R E P O R T

MINISTRY OF FORESTS

Evaluation of CAN/CSA-S6-00 (2000 Canadian Highway Bridge Design Code)

ASSOCIATED ENGINEERING

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Ministry of Forests
3rd Floor
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Re: EVALUATION OF CAN/CSA-S6-00

Dear Mr. Chow:

We are pleased to submit three copies of our final report evaluating the effect of the new Canadian Highway Bridge Design Code (CAN/CSA-S6-00) on the design of bridges for the Ministry of Forests.

Should you have any further questions, please feel free to contact me.

Respectfully submitted,

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EXECUTIVE SUMMARY

With the introduction of the new Canadian Highway bridge Design Code (CAN/CSA-S6-00), numerous revisions have been made to CAN/CSA-S6-88. The Ministry of Forests retained Associated Engineering (B.C.) Ltd. to review and comment on the effect that these revisions would have on typical forestry bridge design as practised in British Columbia.

The clauses that have been reviewed include:

- Clause 1 - General
- Clause 2 - Durability
- Clause 3 - Loads
- Clause 4 - Seismic design
- Clause 5 - Methods of analysis
- Clause 6 - Foundations
- Clause 7 - Buried structures
- Clause 8 - Concrete structures
- Clause 9 - Wood Structures
- Clause 10 - Steel structures
- Clause 11 - Joints and bearings
- Clause 12 - Barriers and highway accessory supports
- Clause 14 - Evaluation
- Clause 15 - Rehabilitation

Clause 13, Movable Bridges, and Clause 16, Fibre Reinforced Structures, were not reviewed as they are not applicable to typical forestry bridges.

Based on this review, the following is a summary of the modifications that should be considered before S6-00 is adopted by the Ministry of Forests:

Clause 1 - General

- Revise the Forest Service Bridge Design and Construction Manual to replace this Clause of S6-00.

Clause 2 - Durability

- Revise the Forest Service Bridge Design and Construction Manual to include relevant durability and inspection access criteria.

Clause 3 - Loads

- Review of the design life and annual reliability index with respect to the whether the values specified are applicable to typical forestry bridge designs.
- Revision of the deflection limitations to include a deflection criteria for all-steel portable bridges.
- Review of the live load factors and their applicability to typical Ministry design trucks.
- Revision of the design trucks.
- Simplification of the Dynamic Load Allowance criteria.

Clause 4 - Seismic Design

- Revise the Forest Service Bridge Design and Construction Manual to include seismic design criteria for forestry bridges.

Clause 5 - Methods of Analysis

- Develop specific design criteria for shear-connected slab bridges.

Clause 6 - Foundations

- Revise the Forest Service Bridge Design and Construction Manual to include the following:
    - Requirements for geotechnical investigations.
    - Guidance for the design and installation of precast concrete footings and steel pipe piles.

Clause 7 - Buried Structures

- Require all buried structures relying on soil structure interaction conform to S6-00 design criteria.

Clause 8 - Concrete Structures

- Revision of Ministry design standards to reflect a minimum allowable concrete strength of 30 MPa.
- Review the revised concrete cover requirements and their applicability to typical forestry bridges.

Clause 9 - Wood Structures

- Preparation of standard guidelines for the design of log stringer bridges.

Clause 10 - Steel Structures

- Preparation of standard requirements to improve durability of steel structures.
- Revision of minimum plate thickness requirements to allow for the use of 9.5 mm thick plate.
- Clarify applicability of longitudinal stiffener requirements when considering construction loading.
- Revision of structural fatigue criteria to suit typical forestry bridge design.
- Further investigation into the applicability/appropriateness of the revised fracture control requirements.

Clause 11 - Joints and Bearings

- The MoF should develop specific design criteria for plain rubber pads.

Clause 14 - Evaluation

- Define which permit vehicle should be considered.
- Development of a table summarizing sawn timber strengths to be used for evaluating existing timber bridges.

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INTRODUCTION

SECTION 1

During 2001, the new Canadian Highway Bridge Design Code, CHBDC-S6-00 (S6-00), was formally released. S6-00 is a complete revision of CAN/CSA-S6-88 (S6-88) the current bridge design code. Given the significant changes included in S6-00, the Ministry of Forests (MoF) retained Associated Engineering (B.C.) Ltd. to review and comment on the effect the changes would have on typical forestry bridge design.

S6-00 has been specifically developed to guide the design of highway bridges. Low volume forestry bridges do not conform to typical highway bridges and the Ministry standards should be revised to account for this difference. This is best done by altering specific clauses that are not applicable to the design of forestry bridges. This report presents a summary of our review with suggested changes to code clauses where appropriate. Due to the comprehensive nature of S6-00, only clauses that are relevant to typical forestry bridge design have been reviewed.

The clauses reviewed include:

- Clause 1 - General
- Clause 2 - Durability
- Clause 3 - Loads
- Clause 4 - Seismic design
- Clause 5 - Methods of analysis
- Clause 6 - Foundations
- Clause 7 - Buried conduit
- Clause 8 - Concrete structures
- Clause 9 - Wood structures
- Clause 10 - Steel structures
- Clause 11 - Joints and bearings
- Clause 12 - Barriers and highway accessory supports
- Clause 14 - Evaluation
- Clause 15 - Rehabilitation

The following clauses have not been reviewed as they are not typically encountered:

- Clause 13 - Movable bridges
- Clause 16 - Fibre-reinforced structures

In addition to reviewing and making recommendations, we have included some suggestions to improve current design practices. These typically cover the design of shear connected slab bridges and log stringer bridges with the intention of encouraging discussion regarding current design practices within these areas.

GENERAL

SECTION 2

Clause 1 of S6-00 details the provisions for the application of the Code and requirements of a general nature for bridges, culverts and related works. The provisions govern basic geometry and hydraulic design while general guidelines are provided for subsidiary components, deck drainage, maintenance and inspection access. Broad guidelines are also provided concerning economic, aesthetic and environmental considerations.

As discussed above, these guidelines pertain to the design and rehabilitation of highway bridges and are therefore not appropriate to the design of forestry bridges. In general, the majority of the issues included within this Clause are effectively dealt within the Forest Service Bridge Design and Construction Manual. However, the following is a brief review of these clauses with comments on the applicability to the design of forestry bridges.

2.1 GENERAL PROVISIONS (CL. 1.5)

These clauses cover the general design philosophy and requirements on which S6-00 is based. The clauses present guidance on bridge design rather than mandating specific requirements and can be adopted without directly affecting the design of typical forestry bridges. The exception is Cl 1.5.2.3 which requires that bridges be designed for a minimum design life of 75 years. This exceeds the MoF requirement of 45 years as stated in the Forest Service Bridge Design and Construction Manual.

Recommendations

Upon modification of the minimum specified design life, the clauses be adopted.

2.2 GEOMETRY (CL. 1.6)

These clauses cover the geometric design of bridge structures and are not appropriate to the design of forestry bridges.

Recommendations

These clauses should be replaced with the requirements outlined in the Forest Service Bridge Design and Construction Manual and the Forest Road Regulations that have specifically been developed for the design of forestry bridges.

2.3 BARRIERS (CL. 1.7)

These clauses cover the design and placement of barriers and guardrails on bridge structures and their approaches and are not appropriate to the design of forestry roads and bridges.

Recommendations

These clauses should be replaced with the requirements outlined in the Forest Service Bridge Design and Construction Manual and the Forest Road Regulations that have specifically been developed for the design of forestry roads and bridges. (For further commentary on the design of barriers to S6-00 refer to Section 13 of this report.)

2.4 AUXILIARY COMPONENTS (CL. 1.8)

These clauses cover the design of expansion joints and bearings, the provision of approach slabs and the allowance for utilities on bridges. Although the majority of forestry bridges do not include such components these clauses are applicable if such components are required.

Recommendations

The clauses as outlined in S6-00 can be adopted.

2.5 DURABILITY AND MAINTENANCE (CL. 1.9)

These clauses cover bridge deck drainage and the maintenance requirements for bridge structures. The clauses covering the drainage of bridge structures (Cl 1.9.2) are not entirely applicable to forestry bridges.

The clauses covering the maintenance of bridge structures are specific to highways bridges and are not applicable to forestry bridges. For further commentary regarding the maintenance of bridge structures refer to Section 3 of this report.

Recommendations

These clauses should be replaced with the requirements of the Forest Service Bridge Design and Construction Manual as modified by the comments included in Section 3 of this report.

2.6 HYDRAULIC DESIGN (CL. 1.10)

These clauses cover the hydraulic design requirements for highway bridge structures such as minimum freeboard allowances and flood return periods. The requirements outlined, however, are excessive for forestry bridges.

Recommendations

These clauses should be replaced with the requirements outlined in the Forest Service Bridge Design and Construction Manual.

DURABILITY

SECTION 3

Clause 2 is a new addition to the Bridge Design Code and provides requirements for durability that shall be considered during the design process in addition to requirements for strength and serviceability. Durability is not a primary design criteria for forestry bridges. However, this does not imply that certain measures cannot be taken to increase the durability of forestry bridges. Such measures could include:

- Standard bridge abutment and span joint details incorporating rubber seals to protect the bearings from run-off from the deck,
- Provision of access to complete close proximity inspections of bridge girders and piers where required.

Further to the above, specific issues dealing with the durability of steel and concrete structures have been addressed in Sections 9 and 11 of this report.

Recommendations

We recommend that this clause be referred to for guidance rather than be adopted in its entirety. If required, the MoF should develop some standard abutment and span joint details that would protect bridge bearings from run-off. These details could be included in the MoF Standard Bridge Drawings. A further requirement that should be included in longer span and multi-span structures is the inclusion of safety lines to facilitate close proximity inspection of the girders deck soffit and bridge piers.

LOADS

SECTION 4

Clause 3 of S6-00 outlines the bridge loads and associated load factors. Only those loads and load factors that are typically encountered in the design of forestry bridges have been reviewed. Where special loading (seismic, ice etc.) needs to be considered, we recommend that S6-00 be adopted.

The following material from Clause 3 of S6-00 has been reviewed:

- Design life and safety reliability index.
- Deflection limitations.
- Load factors and load combinations.
- Live loads - CL-W Vehicle.
- Dynamic load allowance.
- Construction loading and load factors.

4.1 DESIGN LIFE AND ANNUAL RELIABILITY INDEX (CL. 3.5.1)

Table 4.1 summarizes the changes to design life and annual reliability index.

Table 4-1

Design Life and Safety Reliability Index

Code	Design Life	Annual Reliability Index	Reliability Index Over Design Life
CAN/CSA-S6-00	75 years	3.75	3.5
CAN/CSA-S6-88	50 years	3.5	3.25
MoF Forest Service Bridge Design and Construction Manual	45 years	?	?

Both the design life and annual reliability index have been increased when compared to S6-88. The MoF guidelines only require a design life for structures of 45 years. The design life and annual reliability index are important design variables as they are used to define the following:

Load Factors (Cl. 3.5.1)

Using the design life and annual reliability index, the load factors and load combinations have been calibrated to ensure a uniform level of reliability. At the Ultimate Limit State, use of the load factors included in S6-00 results in a probability of approximately 1% that the design load will be exceeded during the 75-year design life of the structure. The live load factors have been calibrated to reflect the variability of trucks on public highways. Should the factors be applied to other live loads, such as the MoF design trucks, a different safety level will be attained.

Fatigue Life of Steel Structures (Cl. 10.17.2.3)

The number of cycles a specific detail is subject to is based on the design life of the structure. With the increase in the design life of structures from 50 to 75 years, there is theoretically an increase in the number of cycles to which fatigue-prone details will be subjected.

Concrete Cover (Cl. 8.11.2.2)

With the increase in bridge design life, concrete durability has to be increased. In conjunction with improving the quality of the concrete, durability is also increased by reducing the potential for the corrosion of the reinforcing. This is accommodated in S6-00 by increasing the clear cover to the reinforcing.

Recommendations

The values defined in S6-00 provide a prescribed uniform level of reliability based on an acceptable probability that the factored loads will be exceeded during a specific period of time for highway bridges subject to the specified design loads. This "level of reliability" may not be appropriate for use in forestry bridge design.

We suggest that the MoF define an appropriate design life and required reliability index. Based on these revised values, appropriate load factors, fatigue life and concrete cover requirements can be adopted.

4.2 DEFLECTION LIMITATIONS (CL. 3.4.4)

The deflection criteria in S6-00 are the same as those contained in S6-88. These limitations are based on the first flexural frequency of the structure. The intention is to limit traffic-induced vibration experienced by pedestrians. The Forest Service and Bridge Design and Construction Manual simplify these limitations to the following:

- Concrete superstructures: Live load deflection *span/350*,
- Steel superstructures: Live load deflection *span/450*.

A comparison of the allowable deflections between MoF and S6-00 guidelines shows that the MoF allowable deflections are typically greater than those allowed by S6-00.

Recommendations

Given that forestry structures are not designed to accommodate pedestrians we suggest that the MoF deflection guidelines for steel girder and concrete slab bridges remain unchanged.

We recommend that the deflection limitations be adjusted to accommodate all-steel portable bridges. These bridges are lighter and shallower than other portable bridge designs and are subject to higher live load deflections. These deflections do not conform to the current MoF guidelines. Given the limited pedestrian use, the deflection criteria should be reduced to *span/300.*

4.3 LOAD FACTORS AND LOAD COMBINATIONS (CL. 3.5.1)

Although the load factors have been modified in S6-00 (refer to comments on Cl. 3.5.1), the changes will result in nominal changes to total factored loads. Table 4.2 summarizes the changes to the load factors. It is important to note that these factors cannot be viewed in isolation, as other parameters (Φ, live load distribution, etc.) also directly affect the design.

Table 4-2

Summary of Load Factors

Load Effect	Load Factor (S6-00)	Load Factor (S6-88)
Live Load	1.7	1.6
Dead Load - Manufactured components included precast concrete and steel girders (excluding wood)	1.1	1.2
Dead Load - Wearing surfaces, based on nominal or specified thicknesses	1.5	1.6

Recommendations

The live load factors included within S6-00 are intended to be applied to the CL-625 design vehicle and are calibrated based on the specified design life and annual reliability index. Application of these factors may not be appropriate to forestry bridges.

Cl. 3.8.3 allows for the use of "site specific" vehicles with appropriate load factors as long as the resulting level of safety is not lower than that provided by S6-00. Based on this clause we recommend that the MoF investigate the suitability of the live load factors when applied to off-highway logging trucks with the expectation that live load factors may be reduced (refer to Section 4.1).

4.4 LIVE LOADS - CL-W LOADING (CL. 3.8.3)

The design vehicle has been revised and is defined as CL-W loading where W represents the total vehicle weight in kN. The typical designation for highways will be CL-625. The CL-W lane load is defined as 80% of the CL-W truck load superimposed on a 9 kN/m lane load. The change in vehicle configuration will not result in a significant change to vehicular live loads when compared to the original CS600 loading.

Recommendations

It is our understanding that the MoF is currently undertaking a study of logging truck axle weights and configurations. On completion of this study, it is anticipated that the axle weights and configurations of the standard design trucks may be altered.

In addition to this work, the MoF may wish to revise the maximum truck eccentricity and associated off-balance wheel loading. We suggest that this be adjusted to reflect the philosophy used in S6-00, and as noted below:

Ultimate Limit State and Serviceability Limit State

- The minimum distance between the wheel centerline and the edge of curb shall be 600 mm (except for the design of slabs where this distance shall be reduced to 300 mm)
- The axle load shall be evenly distributed between both wheels (i.e., no allowance for off-balance wheel loading)

Fatigue Limit State

- A single design vehicle located at the centre of the travelled lane.
- The axle load shall be evenly distributed between both wheels (i.e., no allowance for off-balance wheel loading).

In order to account for the possibility that there could be more than two trucks on a bridge simultaneously, the MoF requires that where the bridge length exceeds 40 m, the bridge shall be designed for two trucks with the distance between them equal to half the length of a single truck. We recommend that this requirement for two trucks be retained.

4.5 DYNAMIC LOAD ALLOWANCE (CL. 3.8.4.5)

S6-88 defined the dynamic load allowance (DLA) as a function of span length. S6-00 has revised this definition, resulting in the DLA being a function of the axle configuration causing the load effect. One of the problems with this definition is that the DLA varies with load effect, span length and location where the force effect is being considered. Tables 4.3 and 4.4 summarizes the DLA for typical logging trucks and span configurations. For the design of decks where only a single axle is typically considered, the DLA remains unchanged at 40%.

Table 4-3

Dynamic Load Allowance (Maximum Flexure)

	Dynamic Load Allowance		
MoF Design Truck	**40%**	**30%**	**25%**
BCFS L75	2 m ≤ span	2 m < span ≤ 15 m	span > 15 m
BCFS L150	2 m ≤ span	2 m < span ≤ 15 m	span > 15 m
BCFS L165	2 m ≤ span	2 m < span ≤ 14 m	span > 14 m

Table 4-4

Dynamic Load Allowance (Maximum Shear)

	Dynamic Load Allowance		
MoF Design Truck	**40%**	**30%**	**25%**
BCFS L75	2 m ≤ span	2 m < span ≤ 10 m	span > 10 m
BCFS L150	2 m ≤ span	2 m < span ≤ 10 m	span > 10 m
BCFS L165	2 m ≤ span	2 m < span ≤ 9 m	span > 9 m

Recommendations

Given the complexity in choosing the appropriate DLA, we suggest it be based on span length, similar to the requirements outlined in S6-88. We recommend the following values be adopted:

- Span < 10 m, DLA = 30%.

- Span $\geq$ 10 m, DLA = 25 %.
- Where only a single axle is used, the value of 40% as required by S6-00 should be adopted. (When using a tandem axle for deck design we recommend a value of 40% be applied to both axles.)

4.6 CONSTRUCTION LOADING AND LOAD FACTORS (CL. 3.16)

Prior to the introduction of S6-00, the issue of construction live loading and the associated load factors were left to the discretion of the engineer. In S6-00, construction and erection loading is a defined load case with required load factors. A construction live load factor of 1.445 (0.85 x 1.7) is to be applied to construction live loads along with the associated dead load factors. In addition, the total factored load effect for construction loads shall not be less than 1.25 times the sum of the unfactored load effects.

Recommendations

We recommend that the load factors as outlined in S6-00 be adopted. The MoF should also specify a minimum erection load to be included in all designs unless specifically noted otherwise. For example, we recommend the following minimum erection load for steel girder bridges with composite precast concrete deck panels:

- Self weight of structure including all deck panels in position (ungrouted).
- Live load of 445 kN (represents typical excavator (80,000 lbs) and payload (20,000 lb). The vertical load can be treated as point load evenly distributed between two girders.

SEISMIC DESIGN

SECTION 5

Clause 4 of S6-00 provides a detailed description of the characteristics and requirements for highway bridge structures to provide a high level of seismic performance during and following a major earthquake. It specifies performance objectives for bridges, and provides information on Canadian seismicity design earthquake loading.

There are fundamental differences in the configuration, details, and operational requirements of highway bridges and forestry/resource bridges. These suggest that the requirements within Clause 4 should be applied with simplification and, arguably, significant relaxation, to forestry bridges. Features of forestry bridges suggesting that the seismic requirements of S6-00 should be simplified and relaxed include:

- They are often single-span, and generally of ‘regular' arrangement.
- They are typically ‘locked in' to the ground, i.e., are restrained by the ground bearing against ballast walls. This provides a good level of inherent seismic resistance.
- They are typically relatively light.
- Loss of span failures, one of the most common causes of highway bridge damage, is unlikely for forestry bridges in light of the above points.
- Traffic volumes are very low compared to highway bridges.
- The design life is less than for highway bridges.
- The consequences of significant earthquake-induced damage and even collapse are much less than for highway bridges. The consequences are mainly economic, rather than loss of life or the severing of an emergency response route.
- In the unlikely event of the need for replacement, this can be accomplished much more quickly than for a highway bridge.
- The forestry bridge may remain useable even with significant damage that requires eventual repair.
- There is unlikely to be the level of geotechnical information available during design to allow many of the requirements of S6-00 to be applied.

Features of forestry bridges that suggest some level of seismic resistance should be provided include:

- Potential for loss of life remains a possibility throughout the life of a road serving both resource and recreational demands.
- The design life of forestry bridges adopted may exceed 45 years.
- The weight of a loaded truck, which may be on a bridge during an earthquake, could be significantly greater than the weight of the bridge. Thus a substantial lateral load may result, and which is unlikely to have been considered in the design of members and connections.

To mitigate the level of seismic damage and to prevent collapse, in particular to loss-of-span collapses, the design features noted below are highly desirable for forestry bridges. These features would provide substantial inherent seismic robustness to forestry bridges. It will be apparent that most modern forestry bridges will possess these features, by virtue of the design and construction details that are common practice in the industry.

- Positive connections between superstructures (girders) and abutments.
- Locked-in superstructures between earth-retaining abutments. Note that this feature, common to forestry bridges, provides a level of protection for both structural demands and local soil failures, and thus mitigates the lack of geotechnical information.
- Positive, robust connections between spans and between girders and piers for multispan bridges. This is of particular importance for non-composite deck, simply supported multispan bridges.
- A complete load path through members and connections for transverse and longitudinal loads from the deck components through to the supports.

5.1 GENERAL APPLICATION OF S6-00

In general, Clause 4 of S6-00 should be referenced and adopted with modification for the seismic design of forestry bridges. Given the competitive nature of this industry, the wide range of seismic design skills among forestry bridge designers, the inherent seismic resistance of most forestry bridges, and the mainly economic consequences of damage, the seismic design process should be straightforward, simple to apply, and not introduce a significant economic penalty to the initial construction.

In this context, we suggest the following application of Section 4 of S6-00. Note that for single span bridges ‘locked' in to the ground, which is the most common configuration, the seismic design is straightforward, and unlikely to affect the cost of most bridges.

5.2 IMPORTANCE CATEGORY (CL. 4.4.2)

Forestry bridges should be considered as "other" bridges unless otherwise specified by the Owner or authority having jurisdiction.

This Category, along with the zonal acceleration level, lead to a "Seismic performance zone" (SPZ) from Table 4.4.4.1. We suggest that the SPZ be limited to a maximum of 2 (highway bridges may be as high as SPZ 4). This parameter, among others, is subsequently used to determine the required minimum level of seismic analysis.

Recommendations

Limit the Seismic Performance Zone (SPZ) to a maximum of 2 for forestry bridges.

5.3 ANALYSIS FOR EARTHQUAKE LOADS (CL. 4.4.5)

Clause 4.4.5.2, Single Span Bridges, will likely cover the majority of new forestry bridges. Single span girder bridges do not require any seismic analysis, and as such S6-00 does not impose a significant design penalty. Connections between the girder and supports could be designed for the zonal acceleration multiplied by the tributary bridge mass. This implies no amplification of the ground motion in the structure. This would provide a relatively modest force for which commonly used connections may prove sufficient. Perhaps of more importance, it would provide a reasonable soil pressure for the design of the precast ballast walls. For longitudinal loads, 100% of the bridge weight times the ground acceleration could be considered to act in compression on each abutment/soil interface.

For multispan forestry bridges, or other bridge configurations, this clause and the above suggested limit for the SPZ would limit the required level of analysis to the "uniform load" method or the "single mode" spectral method, both of which are relatively straightforward. While either method can be formulated and solved by hand or spreadsheet, it is more likely that dynamic analyses packages would be used for the spectral method, and likely to prove in practice to be as straightforward as the uniform load method. It should be noted that a number of forestry bridges will be considered ‘irregular' by virtue of differing spans or pier heights, such that a spectral method would be required by S6-00. The necessity of this is debatable, but providing for seismic loads may in fact provide a useful and economic lower bound for the lateral design of piers and girder/pier connections. Please also refer to the discussion below for Clause 4.4.7.1.

Recommendations

- For longitudinal loads, check ballast wall flexural strength assuming a soil pressure derived from 100% of the bridge weight times the design ground acceleration. A triangular soil pressure distribution, with zero stress at grade, could be adopted.
- Adopt an analysis method (uniform or spectral) based on S6-00 Clause 4.4.5 and as limited by parameters discussed above.

5.4 SITE EFFECTS (CL. 4.4.6)

There is typically not enough site information to allow a refined selection of the site coefficient "S" within this section.

Recommendations

The following values of S may be assumed in design:

- S=1.2 for bridges on footings.
- S=1.5 for bridges on piles.
- S=1.0 only if adequate site information is available to justify this lower value.

5.5 ELASTIC SITE RESPONSE COEFFICIENT (CL. 4.4.7)

We suggest that an "Importance factor" of 1.0 be adopted for forestry bridges unless a higher value, probably not greater than 1.5, is specified by the Owner or Authority.

Clause 4.4.7.1 provides the seismic response coefficient, C_{sm}, which is multiplied by the bridges' distributed weight for an equivalent static uniformly distributed seismic load.

The upper limit on C_{sm} is an amplification of 2.5 times the ground acceleration, and the designer is free to simply adopt this as a design force in lieu of refinement if desired. Given the lack of soil information and the uncertainty in calculating the bridge's period of vibration, this appears to represent a reasonable analysis approach for lateral forces for all but very irregular forestry bridges.

One potential area of uncertainty with this approach would be in proportioning the total transverse seismic force between supports at either end of a given span. A lower limit of 50% of each adjacent span is suggested, unless the designer refines this distribution through analysis. For longitudinal connection or restrainer loads, a reasonable distribution based on an analysis and relative stiffnesses could be adopted.

Recommendations

- Adopt an Importance Factor of 1.0 unless otherwise specified by the bridge Authority.
- Distribute transverse seismic loads among supports based on not less than 50% of the span length on each side of a support as a tributary mass (or weight) unless refined by suitable analysis.

5.6 RESPONSE MODIFICATION FACTORS (CL. 4.4.8)

This section could be adopted as is. Note that S6-00 would allow earthquake-induced plastic ‘hinging' in foundation elements, which is appropriate and should be considered as a useful design strategy.

5.7 RECOMMENDATIONS

This section could be adopted as written.

5.8 LOAD COMBINATIONS (CL. 4.4.9)

Seismic loads should be taken to act along with only dead loads. Given the low traffic volumes, the low probability of having a fully loaded truck on a given bridge during the brief time of strong shaking, the complexities of structure-vehicle interaction, and the robust nature of most logging bridges, a live load allowance in this combination appears conservative.

We suggest that directional combinations could be neglected in the design of forestry bridges. A lateral design in the two principal directions, performed independently, appears appropriate.

Recommendations

- Apply seismic loads in combination with dead loads only.
- Perform seismic design of regular bridges in two separate directions, neglecting directional combinations.

5.9 FORCES AND SUPPORT LENGTHS (CL. 4.4.10)

Forces prescribed in this section are generally as suggested in the discussions above. For irregular bridges, design forces could be modified by member nominal capacities acting as structural ‘fuses'.

Recommendations

Adopt this section as required for seismic design.

5.10 FOUNDATIONS (CL. 4.6)

This section does not appear applicable for the seismic design of most forestry bridges, in that limited geotechnical investigation or assessment is typically carried out. Soil or foundation failures may nonetheless occur, and the Owner or Authority having jurisdiction should address this risk to their satisfaction.

For irregular bridges, Clause 4.6 would lead, in some cases, to larger abutment design pressures than considered in the above methods. For ballast wall design, if the designers recognize that yielding and some permanent deformation may occur in the reinforced concrete ballast walls, then the higher pressures implicit in this section could be ignored, providing a structural collapse is unlikely to occur.

Approach slabs are rarely, if ever, incorporated into forestry bridges. On highway bridges they serve a continuous serviceability function as well as facilitating the use of such bridges by emergency vehicles in the days following a major earthquake.

Recommendations

- The Authority and Owner recognize that seismically induced geotechnical risks to the crossing may exist and be addressed where appropriate.
- Adopt structural aspects of this Clause as appropriate.
- Neglect provisions related to approach slabs unless otherwise directed by the Owner.

5.11 CONCRETE (CL. 4.7)

This clause essentially provides detailing requirements for reinforcing steel within concrete elements, and would apply mainly to substructures (piers). These provisions can be adopted as required for the design of ‘irregular' forestry bridges, i.e., those classified as SPC 2 in the previous discussions.

Clause 4.7.1 deals with columns that may be expected to ‘hinge' during an earthquake. The detailing provisions are reasonable, but a comparison of some formulae as implemented in S6-00 for highway bridges against the original formulae in the literature suggests that they are likely to be conservative even for highway bridges. We suggest that some relaxation for forestry bridges.

Recommendations

That the Ministry consider adopting the following:

- Minimum column reinforcing steel ratio of 0.75% (rather than 1%).
- V_c not be taken as zero. For irregular bridges with concrete decks or heavy superstructures, and where R=3 (single column) or R=5 (multi-column) was used, to determine design forces, a minimum $V_c = 0.1 f_c'^{0.5}$ could be adopted.
- The effective area of the column used to resist seismic shears be taken as $0.8A_g$. The f_c' term in formulae within clause 4.7.4.1.4 be omitted.
- The 1.25 factor in formulae within clause 4.7.4.1.4 be omitted. Alternately, the 1.25 factor could be retained and the axial load taken as unfactored dead plus

seismic load. Note that the seismic component may be obtained from analysis or from a plastic hinging mechanism assuming nominal (normal design) material strengths.

- For columns with a ratio of the length between the maximum moment to a point of inflection, divided by the column cross sectional dimension (diameter or width) greater than 10, then the detailing provisions of S6-00 Clause 4 be adopted without modification. This is because local ‘plasticity' demand in column hinge zones increase, or become more concentrated, at a rapidly increasing rate as aspect ratios reach or exceed this level. We suggest that columns be proportioned to limit this aspect ratios to 15 or less.

5.12 STEEL (CL. 4.8)

Detailing of steel elements for seismic resistance in ‘irregular' forestry bridges can be based on the requirements in this section. The majority of forestry bridges will not require reference to this section at all. We expect that of those irregular bridges, where the designer is faced with this section, that the provisions of Cl. 4.8.4.3.3, Concentrically Braced Frames with Nominal Ductility, would be appropriate. Design experience indicates that ‘tension only' bracing would often be adopted, such that a connection design force and detailing to allow the tension brace to develop A_gF_y would be required. This will require proportioning the member and bolt holes to ensure that $0.85*A_{ne}F_u$ exceeds AgFy (Cl. 10.8.2).

Recommendations

For ‘irregular' bridges where the designer is faced with this section, that the provisions of Cl. 4.8.4.3.3, Concentrically Braced Frames with Nominal Ductility, be adopted. This need not constrain the Designer from adopting other viable design strategies.

5.13 OTHER SECTIONS

Other sections, such as ‘joints and bearings' and ‘base isolation' are not expected to be applied during the design of forestry bridges. These provisions are available, and can be adopted, at the discretion of the Owner and designer.

METHODS OF ANALYSIS

SECTION 6

Clause 5 covers the analysis of bridges subject to the loads described in Clause 3. When compared to S6-88, this clause is more extensive and has undergone a number of significant revisions. In many cases the net live load distribution to girders increases in S6-00 when compared to S6-88.

Although the majority of Clause 5 is not applicable to forestry bridges, the clause covering the distribution of live loads in shear connected slab bridges is often relied upon. This clause outlines the methods used for distributing live loads in shear connected slab bridges and our recommendation for changes to the design of such forestry bridges.

Longitudinal Bending Moments and Shear in Multi-Spine Bridges (Cl. 5.7.1.3 and Cl. 5.7.1.5)

S6-88 defined the longitudinal moment and shear per slab by applying a fraction (S/D) of a line of wheels to the slab where S represented the centre to centre spacing of the slabs and D was a value defined by an equation in S6-88. When calculating the shear distribution, the entire wheel load, directly over where the shear was being considered, was to be used and could not be reduced using the S/D fraction.

S6-00 introduced a new methodology for determining the primary longitudinal moment and shear in slabs. The method defines a parameter (β) based on the flexural and torsional stiffness of the slabs and the overall bridge geometry. The load distribution is based on β and an associated lane width correction factor.

Unfortunately the distribution factors presented only cover load distribution for bridges comprising two or more lanes.

Recommendations

We suggest that appropriate load distribution factors for both longitudinal moment and longitudinal shear be developed for single-lane shear connected slab bridges.

In addition to the development of these load distribution factors, we suggest that the MoF develop a standard set of design guidelines for slab bridges. Such guidelines could cover the following:

- Moment distribution (all limit states),
- Shear distribution (all limit states),
- Torsional loading in shear connected slabs,
- Torsional behaviour of beams versus slabs,
- Standard shear connector design for L75, L100, L150, and L165 logging trucks.

The MoF may also wish to reconsider the onerous design requirements for welded shear connected slab bridges. Based on the research commissioned by the MoF, the requirement that slabs be designed as though all shear connectors have failed at the ULS is overly conservative and directly affects the cost of the bridge.

FOUNDATIONS

Clause 6 of S6-00 provides minimum requirements for the design of foundations and the estimation of earth pressures on retaining structures. The requirements of this Clause are not applicable to forestry bridge substructures.

Recommendations

This Clause should be adopted for the design of bridge foundations with the following exceptions:

- Geotechnical investigation requirements should be based on the requirements outlined in the Forest Service Bridge Design and Construction Manual,
- Design and installation of precast concrete footings and steel piles based on the requirement outlined in the Forest Service Bridge Design and Construction Manual.

The Forest Service Bridge Design and Construction Manual should be revised to include guidance on the design and installation of precast concrete footings and steel piles.

BURIED STRUCTURES

Clause 7 of S6-00 covers the design of buried structures which include soil-metal structures, metal box structures and reinforced concrete structures. The majority of these structures are pre-engineered and sizes are based on hydraulic design criteria.

Given that the majority of these systems are pre-engineered structures and purchased as such, the suppliers should ensure that the design and installation requirements conform to the requirements of S6-00. No exceptions or changes are suggested.

CONCRETE DESIGN

SECTION 9

Clause 8 covers the design of structural components made of precast or cast-in-place concrete reinforced with prestressed or non-prestressed steel. This section replaces both Clauses 8 and 9 of S6-88. Numerous changes have been made when compared to S6-88; these changes have been summarized and include the following:

- Concrete Strength,
- Concrete Modulus of Elasticity,
- Prestressing Steel Modulus of Elasticity,
- Prestressing Requirements,
- Material Resistance Factors,
- Flexural and Axial Capacity,
- Shear and Torsion,
- Durability,
- Multibeam Decks.

9.1 CONCRETE STRENGTH (CL. 8.4.1.2)

S6-00 requires a minimum concrete strength of 30 MPa for non-prestressed concrete elements and 35 MPa for prestressed concrete elements. This differs form S6-88 which had no limitations. The inclusion of a minimum strength of concrete implies a better quality concrete with an increased durability.

A review of the MoF Standard Drawings and the Forest Service Bridge Design and Construction Manual shows that the minimum concrete strength typically specified is 30 MPa. (Also refer to Section 9.9 for further comments on minimum concrete strength requirements.)

Recommendations

We recommend that S6-00 be adopted. It is very similar to the current requirements of the Forest Service Bridge Design and Construction Manual

9.2 CONCRETE MODULUS OF ELASTICITY (CL. 8.4.1.7)

The Modulus of Elasticity has been revised but will not result in any significant changes.

Recommendations

We recommend that the revised method of calculating the Modulus of Elasticity for concrete be adopted.

9.3 ELASTIC MODULUS OF PRESTRESSING STEEL (CL. 8.4.3.3)

Elastic modulii are higher than in S6-88. This increases the elastic shortening loss slightly.

Recommendations

The elastic modulii in S6-00 be adopted.

9.4 MATERIAL RESISTANCE FACTORS (CL. 8.4.6)

The material resistance factors have been revised and are applied directly to the associated materials. This is different to S6-88 where a resistance factor was applied to a nominal section resistance (flexure, shear or torsion etc.). This revision has resulted in the S6-00 design philosophy becoming consistent with the design philosophy adopted in CSA A23.3, the code governing the design of concrete structures.

Recommendations

The material resistance factors as specified in S6-00 be adopted.

9.5 LIMIT STATES (CL. 8.5)

Although this clause is more descriptive than what was defined in S6-88, there are no major changes in the requirements that have to be met to satisfy the various Limit States. The changes to crack control and deformation limitation calculations are dealt within a separate section of this report.

Recommendations

The methodology as outlined in S6-00 be adopted.

9.6 PRESTRESSING REQUIREMENTS (CL. 8.7)

Prestressing Strand and Jacking Stress (Cl. 8.7.1)

Only low relaxation strand is permitted for new bridges. Formulae for stress relieved strand losses are provided for evaluation of existing bridges in Clause 14. Jacking stress for low relaxation strand has been reduced from $0.80f_{pu}$ to $0.78f_{pu}$ in pretensioned members. Jacking stress for post-tensioned members remains at $0.80f_{pu}$ (see Table 9.1). Stress at transfer is reduced slightly from $0.75f_{pu}$ to $0.74f_{pu}$. For post-tensioning anchorages, the stress at transfer is further limited to $0.70f_{pu}$. Final effective prestress is specified as $0.45f_{pu}$ in S6-00, whereas there was no requirement in S6-88.

The precast industry has, in the past, expressed concerns over high specified jacking stresses. Industry preference is to use $0.75f_{pu}$ for new designs. The new lower transfer stress limit and minimum final effective prestress are nearly always satisfied.

Prestress Losses (Cl. 8.7.4)

Based on a sample design, the total loss calculated under S6-00 for pretensioned girders is expected to be slightly lower than under S6-88, although the difference is not expected to be significant. Loss at transfer in S6-00 is higher owing to a new initial steel relaxation loss and a higher elastic shortening loss resulting from higher elastic modulus for steel. The initial relaxation is time dependant. Precast fabricators routinely use a one day production cycle; consequently, the effect of initial relaxation would be minimal. Loss at final overall is lower for owing to a lower creep loss. Steel relaxation is higher, but the magnitude of the loss is low. Shrinkage loss remains the same in both codes.

The new loss formulae are expected to be slightly more favourable than S6-88.

Recommendations

We recommend that S6-00 be adopted.

Table 9-1

Prestressing Tendon Stress Limits (Low Relaxation Strand)

	S6-88	**S6-00**
At jacking	$0.80f_{pu}$	$0.78f_{pu}$
At transfer	$0.75f_{pu}$	$0.74f_{pu}$

Recommendations

The methodology as outlined in S6-00 be adopted.

9.7 FLEXURAL AND AXIAL LOADS (CL. 8.8)

Given that concrete compressive members (columns/piles) are not commonly used in the design of forestry bridges, a comparison of the clauses specifically relating to axial loads has not been conducted. The following discussion covers flexural design:

Assumptions at the Ultimate Limit States (Cl. 8.8.3)

Given the replacement of resistance factors with material resistance factors, the approach to flexural and axial resistance calculations have changed. These changes reflect the design philosophy adopted in CSA A23.1, the code governing the design of concrete structures.

In addition to a change in design philosophy, the equivalent rectangular stress distribution has been redefined. The equivalent uniform compressive stress is defined as $\alpha_1 \Phi_c f_c'$ where previously it was defined as $0.85f_c'$ and the depth over which this compressive stress acts is defined as $\beta_1 c$ where c is the depth to the neutral axis. The changes to β_1 and α_1 are summarized in Table 9.2

Table 9-2

Equivalent Rectangular Stress Distribution

	S6-88	S6-00
β_1	0.85 (f_c' = 35 MPa, β_1 = 0.85)	0.85-0.0015$f_c' \leq$ 0.67 (f_c' = 35 MPa, β_1 = 0.80)
α_1	0.85 if $f_c' \leq$ 30 MPa 0.85-0.008(f_c'-30)$\geq$ 0.65 if f_c' > 35 MPa (f_c' = 35 MPa, α_1 = 0.81)	0.97-0.0025$f_c' \geq$ 0.67 (f_c' = 35 MPa, α_1 = 0.88)

Although these are significant changes to the design philosophy, the effect on section capacities is nominal.

A further modification, which will also have a minor effect on section capacities, is the increase in ultimate compressive strain from 0.003 to 0.0035.

Tendon Stress at Ultimate Limit States (Cl. 8.8.4.2)

Unlike S6-88 where a simplified equation was provided to calculate f_{ps}, S6-00 requires that strain compatibility be used. The stress in the tendon, f_{ps}, is then calculated using a representative stress-strain curve for the prestressing steel. This change will not result in any significant changes to section capacities but does make the calculation of the flexural capacity an iterative process.

Minimum Reinforcement (Cl. 8.8.4.3)

The minimum reinforcement ratio of 1.4/F_y, as stated in S6-88, results in a higher reinforcement ratio than that required using the method outlined in S6-00. Minimum flexural reinforcing seldom governs; as a result, this change will have little influence on typical section designs.

Maximum Reinforcement (Cl. 8.8.4.5)

The definition of the maximum reinforcement ratio has been revised. However, the method outlined in S6-00 results in similar reinforcement ratios when compared to S6-88.

Prestressed Concrete Stress Limitations (Cl. 8.8.4.6)

The significant change to this clause is the removal of the compressive and tensile stress limitations at SLS. This clause therefore covers a range of designs from the use of non-prestressed reinforcing to fully prestressed.

When the tensile stress exceeds f_{cr}, the guidelines in Cl. 8.12 (Control of Cracking) govern.

Recommendations

The methodology as outlined in S6-00 be adopted.

9.8 SHEAR AND TORSION

This clause has been completely revised and is generally based on the Modified Compression Field Theory. This method, also known as the General Method, has previously been included in CSA A23.3, the code governing the design of concrete structures, and the Ontario Highway Design Bridge Code.

When designing plain reinforced concrete sections, the method can be simplified by assuming a 45° truss model as per S6-88. However, the design of prestressed concrete sections requires that the Sectional Design Model as outlined in Cl. 8.9.3 be adopted.

If the simplified method is used, there are no significant changes to section capacities when compared to S6-88.

The use of the Sectional Design Method for prestressed concrete components results in increased stirrup spacing and an increase in the maximum factored shear stress that can be applied to a section when compared to S6-88. Previously, the maximum factored shear stress was limited by v_s ($v_s \leq 0.66\sqrt{f_c'}b_w d$). With the introduction of S6-00 the maximum factored shear stress is now limited to *0.25* $\Phi_c f_c'$. For 35 MPa concrete, this represents a 68% increase in the maximum factored shear stress in a prestressed concrete beam.

Recommendations

The methodology as outlined in S6-00 be adopted.

9.9 DURABILITY (CL. 8.11)

S6-00 directly addresses the durability of concrete structures. Although numerous issues are dealt with, only issues that directly affect forestry bridges have been reviewed. These include:

- Concrete quality,
- Concrete cover and tolerances.

In order to make comparisons with current MoF guidelines, the assumption that typical concrete components in forestry bridges are not subject to aggressively corrosive environments has been used.

Concrete Quality (Cl. 8.11.2.1)

In an effort to increase concrete durability, minimum concrete strengths have been specified for components exposed to aggressive environments. These concrete strengths are higher than those typically used in forestry bridge design as well as required for Exposure Class C1 as defined in CSA A23.1. Table 9.3 summarizes the changes.

Table 9-3

S6-00 Minimum Concrete Strengths

Deterioration Mechanism	Exposure	Minimum Concrete Strength (MPa)	MoF Requirements (MPa)
Chloride induced corrosion (marine)	Airborne salts	45	35
	Tidal and splash spray	40	35
Chloride induced corrosion (other)	Cyclic wet/dry	45	35
Freeze-thaw	Unsaturated	40	35

If we assume that the majority of forestry bridges are not subject to an aggressive environment but may be subject to freeze-thaw, the minimum concrete strength would have to be increased from 35 to 40 MPa.

Concrete Cover and Tolerances (Cl. 8.11.2.2)

The typical minimum cover requirements have been increased in S6-00. Table 9.4 summarizes these changes assuming that the elements listed are not subject to an aggressive corrosive environment.

A further complication is the inclusion of tolerances in the specified covers.

Table 9-4

Typical Concrete Cover Requirements

Element	S6-00	S6-88
Top of precast concrete decks	60	50
Soffit of precast concrete decks	40	25
Pile caps	50	30
Rear face of ballast wall	55	35
Front face of ballast wall	50	25
Footings	55	35

Should the Ministry wish to comply with the concrete strength and cover requirements of S6-00, revision of the Standard Drawings would be required. Although an increase in concrete strength will not have a significant effect on the design or fabrication costs, the increase in concrete cover would result in the following:

- Redesign of the majority of the standard components,
- Increased component costs,
- Increased component weights,
- Increased freight costs.

Recommendations

We recommend that the current criteria remain and the Standard Drawings not be revised, given that forestry bridges are not normally exposed to aggressively corrosive environments and their design life is significantly less than 75 years.

However, where structures are subject to an aggressive environment, the requirements of S6-00 be adopted.

9.10 REINFORCEMENT DETAILS (CL. 8.14 AND 8.15)

Detailing of reinforcement, including splice and lap lengths has been revised to reflect current practices included in CSA A23.1 and A23.3. Although the design philosophy has changed, this will not result in any significant changes to typical details encountered in forestry bridge design.

Recommendations

The methodology as outlined in S6-00 be adopted.

9.11 ANCHORAGE OF ATTACHMENTS (CL. 8. 16.7)

This is a new clause covering the design of anchor bolts.

Recommendations

The methodology as outlined in S6-00 be adopted.

9.12 BOX GIRDER BOTTOM FLANGE THICKNESS (CL. 8.20.3.2.)

Minimum bottom flange thickness for precast girders is specified as 100 mm, whereas there was no limit in S6-88. Adopting this clause would render all Ministry of Transportation standard twin cell box girders non-compliant as these have 90 mm bottom flanges. The current sections are adequate given the shorter design life assumed for forestry bridge design. In addition, increasing the bottom flange thickness would increase weight. Also of interest is that the Ministry of Transportation has initially chosen to ignore this requirement. We note however, that the cells are formed with stay-in-place forms, so if the code-compliant bottom flange thickness were adopted, it would not require precasters to reinvest in new permanent formwork.

Recommendations

The requirement of this clause not be adopted.

9.13 MULTIBEAM DECKS (CL. 8.21)

This clause requires that bridges comprising of precast girders or slab units, placed side by side (typical shear connected slabs), transfer the live load shear among the units by one of the following:

- 150 mm thick concrete structural slab,
- Grouted shear keys with lateral post-tensioning (as adopted in AASHTO LRFD),
- Other approved means.

The intention of this clause is to limit cracking in the grouted shear connector and the subsequent intrusion of de-icing salts and deterioration of the joint.

Recommendations

The requirements of this clause not be applied to bridges having welded shear connectors. Based on current bridge inspection information, little deterioration of the welded shear connectors has been identified.

The MoF may wish to review the bridge inspection information available and subsequently reconsider the use of the grouted shear keys based on the requirements of this clause.

TIMBER BRIDGES

SECTION 10

Clause 9 covers the design and construction of wood structures. There are some significant changes and revisions to this clause. These changes have been summarized and include the following:

- Modification of resistance factors,
- Modification of size effect factors,
- Modification of load sharing factor,
- Modification of factored shear,
- Revision of specified material strengths.

10.1 RESISTANCE FACTORS (CL. 9.4.4)

The only change to the resistance factors is the increase from 0.8 to 0.9 in flexure for glued-laminated timber.

Recommendations

The resistance factors as specified in S6-00 be adopted.

10.2 LOAD DURATION FACTOR (CL. 9.5.3)

There is no change to the load duration factor when compared to S6-88

10.3 SIZE EFFECT FACTORS (CL. 9.5.4)

10.3.1 Sawn Timber Stringers

The size effect factor for flexure has been revised to represent varying thicknesses of sawn timber stringers. The values for timber stringers typically used in bridge construction have not been altered significantly. The typical size effect factor for flexure has increased from 0.8 to 0.9.

The size effect factor for shear has been revised significantly and is now based on the volume of the sawn timber stringer under consideration. This results in a reduction in the size effect factor for shear. The greater the volume of the beam, the more significant the reduction. This change results in a 25% reduction in the size effect factor when compared to that calculated using S6-88.

10.3.2 Glued-Laminated Timber

The size effect factor for Glued-Laminated beams in flexure is no longer based on the volume of the beam and is specified as 1.0. This typically results in a 25% increase in the size effect factor when compared to that calculated using S6-88. This will result in an increase to the flexural capacity.

The size effect factor for Glued-Laminated beams in shear is still based on the volume of the beam but has been altered significantly. This change results in a reduction of approximately 40% when compared to that calculated using S6-88. This will result in a reduction of the section shear capacity.

Recommendations

The size effect factors as specified in S6-00 be adopted.

10.4 LOAD SHARING FACTOR (CL. 9.5.6)

The load sharing factor has been modified and is consistent with the Ontario Highway Bridge Design Code. The revised methodology generally results in increased load sharing factors when compared with S6-88.

Recommendations

The load sharing factors as specified in S6-00 be adopted.

10.5 FLEXURAL RESISTANCE (CL. 9.6)

The methodology used to calculate the flexural resistance of timber sections has not changed significantly. The section capacities will change slightly due to revised resistance and modification factors as noted in the previous sections.

10.5.1 Sawn Timber Stringers

Based on the changes to the specified strengths and the various modification factors, as outlined in S6-00, the calculated flexural resistance for sawn timber stringers typically decreases by approximately 5%.

10.5.2 Glued-Laminated Timber Girders

Based on the changes to the specified strengths and various modification factors, as outlined in S6-00, the calculated flexural resistance of glued-laminated timber girders increases by 50-75%.

Recommendations

The methodology as outlined in S6-00 be adopted.

10.6 SHEAR RESISTANCE (CL. 9.7)

The methodology for calculating the factored shear load has changed significantly. Research has indicated that the shear resistance of timber stringers is dependant on both the volume of the beam and the shear loading pattern. The methodology for calculating the factored shear force has been altered to account for this. One result of these changes is that it is not possible to directly compare factored shear loads and resistances between S6-88 and S6-00.

For shorter bridges using multiple sawn timber stringers, the changes typically result in an increased shear capacity. For longer span glued-laminated girders, the changes typically result in a reduced shear capacity.

Recommendations

The methodology as outlined in S6-00 be adopted.

10.7 SAWN WOOD (CL. 9.11)

This clause outlines the specified strengths and properties for sawn wood. There is a noticeable reduction in specified flexural and shear strengths for sawn timber. These modifications will result in an equivalent reduction in section strengths when designing new structures. Table 10.1 summarizes the changes to the specified strengths for typical wood species.

Recommendations

The specified strengths as stated in S6-00 be adopted for the design of new structures.

10.8 GLUED-LAMINATED TIMBER (CL. 9.12)

This clause outlines the specified strengths and properties for glued-laminated timber. A comparison between S6-00 and S6-88 shows that the specified flexural strength has increased by 18% while the specified shear strength has increased by 25% (refer to Table 10.1).

Recommendations

The specified strengths as stated in S6-00 should be adopted for the design of new structures.

Table 10-1

Specified Timber Strengths

Material	Flexure		Shear	
	S6-88	S6-00	S6-88	S6-00
D/Fir Select Structural	24	19.5	1.1	0.9
D/Fir No. 1	20	15.8	1.1	0.9
D/Fir No. 2	20	9	1.1	0.9
Glued-Laminated (24f-E)	23.4	27.5	1.1	1.4

10.9 LOG STRINGER BRIDGE DESIGN

Unfortunately neither S6-88 or S6-00 effectively covers the design of log stringer bridges. Typically engineers have resorted to engineering judgement or tables developed by the MoF and FERIC. This has resulted in a non-uniform approach to the design of log stringer bridges.

In order to address this issue, guidelines should be developed to aid engineers in the design of log stringer bridges. These guidelines can be included in the Forest Service Bridge Design and Construction Manual and would form the basis for log stringer bridge design. All future designs would conform to these guidelines.

STEEL DESIGN

SECTION 11

Clause 10 of S6-00 covers the design of structural steel bridges, including structural steel components, welds, bolts and other requirements related to erection and fabrication. Numerous changes and revisions have been made when compared to S6-88. Only those changes and revisions that are relevant to forestry bridge design are discussed, these include:

- Resistance factors,
- Durability,
- Design detail requirements,
- Compression members,
- Beams and girders,
- Composite beams and girders,
- Shear connectors,
- Structural fatigue requirements,
- Fracture critical components,
- Bolted connections,
- Fabrication requirements,
- Construction requirements.

11.1 RESISTANCE FACTORS (CL. 10.5.7)

Although the design philosophy has not changed, the resistance factors have been revised to reflect

- Increased design life of 75 years,
- Safety reliability index of 3.75,
- More recent information on the reliability of the various materials.

Table 11.1 summarizes the changes to the various resistance factors (only factors typically used in design have been included).

Table 11-1

Resistance Factors

Factor	S6-00	S6-88
Flexure	0.95	0.9
Shear	0.95	0.9
Compression	0.9	0.9
Tension	0.95	0.9
Concrete in Composite Construction	0.75	0.7
Bolts	0.8	0.67
Welds[1]	0.67	0.671
Shear Connectors	0.85	0.8

[1]S6-88 referred engineers to CSA W59

Recommendations

We recommend that the revised resistance factors be adopted.

11.2 DURABILITY (CL. 10.6)

Prior to the introduction of S6-00, the issue of durability was left to the discretion of the engineer. S6-00 requires that as a minimum, the corrosion protection measures as specified be implemented.

Table 11.2 summarizes the requirements of S6-00 as they would apply to forestry bridges. We have not included exposure to airborne chlorides, light industrial atmosphere or heavy industrial atmosphere as these are not typical environments encountered with the Forest Industry.

Table 11-2

S6-00 Minimum Requirements for Corrosion Protection

	Environmental Exposure Condition			
	No Direct Chlorides			
Component	**Wet, Rarely Dry**	**Dry, Rarely Wet**	**Cyclical Wet/Dry**	**Marine**
All superstructure	Coat	Uncoated weathering steel	Uncoated weathering steel	Coat
Structure with clearance over stagnant water $\leq$ 3.0 m or over fresh water $\leq$ 1.50 m	Coat	Coat	Coat	Coat
Faying surfaces of joints	None	None	None	None
Substructure	Coat	Uncoated weathering steel	Uncoated weathering steel	Coat
Bearings	Galvanize, metallize, coat	Galvanize, metallize, coat	Galvanize, metallize, coat	Galvanize, metallize, coat
Railings	Galvanize	Galvanize	Galvanize	Galvanize

S6-00 also requires that all elements subject to water runoff and within 3000 mm from fixed or expansion joints be coated with an approved system.

In addition to corrosion protection, structures should also be detailed to facilitate inspection and minimize any potential water pooling effects.

Recommendations

The corrosion protection requirements of S6-00 have been included to increase the likelihood that steel structures attain the required 75-year design life. These requirements may not be appropriate for forestry bridges which have a shorter specified design life. The durability requirements in S6-00 could be simplified as follows:

- All steel plate to be uncoated weathering steel except for bridges in a marine environment (within 1 km of the coast or subject to salt spray or fog).
- Where bridges are situated in a marine environment (located within 1 km of the coast and subject to salt spray or salt fog) the steel structure shall be coated with an approved protective coating.
- If the steel is not weathering steel, it should be coated with an approved protective coating.
- Steel substructure to be coated with one coat of bituminous paint. Allowance shall also be made for a 3-mm section loss due to corrosion.
- Angle and tees should be orientated so that vertical legs or webs are extending downwards wherever practical.
- Steel decks shall be waterproofed and provided with a skid-resistant wearing surface.

11.3 DESIGN DETAIL REQUIREMENTS (CL. 10.7)

This is a new addition to the code and is intended to cover typical detailing requirements for steel bridges. The following issues are applicable to forestry bridges:

- Minimum section thickness,
- Camber requirements.

Minimum Section Thickness (Cl. 10.7.2)

Prior to the introduction of S6-00 there was no guidance to the minimum thickness of steel section/plate that could be used. With the introduction of this clause, all webs of fabricated steel plate girders, gusset plates for main members, end diaphragms and their connections must be fabricated from plate that is at least 10 mm thick.

Camber (Cl. 10.7.4)

This clause requires that drawings include deflection due to dead load of steel alone and the deflection due to full dead load, including steel deck and wearing surface. This information is useful in the construction of highway bridges, for example in setting screed elevations for cast-in-place concrete decks.

Recommendations

The camber requirements as outlined in S6-00 be adopted with the following changes:

- Minimum steel plate thickness be reduced to 9.5 mm (3/8") as this is a commonly stocked steel plate.
- Only the total required camber be shown on the drawing, which is sufficient for fabrication.

11.4 TENSION MEMBERS (CL. 10.8)

This clause has been substantially expanded from that originally contained in S6-88. The design philosophy has been revised to reflect what is included in CSA S16.1, the code governing the design of steel structures. With these changes, there is a reduction in the tensile capacity of certain connections. This reduction will not have a significant impact on typical tension elements in forestry bridges.

Recommendations

The methodology as outlined in S6-00 be adopted.

11.5 COMPRESSION MEMBERS (CL. 10.9)

This clause has been substantially expanded when compared to S6-88. The following changes are relevant to forestry bridges:

- Definition of four classes of sections,
- Flexural buckling,
- Flexural-torsional buckling.

Width to Thickness Ratio of Elements in Compression (Cl. 10.9.2)

Previously, S6-88 only included three section classifications, compact, non-compact and other. With the introduction of S6-00, this has been increased to four classifications, Classes 1, 2, 3, and 4. Class 1 and Class 2 sections can attain plastic capacity while Class 2 sections will not necessarily allow for moment redistribution (this is a new addition). Class 3 sections are able to attain a capacity based on the yielding of the steel (these sections were originally designated "non-compact"). Class 4 sections exceed the limits of Class 3 sections and are to be treated on a case by case basis.

Flexural Buckling (Cl. 10.9.3.1)

The four equations defining the compressive resistance of sections based on the appropriate buckling curve have been replaced with a single equation.

Flexural-Torsional Buckling (Cl. 10.9.3.2)

A new clause has been included to calculate the flexural-torsional buckling load for various sections. Previously this check was required, but no guidance was given on how it was to be completed. The equation included in S6-00 is a polynomial that is solved for the lowest root, i.e., least load.

Recommendations

The methodology outlined in S6-00 should be adopted.

11.6 BEAMS AND GIRDERS (CL. 10.10)

Numerous changes have been made to the clauses governing the design of beams and girders in flexure and shear. The changes include:

- Guidance for the design of mono-symmetric sections,
- Revised design criteria for stiffened plate girders,
- Revised shear design criteria,
- Revised requirements for transverse stiffeners.

Width to Thickness Ratios for Flexural Members (Cl. 10.10.2.1)

The girder *h/w* limitations presented in Clause 10.9 are based on doubly-symmetric sections. S6-00 has also clarified that the effective height of the web for mono-symmetric sections is equal to twice the depth of compression in the web.

Flexural Capacity of Laterally Unbraced Members (Cl. 10.10.2.3)

Some minor modifications have been made to the methodology and formulae used to calculate the flexural capacity of laterally unsupported sections. These changes allow for the inclusion of mono-symmetric sections. Prior to the introduction of S6-00, engineers were directed to the Structural Stability Research Council's *Guide to Stability Design Criteria for Metal Structures*.

Stiffened Plate Girders (Cl. 10.10.4)

Although there is no change to the *h/w* limits, that define when longitudinal stiffeners are required, the definition has been revised to clarify how these limits are applied to mono-symmetric sections.

If this clause is applied to mono-symmetric sections during construction (non-composite section), it results in webs requiring longitudinal stiffeners where previously they were not required. This is because d_c is greater than *h/2* for mono-symmetric sections. The clause does not address this issue directly but the assumption would be that girders are required to meet all code requirements at all stages of construction.

The moment reduction factor applied to slender webs, although similar to that included in S6-88, is now applied to less slender webs. This change will not have a significant effect on typical girder sections.

Shear Resistance (Cl. 10.10.5)

Significant changes have been made to this clause. S6-00 has divided the shear capacity of steel webs into two parts, shear buckling capacity (F_{cr}) and tension field post buckling capacity (F_t). If the web is unstiffened, only the shear buckling capacity may be assumed.

For stiffened webs, both the shear buckling capacity and tension field post buckling capacity can be assumed. For end panels, only the shear buckling capacity can be assumed as there is no anchorage for the tension force required to develop the tension field capacity.

Intermediate Transverse Stiffeners (Cl. 10.10.6)

Prior to the introduction of S6-00, there was no clear indication as to when transverse stiffeners were required. S6-00 clearly defines the when transverse stiffeners are required by introducing two new criteria:

- Handling requirements: If $h/w \geq 150$, transverse stiffeners are to be included to allow for efficient handling of girders with slender webs during fabrication and erection.
- Unstiffened web capacity: If the shear strength of the unstiffened web is less than the factored shear, stiffeners are not required as long as $h/w \leq 150$.

The formula defining the minimum stiffener size has been revised to include an allowance for the strengthening effect of the web. This has resulted in a significant reduction in the size of transverse stiffeners when compared to designs based on S6-88.

Bearing Stiffeners (Cl. 10.10.8)

The only significant change to this section has been the revision of the web crippling and web yielding formulas. These formulas are similar to those included in CAN/CSA S16.1-01.

Lateral Bracing (Cl. 10.9)

In line with what was previously included in S6-88, lateral bracing systems are to be designed to resist a lateral load equivalent to at least 1% of the compression flange force at the location under consideration in addition to any additional loads. In addition to this, S6-00 requires that lateral bracing systems be designed for forces they attract in maintaining compatibility of deformations of girders under vertical loading.

This additional force resulting from deformation compatibility will only present a problem in longer non-composite systems where the deck is unable to resist lateral loads. In these bridges, the lateral plan bracing is required to ensure lateral stability of the structural system. In order to prevent premature failure additional, the compressive loads that result from deformation compatibility need to be accounted for.

Where a composite deck system is used, lateral plan bracing is not required once construction is complete. Any subsequent failure of the plan bracing, due to excessive compressive loads resulting from deformation compatibility, will not affect the structural integrity of the bridge.

Unlike S6-88, no spacing limitation has been placed on intermediate diaphragms.

Recommendations

The methodology outlined in S6-00 be adopted with the following changes or clarifications:

- Clarification should be provided by the MoF on the applicability of the longitudinal stiffener requirements when considering construction loading.
- Where the designer chooses to exceed the current maximum diaphragm spacing of 7.5 m, care should be taken to ensure that all applicable buckling criteria are satisfied especially when considering construction loading.

11.7 COMPOSITE BEAMS AND GIRDERS (CL. 10.11)

A number of changes have been made to the clauses relating to the design of composite beams and girders. These changes are summarized below:

Control of Permanent Deflections (Cl. 10.11.4)

The maximum stress in the girders due to serviceability dead and live loads has been limited to $0.90F_y$. Prior to the introduction of S6-00 there was no explicit stress limit at the Service Limit State.

Flexural Capacity of Composite Beams and Girders (Cl. 10.11.5-7)

Some significant changes have been introduced when calculating the flexural capacity of composite beams and girders. Prior to the introduction of S6-00, full plastic stress distribution could only be assumed if the depth of the plastic neutral axis (PNA) in the web was less than $685w/\sqrt{Fy}$. If the PNA exceeded this limit, an elastic stress distribution was assumed based on first yield. With the introduction of S6-00, the following changes have been made:

- The maximum depth of the PNA in the web has been increased to $850w/\sqrt{Fy}$.
- If the PNA exceeds this limit, a plastic stress distribution can still be assumed if the guidelines outlined in the code are followed.

These changes will have a significant effect on the design of the more heavily loaded longer span bridges (>40 m) where sections have typically been designed using an elastic stress distribution. The ability to calculate a flexural resistance based on a plastic stress distribution will result in lighter steel sections for longer span bridges. This may eventually result in Service Limit State stresses controlling the design of these structures.

Shear Connectors (Cl.10.11.8)

This clause is similar to that included in S6-88. The maximum spacing between shear connectors has again been limited to 600 mm. Typical MoF Standard Composite Deck Panels have a shear connector spacing of 1100 mm.

Recommendations

The methodology outlined in S6-00 be adopted with the following revision:

- The maximum shear connector spacing may exceed 600 mm when using standard MoF precast concrete composite deck panels.

11.8 STRUCTURAL FATIGUE (CL. 10.17)

Although the philosophy behind fatigue design has not changed between S6-88 and S6-00, there are some significant changes to S6-00 based on research conducted in the USA, and now included in the AASHTO LRFD Bridge Design Code.

Fatigue Stress Range and Design Criteria (Cl. 10.17.2.1-2)

The calculated fatigue stress range has been reduced from 100% to 52% of the effect of a single CL625 design truck. This adjustment recognizes that the weight of the majority of loaded trucks is less than that of the design truck and that the weight of the design truck is heavier than typical legal axle limits.

Number of Cycles (Cl. 10.17.2.3)

The number of cycles that a detail is subject to is based on the following information:

- Design life,
- Number of trips per day,
- Number of cycles per trip (single cycle for spans greater the 12 m and two cycles for spans less than 12 m).

S6-88 previously defined the number of cycles, ranging from 100,000 to 2,000,000+ and the MoF adopted 500,000 cycles as a standard requirement.

Fatigue Stress Range Resistance (Cl. 10.17.2.3)

The fatigue stress range resistance is based on the calculated number of cycles with a minimum value of 50% of the constant amplitude fatigue threshold stress range. Table 11.3 shows a comparison between S6-00 and S6-88 for the MoF required 500,000 cycles.

Table 11-3

Comparison of Fatigue Stress Range Resistance (500,000 cycles)

Category	S6-00 (MPa)	S6-88 (MPa)
A	254	250
B	198	190
B1	158	190
C	142	130
C1	142	130
D	113	110
E	89	85

A review of Table 11.3 shows that two additional subcategories have been created. Category B1 involves specific full or partial penetration welds while category C1 refers to the fillet weld at the toe of a transverse stiffener and the transverse stiffener-to-flange weld. In addition to these changes, the allowance fatigue stress is no longer reduced by 25% if the detail under consideration is fracture critical.

Fatigue Resistance of Stud Shear Connectors (Cl. 10.17.2.5)

The calculation of the permissible range of interface shear for stud type shear connectors has been revised. These changes have a nominal effect on the resistance of a single shear connector if the total number of cycles remains at 500,000.

Distortion Induced Fatigue (Cl. 10.17.3)

This clause requires that all diaphragms and lateral bracing be connected to both the tension and compression flanges using transverse connection plates. The intention of this clause is to limit distortion fatigue in the girder web. Given that the lateral plan brace is typically connected to the top flange, it does not induce distortion fatigue in the web.

In addition to this change, the minimum force that the connection is to be designed for has been increased from 80 kN to 90 kN.

Recommendations

The methodology outlined in S6-00 be adopted with the following changes or clarifications:

- **Calculated Fatigue Stress Range:** The calculated fatigue stress range should be based on 100% of the design logging vehicle (the use of the BCFS L150 truck for fatigue design on BCFS L165 bridges should remain unchanged). Unlike highway structures that are subject to mixed traffic, logging bridges will typically be subject to the fully loaded design truck. This reduction factor is therefore inappropriate for logging bridge design.

- **Number of Cycles:** The MoF should define the number of cycles to be used for fatigue calculations. For simplification, the existing number of 500,000 cycles can be adopted. However, shorter structures (less than 12 m) are subjected to more as they are generally governed by axle loads rather than whole trucks and an increase may be appropriate for permanent bridges.

  If the MoF is not comfortable with this simplification, the number of cycles could be based on span length. Such a definition such as 500,000 cycles for spans >12 m and 1,000,000 cycles for spans <12 m could be adopted.

- **Fatigue Stress Range Resistance:** The fatigue stress range resistance as defined in S6-00 be adopted with no changes.

- **Distortion Induced Fatigue:** Attaching the lateral plan brace to the top flange as typically practiced is an acceptable detail for simple span structures. If the lateral plan bracing is attached to the web, the guidelines as specified in S6-00 should be adopted.

11.9 SPLICES AND CONNECTIONS (CL. 10.18)

Bolted Connections (Cl. 10.18.2)

Although the design philosophy has not changed, both the material resistance factor for bolts and the friction parameters for various surfaces have been revised. Table 11.4. tabulates the results of these changes for a typical M22 bolt.

Table 11-4

Bolt Connection Resistance

Connection	S6-00 (kN)	S6-88 (kN)
Ultimate shear capacity of single M22 in double shear	302.8	253.6
Slip-critical shear capacity of single M22 in double shear	90.5	96.8

A review of Table 11.4 shows that the slip-critical resistance has decreased but the ultimate shear capacity has increased.

Welds (Cl. 10.18.3)

Weld design is similar to S6-88 which required that all weld be designed in accordance with CSA W59.

Detailing of Bolted Connections (Cl. 10.18.4)

This clause is more extensive than that previously found in S6-88 with more instructions given regarding the detailing of bolted connections.

Recommendations

The methodology outlined in S6-00 be adopted.

11.10 ANCHOR BOLTS (CL. 10.19)

This is a new clause that covers the design of anchor bolts. In addition to satisfy the requirements of this clause, the requirements outlined in Clause 8.16.7 must also be satisfied.

Recommendations

The methodology outlined in S6-00 be adopted.

11.11 PILES (CL. 10.22)

This is a new clause that covers the design of steel pipe piles. In addition to satisfying the requirements of this clause, the requirements outlined in Section 6 and 10 must also be satisfied.

Recommendations

The methodology outlined in S6-00 be adopted.

11.12 FRACTURE CRITICAL COMPONENTS (CL. 10.23)

Typical steel forestry bridges are comprised of twin girder systems. Based on this configuration, the girders are considered fracture critical components as the failure of a single girder will result in the collapse of the bridge. In order to control potential fracture, S6-88 reduced the allowable fatigue stress range by 25% and specified a minimum average energy absorption and test temperature for the Charpy Impact Test (refer to Table 11.5). (This test was required on a per plate frequency.) This test frequency requirement has typically not been consistently applied by engineers and fabricators.

One of the reasons for the inconsistent application of the per plate testing frequency requirement is that Grade 350 AT Category 3 steel meets the S6-88 requirements if a per heat testing frequency is assumed. As a result, sourcing and purchasing of steel plates are simplified and mill ordering is not required.

In S6-00 fracture control and fatigue have been separated. Fracture critical elements are no longer subject to the reduced allowable fatigue stress but do require an increased notch toughness as indicated in Table 11.5. These revisions result in Grade 350 AT category 3 plate no longer conforming to the requirements for fracture critical components even when a per heat testing frequency is completed.

Table 11-5

Charpy Impact Test Criteria

	Test Temperature (°C)	**Minimum Energy Absorption (Joules)**	**Frequency of Tests**
CSA G40.21-Grade 350 AT Cat 3	-29	27	per heat
S6-88	-30	27	per plate
S6-00	-20	40	per plate

Based on discussions with steel fabricators and suppliers, the following concerns have been raised regarding the adoption of the requirements contained in S6-00:

- The cost premium to supply steel conforming to the S6-00 requirements is an extra $1.50 per 100 pounds of steel supplied. This will result in an estimated overall increase in the fabrication cost of steel girders of approximately 3%.
- Numerous fabricators have inventoried significant quantities of steel plate that would not conform to the new specifications.
- All steel plate would have to be specially mill ordered, which may place undue financial hardship on the steel fabricators who wish to inventory material.

Welding requirements for fracture critical elements typically refer the fabricators to the relevant CSA Standards or the equivalent American Welding Society Standards. Given that these represent standard practices, adoption of these clauses should not pose any hardship on the majority of CSA W47.1 Division 1 or Division 2.1 certified steel fabricators.

Recommendations

Since the new notch toughness requirements are not reflected in any of the standard categories included in CSA G40.21 (Structural Quality Steels), the MoF has the following options:

- Rather than adopt the requirements as outlined in S6-00, the MoF could maintain the present practice. This would require the use of 350 AT category 3 steel (tested on a per heat frequency) and a 25% reduction in the allowable fatigue stress range.
- Adopt the requirements as outlined in S6-00.

Given the complex nature of this issue, the MoF must further investigate this issue prior to making any decisions. The investigation should address the following issues:

- Why the methodology has been revised?
- Is the safety of the structure compromised if the present practice is maintained?
- Effect of per plate versus per heat testing frequency on the quality of the steel plate?
- How will the change affect the fabrication costs of steel girders?
- What are the implications of the change for steel fabricators?
- What impact will mill ordering have on planning and construction scheduling?

The clause outlining the welding requirements for fracture critical components as outlined in S6-00 be adopted.

11.13 CONSTRUCTION REQUIREMENTS FOR STRUCTURAL STEEL (CL. 10.24)

This is a new clause that has been added to ensure that the construction complies with the design philosophy of Clause 10. This Clause draws together information that is typically included in Contract Specifications, numerous other guidelines and standards covering the construction and fabrication of steel bridges.

Submission (Cl. 10.24.2)

The contractor is required to submit erection diagrams, shop details, welding and erection procedure drawings with calculations to the owner. These requirements may be onerous for typical forestry bridge construction given the repetitive nature of the work.

Materials (Cl. 10.24.3)

This clause requires that the materials supplied conform to what was assumed during the design. The majority of what is stated can typically be summarized within the drawing notes and included in the MoF Forest Service Bridge Design and Construction Manual.

Fabrication (Cl. 10.24.4)

This clause summarizes standard fabrication practices.

Welded Construction (Cl. 10.24.5)

This clause summarizes the typical welding practice that is expected from companies certified to CSA W47.1 Division 1 or Division 2.1. S6-00, like S6-88, requires that companies fabricating steel girders be certified to Division 1 or Division 2.1 of CSA W47.1.

Bolted Construction (Cl. 10.24.6)

This clause summarizes typical bolted construction practices.

Tolerances (Cl. 10.24.7)

This clause summarizes the tolerance requirements for steel fabrication and construction.

Quality Control (Cl. 10.24.8)

This clause summarizes the requirements for weld testing and repair. The requirements outlined are more stringent than what is typically practiced for forestry bridges.

Transportation and Delivery (Cl. 10.24.9)

This clause requires that all girders be transported with the webs in the vertical plane. The intent of this clause is to limit web distortion during transportation. This requirement may be impractical at times. Should contractors wish to transport girders with the webs in the horizontal plane, an engineered lifting procedure can be developed in addition to providing sufficient dunnage to support the web.

Erection (Cl. 10.24.10)

This clause summarizes the typical requirements for the erection of steel bridges. The only item in this clause that may pose some difficulties for the MoF is the requirement that all field welders be certified to Division 1 or Division 2.1 of CSA W47.1.

Recommendations

The methodology outlined in S6-00 be adopted with the following changes or clarifications:

- Weld inspection requirements conform to Forest Service Bridge Design and Construction Manual.
- Girders can be transported with webs in horizontal position only when approved by the Engineer.
- Field welder certification to be in accordance with MoF requirements.

JOINTS AND BEARINGS

SECTION 12

Clause 11 covers the design of joints and bearings. No significant changes have been made to the design and specification of elastomeric bearing pads. It should be noted that when S6-00 is compared to BS5400 (British Bridge Design Code), AASHTO LRFD and the Goodco Bearing catalogue, it is the only Code that does not modify the shape factor for plain bearing pads. Most codes typically increase the shape factor by 100%. This has the effect of reducing the allowable stress on the plain elastomeric bearing pads.

Recommendation

The MoF give consideration to developing specific design criteria for plain rubber pads.

BARRIERS AND HIGHWAY ACCESSORY SUPPORTS

SECTION 13

Clause 12 of S6-00 covers the design requirements for typical traffic barriers. A new requirement in S6-00 is that all barriers be crash tested or evaluated based on performance when subject to collisions. The typical barrier details that have been adopted by the MoF do not conform to any specific design criteria and do not meet the design requirements of either S6-00 or S6-88.

Recommendation

The methodology outlined in S6-00 be replaced with the requirement that all barriers conform to typical MoF standard barrier designs.

EVALUATION OF EXISTING BRIDGES

SECTION 14

Clause 14 provides guidance on the evaluation of existing structures given the availability of specific information relating to various aspects of the evaluation. The philosophy is similar to that used in Clause 12 of S6-88. However, this Clause has undergone some significant changes.

The approach is generally applicable to the evaluation of typical forestry bridges and can be adopted. However, some clarification should be given to the following items to create consistent load rating criteria.

- Permit vehicle loads,
- Material strengths,
- Resistance adjustment factors,
- Evaluation of log stringer bridges.

14.1 PERMIT VEHICLE LOADS (CL. 14.8.2)

Forestry bridges are evaluated using standard design logging trucks (or a scaled multiple thereof). These loads are likely to be less variable than standard highway vehicles, the three-level approach outlined in Cl. 14.8.1 is not appropriate. This clause implies that the live load evaluation has to be treated as a permit load and hence falls into one of the following categories:

- Permit annual (or project),
- Permit bulk haul,
- Permit controlled,
- Permit single trip.

For each permit load, S6-00 then defines the target reliability index (β) and the associated live load factors. Given that the live load factors vary for each type of permit load, the Live Load Reduction Factor will vary for the same bridge depending on the type of permit load assumed.

Recommendation

Logging trucks do not directly conform to any of these categories. Therefore, we suggest that the MoF adopt a vehicle category when evaluating steel, concrete or timber bridges (excluding log stringer bridges).

14.2 MATERIAL STRENGTHS (CL. 14.6)

Tables are provided to aid the Engineer in estimating material strengths if no design drawings are available. Unfortunately, no information is provided for timber structures and Engineers are referred to Clause 9 of S6-00. Given that timber strengths have been continually reduced over the last 20 years with the release of each new code, evaluation of these structures based on the new code always yields a deficient structure.

To address this discrepancy, the MoF should develop a table similar to the one included in Cl. 14.6.3. Such a table would allow the designer to estimate specified material strengths based on the age of the structure.

Recommendations

The methodology outlined in S6-00 be adopted and a table summarizing specified timber strengths based on age be developed.

14.3 RESISTANCE ADJUSTMENT FACTORS (CL. 14.13.2)

These factors allow for the modification of the calculated section resistance if the components show no visible sign of defect or deterioration. Where no value is specified, unity is assumed.

Recommendation

The resistance adjustment factors as specified in S6-00 be adopted.

14.4 EVALUATION OF LOG STRINGER BRIDGES (NOT INCLUDED IN S6-00)

As discussed in Section 10 of this report, S6-00 does not effectively cover the design or evaluation of log stringer bridges. We suggest that the guidelines outlined in Section 6 be adopted for the evaluation of existing log stringer bridges.

All evaluations are to be based on actual stringer sizes (discounted to account for measured rot). Since the MoF requires all timber structures to be inspected every two years, the 1-2 year allowable stress could be used for evaluation purposes (see Table 10.1).

REHABILITATION

SECTION 15

Clause 15 of S6-00 defines the minimum requirements for the rehabilitation of existing bridges. We recommend that this Clause be adopted without any changes.

SUMMARY

SECTION 16

This Section provides a summary of the recommended changes or recommendations for further study. Where clarification is required, refer to the relevant Section in this report for a more detailed discussion.

16.1 CLAUSE 1 - GENERAL

Revise the Forest Service Bridge Design and Construction Manual to include the parts of Clause 1 in S6-00 that are relevant to forestry bridge design and construction.

16.2 CLAUSE 2 - DURABILITY

Revise the Forest Service Bridge Design and Construction Manual to include the durability criteria that are applicable to forestry bridge design. Such criteria may include:

- Standard bridge abutment and span joint details incorporating rubber seals to protect the bearings from deck run-off,
- Provision for access to complete close proximity inspections of bridge girders and piers.

16.3 CLAUSE 3 - LOADS

Design Life and Annual Reliability Index (Cl. 3.5.1)

The values defined in S6-00 ensure a prescribed uniform level of reliability based on an acceptable probability that the factored loads will be exceeded during a specific time period for typical highway bridges subject to the specified design loads. This “level of safety” may not be appropriate for use in forestry bridge design.

We suggest that the MoF define an appropriate design life and annual reliability index. Based on these revised values, load factors, fatigue life and concrete cover requirements can be developed.

Deflection Limitations (Cl. 3.4.4)

Forestry bridges are not designed to accommodate pedestrians. Therefore, we suggest that the MoF guidelines for steel girder and concrete slab bridges remain unchanged.

We recommend that the deflection limitations be relaxed further to accommodate all-steel portable bridges. These bridges are lighter and shallower than existing portable bridge designs and are subject to higher live load deflections. These deflections do not conform to the current MoF guidelines. For this type of bridge, the deflection criteria should be reduced to *span/300*.

Load Factors and Load Combinations (Cl. 3.5.1)

The live load factors included within S6-00 are intended to be applied to the CL-625 design vehicle and are calibrated based on the specified design life and annual reliability index. Based on this definition, the factors may not be appropriate for forestry bridge design.

Cl. 3.8.3 allows for the use of "site specific" vehicles with appropriate load factors as long as the resulting level of safety is not lower than that provided by S6-00. Based on this clause we recommend that the MoF investigate the suitability of the live load factors when applied to off-highway logging trucks.

Live Loads - CL-W Loading (Cl. 3.8.3)

In addition to the axle weight and configuration study currently being undertaken, the MoF may wish to revise the maximum truck eccentricity and associated off-balance wheel loading. We suggest that this be adjusted to reflect the philosophy used in S6-00, and as noted below.

Ultimate Limit State and Serviceability Limit State

- The minimum distance between the wheel centerline and the edge of curb shall be 600 mm (except for the design of slabs where this distance shall be reduced to 300 mm).

- The axle load shall be evenly distributed between both wheels (i.e., no allowance for off-balance wheel loading).

Fatigue Limit State

- A single design vehicle located at the centre of the travelled lane.
- The axle load shall be evenly distributed between both wheels (i.e., no allowance for off-balance wheel loading).

In order to account for the possibility that there could be more than two trucks on a bridge simultaneously, we recommend that the requirement for two trucks be retained for bridges where the length exceeds 40 m.

Dynamic Load Allowance (Cl. 3.8.4.5)

With the complexity of choosing the appropriate DLA, we recommend the following values be adopted:

- Span < 10 m, DLA = 30%
- Span ≥ 10 m, DLA = 25 %
- Where only a single axle is used, the value of 40% as required by S6-00 be adopted (when using a tandem axle for deck designs we recommend a value of 40% be applied to both axles).

Construction Loading and Load Factors (Cl. 3.16)

We recommend that the load factors as outlined in S6-00 be adopted. The MoF should also specify a minimum erection load to be included in all designs unless specifically noted. We recommend the following for the typical erection load:

- Self weight of structure including all deck panels in position and ungrouted.
- Vertical load of 445 kN (represents typical excavator (80,000 lbs) and payload (20,000 lb). The vertical load can be treated as a point load evenly distributed between two girders.

16.4 CLAUSE 4 - SEISMIC DESIGN

With the complexity of this clause, the recommendations noted below are to be read in conjunction with the discussion included in Section 5 of this report.

Importance Category (Cl. 4.4.2)

Forestry bridges should be considered as "other" bridges and the Seismic Performance Zone (SPZ) be limited to a maximum of 2 for forestry bridges.

Analysis for Earthquake Loads (Cl. 4.4.5)

The following is recommended:

- For longitudinal loads, check ballast wall flexural strength assuming a soil pressure derived from 100% of the bridge weight times the design ground acceleration. A triangular soil pressure distribution, with zero stress at grade, could be adopted.
- Adopt an analysis method (uniform or spectral) based on S6-00 Clause 4.4.5 and as limited by parameters discussed in Section 5 of this report.

Site Effects (CL. 4.4.6)

In lieu of better information, the following values could be assumed:

- S=1.2 for bridges on footings.
- S=1.5 for bridges on piles.
- S=1.0 only if adequate site information is available to justify this lower value.

Elastic Site Response Coefficient (Cl. 4.4.7)

We recommend the following:

- Adopt an Importance Factor of 1.0 unless otherwise specified by the Bridge Authority.

- Distribute transverse seismic loads among supports based on not less than 50% of the span length on each side of a support as a tributary mass (or weight) unless refined by suitable analysis.

Load Combinations (Cl. 4.4.9):

We recommend the following:

- Apply seismic loads in combination with dead loads only.
- Perform seismic design of regular bridges in two separate directions, neglecting directional combinations.

Foundations (Cl. 4.6):

We recommend the following:

- The Authority and Owner should recognize that seismically induced geotechnical risks to the crossing may exist and should be addressed where appropriate.
- Adopt structural aspects of this Clause as appropriate.
- Neglect provisions related to approach slabs unless otherwise directed by the Owner.

Concrete and Steel (Cl 4.7-4.8):

For a detailed discussion of these clauses, refer to Section 5.11 of this report.

16.5 CLAUSE 5 - METHODS OF ANALYSIS

Longitudinal Bending Moments and Shear in Multi-Spine Bridges (Cl. 5.7.1.3 and Cl. 5.7.1.5)

We suggest that appropriate load distribution factors for both longitudinal moment and shear be developed for single-lane shear connected slab bridges.

In addition, we suggest that the MoF develop a standard set of design guidelines for slab bridges to cover the following:

- Moment distribution (all limit states),
- Shear distribution (all limit states),
- Torsional loading in shear connected slabs,
- Torsional behaviour of beams versus slabs,
- Standard shear connector design for L75, L100, L150, and L165 logging trucks.

The MoF may also wish to reconsider the onerous design requirements for welded shear connected slab bridges. Based on the MoF research conducted the requirement that slabs be designed as though all shear connectors have failed at the ULS is excessively conservative.

16.6 CLAUSE 6 - FOUNDATIONS

Adopt this clause for the design of non-typical bridge foundations. Typical forestry bridge foundation design should be based on guidelines outlined in the Forestry Bridge Design and Construction Manual. These guidelines should include:

- Requirements for geotechnical investigations,
- Guidance for the design and installation of precast concrete footings and steel pipe piles.

16.7 CLAUSE 7 - BURIED STRUCTURES

The MoF adopt the requirement that all buried structures relying on soil structure interaction conform to the design criteria included in S6-00.

16.8 CLAUSE 8 - CONCRETE STRUCTURES

Concrete Strength (Cl. 8.4.1.2)

We recommend that a minimum concrete strength of 30 MPa be adopted.

Durability (Cl. 8.11)

Revision of the minimum concrete strength and cover requirements in S6-00 will result in the current Standard Drawings having to be revised, resulting in increased component weights and associated costs. We suggest that the current criteria remain based on the limited exposure to a corrosive environment and limited vehicular use of the typical structures.

Where structures are subject to a corrosive environment, we suggest that the requirements of S6-00 be adopted.

Box Girder Bottom Flange Thickness (Cl. 8.20.3.2)

We recommend that this clause not be adopted.

Multibeam Decks (Cl. 8.21)

We recommend that the requirements of this clause not be applied to bridges using welded shear connectors. The MoF may wish to reconsider the use of the grouted shear keys based on the requirements of this clause.

16.9 CLAUSE 9 - WOOD STRUCTURES

Log Stringer Bridge Design

We recommend that the MoF develop standard design guidelines for log stringer bridges.

16.10 CLAUSE 10 - STEEL STRUCTURES

Durability (Cl. 10.6)

The requirements of S6-00 are not appropriate and could be simplified to the following:

- All steel plate to be uncoated weathering steel except for bridges in a marine environment (within 1 km of the coast or subject to salt spray or fog).
- Where bridges are situated in a marine environment (located within 1 km of the coast and subject to salt spray or salt fog) the steel structure shall be coated with an appropriate protective coating.
- If the steel does not conform to weathering steel, it shall be coated with an appropriate protective coating.

- Steel substructure to be coated with one coat bituminous paint. Allowance shall be made for a 3-mm section loss due to corrosion.
- Angles and tees should be orientated so that the vertical legs or webs are extending downwards wherever practical.
- Steel decks shall be waterproofed and provided with a skid-resistant wearing surface.

Design Detail Requirements (Cl. 10.7)

We recommend that S6-00 be adopted with the following changes:

- Minimum steel plate thickness be reduced to 9.5 mm (3/8") as this is a commonly used plate.
- The total required camber be shown on the drawing for simplicity.

Beams and Girders (Cl. 10.10)

The methodology outlined in S6-00 should be adopted with the following changes or clarifications:

- Clarification should be provided by the MoF on the applicability of the longitudinal stiffener requirements when considering construction loading.
- Where the designer chooses to exceed the current maximum diaphragm spacing of 7.5 m, care should be taken to ensure all buckling criteria are satisfied.

Composite Beams and Girders (Cl. 10.11)

The methodology outlined in S6-00 should be adopted with the following revision:

- The maximum shear connector spacing may exceed 600 mm when using standard MoF precast concrete composite deck panels.

Structural Fatigue (Cl 10.17)

The methodology outlined in S6-00 should be adopted with the following changes or clarifications:

- **Calculated Fatigue Stress Range:** The calculated fatigue stress range should be based on 100% of the design logging vehicle. Unlike highway structures that are subject to mixed traffic, logging bridges will typically be subject to the fully loaded design truck. This reduction factor is therefore inappropriate for logging bridge design.

- **Number of Cycles:** The MoF should define the number of cycles to be used for fatigue calculations. For simplification, the existing number of 500,000 cycles can be adopted. Shorter structures (less than 12 m) are subject to more as they are generally governed by axle loads rather than whole trucks.

  If the MoF is not comfortable with this simplification, the number of cycles could be based on span length. Definition such as 500,000 cycles for spans >12 m and 1,000,000 cycles for spans <12 m could be adopted.

- **Fatigue Stress Range Resistance:** The fatigue stress range resistance as defined in S6-00 be adopted with no changes.

- **Distortion Induced Fatigue:** Attaching the lateral plan brace to the top flange as is typically practiced and is an acceptable detail for simple span structures.

Fracture Critical Components (Cl. 10.23)

The new notch toughness requirements are not reflected in any of the standard categories included in CSA G40.21 (Structural Quality Steels). As a result, the MoF has the following options:

- Maintain the present requirements. This would require the use of 350 AT category 3 steel (tested on a per heat frequency) and a 25% reduction in the allowable fatigue stress range.
- Adopt the requirements as outlined in S6-00.

With the complex nature of this problem, we suggest the MoF further investigate this issue prior to making any decisions. The investigation should address the following issues:

- Why the methodology has been revised?
- Is the safety of the structure compromised if the present practice is maintained?
- Effect of per plate versus per heat testing frequency on the quality of the steel plate?
- How the change will affect the fabrication costs of steel girders?
- What are the implications of the change for the steel fabricators?
- What impact will mill ordering have on planning and construction scheduling?

Construction Requirements for Structural Steel (Cl. 10.24)

The methodology outlined in S6-00 be adopted with the following changes or clarifications:

- Weld inspection requirements conform to Forest Service Bridge Design and Construction Manual.
- Girders can be transported with webs in horizontal position if approved by the Engineer.
- Minimum field welder certification to be in accordance with MoF requirements.

16.11 CLAUSE 11 - JOINTS AND BEARINGS

We recommend that the MoF give consideration to developing specific design criteria for plain rubber pads.

16.12 CLAUSE 12 - BARRIERS AND HIGHWAY ACCESSORY SUPPORTS

The methodology outlined in S6-00 be replaced with the requirement that all barriers conform to typical MoF standard barrier designs.

16.13 CLAUSE 14 - EVALUATION

Permit Vehicle Loads (Cl. 14.8.2)

As the typical logging trucks do not directly conform to any of the specified categories defined in S6-00, we suggest that the MoF adopt a vehicle category for evaluating steel, concrete or timber bridges (excluding log stringer bridges).

Material Strengths (Cl. 14.6)

The methodology as outlined in S6-00 be adopted and a table summarizing specified timber strengths based on age be developed.

Evaluation of Log Stringer Bridges (not included in S6-00)

Refer to Clause 9 for proposed method for evaluation of existing log stringer bridges.

16.14 CLAUSE 15 - REHABILITATION

This Clause should be adopted without any changes.