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November 24, 2015
File: 20152030.00.A.01.00

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Chief Engineer, Engineering Branch
Ministry of Forests, Lands and Natural Resource Operations
PO Box 9510, Stn Prov Govt
3rd Floor - 1520 Blanshard Street
Victoria BC V8W 3K2

Re: REVIEW OF 2014 CANADIAN HIGHWAY AND BRIDGE DESIGN CODE WITH RESPECT TO MFLNRO DESIGN STANDARDS

Dear Mr. Chow:

The **Ministry of Forests Lands and Natural Resource Operations (Ministry)** retained **Associated Engineering** to review the 2014 Canadian Highway Bridge Design Code (S6-14) and identify changes from the 2006 Code that may affect Ministry design/construction standards or the design of single lane forestry bridges. The work included:

- A review of all sections except for:
  - Section 7 - Buried Structures.
  - Section 13 - Moveable Bridges.
  - Section 15 - Rehabilitation and Repair.
  - Section 16 - Fibre-reinforced Structures.
  - Section 17 - Aluminum Structures.
- Provide a brief synopsis of changes to Section 4 – Seismic Design; however, since seismic design is not typically required for forestry bridge.

We have attached a tabular summary of the changes along with our recommendations to this letter. Based on the review, the Ministry should consider the following:

1. With the withdrawal of CSA G164 – Hot Dip Galvanizing of Irregularly Shaped Articles, an alternative reference is required (S6-14 Cl. 1.2).
2. Cl. 3.8.3.2 introduces a new class of owner specified design vehicles, “Special Loads”. Since the Ministry requires design of bridges for owner specific design loads e.g. L100, L150 and L165, we recommend that the Ministry clarify the Live Load Factors to be used when designing bridges for these loads. To be consistent with current design practices, we recommend classification of these that these loads as Normal Traffic (S6-14 Table 3.2).

3. Cl. 10.7.3 recommends the incorporation of jacking diaphragms to facilitate bearing replacement. The Ministry may consider requiring bridges over a certain length, or where laminated bearing pads are used, include provisions for jacking to allow bearing replacement.
4. Cl. 10.11.8.3.4 does not allow the use of channel shear connectors with full depth precast concrete deck panels. This is a typical retrofit detail adopted by the Ministry and we recommend that the Ministry not adopt this requirement. Further, the resistance of channel shear connectors as defined within this clause is not appropriate for the larger channel sections typically used in retrofit projects. We recommend that the Ministry develop standard channel shear connector details and provide guidance on how to calculate the resistance of the channel shear connectors.
5. The design guidelines included in Cl. 10.16 for the design of orthotropic steel decks are not applicable to the design of typical all-steel portable bridge decks. While there is a reliance on the Standard Drawings, which mandate a minimum deck thickness (15.9 mm), the Ministry may consider supplementing the drawings with additional design guidelines.
6. While Cl. 10.17.2.7 does not change the design criteria for stud shear connectors, the Ministry may consider explicitly stating the fatigue design criteria similar to that stated for the design of steel plate girders on drawings STD-EC-030-01 and 040-01.
7. Consider developing guidance for the evaluation of bridges in accordance with Section 14. This would ensure consistency between various evaluators with respect to classification of vehicles, behaviour of various structure and material types and calculation of section resistances. This would likely take the form of prescriptive guidelines such as:

    Evaluation of single span twin steel girder bridges.
    - *Evaluation vehicle:*
        - *BCL625 – Classification: Normal Traffic*
        - *L100, L150, L165 – Classification: Permit Annual Traffic*
    - *System Behaviour: S1*
    - *Element Behaviour (Flexure): E3*
    - *Element Behaviour (Shear): E3*
    - *Inspection Level:*
        - *Annual Routine Inspection: INSP2*
        - *Close Proximity Inspection: INSP3*

    We also recommend the Ministry provide guidance on when the evaluation of deck and substructure components, which typically don't meet code design criteria.

We trust that this brief letter and attached tabular summary addresses the Ministry's needs. If you have any questions, please call me.

Yours truly,

PROFESSIONAL
PROVINCE OF BRITISH COLUMBIA
J. HENLEY
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2015.11.24

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JH/skn

Attachments
- Tabular Summary of Review of S6-14

Section 1 – General

Only one clause has been changed with Section 1 of S6-14.

1.2 Reference publications CAN/CSA-G164-M92 (withdrawn). Hot dip galvanizing of irregularly shaped articles.	**Change(s) from S6-06:** Reference withdrawn. **Recommendation to Ministry:** We recommend the Ministry consider referencing the following ASTM standards: 1. ASTM A123 / A123M - Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products 2. ASTM A385 / A385M - Standard Practice for Providing High-Quality Zinc Coatings (Hot-Dip) 3. ASTM A143 / A143M - Standard Practice for Safeguarding Against Embrittlement of Hot-Dip Galvanized Structural Steel Products and Procedure for Detecting Embrittlement

Section 2 – Durability

No changes have been made to this section in S6-14.

Section 3 – Loads

In addition to minor editorial changes made in Section 3 of S6-14, S6-14 introduces the concept of a “Special Load” which is intended to represent owner specific vehicles that exceed the CL-625 loading.

3.2 Definitions **Axle unit** — any single-axle, tandem, or tridem. **Short Span** — a span where axle unit loads govern design. **Special Loads** — permit vehicle loads to transport indivisible loads, or military loads, on a designated route with or without controls and supervision, that exceed the CL-625 loading. Note: These do not include vehicles under bulk haul permit programs. **Tandem** — any two consecutive axles whose centres are more than 1.00 m apart, articulated from a common attachment to the vehicle, and designed to automatically equalize the load between the two axles. **Tridem** — any three consecutive axles that have their consecutive centres equally spaced at more than 1.00 m apart, articulated from a common attachment to the vehicle and designed to automatically equalize the load between the three axles.	**Change(s) from S6-06:** These are new definitions added to clause 3.2. **Recommendation to Ministry:** We recommend that the Ministry adopt the changes however; design guidelines should clearly state that the BCFS L100, L150 and L165 are classified as normal traffic for design purposes.

3.5 Load Factors and Load Combinations

3.5.1 General

Table 3.1
Load Factors and Load Combinations

Loads	Permanent loads D	E	P	Transitory loads L*	K	W	V	S
Ultimate limit states‡								
ULS Combination 1	α_D	α_E	α_P	Table 3.2	0	0	0	0
ULS Combination 2	α_D	α_E	α_P	Table 3.2	1.15	0	0	0
ULS Combination 3	α_D	α_E	α_P	Table 3.2	1.00	0.45§	0.45	0
ULS Combination 4	α_D	α_E	α_P	0	1.25	1.40§	0	0
ULS Combination 5	α_D	α_E	α_P	0	0	0	0	0
ULS Combination 6**	α_D	α_E	α_P	0	0	0	0	0
ULS Combination 7	α_D	α_E	α_P	0	0	0.75§	0	0
ULS Combination 8	α_D	α_E	α_P	0	0	0	0	0
ULS Combination 9	1.35	α_E	α_P	0	0	0	0	0

Change(s) from S6-06:
Table modified to include special loads.

Wind load factor in ULS 4 is reduced from 1.5 to 1.4.

Wind load factor in ULS 7 is reduced from 0.8 to 0.75 when combine wind load and ice accretion load.

Recommendations to Ministry:
We recommend the Ministry adopt the changes.

Table 3.2
Live load factors ultimate limit states
(See Clause 3.5.1 and Table 3.1.)

Load	Live load factor: Normal traffic	Special loads mixed with normal traffic: Short spans	Special loads mixed with normal traffic: Other spans	Special loads travelling alone on bridge under supervision: Short spans	Special loads travelling alone on bridge under supervision: Other spans
ULS combination 1	1.70*	1.70	1.50	1.50	1.35
ULS combination 2	1.60	1.60	1.40	1.40	1.25
ULS combination 3	1.40	1.40	1.25	1.25	1.10

*Also to be applied to the barrier loads.

Change(s) from S6-06:
New definition of live load factors that includes owner specified special loads.

Recommendations to Ministry:
We recommend that the Ministry adopt the changes however; design guidelines should clearly state that the BCFS L100, L150 and L165 are classified as normal traffic for design purposes.

3.8 Live Loads

3.8.3.2 Special Loads

3.8.3.2.1 Special Trucks

Special Trucks shall be provided by the authorities responsible for their operation and shall include detailed configurations of the design vehicles including all inter-axle spacings, axles loads, distance between wheel lines, overall width of the vehicles, and sizes of wheel foot prints in loaded condition. Clearance envelope of the Special Truck shall be assumed to extend 0.30 m on each side beyond the overall width of the special trucks.

3.8.3.2.2 Special Lane Load

The Special Lane load consists of a Special Truck with each axle reduced to 85% of its value, superimposed within a uniformly distributed load of 9 kN/m.

Change(s) from S6-06:
To accommodate owner specified design vehicles, S6-14 includes a new live load designation - special loads.

Recommendations to Ministry:
We recommend that the Ministry adopt the changes however; design guidelines should clearly state that the BCFS L100, L150 and L165 are classified as normal traffic for design purposes.

3.8.8 Barrier Loads

3.8.8.1 Traffic Barriers

Table 3.7
Loads on Traffic Barriers

Performance level	Transverse load, kN	Longitudinal load, kN	Vertical load, kN
TL-1	25	10	10
TL-2	100	30	30
TL-4	210	70	90
TL-5	50	20	10

Change(s) from S6-06:
S6-14 has renamed the barrier performance levels to correspond to those used by AASHTO. Further, they have added a lower performance level barrier for use on low volume roads. The loading for TL-2, TL-4 and TL-5 are the same as the previously designated PL-1, PL-2 and PL-3 barriers respectively.
Recommendations to Ministry:
The TL-1 barrier loads reflect the Minstry's CL-2 barrier design criteria. See Section 12 for further discussion and recommendations.

Section 4 – Seismic Design

S6-14 has preserved the Force-based Design (FBD) approach to seismic design from S6-06 for areas of low seismicity and simple structural systems. For most types of bridges, the new Performance-based Design (PBD) approach applies. The PBD approach is philosophically different from FBD, seeking to directly quantify the nonlinear stresses and strains in structure elements resulting from earthquake-induced displacements, rather than assume linear behaviour for an assumed structure ductility level.

The new PBD approach to seismic design may apply to several ‘special’ bridge crossings/replacements in the future, but does not generally apply to standard single-span, single-lane forestry bridges.

The previously developed seismic design guidelines remain applicable for typical single span forestry bridges.

Section 5 – Methods of Analysis

Section 5 of S6-14 has undergone extensive revisions, the majority of which are focussed on the simplified method of analysis for longitudinal load effects. The simplified method of analysis is based on the beam analogy method in which the bridge is considered as a beam for determining the longitudinal distribution of load effects. The transverse distribution of the longitudinal load effects across the bridge width is determined by multiplying the one-lane longitudinal load effect by two factors:

- F_T which accounts for the transverse redistribution among the longitudinal elements
- F_S, which account for the effects of skewed geometries.

Notwithstanding these extensive revisions, Section 5 remains of limited value when considering the analysis of typical single lane forestry bridges since:

- It focusses on providing simplified methods for analysing multi-lane bridges with limited/no guidance provided for single lane bridges.
- Twin girder and concrete/timber deck bridges are statically determinate and calculation of live loads can be completed using simple statics.

We therefore recommend the Ministry continue to rely on the previously developed guidelines for calculating live load distribution in shear-connected slab bridges and using statics to calculate live load distribution on twin steel girder bridges.

As noted, S6-14 Section 5 has increased the applicability of the simplified methods of analysis to structures that have large skews. The effect of skew on typical single lane twin steel girder concrete/timber deck bridges is typically nominal and can be ignored. The work completed by Baidar Bakht indicated a nominal magnification of forces due to skewed effects on shear-connected slab bridges with a recommendation that the effects be ignored.

Given the extensive revisions to this Section, we have not presented a comparison of clauses with associated comments and recommendations. We recommend the Ministry adopt of Section 5 for analysis multi-lane bridges and multi-girder steel and concrete/timber deck bridges. The Ministry should continue to use the guidance developed by Baidar Bakht for the analysis of single lane shear-connected slab bridges.

The Ministry should also recognise that there is still a lack of clarity when considering how to address the design of multi-lane forestry bridges especially when considering off-highway loads which are wider than highway legal loads. We recommend that the Ministry develop design guidelines for multi-lane bridges.

Section 6 – Foundations and Geotechnical Systems

The approach to geotechnical resistance for design of foundations and geotechnical systems has expanded from S6-06 to include a new coefficient called the ‘Consequence Factor’. This coefficient is applied to the geotechnical resistance based on the consequence of limit state exceedance. Classifications are ‘high’, ‘typical’ and ‘low’, and have values of 0.9, 1.0 and 1.15, respectively.

Based on the definitions of the three consequence factors, standard forestry bridges will generally receive a factor of 1.0 to the geotechnical resistance. In addition, foundation engineering practises in the forestry sector do not adhere to the S6-06 code, and as such, changes to this section of S6-14 have no effect of the design of typical spread footings or steel piles.

We therefore recommend the Ministry do not change the current approach to the design of precast concrete footing substructures or steel piles. When considering the design of other foundations systems, we recommend the adoption of S6-14.

Section 8 – Concrete Structures

No significant changes were made to Section 8 and we recommend the Ministry’s design and fabrications standards for concrete components remain unchanged. We recommend the Ministry adopt Section 8 for the design of other concrete components not covered within the Standard Drawings with the possible exception of cover requirements.

Below, we have highlighted the non-grammatical changes to Section 8 for reference.

8.10 Strut-and-tie Model

8.10.5.1 Stress Limits in Node Regions

……

(c) $0.76\alpha_1\varphi_c f_c'$ in node regions anchoring tension ties in more than one direction.

Change(s) from S6-06:
This Clause was modified by the Supplements to S6-06 and has not subsequently been changed in S6-14. The revision relates to the stress limits change in nodal regions from $\alpha_1 f_c'$ to $0.76\alpha_1\varphi_c f_c'$.

Recommendations to Ministry:
We recommend the Ministry adopt the change.

Table 8.5 Minimum Concrete Covers and Tolerances

Environmental exposure	Component	
De-icing chemicals; spray or surface runoff containing de-icing chemicals; marine spray	(1)	Top of bottom slab for rectangular voided deck
	(2)	Top surface of buried structure with less than 600 mm fill† Top surface of bottom slab of buried structure

Change(s) from S6-06:
Components (2)(a) and (2)(b) are now combined into a single Component (2). This adjustment was by the Supplements to S6-06 and has not subsequently been changed in S6-14.

Recommendations to Ministry:
This change does not affect the minimum concrete covers specified on the standard drawings.

8.14.6 Maximum Spacing of Reinforcement for Shear and Torsion

Change(s) from S6-06:
Redundant Subclauses (a) and (b) removed.

Recommendations to Ministry:
We recommend the Ministry adopt the change.

Section 9 – Wood Structures

A number of changes have been made to Section 9; however, we have only commented on changes that effect the calculation of sawn stringer and glulam girder section resistances, which are required to complete load evaluation calculations as the Ministry does not typically incorporate timber elements into new structures with the exception of timber decks for which standard designs exist. For the design of other timber structures, we recommend adoption of S6-14.

9.6.1 Flexural Resistance The factored resistance, M_r , of glued-laminated members shall be the lesser of $M_r = \phi k_d k_{\ell s} k_m f_{bu} S$ and $M_r = \phi k_d k_m k_{sb} f_{bu} S$	**Change(s) from S6-06:** Separate formula is given for glue-laminated members. This Clause was modified by the Supplements to S6-06 and has not subsequently been changed in S6-14. The calculation of the flexural capacity of glulam girders now considers two cases: 1) Flexural capacity dependant on load duration, lateral stability and load sharing effects. 2) Flexural capacity dependant on load duration, load sharing and size effects. **Recommendations to Ministry:** We recommend the Ministry adopt the change.
9.6.2 Size Effect The value of k_{sb} for glued-laminated members shall be calculated as follows: $k_{sb} = \left(\frac{130}{b}\right)^{\frac{1}{10}} \left(\frac{610}{d}\right)^{\frac{1}{10}} \left(\frac{9100}{L}\right)^{\frac{1}{10}} \le 1.3$ Where: b = the beam width (for single-piece laminations) or the width of the widest piece (for multiple-piece laminations), mm. d = the beam depth, mm. L = the length of beam segment from point of zero moment to point of zero moment, mm. The value of k_{sb} for sawn wood members shall be obtained from Table 9.4. The value of k_{sb} for members other than glued-laminated or sawn wood members shall be 1.0.	**Change(s) from S6-06:** New formula for size effect on glue-laminated member is given. This Clause was modified by the Supplements to S6-06 and has subsequently been changed again in S6-14. **Recommendations to Ministry:** We recommend the Ministry adopt the change.
9.8.1 General $\left(\frac{P}{P_r}\right)^2 + \frac{M_c}{M_r} \le 1.0$ (for uniaxial bending)	**Change(s) from S6-06:** The formula for uniaxial bending revised. This Clause was modified by the Supplements to S6-06 and has not subsequently been changed in S6-14. **Recommendations to Ministry:** We recommend the Ministry adopt the change.

9.10 Compression at an Angle to Grain The factored compressive resistance, R_r, for loads applied at an angle θ to the grain shall be calculated as follows: $$R_r = \phi k_d \frac{A\left(k_{sp}f_{pu}\right)\left(k_{sq}f_{qu}\right)}{\left(k_{sp}f_{pu}\right)\sin^2\theta + \left(k_{sq}f_{qu}\right)\cos^2\theta}$$ where φ = 0.8 f_{pu}, f_{qu} = obtained from Tables 9.12 to 9.16. k_d = as specified in Clause 9.5.3. k_{sp} = as specified in Clauses 9.8.2.2 and 9.8.2.3. When the larger dimension or the diameter of the bearing area is less than 150 mm, no part of the bearing area is closer than 75 mm to the end of the member, and the bending moments at the bearing section do not exceed $0.4M_r$, k_{sq} shall be obtained from Table 9.10. For all other cases, k_{sq} shall be taken as 1.0.	**Change(s) from S6-06:** New modification factors included in the formula. **Recommendations to Ministry:** We recommend the Ministry adopt the change.
9.12.2 Specified Strengths and Moduli of Elasticity The specified strengths and moduli of elasticity for glued-laminated Douglas fir timber shall be obtained from Table 9.15.	**Change(s) from S6-06:** Slight adjustments to the negative bending moment strengths and moduli of elasticity. The moduli of elasticity were modified by the Supplements to S6-06 and have not subsequently been changed in S6-14. **Recommendations to Ministry:** We recommend the Ministry adopt the change.
9.17 Durability **9.17.1 General**	**Change(s) from S6-06:** Requirement of approval by Health Canada's Pest Management Regulatory Agency removed **Recommendations to Ministry:** We recommend the Ministry adopt the change.
9.17.2 Pedestrian Contact	**Change(s) from S6-06:** Requirement of approval by Health Canada's Pest Management Regulatory Agency removed. **Recommendations to Ministry:** We recommend the Ministry adopt the change.
9.17.5 Pressure Preservative Treatment of Laminated Veneer Lumber Treatment shall be in accordance with AWPA U1 and T1.	**Change(s) from S6-06:** Treatment standards revised to refer to AWPA U1 and T1. **Recommendations to Ministry:** We recommend the Ministry to further investigate.

9.17.6 Pressure Preservative Treatment of Parallel Strand Lumber Treatment shall be in accordance with AWPA U1 and T1.	**Change(s) from S6-06:** Treatment standards revised to AWPA U1 and T1. **Recommendations to Ministry:** We recommend the Ministry to further investigate.
9.17.12 Stress-Laminated Timber Decking	**Change(s) from S6-06:** Requirement of net retention removed. **Recommendations to Ministry:** We recommend the Ministry adopt the change.

Section 10 – Steel Structures

The majority of the changes to Section 10 are grammatical changes to promote consistency between Clauses. We have only highlighted grammatical changes that alter the interpretation of a specific Clause. We have also highlighted changes made by the three Supplements to S6-06, the majority of which have been carried over into S6-14.

The most significant changes that will impact the design of typical single lane forestry bridges relate to:

- Creating consistency between Section 8 and 10 when calculating the compressive resistance of concrete.
- Calculation of allowable fatigue stress ranges.
- Calculation of the fatigue resistance of stud type shear connectors.

10.7.2 Minimum Thickness of Steel The minimum thickness of steel shall be as follows: (a) gusset plates for main members and all material in end floor beams and end diaphragms and their connections: 9.5 mm; (b) closed sections, e.g., tubular members or closed ribs in orthotropic decks that are sealed against entry of moisture: 6 mm; (c) webs of rolled shapes: 6 mm; (d) webs of plate girders and box girders: 9.5 mm; and (e) other structural steel except for fillers, railings, and components not intended to resist loads: 8 mm.	**Change(s) from S6-06:** Reduced minimum thickness of gusset plate and girder web plate from 10 mm to 9.5 mm. **Recommendations to Ministry:** We recommend the Ministry adopt the change. This change reflects standard practice that is currently in use by the Ministry.
10.7.3 Floor Beams and Diaphragms at Piers and Abutments Floor beams and diaphragms at piers and abutments shall be designed to facilitate jacking of the superstructure unless the main longitudinal members are designed to be jacked directly.	**Change(s) from S6-06:** No changes. **Recommendations to Ministry:** The Ministry may consider requiring designs of bridges with spans over a specified length to include jacking provisions.
10.8.1 Tension Members **10.8.1.3 Cross-sectional Areas** **10.8.1.3.1 General** Deductions for fastener holes shall be made using a hole diameter 2 mm greater than the specified hole diameter for punched holes. This allowance shall be waived for drilled holes or holes that are subpunched and reamed to the specified hole diameter.	**Change(s) from S6-06:** Deduction now only applies to punched holes, whereas in S6-06, it applied to all holes. **Recommendations to Ministry:** We recommend the Ministry adopt the change.

10.9.4 Axial Compression and Bending **10.9.4.1 Cross-Sectional and Member Strengths — All Classes of Sections except Class 1 and 2 Sections of I-Shaped Members** **10.9.4.4 Member Strength and Stability — Class 1 and Class 2 Sections of I-Shaped Members**	**Change(s) from S6-06:** Design of Class 2 Sections of I-shaped members is now covered by Cl .10.9.4.4, i.e., Class 2 Sections are treated similarly to Class 1 Sections. **Recommendations to Ministry:** We recommend the Ministry adopt the change.
10.9.5 Composite Columns	This Section has been revised but we have not completed a review since typical single lane forestry bridges do not comprise composite columns. However, where the design of a composite column is required, we recommend the adoption of the S6-14 requirements.
10.10 Beams and Girders **10.10.2.3 Laterally Unbraced Members** $\omega_2 = \frac{4M_{max}}{\sqrt{M_{max}^2 + 4M_a^2 + 7M_b^2 + 4M_c^2}} \le 2.5$ Where: M_{max} = maximum absolute value of factored bending moment in unbraced segment, N·mm. M_a = factored bending moment at one-quarter point of unbraced segment, N·mm. M_b = factored bending moment at midpoint of unbraced segment, N ·mm. M_c = factored bending moment at three-quarter point of unbraced segment, N·mm.	**Change(s) from S6-06:** This Clause was modified by the Supplements to S6-06 and has not subsequently been changed in S6-14. The revision relates to the calculation of w_2 factor. **Recommendations to Ministry:** We recommend the Ministry adopt the change.
10.11 Composite Beams and Girders **10.11.5.2.2 Compressive resistance of concrete** $C_c = \alpha_1 \varphi_c b_e t_c f_c'$	**Change(s) from S6-06:** The compressive resistance of the concrete deck (C_c) has been revised to be consistent with Section 8. **Recommendations to Ministry:** We recommend the Ministry adopt the change.
10.11.5.3 Negative Moment Regions	**Change(s) from S6-06:** The calculation of negative moment resistance includes a requirement that the lateral torsional buckling resistance be calculated when determining the resistance of a composite section in the negative moment region. **Recommendations to Ministry:** We recommend the Ministry adopt the change. Further, the Ministry may wish to develop some guidelines for the detailing and calculation of negative moment resistances in steel girders with composite precast concrete decks.
10.11.6 Class 3 Sections	**Change(s) from S6-06:**

10.11.6.2.2 Moment Resistance of Slender Members	The compressive resistance of the concrete deck (C_c) has been revised to be consistent with Section 8. **Recommendations to Ministry:** We recommend the Ministry adopt the change.
10.11.6.3 Negative Moment Regions **10.11.6.3.1.2** **"….,**or a more detailed analysis of its lateral torsional buckling resistance."	**Change(s) from S6-06:** New phrase added that allows the calculation of negative moment resistance be calculated using a more detailed analysis. **Recommendations to Ministry:** We recommend the Ministry adopt the change. Further, the Ministry may wish to develop some guidelines for the detailing and calculation of negative moment resistances in steel girders with composite precast concrete decks.
10.11.8.3 Shear connector resistance **10.11.8.3.1 General** $P = \alpha_1 \varphi_c f_c b_e t_c + \varphi_r A_r f_y$	**Change(s) from S6-06:** The compressive resistance of the concrete deck (C_c) has been revised to be consistent with Section 8. **Recommendations to Ministry:** We recommend the Ministry adopt the change.
10.11.8.3.3 Stud Connectors in Full-Depth Precast Panels	**Change(s) from S6-06:** This Clause was added by the Supplements to S6-06 and has not subsequently been changed in S6-14. The Clause provides guidance for the design of shear connectors with full depth precast concrete deck panels. **Recommendations to Ministry:** We recommend that the Ministry adopt these requirements.

10.11.8.3.4 Channel Connectors in Cast-in-Place Deck Slab In solid slabs of normal-density concrete, the factored shear resistance for channel shear connectors shall be taken as.	**Change(s) from S6-06:** This Clause was added by the Supplements to S6-06 and has not subsequently been changed in S6-14. This Clause does not allow the use of channel shear connectors with full depth precast concrete deck panels. **Recommendations to Ministry:** Since this is a typical retrofit detail used by the Ministry, the Ministry should consider reviewing this requirement and developing a suitable standard detail. Further, the resistance for a channel shear connector as calculated using this Clause does not apply to the larger channel sections used for deck retrofit projects. We therefore recommend the Ministry develop standard design criteria for channel shear connectors.
10.11.8.4 Longitudinal Shear	**Change(s) from S6-06:** The compressive resistance of the concrete deck (C_c) has been revised to be consistent with Section 8. **Recommendations to Ministry:** We recommend the Ministry adopt the change.
10.12 Composite Box Girders	This Section has been revised but we have not completed a review since typical single lane forestry bridges do not comprise composite box girders. However, where the design of a composite box girder is required, we recommend the adoption of the S6-14 requirements.
10.13 Horizontally Curved Girders	This Section has been revised, but we have not completed a review since typical single lane forestry bridges do not comprise horizontally curved girders. However, where the design of a horizontally curved girder is required, we recommend the adoption of the S6-14 requirements.
10.14 Trusses	This Section has been revised but we have not completed a review since typical single lane forestry bridges do not comprise trusses. However, where the design of a truss is required, we recommend the adoption of the S6-14 requirements.
10.15 Arches	This Section has been revised but we have not completed a review since typical single lane forestry bridges do not comprise arches. However, where the design of an arch is required, we recommend the adoption of the S6-14 requirements.

10.16 Orthotropic Decks	**Change(s) from S6-06:** While there are no significant changes from S6-06. The Ministry should note that this Clause is not applicable to the design of orthotropic steel decks incorporated into all-steel portable bridges as defined by the Ministry Standard Drawings STD-E-090-01 and 02. Since these are temporary structures, the typical requirements for the design of orthotropic steel decks has been relaxed. **Recommendations to Ministry:** The Ministry may wish to develop design guidelines for orthotropic steel decks. This could include providing supplementary information on standard drawings.
10.17 Structural Fatigue	The Ministry has already adopted alternative fatigue design guidelines that address the difference between highway and off-highway loading. These requirements are not addressed in the subsequent review unless we are recommending specific changes.
10.17.2.3 Fatigue Stress Range Resistance **10.17.2.3.1 Fatigue Stress Range Resistance of a Member or Detail** $F_{sr} = \left(\frac{\gamma}{N_c}\right)^{1/3}$ If $F_{sr} = \left(\frac{\gamma}{N_c}\right)^{1/3} < F_{srt}$, $F_{sr} = \left(\frac{\gamma'}{N_c}\right)^{1/5} \geq \frac{F_{srt}}{2}$	**Change(s) from S6-06:** The calculation of the Fatigue Stress Range Resistance (F_{sr}) has been revised. **Recommendations to Ministry:** We recommend the Ministry adopt the changes.
10.17.2.7 Fatigue Resistance of Stud Shear Connectors Stud shear connectors shall be designed for the following stress range: $\tau_{rs} = 0.52 C_L \frac{V_{sc} Q}{A_{sc} I_t} \frac{s}{n} \leq F_{sr}^D$	**Change(s) from S6-06:** This Clause was modified by the Supplements to S6-06 and has not subsequently been changed in S6-14. **Recommendations to Ministry:** Similar to the calculation of the fatigue stress range f_{sr} the Ministry should explicitly revise this clause to require the design for the full range of the design shear force rather than a reduced range. The formula should be revised to: $\tau_{sr} = \frac{V_{sc} Q s}{A_{sc} I_n n} \leq F_{sr}^D$ Where: V_{sc} = range of design shear force at the section along the length of the beam where the shear resistance of shear connectors is being evaluated for BCL-625, L100, L150 and L165 design vehicles.

10.18.2.3 Bolted Joints in Shear **10.18.2.3.1 General** Bolted joints required to resist shear between the connected parts shall be designed as bearing-type connections at ULS with pretensioned bolts as specified in Clause A10.1.6.4. Joints of primary members subjected to stress reversal shall be designed as slip-critical connections. To prevent cyclic slip, the design load level shall be the net difference between permanent and transitory loads at SLS and FLS that causes stress reversal.	**Change(s) from S6-06:** This Clause was modified by the Supplements to S6-06 and has only undergone minor changes to wording in S6-14. The revision allows the use of bearing type connections for girder splices that are not subject to stress reversal. As a result, girder splices on single span bridges can be designed as bearing type connections. **Recommendations to Ministry:** We recommend the Ministry adopt the changes.
10.18.2.3.2 Slip resistance at the service load levels $V_s = 0.53 c_s k_s mnA_b F_u$	**Change(s) from S6-06:** The calculation of the slip resistance of a bolted connection has been revised through the adjustment of the mean slip coefficient (k_s) and the slip resistance factor (c_s). Further previously classified Class C contact surfaces (Hot-dip galvanized with hand wired brush surfaces) have been incorporated into Class A contact surfaces. **Recommendations to Ministry:** We recommend the Ministry adopt the changes.
10.18.3 Welds **10.18.3.1 General**	**Change(s) from S6-06:** Included matching electrode table for ASTM A709 steels. ASTM A709 is the American specification for weather steel incorporated in bridge structures and is similar to the CSA G40.21 specifications. **Recommendations to Ministry:** We recommend the Ministry adopt the changes.
10.18.4 Detailing of Bolted Connections	**Change(s) from S6-06:** A number of minor edits have been made in the text to improve clarity. **Recommendations to Ministry:** We recommend the Ministry adopt the changes.
10.23.3 Fracture Toughness **10.23.3.2 Primary Tension Members**	**Change(s) from S6-06:** Charpy V-notch testing required on a per plate frequency as opposed to per heat frequency as previously required. **Recommendations to Ministry:** Since typical forestry bridges do not include primary tension members, this requirement likely has no effect on typical fabrication practices. However, the Ministry has relaxed the requirements for fracture critical members from per plate to per heat frequency. Therefore, per heat frequency may be justified for primary tension members.
Table 10.12	**Change(s) from S6-06:**

Impact test temperatures and Charpy impact energy requirements for base metal and weld metal in primary tension members. **Table 10.13** Impact test temperatures and Charpy impact energy requirements for primary tension members.	Minor changes to test temperature and minimum energy requirements. Added minimum energy requirements for ASTM A709 steel. **Recommendations to Ministry:** We recommend the Ministry adopt the changes.
Annex A10.1 (normative) ***Construction requirements for structural steel.*** **A10.1.1 General** **A10.1.1.1** This Annex specifies requirements for the construction of structural steel for highway bridges and applies unless otherwise specified by the Regulatory Authority. The requirements specified in these Clauses are provided to ensure compliance with the design philosophy of this Section.	**Change(s) from S6-06:** This was previously Clause 10.24 and has now been removed from the Section and included as an Annex. Minor edits have been made to improve clarity and reduce the potential for conflicting requirements within Section 10. Since the Ministry has already developed standard steel fabrication specifications that reference CSA W59 (Standard Bridge Material Templates), this Annex is not applicable to the fabrication of typical forestry type steel girder bridges. Notwithstanding, we have reviewed and commented on the changes for completeness. We recommend that the Ministry review this Annex and modify the Standard Bridge Material Templates where it is thought that the Annex provides value to the Ministry.
A10.1.1.2 Fabricators shall have a comprehensive, documented quality management system (QMS). The quality standard shall be an industry recognized certification program specific to steel bridge fabrication acceptable to the Regulatory Authority. The QMS shall address the requirements of Section 10 and shall include a documented fracture control plan. For single-span girder bridges consisting of unspliced rolled sections or single span pedestrian bridges, certification requirements may be waived or modified by the Regulatory Authority. **Note:** *A quality management system certified by the Canadian Institute of Steel Construction, in the category of steel bridges, is compliant with this requirement.*	**Change(s) from S6-06:** Included requirement for fabricators to have a Quality Management System **Recommendations to Ministry:** This is a policy issue that the Ministry (as an owner) may wish to address.
A10.1.4.3.4 Bent Plates The following requirements shall apply to bent plates: (c) Hot bending at a plate temperature not greater than 650 °C shall be used to form radii less than those specified for cold bending. Accelerated cooling using compressed air or water may be used for a hot bent component only when its temperature is below 315 °C. Hot bending of steel with specified yield strength exceeding 350 MPa shall not be permitted without Approval.	**Change(s) from S6-06:** Revised requirements for hot bending of plate. **Recommendations to Ministry:** We recommend the Ministry adopt the changes.

A10.1.4.5.5 Accuracy of Holes **A10.1.4.5.6 Accuracy of Hole Group**	**Change(s) from S6-06:** New Clauses added. **Recommendations to Ministry:** We recommend the Ministry adopt the changes. **Change(s) from S6-06:** New Clauses added. **Recommendations to Ministry:** We recommend the Ministry adopt the changes.
A10.1.4.7.2 Heat Curving of Rolled Beams and Welded Girders	**Change(s) from S6-06:** Revised requirements for heat curving steel girders. **Recommendations to Ministry:** We recommend the Ministry adopt the changes.
A10.1.4.8 Identification Marking	**Change(s) from S6-06:** This Clause was modified by the Supplements to S6-06 and has not been revised in S6-14. More specific guidelines are provided to protect fracture critical or primary tension members from damage during application of identification marking. **Recommendations to Ministry:** We recommend the Ministry adopt the changes.
A10.1.5 Welded Construction **A10.1.5.1 General** All welding procedures, including those related to quality of work, techniques, repairs, and qualifications, shall comply with CSA W47.1 and CSA W59, except where modified by Clauses A10.1.5.2 to A10.1.5.7 of this Section.	**Change(s) from S6-06:** This Clause was modified by the Supplements to S6-06 and has not been revised in S6-14. Reference to CSA W47.1 added. **Recommendations to Ministry:** We recommend the Ministry adopt the changes.
A10.1.5.4 Submissions CWB-accepted welding procedure specifications, data sheets, and repair procedures for prequalification shall be submitted to the Owner in compliance with the Plans.	**Change(s) from S6-06:** This Clause was modified by the Supplements to S6-06 and has not been revised in S6-14. Requires CWB approved welding processes as required by CSA W47.1. **Recommendations to Ministry:** We recommend the Ministry adopt the changes.

A10.1.5.5 Certification of Fabrication Companies Any company undertaking welded fabrication and/or welded erection (including steel piles, railings and guards, or other welded attachments) shall be certified to Division 1 or 2 of CSA W47.1.	**Change(s) from S6-06:** This Clause was modified by the Supplements to S6-06 and has not been revised in S6-14. Clause modified to require companies completing any welded fabrication to be certified to CSA W47.1 Division 1 or 2. **Recommendations to Ministry:** We recommend the Ministry review these requirements and consider whether the current allowance that certain miscellaneous metal fabrication be completed by CSA W47.1 Division 3 companies is suitable.
A10.1.5.6 Backing Bars	**Change(s) from S6-06:** Added Clause. **Recommendations to Ministry:** We recommend the Ministry adopt the changes.
A10.1.6.3 Storage of Bolt	**Change(s) from S6-06:** Added Clause. **Recommendations to Ministry:** We recommend the Ministry adopt the changes.
A10.1.6.9 Installation of ASTM F1852 or ASTM F2280 Bolts	**Change(s) from S6-06:** Added Clause. **Recommendations to Ministry:** We recommend the Ministry adopt the changes.
A10.1.7 Tolerances **A10.1.7.2 Abutting joints** When compression members are butted together to transmit loads in bearing, the contact faces shall be . At joints where loads are not transferred in bearing, the nominal dimension of the gap between main members shall not exceed 10 mm, with a tolerance of ±5 mm from the nominal dimension.	**Change(s) from S6-06:** This Clause was modified by the Supplements to S6-06 and has not been revised in S6-14. Clause modified to include tolerance on joint dimensions that that do not transmit loads through bearings. **Recommendations to Ministry:** We recommend the Ministry adopt the changes.
A10.1.7.5 Fabricated components …….Additional fabrication tolerances shall be as follows: (a) alignment or position of secondary members: ±6 mm; (b) width of girder flanges: ±(*b*/100), where *b* is the flange width in mm, but not less than 5 mm and not greater than 25 mm; (c) width of stiffeners and plates for secondary members: -3 mm, +10 mm; and (d) misalignment of stiffeners on opposite faces of a web shall be less than one third of the web thickness for bearing stiffeners and half the web thickness for intermediate stiffeners.	**Change(s) from S6-06:** Modified Clause to include additional fabrication tolerances. **Recommendations to Ministry:** We recommend the Ministry adopt the changes.

A10.1.7.6 Control of Camber	**Change(s) from S6-06:** Added Clause. **Recommendations to Ministry:** We recommend the Ministry adopt the changes.
A10.1.7.7 Control of Sweep or Horizontal Curvature	**Change(s) from S6-06:** Added Clause. **Recommendations to Ministry:** We recommend the Ministry adopt the changes.
A10.1.8 Quality Control **A10.1.8.1 Qualification of Inspectors** Visual welding inspectors shall comply with the requirements of CSA W178.2 level 2 minimum. Non-destructive testing personnel (other than visual) shall comply with CAN/CGSB-48.9712 level 2 minimum.	**Change(s) from S6-06:** Modified Clause to include specific qualification criteria for welding inspectors. **Recommendations to Ministry:** We recommend the Ministry adopt the changes.
A10.1.8.2 Non-Destructive Testing of Welds	**Change(s) from S6-06:** This Clause was modified by the Supplements to S6-06 to allow radiographic or ultrasonic inspection of groove welds in flanges and webs. **Recommendations to Ministry:** We recommend the Ministry adopt the changes. (Note: This is consistent with Ministry practice that allows for ultrasonic inspection of web and flange splices.)
A10.1.9 Transportation and Delivery ………Where girders cannot be shipped with their webs in the vertical plane, static and dynamic forces during handling, transport, and storage shall be determined using a dynamic load allowance of at least 100%, unless a lower value can be justified. Computed stresses shall satisfy the provisions of Clause 10.10. Fatigue stresses due to dynamic flexure during transport shall also be considered.	**Change(s) from S6-06:** This Clause was modified by the Supplements to S6-06, and has not been revised in S6-14, to allow transportation of girders on their sides. **Recommendations to Ministry:** We recommend the Ministry adopt the changes.

Section 11 – Joints and Bearings

There have been no revisions to Section 11 and it remains applicable. The Ministry, however, may wish to provide additional design guidance related to the maximum bearing pressures and accommodation of rotation for plain elastomeric pads.

Section 12 – Barriers and Highway Accessory Supports

Section 12 has been revised to reflect the replacement of the Performance Level (PL) classification system with the AASHTO Test Level (TL) barrier classification system. The following barriers classifications have been adopted: TL-1, TL-2, TL-4 and TL-5. In order to minimize the impact on existing barrier systems in service, the code has established equivalencies to existing PL-type barriers, which are as follows:

- TL-2 is equivalent to PL-1.
- TL-4 is equivalent to PL-2.
- TL-5 is equivalent to PL-3.

TL-1 barriers are a new, lower standard barrier classification that has not been previously codified in Canada. This type is for use on low-volume road bridges, which is defined by the code as

"A bridge on a road with a maximum roadway width of 8.6 m, a maximum deck height above ground or water surface of 5.0 m, and either a maximum design speed of 80 km/h combined with a maximum AADT of 100 or a maximum design speed of 50 km/h combined with a maximum AADT of 400."

When compared to the 'Containment Level' criteria developed by Associated Engineering for the Ministry, TL-1 barriers provide a slightly higher level of containment than the CL-2 barriers. The Ministry should also recognise that the height of the CL-2 and CL-3 barriers and the location at which the design loads are applied are different than those specified for the TL-1 and TL-2 barriers.

The design forces for the TL barriers included in S6-14 are provided to facilitate the design of the barrier to deck anchorages and not the barrier whose capacity is confirmed through crash testing. As a result, S6-14 has reduced these forces by approximately 40% when compared to the equivalent design forces provided by AASHTO to facilitate the initial design of the barriers prior to crash testing. To allow a more direct comparison the following tables summarize the Ministry design forces along with the 40% reduction to allow a more direct comparison with S6-14.

Ministry Factored Barrier Design Forces

Barrier Classification	Factored Transverse Load (kN)	Factored Longitudinal Load (kN)	Factored Vertical Load (kN)
CL-2	45	20	20
CL-3	120	40	20

Ministry Factored Barrier Design Forces Reduced by 40%
(Allow Direct Comparison with S6-14)

Barrier Classification	Factored Transverse Load (kN)	Factored Longitudinal Load (kN)	Factored Vertical Load (kN)
CL-2	32	14	14
CL-3	85	28	14

S6-14 Factored Barrier Deck Anchorage Design Forces

Barrier Classification	Factored Transverse Load (kN)	Factored Longitudinal Load (kN)	Factored Vertical Load (kN)
TL-1	42.5	17	17
TL-2	85	34	17
TL-4	170	51	51
TL-5	357	119	153

When the Ministry barrier design forces were developed, it was decided not to account for the 40% reduction and the barriers and connection to the deck were tested to the full factored loads given an incomplete understanding of the dynamic behaviour of the barrier and connection.

With the recent adoption of standard CL-1, CL-2 and CL-3 barriers, Section 12 is no longer applicable to typical forestry bridges.

All relevant clauses in Section 12 make reference to the ‘Test Level’ barrier classification. For brevity, not all relevant clauses are presented in the below clause assessment, as they are nearly identical to those in S6-06, but with ‘Test Level’ replacing the previously-presented ‘Performance Level’ designation.

The MFLNRO work conducted in testing and development of the current ministry CL-1, CL-2 and CL-3 barriers standards predates the introduction of the AASHTO Test Level (TL) classification system and associated strength parameters within CSA S6. Although the ministry CL-1, CL-2 and CL-3 barriers are not fully consistent with S6-14, we do not believe that further work on barriers is required in the context of industrial resource roads. The Ministry should recognize that the CL-3 is likely equivalent to the TL-2 from a strength perspective but the geometry (height and railing spacing) does not meet the requirements of S6-14.

12.4.3.4.2 Crash Test Requirements for Traffic Barriers

Except as specified in Clauses 12.4.3.4.4 and 12.4.3.4.5, traffic barriers shall meet the crash test requirements of the optimum test level determined in accordance with Clause 12.4.3.2, or of a more severe test level if considered desirable.

The crash test requirements for traffic barriers for Test Levels 1, 2, 4, and 5 shall be the crash test requirements specified in the NCHRP Report 350 or the *AASHTO Manual for Assessing Safety Hardware*.

The crash test requirements for performance levels other than Test Levels 1, 2, 4, and 5 shall be Approved in accordance with Clause 12.4.3.2.2.

Change(s) from S6-06:
The *AASHTO Manual for Assessing Safety Hardware* is now a recognized document for crash test criteria.

Recommendations to Ministry:
With the development of standard barriers (CL-1, Cl-2 and CL-3), this clause is not applicable. However, the Ministry may wish to clarify the requirements for the design of barriers where standard barriers are not adopted.

12.4.3.4.4 Alternative Crash Test Requirements
A traffic barrier or traffic barrier transition shall be assumed to have met the requirements of Clauses 12.4.3.4.2 and 12.4.3.4.3, respectively, if it has been crash tested to requirements that test its geometry, strength, and behaviour to an equivalent or more severe level than the requirements of Clauses 12.4.3.4.2 and 12.4.3.4.3, respectively.

The crash test requirements for longitudinal barrier Performance Levels 1, 2, and 3 of the *AASHTO Guide Specifications for Bridge Railings* shall be taken as meeting the crash test requirements for Test Levels 2, 4, and 5, respectively.

Change(s) from S6-06:
The code now recognizes ‘Test Level’ barriers as the standard, but makes a provision for existing barriers designed and tested in accordance with ‘Performance Levels’ to be accepted as an alternative.

In S6-06, ‘Performance Level’ barriers were the standard, with this clause providing acceptance for ‘Test Level’ types as an alternative.

Recommendations to Ministry:
None.

Section 14 – Evaluation

This Section is identical to S6-06 with Supplements No. 1, No. 2 and No. 3. Minor grammatical changes were made in Supplements No. 1, 2 and 3 to correct errors and improve consistency between Clauses. These revisions have no effect on the determination of demands and resistances.

We recommend the Ministry adopt Section 14 and provide guidance on the evaluation of forestry bridges. This is of particular importance to timber structures where S6-14 is not clear on the application of Element Behaviour Categories and the applicability of the size factor for timber in shear. Guidance on the classification of off-highway trucks is also required. This guidance would ensure different evaluators are able to replicate evaluations.