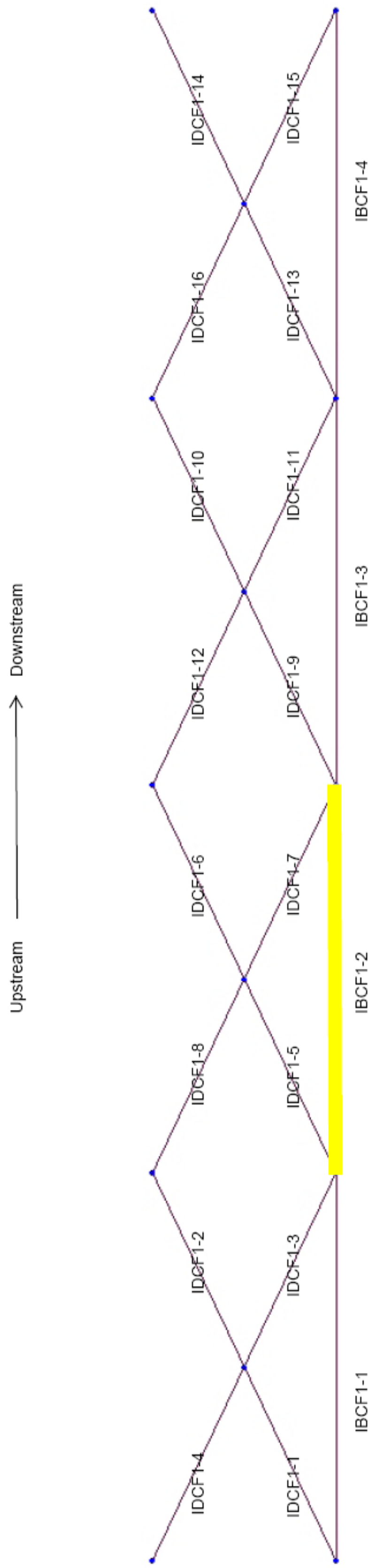

Figure 14: Petersen Creek Bridge Intermediate Bracing View Naming Conventions.

5.2 Live Load Capacity Factors

The following Figures 15 to 18 show the live load capacity factors (LLCFs) that have been calculated along Petersen Creek Bridge's members based on the requirements of Section 14 of the CHBDC S6-06. Specifically, LLCFs are calculated based on Clause 14.15.2, 'Ultimate Limit States'. LLCFs greater than 1 are deemed adequate for the prescribed live loading and LLCFs less than 1 generally require posting.

The location along a member which is considered to govern its design is the position where the member is most highly loaded relative to its resistance.

Due to overall bridge symmetry (i.e. no skew effects), symmetrical transient loading present, and simply supported bearing conditions, symmetrical force diagrams are produced within this bridge. In such cases, half-spans of the members need only to be shown.

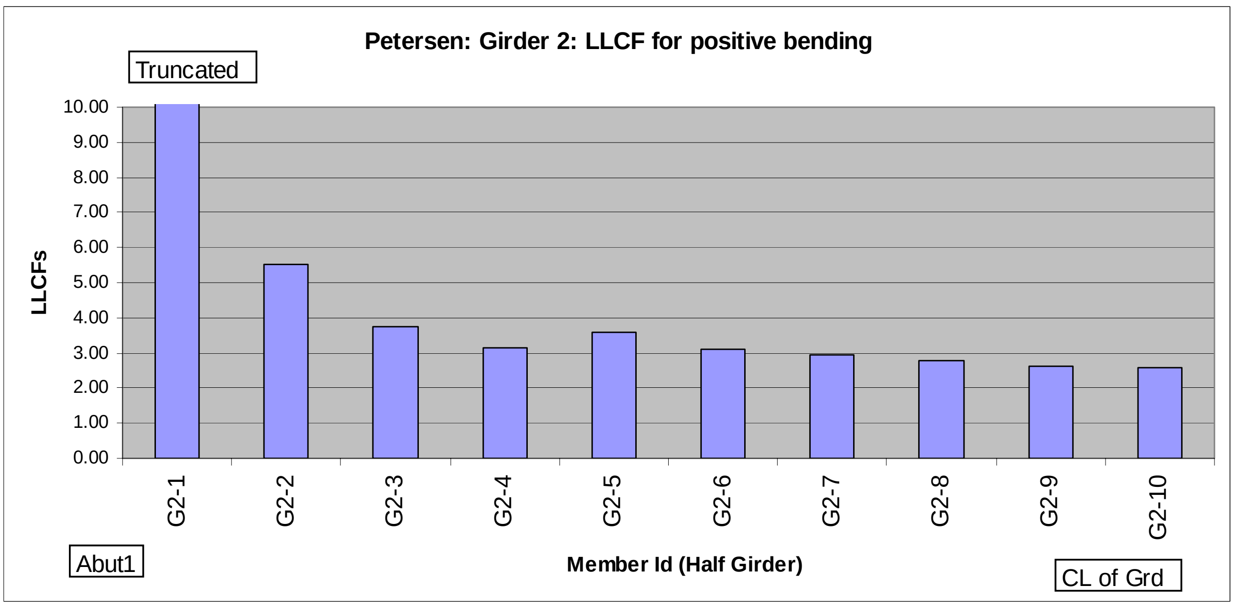

Figure 15: LLCF in Positive Flexure for a Half-Girder (Petersen Creek Bridge).

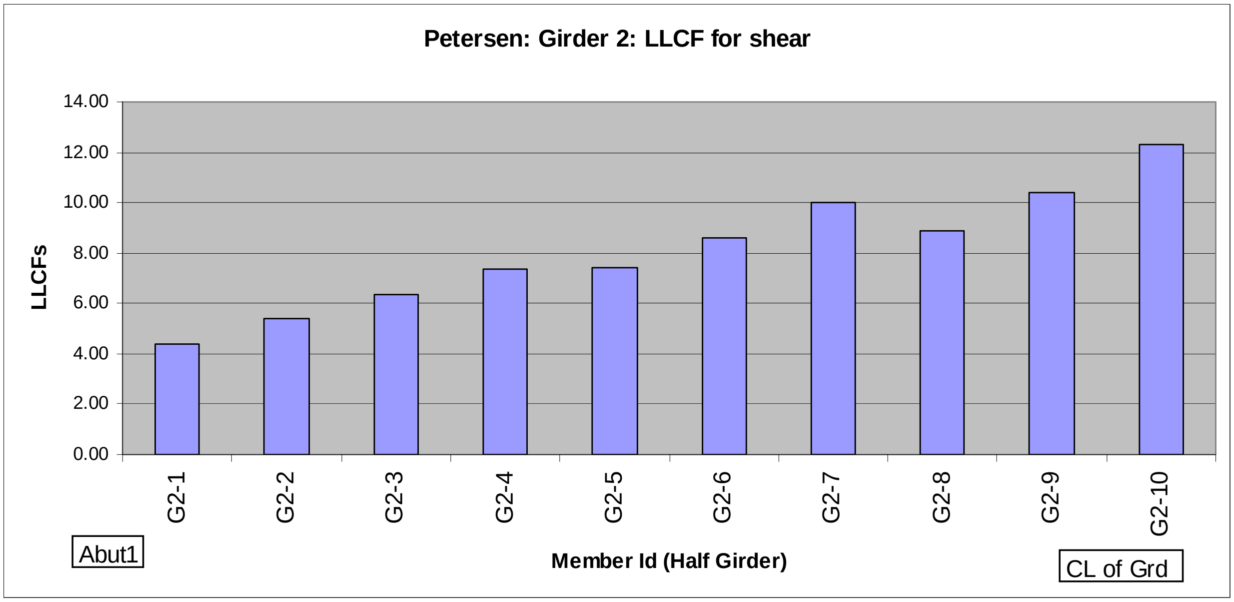

Figure 16: LLCF in Shear for a Half-Girder (Petersen Creek Bridge).

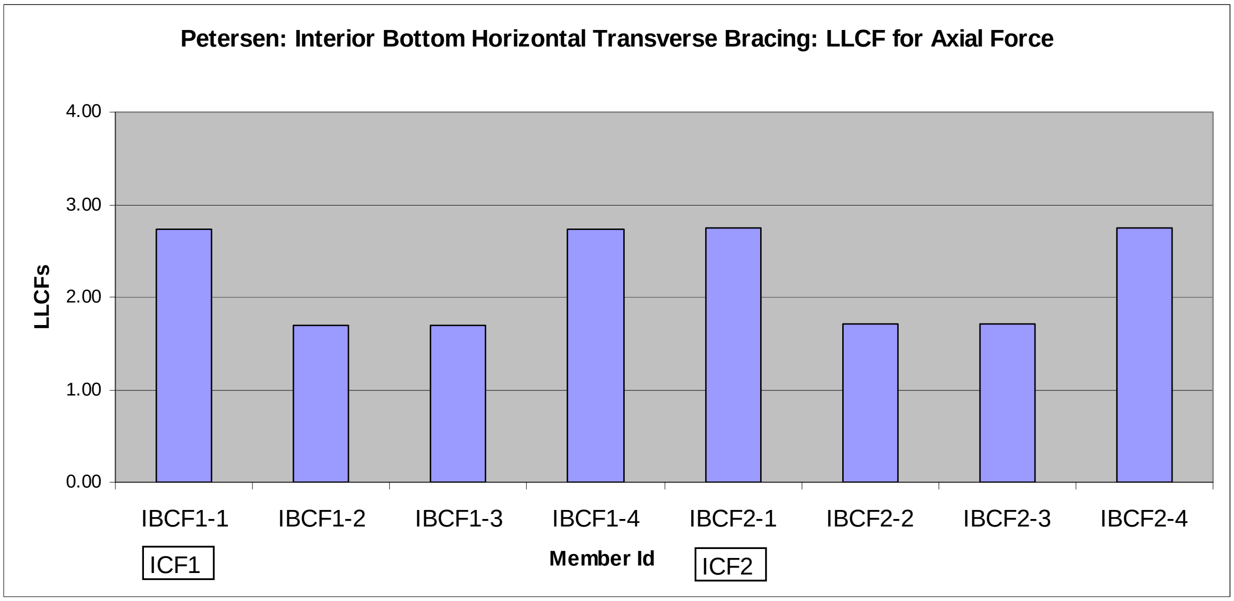

Figure 17: LLCF for Axial Force in All Intermediate Bottom Horizontal Transverse Bracing (Petersen Creek Bridge).

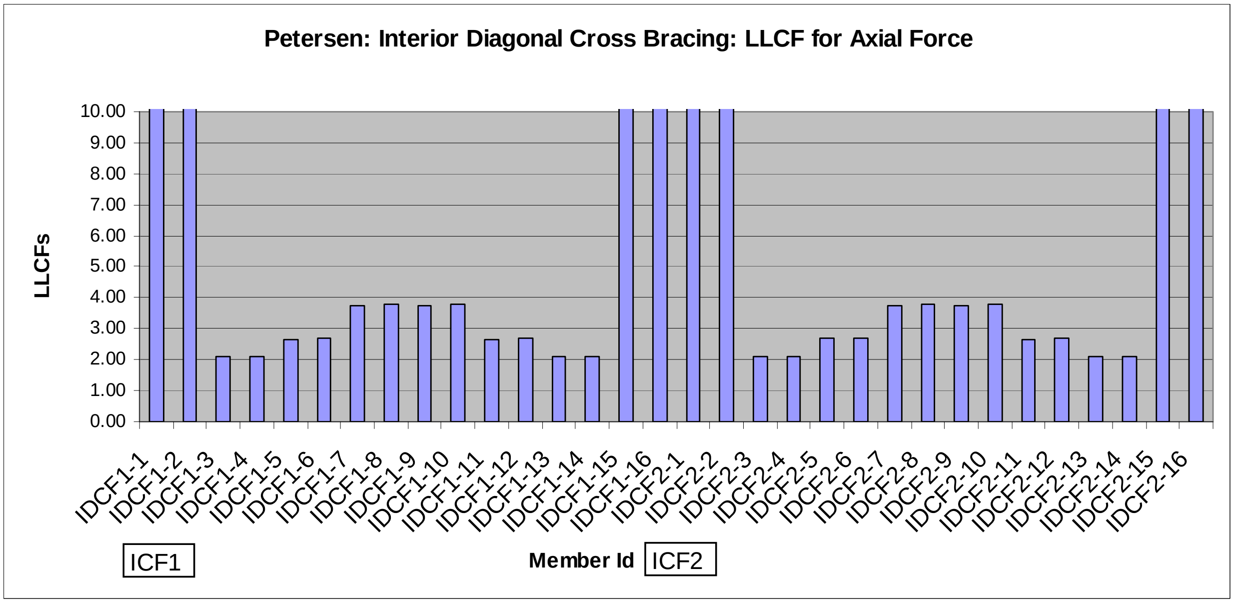

Figure 18: LLCF for Axial Force in All Intermediate Diagonal Cross-Bracing (Petersen Creek Bridge).

For each type of member in this bridge, Table #2 provides its lowest calculated LLCF value and the location of that LLCF. Also, refer to Figures 13 to 14 for the locations of the lowest LLCFs in plan/elevation.

Title:	Petersen Creek Bridge	Completed by:	A. Rafiquzzaman, Ph.D.		Date:	2/11/2010
Subject:	Lowest LLCFs	Checked by:	H. Hawk, M.Sc., P.Eng.		Date:	2/11/2010
					Revision:	2
Member ID	Location		LLCFa Axial	LLCFp Bending(+)		LLCFs Shear
G2-10	Interior girder midspan (ICF1-ICF2)			2.60		
G2-20	Interior girder at abutment (ICF2-Abut2)					4.25
IBCF1-2	Intermediate bottom cross frame (ICF2)		1.70			
IDCF2-3	Intermediate diagonal cross frame		2.08			

Table #2: Lowest LLCF for Each Type of Member Within Petersen Creek Bridge.

5.3 Deck

The following Table #3 shows that the Petersen Creek Bridge satisfies the requirements for using the empirical deck design method of Clause 8.18 of the CHBDC S6-06. Clause 14.14.1.3.1 states that if a bridge meets the requirements for using the empirical deck design method then the deck shall be deemed to have adequate resistance to meet the loading requirements of an Evaluation Level 1 truck/lane, assuming that the physical condition of the deck is adequate as well of course. Therefore, no further calculations for the deck are required except for checking the deck's cantilever overhangs for wheel load induced bending effects. Petersen Creek Bridge's deck cantilevers, however, are not long enough to be of any concern.

Since, however, severe deck thickness deterioration (i.e. delaminations) has been noted for this bridge, punching shear calculations are also included in Table #3. No punching shear issues were determined through the below calculations. Delcan has conservatively assumed here that Petersen Creek Bridge's concrete deck is fully delaminated to below the level of the centroid of its top mat of reinforcement.

Title:	Petersen Creek Bridge	Completed by:	P. Phillips, P.Eng.		Date:	2010/02/10	CHBDC
Subject:	Deck Design	Checked by:	H. Hawk, M.Sc., P.Eng.		Date:	2010/02/10	S6-06
(Empirical Deck Design) General							8.18.4.1
Clause a)	Composite slab with parallel supporting beams					OK	
Clause b)	Actual ratio of the spacing of the girders to the thickness of the slab			10.286			
Clause b) (max)	Maximum ratio of the spacing of the girders to the thickness of the slab			18		OK	
Clause c)	Actual spacing of the girders			1.829	[m]		
Clause c) (max)	Maximum spacing of the girders			4	[m]	OK	
Clause d)	Longitudinal negative moment deck rebar for continuous spans					N/A	
Diaphragms and Edge Stiffening							
Spacing of cross frames/intermediate diaphragms				6.299	[m]		8.18.5
Maximum spacing of cross frames/intermediate diaphragms				8	[m]	OK	
Edge stiffening						OK	8.18.6
(Empirical Deck Design) Cast-in-place deck slabs							8.18.4.2
Transverse rebar ratio (min)			ρ_{min} =	0.003			
Top transverse rebar ratio			ρ_{top} =	0.010		OK	
Bottom transverse rebar ratio			ρ_{bottom} =	0.010		OK	
Longitudinal rebar ratio (min)			ρ_{min} =	0.003			
Top longitudinal rebar ratio			ρ_{top} =	0.003		OK	
Bottom longitudinal rebar ratio			ρ_{bottom} =	0.004		OK	
Cantilever factored resistance (negative moment)			Cantilever Bending is Not Applicable				
At centreline of exterior girder							
Live Load Capacity Factor			F =				
At edge of exterior girder flange							
Live Load Capacity Factor			F =				
Deck punching shear resistance							8.9.3.4
Factored resistance of deck slab			R_r =	189.133	[kN]		
Factored truck wheel load			R_f =	182.525	[kN]	OK	

Table #3: Deck Calculation Summary for Petersen Creek Bridge.

6.0 SEISMIC EVALUATION

Petersen Creek Bridge is a single span bridge. Seismic performance can, therefore, be assessed by examining the available lengths of the bearing seats on this bridge's abutments. This will ensure that the bridge span will not drop if exposed to seismic loading.

Clause 4.4.5.1 of the CHBDC states that: "Bridges in Seismic Performance Zone 1 need not be analyzed for seismic loads, regardless of their importance and geometry. However, the minimum requirements specified in Clauses 4.4.10.2 and 4.4.10.5 shall apply." Based on Table A3.1.1 of the CHBDC, Petersen Creek Bridge would be considered to be in Acceleration-Related Seismic Zone 0. Also, as a lifeline structure, Petersen Creek Bridge would be considered to be in Seismic Performance Zone 2. For single span bridges in Seismic Performance Zone 2, analysis is also not required, but the attachment of the superstructure to the substructure must be able to resist 10% of the weight of the bridge applied as a horizontal load just above the level of the bearings (Clause 4.4.10.2) and the bearing seat length as defined in Clause 4.4.10.5 must be available at each expansion bearing.

For Petersen Creek Bridge, the bearings consist of base plates and low rockers bolted into the abutments and full weldment of the upper bearing plate to the superstructure. The superstructure weight is approximately 1350 kN; therefore the system needs to resist a horizontal load of 135 kN. The anchor bolt capacity is approximately 75 kN per bolt and there are 10 bolts. Weld resistance is approximately 350 kN per bearing and there are 5 bearings. All values are well in excess of the required resistance. At the expansion end of the bridge, bearing seat lengths need to be 231.5 mm long (Clause 4.4.10.5); the length provided at each expansion bearing is 375 mm.

7.0 SUMMARY OF RECOMMENDATIONS

There are no major load capacity issues with Petersen Creek Bridge and no posting is required. No LLCFs for Petersen Creek Bridge are less than 1 for the ultimate limit state.

APPENDIX A – ANALYSIS FILES (ON CD)

The CD accompanying this report includes the following documents:

1) .pdf file of this load rating report.
2) .txt printout of the Midas Civil model.
3) .mcb Midas Civil model.
4) .pdf printouts of all of the design spreadsheets.