Bridge Load Evaluation Manual

Version 1.1

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BRIDGE LOAD EVALUATION MANUAL

Technical Standards Branch

Alberta Transportation

December 2016

P R E F A C E

The Bridge Load Evaluation Manual (Manual) is a documentation of Alberta Transportation's bridge load evaluation guidelines, practices and policies and is intended to supplement the requirements of CSA S6-14.

The determination of the load carrying capacities of the Province's bridges is a major ongoing activity, and the carrying out of consistent, thorough and properly documented bridge load evaluation results is an important factor in achieving load evaluation results that maximize the load carrying capacities of the Province's bridges while maintaining an adequate level of safety. It is intended that this document will help facilitate the achievement of this goal.

It is not the intent of this Manual to limit progress or overrule the exercise of proper engineering judgment in the carrying out of bridge load evaluations. Consultants should satisfy themselves that the requirements of both this Manual and CSA S6-14 are appropriate for a specific bridge load evaluation.

Approved:

_____   Date: _____2015 · 06 · 03_____

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ACKNOWLEDGEMENTS

LIST OF CHANGES

The following page is reserved for documenting changes to this version of the Bridge Load Evaluation Manual. When changes are completed to the document, the following actions will be completed:

- The version of the document will be updated;

- A revision triangle will be placed next to the change in the document;

- A basic description and the date of the change will be summarized below.

Document Revision	Date	Description
1	December 07, 2016	Error in short span factor in Section 8.3.1.1.3 corrected.

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APPENDICES:

1.   Introduction

This Section 1 (Introduction) supplements CSA S6-14, Section 14.1 (Scope). It provides information on:

- The scope and purpose of the Manual;
- The circumstances under which a bridge load evaluation may be required by AT; and
- The types of bridge superstructures (Span Types) in AT's bridge inventory.

1.1   General

All bridge load evaluations carried out on bridges under the management of Alberta Transportation (AT) shall be carried out in accordance with the provisions of CSA S6-14, Canadian Highway Bridge Design Code (CSA 2014) and this Manual. In case of conflict this Manual shall govern. The requirements set forth in this Manual are considered minimum requirements and AT reserves the right to modify these requirements on a bridge specific basis.

The goal of each bridge load evaluation shall be to maximize the load carrying capacity of the bridge while maintaining an adequate level of safety. The expectation is that Consultants will refine their load evaluation practices to meet this goal. However, the following tasks are considered to be beyond the scope of a typical load evaluation.

- The field determination of bridge geometric and material properties;
- The use of non-elastic methods of analysis such as strut and tie modeling and concrete slab yield line theory;
- The use of the Sophisticated Method (see Section 6.9.2 of this Manual) for non-permit truck loads;
- The determination of the resistance of members with details not covered by CSA S6-14 or by this Manual; and
- The determination of the resistance of members that have been damaged or deteriorated by effects not covered by this Manual.

This Manual has been developed specifically to provide guidance in the load evaluation of bridges in Alberta Transportation's bridge inventory and unless otherwise noted in this Manual assumes that these bridges have adequate redundancy and ductility to provide warning of failure prior to collapse. This should be verified by users of this Manual on a bridge specific basis.

The text in this Manual shall be taken as being prescriptive unless it is in italics in which case it shall be taken as being advisory only.

Commentary 1.1

The purpose of this Bridge Load Evaluation Manual is to:

- *Document AT's bridge load evaluation guidelines, practices and policies;*
- *Provide guidance on how to load evaluate bridge Span Types contained in AT's bridge inventory;*
- *Promote consistent and thorough bridge load evaluation results from Consultants; and*
- *Establish bridge load evaluation reporting requirements.*

This Manual is organized to parallel and supplement CSA S6-14, Section 14 (Evaluation).

Information on AT's historical and current bridge load evaluation practices can be found in Appendix A of this Manual.

1.2   Need for a Bridge Load Evaluation

A bridge load evaluation may be required by AT as a result of:

- A bridge's load carrying capacity being unknown;
- A bridge being subjected to increased loading; or
- A change in the load carrying capacity of a bridge.

Commentary 1.2

Examples of when a bridge load evaluation may be required are as follows:

- *An increase in legal non-permit truck loads places the adequacy of a bridge's load carrying capacity in question. This occurs when the load effects due to increased legal non-permit truck loads exceed the load effects due to the bridge's design truck by a specified margin. Historically this margin has been taken to be in the order of 30%. However, since the introduction of limit state bridge design codes in the 1970s and 1980s, the reserve capacities designed into bridges has been reduced. Therefore, the use of a margin of less than 30% may be more appropriate in the future.*
- *Damage or deterioration places a bridge's load carrying capacity in question. Damaged or deteriorated bridge members are typically repaired to restore their load carrying capacities but in some cases a bridge may need to temporarily carry traffic in a damaged or deteriorated condition. In these cases the effects of the damage or deterioration on the bridge's load carrying capacity need to be considered based on their effects on bridge redundancy, bridge member ductility, the distribution of loads between bridge members, bridge member stability and bridge member strength.*
- *Additional load other than increased legal truck load is added to a bridge and places its load carrying capacity in question. During bridge maintenance or rehabilitation significant temporary construction loads such as scaffolding or hoarding can be placed on a bridge and the adequacy of the bridge to carry these loads needs to be verified.*
- *The adequacy of a bridge to carry a permit truck load is in question. This occurs when the standard permit truck configurations for which the bridge has been load evaluated by AT's Transport Engineering Section do not adequately represent the truck configuration for which the permit application is being made.*
- *A bridge assessment or rehabilitation is being carried out and the bridge's load carrying capacity is unknown. Knowledge of a bridge's load carrying capacity is required in order to make decisions regarding its remaining service life or the investment of funds to extend its service life. Also, a bridge rehabilitation can require the addition of dead load to a bridge and/or modify its lateral distribution of load characteristics which can place its load carrying capacity in question.*
- *A bridge is being strengthened. The load carrying capacity of the strengthened bridge needs to be determined to verify that the strengthening has provided the required load carrying capacity.*
- *Any other circumstances that place the adequacy of a bridge's load carrying capacity in question.*

1.3   Bridge Span Types

This Manual supplements the bridge load evaluation information provided in CSA S6-14 for the following AT bridge span types.

- Steel Truss Bridges;
- Steel Girder Bridges;
- Reinforced Concrete Bridges;
- Precast Reinforced Concrete Bridges;
- Precast Prestressed Concrete Bridges;
- Timber Bridges; and
- Other Bridges.

Commentary 1.3

The following gives a brief description of the bridge Span Types found in AT's bridge inventory (BIS 1992). Further information on these bridge Span Types can be found in Appendix B of this Manual.

Steel Truss Bridges – bridges with spans that are through trusses, pony trusses or deck trusses. AT's BIS identifies these Span Types as "TH", "PT" and "DT" respectively. These bridges are typically simple span bridges, although some are continuous spans.

Steel Girder Bridges - bridges with spans that are riveted girders, rolled beams or welded girders. AT's BIS identifies these Span Types as "RG", "RB" and "WG" respectively. They may be simple or continuous spans. They also may be composite or non-composite with the bridge deck. Welded girders may be "I" girders or "box" girders.

Reinforced Concrete Bridges – bridges with spans that are cast-in-place concrete slabs, voided slabs, T-girders or box girders. AT's BIS identifies these Span Types as "CS", "CV", "CT" and "CX" respectively. They may be simple or continuous spans.

Precast Reinforced Concrete Bridges – bridges with spans that are precast reinforced concrete channel girders placed side by side. AT's BIS identifies these Span Types as "GR", "HC", "HH", "PA", "PE", "PES", "PG" or "VH". The girders may be unconnected or connected together with concrete shear keys or steel connectors. The bridges are typically simple span bridges. Span Type "HC", "HH" and "PG" bridges that have received a concrete overlay are identified as Span Types "HCO", "HHO" and "PGO" respectively.

Precast Prestressed Concrete Bridges – bridges with spans that are precast prestressed concrete channel, box, "T" or "I" girders. They may be simple or continuous spans. They may also be composite or non-composite with the bridge deck. AT's BIS identifies channel girder Span Types as "FC", "FM", "VF", or "LF"; box girder Span Types as "PB", "PM", "RD", "RM", "SC", "SCC", "SL", "SLC", "SLW", "SM", "SMC", "VM" or "VS"; "T" girder Span Types as "CBC", "CBT", "DBC" or "DBT"; and "I" girder Span Types as "NU", "PO" or "OM". Span Type "PM", "SM" and "VS" bridges that have received a concrete overlay are identified as Span Types "PMO", "SMO" and "VSO" respectively.

Timber Bridges – bridges with spans that are treated or untreated timber stringers. AT's BIS identifies these Span Types as "TT" and "UT" respectively. These bridges are typically simple span bridges.

Other Bridges – bridges with Span Types other than those defined in this section. These bridges include steel rigid frame bridges (Span Type "FR"), steel tied arch bridges (Span Type "SSA"), steel suspension bridges (Span Type "SSS"), reinforced concrete arch bridges (Span Type "CA"), reinforced concrete frame bridges (Span Type "CF"), cast-in-place prestressed concrete box girder bridges (Span Type "CXP"), precast prestressed concrete frame bridges (in AT's BIS, these are assigned to Span Type "FC", which is the same as the Span Type assigned to precast, prestressed concrete "FC" channel girder bridges) and precast prestressed concrete trellis supported slabs (Span Type "TSS").

2. Evaluation Procedures

This Section 2 (Evaluation Procedures) supplements CSA S6-14, Section 14.4 (General Requirements) and Section 14.5 (Evaluation Procedures). It provides information on:

- Minimum qualification requirements for Consultants and Bridge Load Evaluators carrying out bridge load evaluations;
- Independent checking requirements for bridge load evaluations;
- Condition inspection requirements for bridge load evaluations;
- Reporting requirements for bridge load evaluations;
- The identification of the bridge members that typically need to be considered in a bridge load evaluation;
- The identification of the limit states that need to be considered in a bridge load evaluation; and
- The identification of the loads that need to be considered in a bridge load evaluation.

2.1 Minimum Bridge Load Evaluation Requirements

2.1.1 Bridge Load Evaluations Used to Justify a Significant Change in the Loading Carried by a Bridge

Bridge load evaluations used to justify a significant change in the magnitude of the load carried by a bridge shall meet the following minimum requirements.

- If the bridge meets either of the following conditions, the bridge load evaluation shall be based on a detailed bridge inspection, in accordance with Section 3.2.3 of this Manual:

  - The bridge is a Major River Bridge with more than 2 spans and the bridge is more than twenty years old; or
  - The latest Level 1 BIM inspection report identified damage or deterioration that could affect the load carrying capacity of a primary load carrying member (see Section 3.2.4 of this Manual).

- If the bridge does not meet either of the above conditions, the Consultant may, at its discretion, base the bridge load evaluation on the latest Level 1 BIM inspection report rather than carrying out a detailed bridge inspection.
- When applying for a permit - controlled (PC) permit, the above-noted bridge inspections (Level 1 BIM inspection or detailed bridge inspection) shall have been completed no more than two years prior to submission of the bridge load evaluation report, unless:

  - Damage or deterioration that could affect the load carrying capacity of a primary load carrying member was noted in previous inspection reports, in which case a detailed bridge inspection shall have been completed no more than six months prior to submission of the bridge load evaluation report; or
  - Otherwise required by AT.

- The bridge load evaluation shall be carried out by AT or by a Consultant prequalified by AT in the category for the "Design for Major Bridge Structures".
- The Bridge Load Evaluator shall be an Engineer experienced in the carrying out of bridge load evaluations.
- The bridge load evaluation shall be independently checked by a Bridge Load Evaluation Checker.

- If the bridge load evaluation is being carried out by a Consultant, the results of the bridge load evaluation shall be documented in a Bridge Load Evaluation Report stamped by both the Bridge Load Evaluator and the Bridge Load Evaluation Checker and meeting the requirements of Section 11 of this Manual.

Commentary 2.1.1

This Manual requires a higher standard of care for bridge load evaluations used to justify a significant change in the loading carried by a bridge than for bridge load evaluations not used to justify a significant change in the loading carried by a bridge. This higher standard of care consists of carrying out an independent check of the load evaluation and more rigorous inspection of the bridge.

A higher bridge inspection standard is required for Major River Bridges with more than two spans and more than twenty years old because;

- *Some of the bridge's primary load carrying members (e.g. interior spans and/or piers) can only be observed from a distance during a Level 1 BIM inspection; and*

- *Bridges more than twenty years old are more likely to have experienced damage or deterioration than newer bridges.*

The requirement that the load evaluation for a permit – controlled (PC) truck be based on a bridge inspection completed no more than two years prior to submission of the bridge load evaluation is intended for bridges that are in good condition and for permit-controlled (PC) trucks that are of moderate weight. However, AT may use its sole discretion to require additional bridge inspections prior to issuing the permit. Considerations that may prompt additional bridge inspections include, but are not limited to:

- *Concern over the condition of the bridge (even if it is not documented in an inspection report); and*
- *The permit-controlled (PC) truck is of sufficiently large weight.*

A bridge load evaluation may be based on a bridge inspection that has previously been carried out by AT under an existing bridge inspection program as long as the timing and level of the inspection satisfies the requirements of this section. Otherwise, the required bridge inspection shall be carried out as part of the bridge load evaluation.

This Section of the Manual is not intended to apply to the permit – annual (PA) and permit – single trip (PS) truck categories. The factored load effects applied to bridges in AT's bridge inventory by these permit truck categories are similar to the factored load effects applied to bridges by non-permit trucks as the magnitudes of both are typically governed by the capacities of the bridges in AT's bridge inventory that were designed for HS20 loading. They are therefore assumed to not significantly change the loading carried by a bridge. The factored load effects are also significantly less than those due to permit-controlled (PC) trucks. This Section of the Manual is also not intended to apply to very small increases in bridge dead load (e.g. chip seal).

Examples of bridge load evaluations used to justify a significant change in the loading carried by a bridge are:

- *Load evaluations carried out to justify a significant increase in bridge dead loads (e.g. new or increased thickness of deck or overlay, etc.);*
- *Load evaluations carried out for permit-controlled (PC) truck loads;*
- *Load evaluations carried out to justify a significant increase in non-permit truck loads; and*

- *Load evaluations carried out to lower the load carrying capacity of a bridge due to damage or deterioration.*

2.1.2 Bridge Load Evaluations Not Used to Justify a Significant Change in the Loading Carried by a Bridge

Bridge load evaluations not used to justify a significant change in the magnitude of the load carried by a bridge shall meet the following minimum requirements:

- The Consultant may, at its discretion, base the bridge load evaluation on the latest Level 1 BIM inspection report provided that no damage or deterioration that could affect the load carrying capacity of a primary load carrying member of the bridge is noted in the report (see Section 3.2.4 of this Manual).
- If the Level 1 BIM inspection report notes any damage or deterioration that could affect the load carrying capacity of a primary load carrying member, the Consultant shall carry out a detailed bridge inspection, in accordance with Section 3.2.3 of this Manual. The detailed bridge inspection shall specifically inspect those areas that had damage or deterioration noted in the Level 1 BIM inspection report. The bridge load evaluation shall be based on the combined findings of the latest Level 1 BIM inspection and detailed bridge inspection reports.
- The bridge load evaluation shall be carried out by AT or by a Consultant prequalified by AT in the Category for the "Design for Major Bridge Structures".
- The Bridge Load Evaluator shall be an Engineer experienced in the carrying out of bridge load evaluations.
- Bridge load evaluations carried out by a Consultant for the permit – annual (PA) and permit – single trip (PS) truck categories shall be independently checked by a Bridge Load Evaluation Checker.
- If the bridge load evaluation is carried out by a Consultant, the results of the evaluation shall be documented in a Bridge Load Evaluation Report stamped by the Bridge Load Evaluator and meeting the requirements of Section 11 of this Manual.

If a bridge is considered to not have adequate ductility and redundancy for it to provide warning of failure prior to collapse, the bridge load evaluation shall be carried out in accordance with the requirements of Section 2.1.1 of this Manual even if the load evaluation is not being carried out to justify a significant change in the loading carried by the bridge.

A bridge load evaluation initially carried out based on its not being used to justify a significant change in the loading carried by a bridge shall not be subsequently used to justify a significant change in the loading carried by the bridge unless further work is carried out to meet the requirements of Section 2.1.1 of this Manual.

Commentary 2.1.2

The rationale for allowing a lesser standard of care for bridge load evaluations not used to justify a significant change in the loading carried by a bridge is:

- *The load evaluation will not be used to increase or decrease the current loading regime on the bridge;*
- *The load evaluation will not be used to change the bridge's load carrying capacity information contained in AT's BIS system;*
- *The load evaluation will not be used to account for a change in bridge load carrying capacity due to damage or deterioration;*

- *The bridge has a successful history of carrying the loading under consideration and is not showing any signs of distress; and*
- *Bridges in AT's bridge inventory typically have adequate ductility and redundancy for the bridge to show signs of distress prior to collapse.*

Examples of bridge load evaluations typically not used to justify a significant change in the loading carried by a bridge are:

- *Load evaluations carried out as part of a bridge assessment; and*
- *Load evaluations for carried out for permit – annual (PA) or permit – single trip (PS) permits.*

2.1.3 Bridge Load Evaluations Carried Out During Bridge Design

Bridge load evaluations carried out during bridge design to verify the adequacy of the design for carrying specific permit trucks shall meet the following minimum requirements. The requirements of CSA S6-14, Section 3.8 for Special Trucks shall not apply.

- The bridge design for non-permit truck loading shall be based on CSA S6-14, Sections 1 to 13 and 16 and the bridge load evaluation shall be carried out for specific permit trucks only.
- The bridge load evaluation shall be based on the design drawings and Inspection Level INSP0. INSP0 "β" values shall be determined in accordance with Section 7.3 of this Manual.
- The bridge load evaluation shall be carried out by AT or by a Consultant prequalified by AT in the Category for the "Design for Major Bridge Structures".
- The Bridge Load Evaluator shall be an Engineer experienced in the carrying out of bridge load evaluations;
- The bridge load evaluation shall be independently checked by a Bridge Load Evaluation Checker.
- The results of the bridge load evaluation shall be shown on the design drawings and documented in the design notes and design check notes submitted to AT as part of the bridge design. The documentation shall be in accordance with the applicable requirements of Section 11.2 of this Manual.

Commentary 2.1.3

It is sometimes the intention of AT, during the design of a bridge, to design the bridge to carry not only non-permit trucks but also specific permit trucks. In this case AT may request that the bridge be designed to carry not only the non-permit trucks but also that the capacity of the bridge be checked for its ability to carry specific permit trucks. This check is typically carried out in accordance with CSA S6-14, Section 14. Although CSA S6-14, Section 14 was not developed for the design of new bridges, it is currently being used to design new bridges for anticipated future permit trucks and the above guidelines are provided for its use in that situation.

Inspection Level INSP0 is the same inspection level as was used in the development of the new bridge design provisions in CSA S6-14. The logic behind and implications of using Inspection Level INSP0 are discussed in Section C14.12.1 of the Commentary to CSA S6-14.

2.2 Bridge Members Typically Requiring Load Evaluation

A bridge load evaluation shall include the evaluation of primary load carrying bridge superstructure members (eg. girders, trusses, etc). The evaluation shall also include an assessment based on engineering judgment of the need to load evaluate any of the other bridge members and connections.

Commentary 2.2

Primary load carrying superstructure members are the bridge members most affected by an increase in the truck load on a bridge and are the members of most concern in a load evaluation. Other bridge members are typically of less concern either because they are secondary load carrying members, they have higher load carrying capacities than the primary load carrying superstructure members or they have a higher dead to live load ratio than the primary load carrying superstructure members.

Bridge members that will typically not require load evaluation may need to be load evaluated if they are damaged or deteriorated. For example, a pier column that, in good condition, would not warrant a load evaluation could require a load evaluation if significant concrete spalling or corrosion of the reinforcement has occurred and reduced its load carrying capacity. In general, if a structural component has a Level 1 BIM inspection condition rating of 3 or less, consideration should be given to whether or not a load evaluation of the component is required.

Bridge members that will typically not require load evaluation are:

- *Concrete decks that meet the empirical deck design method requirements in CSA S6-14, Section 8.18.4. These decks typically have higher load carrying capacities than the primary load carrying superstructure members. Note however that this does not include the cantilever portions of decks.*
- *Girder and truss bracing members that are not primary load carrying members. These members can typically be considered to be secondary load carrying members that do not require load evaluation. However, if they are being counted on to provide an improved distribution of truck loads between primary load carrying members, such as in a Sophisticated analysis, or are part of a primary load carrying system with curved or kinked girders, they need to be considered as primary load carrying bridge superstructure members and included in the bridge load evaluation.*
- *Bearings. Bearings typically have higher load carrying capacities than the primary load carrying superstructure members.*
- *Vertical substructure members not susceptible to buckling. These members, e.g. concrete or steel pier shafts and columns, typically have higher load carrying capacities and higher dead to live load ratios than the primary load carrying superstructure members.*
- *Horizontal substructure members. These members, e.g. concrete or steel pier caps, with span to depth ratios of less than 3 typically have higher load carrying capacities and higher dead to live load ratios than the primary load carrying superstructure members. However, this does not include the cantilever portions of the pier caps.*
- *Foundations. Foundation members, such as piles, typically have higher dead to live load ratios than the primary load carrying superstructure members. Also, foundations typically have higher load carrying capacities under short term loading, i.e. truck loads, than under permanent loads, i.e. dead loads. Therefore, bridge foundations are typically less affected by increased truck loads than primary load carrying superstructure members. However, if the total load on the bridge foundation increases by more than 30% or if the foundation is founded in compressible soils, e.g. friction piles in soft clay, AT should be notified and additional consideration given to the adequacy of the bridge foundation.*

Additional guidance on bridge members and connections that may not require load evaluation is provided in Section 9 of this Manual.

2.3   **Bridge Load Evaluation Limit States**

All bridge load evaluations shall be carried out at the Ultimate Limit State. Other limit states shall also be considered when required by this Manual or by AT on a bridge specific basis.

Commentary 2.3

Bridges are typically load evaluated for life safety at the ultimate limit state only. This practice is based on the assumption that the benefits to the owner of being able to carry increased truck loads on an existing bridge outweigh any potential decrease in the bridge's service life associated with exceeding the serviceability or fatigue limit state requirements. If this is not the case, the bridge may also need to be load evaluated for selected serviceability and fatigue limit states.

2.4   **Bridge Loads Requiring Consideration**

Permanent loads and truck loads shall be considered in all bridge load evaluations. The need to consider other transitory and exceptional loads shall be determined on a bridge specific basis.

Commentary 2.4

A bridge may be subjected to increased wind loads if additional material such as scaffolding or hoarding is added to the bridge. In this case the bridge may need to be load evaluated for its ability to carry the increased wind loads as well as the weight of the additional material.

2.4.1 Truck Loads Requiring Consideration

2.4.1.1 Non-Permit Truck Loads

Bridge loads evaluations carried out for non-permit trucks shall, as a minimum, consider the CS1, CS2 and CS3 truck configurations specified in Section 6.1 of this Manual. In addition, the Consultant shall consider additional non-permit truck types which may cross the bridge (eg. forestry trucks, permit overweight trucks, etc.) as required by AT on a bridge specific basis.

2.4.1.2 Permit Truck Loads

Bridge load evaluations carried out for permit trucks need only consider the specific permit truck loads and configurations that the bridge load evaluation is being carried out for.

3.   Existing Information and Condition Inspection

This Section 3 (Existing Information and Condition Inspection) supplements CSA S6-14, Section 14.6 (Condition Inspection). It provides information on:

- The requirements for obtaining existing bridge information;
- The requirements for obtaining bridge condition information; and
- The requirements for assessing the effects of damage or deterioration on a bridge's load carrying capacity.

3.1   Existing Information

All available drawings, including design drawings, shop drawings, as-constructed / Record drawings, and maintenance/rehabilitation drawings shall be considered in determining bridge geometric properties, material properties and member resistances. The Bridge Load Evaluator shall verify that the drawings accurately represent the bridge based on the available bridge condition inspection reports, a review of AT's bridge correspondence file and /or a bridge site condition inspection. If one of the design drawings is an AT Standard Drawing the Bridge Load Evaluator shall verify that the correct version of the standard drawing is being used. Any missing geometric information required for the bridge load evaluation shall be measured in the field.

Commentary 3.1

It is essential that bridges be load evaluated based on the correct geometric and material properties. Bridge geometric and material properties are typically shown on bridge drawings available in AT's bridge drawing system. These drawings include design drawings whose drawing numbers end in "P", as-constructed / Record drawings whose drawing numbers end in "C", shop drawings and AT Standard Drawings whose drawings numbers start with "S". It is important to note that details shown on design drawings or AT Standard Drawings may have been altered either on the shop drawings or on the as-constructed / Record drawings. Therefore, whenever possible, bridge load evaluations should be based on shop drawing and/or as-constructed / Record drawing information. It should also be noted that AT Standard Drawings were updated regularly. It should therefore be confirmed that the correct versions of AT Standard Drawings are being used in a bridge load evaluation.

Bridge details may have been altered subsequent to initial bridge construction as a result of maintenance or rehabilitation activities. These alterations may have been recorded on bridge drawings in AT's bridge drawing system or a record of them may be found in AT's bridge correspondence file.

If geometric information is not available on bridge drawings in AT's bridge drawing system or in the AT bridge correspondence file, the information will need to be obtained from field measurements. For steel and timber members this will typically only require measurement of the external dimensions and spacings of the members. However, for concrete members this will require not only measurement of the external dimensions and spacings of the members but also a determination of their internal reinforcement. Methods that can be used to identify the details of internal reinforcement include the use of pachometers, ultrasonics and radiography. Sometimes the details of internal reinforcement in precast concrete members can be determined by associating identification marks on the member with a standard drawing.

3.2   **Condition Inspections**

3.2.1 <u>General</u>

All bridge load evaluations shall be based on the results of the most recent bridge condition inspection that is sufficiently detailed to verify the condition of the bridge.

3.2.2 <u>Level 1 BIM Inspections</u>

AT carries out regular Level 1 BIM inspections (BIM 2008) on all of the bridges in their bridge inventory. Level 1 BIM inspections are visual condition inspections carried out without the benefit of access equipment. They are carried out once every twenty-one to fifty-seven months for each bridge. As a minimum, the condition ratings in the latest Level 1 BIM inspection report should be considered in the bridge load evaluation. AT's "BIM Inspection Manual" provides more information on the Level 1 BIM inspection program and condition ratings.

Level 1 BIM inspection reports for bridges in AT's bridge inventory are available from Alberta Transportation. Section 2.1 of this Manual provides guidance on when an existing Level 1 BIM inspection report can be considered to be adequate for a bridge load evaluation and when an additional inspection is required.

Commentary 3.2.2

> *Although a Level 1 BIM inspection does not provide for a hands-on inspection of all bridge members the primary load carrying members can generally be seen either from the ground or from the bridge deck.*

> *As specified in Section 2.1 of this Manual there are two situations where the Level 1 BIM inspections reports may not be considered adequate for determining the condition of a bridge. The first situation occurs with major river bridges whose interior spans and/or piers may not be clearly visible from the riverbanks. In this case a Level 1 BIM inspection can not be expected to adequately identify the condition of these members and the members should be load evaluated based on Inspection Level INSP1 in accordance with Section 7.3 of this Manual. The second situation occurs when the information provided on the Level 1 BIM inspection report may no longer be current or valid. Unless otherwise specified in this Manual, this is assumed to have occurred if the previous Level 1 BIM inspection is more than two years old or if the bridge is in such poor condition that deterioration could be occurring at an accelerated rate.*

3.2.3 <u>Detailed Bridge Inspections</u>

If, in accordance with Section 2.1 of this Manual a Level 1 BIM inspection is not sufficient, a detailed bridge inspection shall be carried out. As a minimum a detailed bridge inspection shall be a visual and hands-on inspection, meaning that during the inspection the inspector shall be within an arm's length distance of all the primary load carrying members that are being load evaluated. If a detailed bridge inspection is only required because damage or deterioration has been noted on the Level 1 BIM inspection report, only those members with damage or deterioration need be inspected during the detailed bridge inspection.

If a bridge member is damaged or deteriorated, a visual and hands-on inspection may not be sufficient and more rigorous inspection techniques may be required (eg. ultrasonic testing of welded cover plates).

Detailed bridge inspections shall be carried out by the Bridge Load Evaluator and/or a Class A Inspector.

The results of the detailed bridge inspection shall be documented in the Bridge Load Evaluation Report. Photos and observations of any damage or deterioration shall be of sufficient clarity and quality so that they can be easily correlated with the assumed effects of the damage or deterioration on the results of the load evaluation.

Commentary 3.2.3

> *A Level 1 BIM inspection is typically sufficient to identify the condition of a bridge. However, in some cases a more detailed bridge inspection is warranted as noted in Section 2.1 of this Manual. The scope of the detailed bridge inspection shall be as required for the Bridge Load Evaluator and Bridge Load Evaluation Checker (if required) to be confident in their knowledge of the condition of the bridge and in determining the load carrying capacity of the bridge.*

> *It is likely that an inspection vehicle (eg. snooper platform or articulated man lift bucket) will be required to provide access during a detailed bridge inspection.*

> *It should be noted that AT also carries out several Level 2 BIM Inspection programs. More information on these programs can be obtained from AT's "BIM Level 2 Inspection Manual" (BIM 2007). These Level 2 BIM inspections are in-depth, quantitative inspections conducted using specialized tools, techniques, and equipment. They gather detailed information on the condition of a particular bridge component, but do not necessarily consider the condition of the entire bridge. As a result, a Level 2 inspection may not be sufficient for the purpose of completing a bridge load evaluation. Specific Level 2 BIM inspections that are of interest to a bridge load evaluation include steel girder inspections for fatigue cracking and other defects and timber member inspections for rot. The results of Level 2 BIM Inspections can be found in AT's bridge correspondence files and should be reviewed.*

> *It is important that photos and observations from the bridge inspection are clear enough to allow the Bridge Load Evaluator and Bridge Load Evaluator Checker (if required) to reliably assess the effects of the observed deterioration on the results of the load evaluation. This means that photos and observations should capture information such as: the locations, extents and depths of corrosion; the locations and severities of member deformations; the locations, lengths, angles and widths of cracks; the locations, extents and depths of spalls; etc.*

3.2.4 Member Damage or Deterioration

3.2.4.1 General

The Bridge Load Evaluator and Bridge Load Evaluation Checker (where required) shall have the specialized knowledge required to assess the damage or deterioration to a bridge member and its effects on the bridge's load carrying capacity or shall obtain specialist advice. Damage or deterioration that can affect a member's load carrying capacity includes section loss, loss of material strength, a reduction in ductility, out of straightness, cracking and spalling.

Commentary 3.2.4.1

> *Further guidance on the determination of the effects of member damage or deterioration on the load carrying capacity of a bridge is provided in Section 9 of this Manual.*

3.2.4.2 Level 1 BIM Condition Rating of 4 or More

If a primary load carrying member has received a Level 1 BIM condition rating of 4 or more the Bridge Load Evaluator and Bridge Load Evaluation Checker (where required) shall confirm whether or not any of the findings

of the Level 1 BIM Inspection Report indicate that the member's load carrying capacity has been affected by its condition.

If it is determined by the Bridge Load Evaluator and Bridge Load Evaluator Checker (where required) that the member's load carrying capacity has been affected by its condition, a separate detailed bridge inspection of the damage or deterioration shall be carried out in accordance with Section 3.2.3 of this Manual. The Bridge Load Evaluator and Bridge Load Evaluation Checker shall then assess the effects of the damage or deterioration on the bridge's load carrying capacity.

Commentary 3.2.4.2

> *In accordance with AT's BIM system, a condition rating of 4 implies that the condition of the member is below the minimum acceptable but is not expected to affect the bridge's load carrying capacity and is a low priority for repairs.*

3.2.4.3 Level 1 BIM Condition rating of 3 or Less

If a primary load carrying member has received a Level 1 BIM Inspection condition rating of 3 or less, a separate detailed bridge inspection of the damage or deterioration shall be carried out in accordance with Section 3.2.3 of this Manual. The Bridge Load Evaluator and Bridge Load Evaluation Checker shall assess the effects of the damage or deterioration on the bridge's load carrying capacity.

Commentary 3.2.4.3

> *A condition rating of 3 implies that the member is showing signs of distress or deterioration and is not functioning as intended. While a condition rating of 3 is not expected to affect a bridge's ability to carry current loading over the short term it could affect its ability to carry loads greater than the current loading.*

> *A condition rating of 2 implies that a hazardous condition or severe distress/deterioration is present in a bridge member. It is expected that a condition rating of 2 will reduce a bridge's load carrying capacity.*

> *A condition rating of 1 implies that a danger of bridge collapse and/or a danger to bridge users is present. It is expected that a condition rating of 1 will significantly reduce a bridge's load carrying capacity and may result in bridge closure.*

4.   Material Strengths

This Section 4 (Material Strengths) supplements CSA S6-14, Section 14.7 (Material Strengths). It provides information on:

- Historical structural steel strengths used by AT;
- Historical cast-in-place reinforced concrete strengths used by AT;
- Historical reinforcing steel strengths used by AT;
- Historical prestressing steel strengths and types used by AT; and
- Historical timber species and grades used by AT.

4.1 Structural Steel Strengths

Structural steel strengths shall be taken from the design drawings and/or shop drawings. If the information varies between the two sets of drawings the shop drawings shall be taken as being correct. If information on the structural steel strength is not shown on the drawings the structural steel strength may be assumed to be as shown in Table 1 for structural sections and steel plate 16 mm thick or less. For thicker steel plate, lower structural steel strengths can apply.

Table 1 – Historic Structural Steel Strengths for AT Bridges

Bridge Age	Before 1905	1905 to 1935	1936 to 1960	1961 to 1977	1978 to 1980	1981 to Present
Yield Strength (psi)	26,000	30,000	33,000	36,000	50,000	50,800
Yield Strength (MPa)	180	210	230	250	345	350

Additional information on historical structural steel strengths, including information on the historical steel strengths of thicker steel plates, can be found on the Canadian Institute of Steel Construction website (www.cisc-icca.ca).

Commentary 4.1

The structural steel strengths given in Table 1 are generally consistent with those given in Section 14 of CSA S6-14. However, the first year that a yield strength of 230 MPa can be assumed has been revised from 1933 in CSA S6-14 to 1936 in this Manual. This is consistent with historic AT practice for load evaluating steel trusses. Also, subsequent to 1977, AT switched to the exclusive use of weathering steel with a minimum yield strength of 345 MPa (50,000 psi) for girders.

Alternatively, structural steel strengths of bridge members may be determined from coupons taken from the members and assessed with the provisions of CSA S6-14, Section A14.1.1.

4.2 Cast-in-Place Reinforced Concrete Strengths

Cast-in-place reinforced concrete strengths shall be taken from the design drawings and/or as-constructed/Record drawings. If the information varies between the two sets of drawings the as-

constructed/Record drawings shall be taken as being correct. AT design drawings typically identify the cast-in-place reinforced concrete design strength by Concrete Class. In the absence of better information the design concrete strengths shown in Tables 2 to 9 may be used for the Concrete Classes historically used by AT.

Table 2 – Historic "Class Pile" Concrete Strengths for AT Bridges

Bridge Age	Before 1970	1970 to 1980	1981 to 2012	2013 to Present
Compressive Strength (psi)	3000	3500	3600	4400
Compressive Strength (MPa)	20.7	24.1	25	30

Table 3 – Historic "Class A" Concrete Strengths for AT Bridges

Bridge Age	Before 1970	1970 to 1980	1981 to Present
Compressive Strength (psi)	3000	3500	3600
Compressive Strength (MPa)	20.7	24.1	25

Table 4 – Historic "Class B" Concrete Strengths for AT Bridges

Bridge Age	Before 1970	1970 to 1980	1981 to Present
Compressive Strength (psi)	3000	3500	3600
Compressive Strength (MPa)	20.7	24.1	25

Table 5 – Historic "Class C" Concrete (or Modified Class C Concrete) Strengths for AT Bridges

Bridge Age	Before 1980	1981 to 2006	2006 to Present
Compressive Strength (psi)	4000	4400	5000
Compressive Strength (MPa)	27.6	30	35

Table 6 – Historic "Class D" Concrete Strengths for AT Bridges

Bridge Age	Before 1970	1970 to 1980	1981 to Present
Compressive Strength (psi)	3500	4000	4400
Compressive Strength (MPa)	24.1	27.6	30

Table 7 – Historic "Class SF" Concrete Strengths for AT Bridges

Bridge Age	All
Compressive Strength (psi)	5000
Compressive Strength (MPa)	35

Table 8 – Historic "Class Modified SF" Concrete Strengths for AT Bridges

Bridge Age	All
Compressive Strength (psi)	7300
Compressive Strength (MPa)	50

Table 9 – Historic "Class HPC" Concrete Strengths for AT Bridges

Bridge Age	All
Compressive Strength (psi)	6500
Compressive Strength (MPa)	45

Commentary 4.2

AT has historically identified cast-in-place reinforced concrete strengths by Concrete Class. Concrete Classes that have historically been used are Pile, Class A, Class B, Class C, Modified Class C, Class D, Class SF, Modified Class SF and Class HPC. However, over time, the minimum specified cast-in-place reinforced concrete compressive strength associated with each Concrete Class has changed.

The cast-in-place reinforced concrete compressive strengths shown for the years 1970 and later are based on the minimum concrete compressive strengths specified in AT's Specifications for Bridge Construction.

For the years prior to 1970 copies of AT's Specifications for Bridge Construction are not available. The cast-in-place reinforced concrete compressive strengths shown for these years are conservative minimum values based on historical AT practices. Also the larger post 28 day strength gains experienced by the coarser ground cements used prior to the mid-1960s provide additional conservatism to the assumed pre-1970 cast-in-place reinforced concrete compressive strengths.

The cast-in-place reinforced concrete compressive strengths shown do not account for the possible acceptance of under-strength concrete based on the assumption that under-strength concrete was not accepted for use in critical load carrying members and that strength gains in the concrete since construction have compensated for any initial under-strength in the concrete.

Alternatively, cast-in-place reinforced concrete compressive strengths of bridge members may be determined from concrete cores taken from the members and assessed in accordance with the provisions of CSA S6-14, Section A14.1.2.

4.3 Reinforcing Steel Strengths

Reinforcing steel strengths shall be taken from the design drawings. If information on reinforcing steel strengths is not shown on the drawings, the reinforcing steel strengths shown in Table 10 may be assumed.

Table 10 – Historic Reinforcing Steel Strengths for AT Bridges

Bridge Age	Before 1955	1956 to 1979	1980 to 1985	1986 to Present
Grade	Structural	Intermediate	Grade 300	Grade 400
Yield Strength (psi)	33,000	40,000	43,500	58,000
Yield Strength (MPa)	230	275	300	400

Commentary 4.3

The reinforcing steel strengths given in this Manual are generally consistent with those given in Section 14 of CSA S6-14. However, the first year that a yield strength of 275 MPa can be assumed has been revised from 1973 in CAN/CSA S6-06 to 1956 in this Manual based on historic AT practice for the procurement of reinforcing steel. In approximately 1985, AT switched their open order for the procurement of reinforcing steel from 300 MPa reinforcing steel to 400 MPa reinforcing steel.

Alternatively, reinforcing steel strengths of bridge members may be determined from coupons taken from the members and assessed in accordance with the provisions of CSA S6-14, Section A14.1.3.

4.4 Prestressing Steel Strengths and Types

4.4.1 Prestressing Steel Wires

Prestressing steel wire strengths shall be taken from the design drawings and/or shop drawings/stressing calculations. If the information on the design drawings varies from that on the shop drawings/stressing calculations, the information on the shop drawings/stressing calculations shall be taken to be correct. In the absence of other information, the ultimate tensile strength of the wires may be taken as 1600 MPa (230,000

psi). The areas of the wires may be taken to be 19 mm^2 for 0.196" diameter wires and 38 mm^2 for 0.276" diameter wires.

Commentary 4.4.1

If the stressing calculations are not included with the shop drawings they can sometimes be found in the AT bridge correspondence file or bridge load evaluation file.

Prestressing steel wires were used in Type "PO" girders and the ultimate tensile strength of the wires was not always shown on the drawings. The minimum ultimate tensile strength of 1600 MPa given in this Manual is consistent with Section 14 of CSA S6-14.

4.4.2 <u>Prestressing Steel Strands</u>

Prestressing steel strand strengths shall be taken from the design drawings and/or shop drawings/stressing calculations. If the information on the design drawings varies from that on the shop drawings/stressing calculations the information on the shop drawings/stressing calculations shall be taken to be correct. In the absence of other information, the ultimate tensile strength of the strands may be taken to be 1724 MPa (250,000 psi) for girders fabricated before 1973 and 1860 MPa (270,000 psi) for girders fabricated in 1973 or later. The areas of the strands may be taken to be 93 mm^2 for 0.5" diameter strand with an ultimate tensile strength of 1724 MPa (250,000 psi), 99 mm^2 for 0.5" diameter strand with an ultimate tensile strength of 1860 MPa (270,000 psi) and 139 mm^2 for 0.6" diameter strand with an ultimate tensile strength of 1860 MPa (270,000 psi).

Commentary 4.4.2

Prestressing steel strands with an ultimate tensile strength of 1724 MPa were typically only used in Type "FC and "PM" girders.

4.4.2.1 <u>Identification of Prestressing Steel Strand Type</u>

Seven wire prestressing steel strands shall be identified as being either stress relieved or low relaxation based on information shown on the design drawings and/or shop drawings/stressing calculations. If the information on the design drawings varies from that on the shop drawings/stressing calculations, the shop drawings/stressing calculations shall be taken as being correct. In the absence of other information, the strands shall be taken to be stress relieved for girders fabricated before 1983 and may be taken to be low relaxation for girders fabricated in 1984 or later.

Commentary 4.4.2.1

When low relaxation prestressing steel strands first became available they were initially substituted one for one with stress relieved strands. Later, the design drawings allowed the option of using either stress relieved or low relaxation strands prior to the exclusive adoption of low relaxation strands. As a result, the girder shop drawings are the best source of information regarding the type of prestressing strands used.

4.5   Timber Species and Grades

Timber species and grades shall be taken from the design drawings. If information on timber species and grades is not shown on the drawings, the timber species and grades shown in Table 11 may be assumed.

Table 11 – Historic Timber Species and Grades Used for AT Bridges

Bridge Member	Timber Species	Timber Grade
Timber Subdeck	Coast Douglas Fir	No 1
Timber Stringers	Coast Douglas Fir	Select Structural
Timber Caps	Coast Douglas Fir	Select Structural
Timber Piles	Douglas Fir or Pine	NA

Commentary 4.5

If the species of timber is unknown it can be identified through the analysis of chips taken from a core.

5.   Permanent Loads

This Section 5 (Permanent Loads) supplements CSA S6-14, Section 14.8 (Permanent Loads). It provides information on:

- The determination of wearing surface dead loads;
- The identification of non-load carrying bridge members;
- The treatment of post-tensioning loads; and
- How to determine the lateral distribution of permanent loads between primary load carrying members.

5.1   Wearing Surface

5.1.1 General

Dead Load Category D2 shall be used for concrete overlay wearing surfaces and Dead Load Category D3 shall be used for asphalt wearing surfaces except as noted in Section 5.1.2 of this Manual.

If a chip seal coat is present a 10 mm thickness shall be added to the thickness of the wearing surface to account for the chip seal coat unless noted otherwise in this Manual.

In accordance with CSA S6-14, Section 14.8.2.1, a minimum thickness of 90 mm shall be assumed for unmeasured asphalt wearing surfaces. The thickness of the chip seal coat, if present, may be assumed to be included in the 90 mm thickness.

Commentary 5.1.1

Dead Load Category D2 is allowed for concrete overlay wearing surfaces as it will be obvious during an inspection if an asphalt wearing surface has been placed over the concrete overlay. Dead Load Category D3 is specified for asphalt wearing surfaces as it may not be obvious during an inspection that an additional asphalt overlay has been added.

Information on the thickness of concrete overlay and asphalt wearing surfaces can sometimes be found in Level 2 BIM Deck Testing Inspection Reports in the AT bridge correspondence file.

5.1.2 Field Measured Wearing Surface

Dead Load Category D2 shall not be used for field measured asphalt wearing surfaces unless the truck load being evaluated is a permit truck and it can be verified that an additional lift of asphalt has not been added to the wearing surface between the time of the field measurement and the time of the passage of the permit truck over the bridge.

The thickness of the field measured asphalt wearing surface shall include the thickness of the chip seal coat if present.

Commentary 5.1.2

The use of Dead Load Category D2 for field measured asphalt wearing surfaces is not recommended when the load evaluation is being carried out for non-permit trucks. This is due to the possibility of an additional lift of asphalt being added to the bridge in the future.

Whether or not an additional lift of asphalt has been added to the bridge can be determined by recording the height of the curbs or barriers above the top of the asphalt at the time the field measurements are taken and then verifying that the height has not changed at the time of passage of the permit truck.

5.2   Non-Load Carrying Bridge Members

5.2.1 <u>Wearing Surface</u>

Asphalt and concrete overlay wearing surfaces shall not be considered to be bridge load carrying members or parts of bridge load carrying members unless otherwise noted in this Manual. A reinforced concrete overlay wearing surface in good condition may be assumed to be capable of transferring load laterally between girders.

Commentary 5.2.1

Wearing surfaces are typically not designed to be composite with the load carrying members at the ultimate limit state. A reinforced concrete overlay may be assumed to be capable of transferring load laterally between girders, in the same manner as a concrete shear key, provided that there are only hairline cracks in the overlay along the joints between the girders.

5.2.2 <u>Curbs and Barriers</u>

Curbs and barriers shall not be considered to be bridge load carrying members or parts of bridge load carrying members unless they were cast integrally with a precast concrete girder, i.e. there is no construction joint between the girder and curb or barrier.

Commentary 5.2.2

Curbs and barriers are typically not considered to be load carrying members as they may not be continuous along the length of the bridge or were not designed to be composite with the load carrying members at the ultimate limit state.

5.3   Post-Tensioning

Load effects in bridge members due to secondary moments caused by internal post-tensioning or due to all load effects caused by external post-tensioning shall be considered to be permanent loads and shall be multiplied by the appropriate load factors given in CSA S6-14, Table 3.3 for secondary prestress effects.

5.4   Lateral Distribution of Permanent Load

The lateral distribution of permanent load between primary load carrying members may be determined in accordance with this section.

Alternatively, the lateral distribution of permanent load between primary load carrying members, added after the deck concrete has set or the girders have been connected together, may be determined using a Sophisticated method of analysis, in accordance with the requirements of Section 6.9.2 of this Manual.

Commentary 5.4

The self-weight of primary load carrying members and the wet weight of the deck concrete is distributed between primary load carrying members based on statics. Permanent load added after the

deck concrete is set or the girders have been connected together has historically been assumed to be evenly distributed between the primary load carrying members. This has typically underestimated the permanent load carried by exterior primary load carrying members and overestimated the permanent load carried by interior primary load carrying members.

5.4.1 Deck on Girder Bridges

For deck on girder bridges with two girder lines, the lateral distribution of permanent load to the girders may be determined by assuming that the deck is continuous across and pinned to the girders.

For deck on girder bridges with more than two girder lines, the lateral distribution of permanent load to interior girders may be determined by assuming that the deck is pinned at the girders (see Figure 5.4.1a).

For deck on girder bridges with more than two girder lines, the lateral distribution of permanent load to exterior girders may be determined by assuming that the deck is pinned at the interior girder and continuous across and pinned to the exterior girder (see Figure 5.4.1b).

Commentary 5.4.1

The requirements of this section provide for a system of permanent load distribution between girder lines of deck on girder bridges that conservatively satisfies equilibrium. Deck on girder bridges are typically Span Types "RG", "RB", "WG", "FR", "CT", "CX", "CXP", "PB", "PO", "OM" and "NU".

5.4.2 Unconnected and Shear Connected Girder Bridges

For unconnected and shear connected girders, the lateral distribution of permanent load between the girders may be determined by assuming that each girder carries only the permanent load applied directly to it, i.e. there is no lateral distribution of permanent load between girders.

Commentary 5.4.2

The requirements of this section provide for a conservative system of permanent load distribution between the girders of unconnected and shear connected girder bridges. Unconnected and shear connected girder bridges are typically Span Types "GR", "HH", "HC", "VH", "PG", "VS", "SM", "SC", "SL", "SLW", "PA", "PE", "PES", "FC", "VF", "LF", "FM", "PM", "VM", "RD", "RM", "DBT" and "CBT".

5.4.3 Slab Bridges

For slab bridges, the lateral distribution of permanent load in the slab may be determined by assuming that each one metre wide width of slab carries only the permanent load applied directly to it.

Commentary 5.4.3

The requirements of this section provide for a conservative system of permanent load distribution in a slab. Slab bridges are typically Span Types "CS", "CV" and abutment roof slabs of other Span Type bridges.

5.4.4 Determination of Skew Effects

For slabs with depths greater than 500 mm, the effects of skew on the distribution of permanent load between primary load carrying members shall be determined in accordance with the requirements of CSA S6-14, Section 5.6.3.

Except for slabs with depths greater than 500 mm, the effects of skew on the distribution of permanent load between primary load carrying members may be ignored if the lateral distribution of permanent load between primary load carrying members is determined in accordance with the requirements of Section 5.4 of this Manual.

Commentary 5.4.4

Skew effects reduce moment in primary load carrying members and can therefore be ignored.

Skew effects increase shears in primary load carrying members particularly near the obtuse corners of the bridge superstructure. However, as the majority of the permanent load (e.g. girder self weight and deck self weight) applied to deck on girder, unconnected girder and shear connected girder bridges is applied when the distribution of permanent load between the girders can be determined based on statics, the effects of skew on the distribution of permanent load shear between primary load carrying girders can be ignored.

Increased shears are more pronounced at the obtuse corners of slab bridges than of girder bridges. This is because the longitudinal and transverse bending stiffnesses of slab bridges are approximately equal (as opposed to the transverse bending stiffness being significantly smaller than the longitudinal bending stiffness in girder bridges) and substantially developed before the permanent loads are applied to the bridge (i.e. after the shoring has been removed). However, slab bridges are typically short span bridges where the live load effects are significantly larger than the permanent load effects. Therefore the consideration of permanent load skew effects is not required. However, if a skewed slab bridge is not a short span bridge, the effects of skew on the magnitude of the permanent load shear need to be considered.
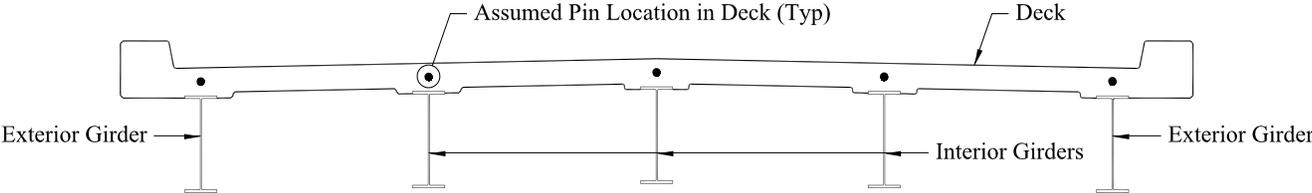
a)   Determination of Lateral Distribution of Permanent Load to Interior Girders
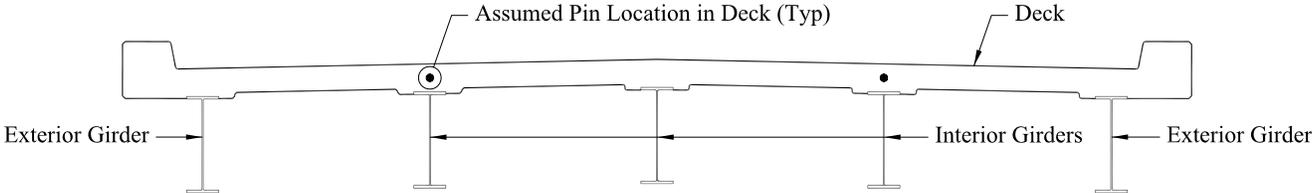
b)   Determination of Lateral Distribution of Permanent Load to Exterior Girders

Figure 5.4.1 Simplified Lateral Distribution of Permanent Load Between Girders

6.   Live Loads

This Section 6 (Live Loads) supplements CSA S6-14, Section 14.9 (Transitory Loads) and Section 14.11 (Lateral Distribution Categories for Live Load). It provides information on:

- The non-permit and/or permit truck configurations that need to be considered;
- The non-permit and/or permit truck dynamic load allowances that need to be considered;
- The lateral non-permit and/or permit truck positions that need to be considered;
- The additional vehicles that need to be considered on the bridge; and
- How to determine the lateral distribution of live loads between primary load carrying members.

6.1   Non-Permit Trucks

A bridge that can safely carry a 28 tonne CS1 truck, 49 tonne CS2 truck and 63.5 tonne (54 tonnes on local roads) CS3 truck can be deemed capable of safely carrying all legal non-permit trucks.

6.1.1 Evaluation Level 1

The Alberta Transportation CS3 truck as shown in Figure 6.1.1 shall be used for Evaluation Level 1 (vehicle trains) in lieu of the CL1 truck given in CSA S6-14, Section 14.9.

6.1.2 Evaluation Level 2

The Alberta Transportation CS2 truck as shown in Figure 6.1.2 shall be used for Evaluation Level 2 (two-unit vehicles) in lieu of the CL2 truck given in CSA S6-14, Section 14.9.

6.1.3 Evaluation Level 3

The Alberta Transportation CS1 truck as shown in Figure 6.1.3 shall be used for Evaluation Level 3 (single-unit vehicles) in lieu of the CL3 truck given in CSA S6-14, Section 14.9.

Commentary 6.1

AT developed the CS1, CS2 and CS3 load evaluation truck configurations to model non-permit single unit trucks (or single-unit vehicles), semi-trailer trucks (or two-unit vehicles) and truck trains (or vehicle trains) respectively, after the issuing of CAN/CSA S6-88 and the CS design truck configuration (Loo and Kornelsen 1997). The CS1 truck configuration was developed to produce load effects that were a conservative representation of the load effects produced by legal non-permit single unit truck configurations having the same weight as the CS1 truck configuration. Similarly the CS2 and CS3 truck configurations were developed to produce load effects that were conservative representations of the load effects produced by legal non-permit semi-trailer truck and truck train configurations having the same weight as the CS2 and CS3 truck configurations respectively.

The CL1, CL2 and CL3 load evaluation trucks were introduced in CAN/CSA S6-00 (CSA, 2000). To avoid the need to perform new load evaluations for the bridges in their bridge inventory, AT made the decision to continue to use the CS load evaluation truck configurations rather than adopt the CL load evaluation truck configurations. A study carried out for AT showed that, except for short spans, the CS1, CS2 and CS3 truck configurations produced similar or conservative load effects relative to the CL3, CL2 and CL1 truck configuration load effects respectively (Ramsay, 2001). In order for the CS1,

CS2 and CS3 truck configurations to produce conservative load effects for short spans relative to the CL3, CL2 and CL1 truck configuration load effects respectively, Section 8.1 of this Manual provides adjusted "Short Spans" live load factors for the CS1, CS2 and CS3 truck configurations.

The single unit truck (single-unit vehicle) designation (CS1) applies to:

- *a single unit truck; and*
- *a truck tractor and a semi-trailer, in the case where the inter-axle spacing between the truck tractor and the semi-trailer is less than 4.5 metres.*

The semi-trailer truck (two-unit vehicle) designation (CS2) applies to:

- *a truck and a trailer;*
- *a truck tractor and semi-trailer, in the case where the inter-axle spacing between the truck tractor and the semi-trailer is 4.5 metres or more;*
- *a mobile crane with a boom dolly;*
- *a truck tractor in combination with a single axle semi-trailer and a semi-trailer;*
- *a truck tractor in combination with a single axle semi-trailer and a pole trailer; and*
- *a truck or truck tractor in combination with a pole trailer.*

The truck train (vehicle train) designation (CS3) applies to:

- *a truck tractor in combination with 2 or more trailers;*
- *a truck and a full trailer with 2 tandem axles, in the case where the inter-axle spacing between the tandem axle groups is 5.0 metres or more;*
- *a truck tractor in combination with a tandem axle semi-trailer and a semi-trailer; and*
- *a truck tractor in combination with a tandem axle semi-trailer and a pole trailer.*

6.1.4 <u>Lateral Position of a Non-Permit Truck in its Travel Lane</u>

The lateral position of a non-permit truck in its travel lane may be based on the centrelines of the truck wheels having a minimum distance of 0.75 m from the edges of the travel lanes rather than the 0.6 m specified by CSA S6-14, Section 3.8.3.1.2.

Commentary 6.1.4

Trucks most often travel in the centres of their travel lanes but also deviate from the centres from time to time. In new bridge design, the minimum distance of the centreline of a truck wheel from the edge of a travel lane is assumed to be 0.6 metres which means that the centrelines of the wheels of side by side trucks are assumed to be a minimum of 1.2 metres apart. This is considered to be overly conservative for bridge load evaluation as it assumes that two side by side trucks with weights near the upper ends of their weight distributions will also have an inter-truck wheel spacing near the lower end of the spacing distribution. This is considered to be an overly conservative assumption as it involves the multiplying out of the probabilities of improbable events resulting in a negligible probability of occurrence. A more likely minimum distance of the centreline of a truck wheel from the edge of a travel lane when two trucks are side by side is assumed to be 0.75 metres, resulting in an inter-truck wheel spacing of 1.5 metres.

Also, for a truck with its weight near the upper end of its weight distribution, a more likely minimum distance of the centerline of the truck wheel from the edge of a curb or barrier is 0.75 m as the truck will be typically located a minimum of the shy distance away from the edge of the curb or barrier.

6.2   **Permit Trucks**

Permit trucks shall be defined by the axle loads, axle spacings, axle widths and number and spacings of wheels per axle submitted by the carrier applying to AT's Transport Engineering Section for a permit.

If the axle loads, axle spacings, axle widths and number and spacings of wheels per axle of a standard permit truck are representative of the axle loads, axle spacings, axle widths and number and spacings of wheels per axle of the actual permit truck under consideration, the ability of a bridge to carry the actual permit truck may be based on its ability to carry the standard permit truck with a gross vehicle weight equal to that of the actual permit truck.

Commentary 6.2

Permit trucks are defined by the following properties which affect the load effects they apply to a bridge:

- *The number of axles and/or axle groups;*
- *The axle spacings;*
- *The weight on each axle group;*
- *The width of and number of wheels on each axle;*
- *The spacing of the wheels on each axle; and*
- *The size of the tires on each axle.*

AT's Transport Engineering Section has carried out load evaluations for most bridges on the provincial highway network for typical permit truck configurations travelling on Alberta roads. These typical permit truck configurations are referred to as standard permit trucks in this Manual. Some of these standard permit trucks configurations are shown in Figure A1 of Appendix A. They are defined by their axle types, axle weights, axle widths and axle spacings. They are conservative representations, e.g. smaller axle spacings, of actual permit trucks and therefore the load capacity of a bridge to carry an actual permit truck will typically be greater than its load capacity to carry the similar standard permit truck of the same weight.

The standard axle types associated with these standard permit trucks are:

- *Two wheel axles used for steering axles with standard tire sizes or carrying axles with super single tires;*
- *Four wheel axles used for 4 wheel single axles, 8 wheel tandem axle groups or 12 wheel tridem axle groups;*
- *Eight wheel axles used for 16 wheel tandem axle groups, 24 wheel tridem axle groups or double wide (2 file) hydraulic trailers;*
- *Wide eight wheel axles used for wide16 wheel tandem axle groups or wide 24 wheel tridem axle groups; and*
- *Twelve wheel axle groups used for 24 wheel tandem axle groups or triple wide (3 file) hydraulic trailers.*

These standard axle types are shown in Figure A1 of Appendix A. Actual tire sizes and wheel spacings on actual axles may vary from the standard axle types but the differences are slight and the standard axle types may be used. However, when actual measured dimensions are available for a specific permit truck, these dimensions may be used for determining the load capacity of the bridges being crossed by that permit truck.

6.2.1 <u>Permit Truck Categories</u>

Permits issued by AT for a permit truck will fall into one of the following categories based on the conditions placed by AT on the permit.

- Permit – Annual, Seasonal or Multi-Trip (PA) – These permit trucks carry a divisible load (e.g. non-processed logs) or permanently mounted equipment (e.g. mobile crane) and are allowed to mix with non-permit traffic without supervision, although, conditions may apply. These conditions may control the lateral position of the truck on the bridge and/or the presence of non-permit traffic on the bridge with the permit truck. The permits are issued on an annual, seasonal or multi-trip basis for prescribed routes. This permit truck category is not equivalent to the PA Category in CSA S6-14. Permits for logging trucks are issued based on the Normal Traffic (Alternative Loading) Category in CSA S6-14 and permits for permanently mounted equipment are issued based on Permit Category PS in CSA S6-14.
- Permit – Single trip (PS) – These permit trucks carry either permanently mounted equipment (e.g. mobile crane) or non-divisible loads (e.g. D8 Cat) and are allowed to mix with non-permit traffic without supervision, although, conditions may apply to the permit. These conditions may control the lateral position of the truck on the bridge and/or the presence of non-permit traffic on the bridge with the permit truck. If approved by AT, the live load factors given in Section 14.13 (Load Factors) of CSA S6-14 may be decreased, as specified in Section 8.2 of this Manual, provided the load evaluation is based on measured load weights that are verified by scaled weights during the transport of the load. Scaled weights are weights that have been obtained from a Government of Alberta scale. This permit truck category is equivalent to the PS Category in CSA S6-14.
- Permit – Controlled (PC) – These permit trucks carry loads which are not divisible and which are too heavy to fall within the Permit – Single Trip (PS) category. They are only allowed to cross a bridge under the supervision of the Bridge Load Evaluator or his agent, with no other traffic allowed on the bridge at the same time. The load is weighed, the position of the load on the truck verified and the truck axle widths and spacings measured prior to the move. Conditions will apply to the permit. These conditions typically include the lateral position of the truck on the bridge and the speed of the truck on the bridge. This permit truck category is equivalent to the PC Category in CSA S6-14.
- Permit – Bulk haul (PB) – This classification is not currently used by AT.

Commentary 6.2.1

Permit – Annual, Seasonal or Multi-Trip includes:

- *Divisible Load – a load consisting of more than one item that can be reduced by removing part of the load. Non-processed logs are the only divisible load allowed under this category; and*
- *Permanently mounted equipment, e.g. mobile cranes, service rigs and mobile substations.*

This permit truck category applies to two types of permit trucks. The first type is logging trucks which have axle weight and gross vehicle weight variabilities similar to those for non-permit traffic. The live load factors for logging trucks are therefore based on the Normal Traffic (Alternative Loading) Category in CSA S6-14. The second type is trucks carrying permanently mounted equipment which have axle weight and gross vehicle weight variabilities similar to those for single trip permit trucks. The live load factors for trucks carrying permanently mounted equipment are therefore based on Permit Category PS in CSA S6-14.

Permit – Single Trip (PS) includes:

- *Component - A piece of a larger object. When a large object can not be moved as a whole unit, it can be broken down into smaller components that can be moved within the weight and dimension limits established by AT. Example: a drilling rig can be broken down into components such as the derrick, mud tank, drilling platform, etc;*
- *Compressor Building – A building that contains a compressor used to pump fluid or gas through a pipeline. The compressor is housed in a building which also contains the pump, motors, cooler, pipes, valves, etc. The cooler is normally placed along one end of the building and may comprise the outside wall along that side of the building and may actually be wider than the building itself;*
- *Equipment – a single piece of machinery that is manufactured to perform a specified function. It may be self propelled or stationary on the work site, but is transported by truck from location to location. It may also include attachments such as the dozer blade on a cat;*
- *Mobile Crane – a crane that is mounted on a self propelled vehicle, including all of the components needed for the operation of the crane such as counter weights, booms, cables, outrigger pads, etc;*
- *Module – an assembly of multiple components into a single unit. The size and weight can be adjusted during the design stage to meet route weight limitations;*
- *Permanently Mounted Equipment – a vehicle on which is mounted exclusively fixed equipment that cannot be removed or altered under normal operational circumstances. A vehicle with fixed equipment cannot have a deck, box, tank or any container to carry a payload, except for fuel needed to operate motors mounted on the vehicle; and*
- *Pressure Vessel – a single container used in the chemical/petro-chemical industries to process high pressure gases. As the walls of the vessel are subject to high stresses due to pressure and temperature, it is common for these items to have thick walls and to be very heavy. During fabrication, the welds must be normalized which requires a highly controlled process in a closed environment. These welds cannot be done in the field, so the entire vessel must be fabricated in a plant. The size of the vessel may influence the capacity of the processing facility or plant. An assembly with multiple pipes or tanks is not a vessel.*

Permit – Controlled (PC) includes:

- *Pressure vessel.*

6.2.2 Lateral Position of a Permit Category PA or PS Truck

6.2.2.1   No Permit Restrictions on Lateral Position of Truck

For Permit Category PA and PS trucks, where the conditions of the permit do not restrict their lateral position on the bridge, the permit truck shall be positioned laterally to produce the most critical load effects in the bridge subject to the following limitations:

- The exterior face of the exterior wheel of the permit truck shall not be placed closer than 0.6 metres from a curb or median; and
- The centre of the permit truck need not be placed more than 1.2 metres to the left of the centreline of the bridge unless otherwise specified by AT.

Commentary 6.2.2.1

A permit truck can travel in any lateral position on a bridge subject to practical limitations. As most permit trucks are wider than non-permit trucks, they may occupy more than one lane, travel with part of the truck over the centreline or on the shoulder of the roadway. Where it is possible for the wheels of

the permit truck to be adjacent to the curb or median, they are likely to be at least 0.6 metres from the curb as the width of the load typically exceeds the width of the permit truck. Note that the condition stating that the centre of the permit truck need not be placed more than 1.2 metres to the left of the centerline is based on the bridge being a two lane bridge. If the bridge is wider than two lanes AT should be contacted to provide additional direction on how the permit truck should be positioned laterally on the bridge.

6.2.2.2 Permit Restriction on Lateral Position of Truck

If the conditions of a permit for a Permit Category PA or PS or truck restrict its lateral position on the bridge, the bridge load evaluation shall account for a lateral deviation of up to 1.2 m in each direction from the prescribed lateral position. This lateral deviation may only be reduced if the lateral deviation of the truck is physically restricted by the bridge geometry or if approved by AT on a bridge specific basis.

Commentary 6.2.2.2

A permit condition restricting the lateral position of the permit truck on the bridge shall only be applied if the permit truck is being accompanied by escort vehicles and if high traffic volumes do not make it difficult for the carrier to properly control other traffic. Care shall be taken to ensure that no other traffic can be on the bridge at the same time as the permit truck as even light cars can force the permit truck onto its own side of the bridge nullifying any gain by having specified that the permit truck travel down the centre of the bridge with no traffic. If the permit truck is escorted by pilot vehicles, there is a higher confidence that the permit truck will in fact travel down the centre of the bridge.

It is impossible for a permit truck to travel exactly down the centreline of the bridge. Therefore, the permit truck should be assumed to deviate by up to 1.2 metres on either side of the centreline.

6.2.3 Lateral Position of a Permit Category PC Truck

If the conditions of a permit for a Permit Category PC truck restrict its lateral position on the bridge, the bridge load evaluation shall account for a lateral deviation of up to 0.6 m in each direction from the prescribed lateral position. This lateral deviation may only be reduced if the lateral deviation of the truck is physically restricted by the bridge geometry or if approved by AT on a bridge specific basis.

Commentary 6.2.3

It is impossible for a permit truck to travel exactly down the centreline of the bridge. Therefore, the permit truck should be assumed to deviate by up to 0.6 metres on either side of the centreline. A reduced maximum lateral truck deviation of 0.6 m from the bridge centerline is allowed as the bridge crossing is independently supervised.

6.3 Real Truck Rationalization Method

Bridge load evaluations for non-permit trucks are initially carried out using the CS1, CS2 and CS3 trucks defined in Section 6.1 of this Manual. However, for some bridges, additional load carrying capacity can be obtained if the load evaluation is carried out using actual legal non-permit truck configurations rather than the CS truck configurations. The actual legal non-permit truck configurations that need to be considered in a bridge load evaluation in lieu of the CS trucks are given in Appendix E of this Manual.

Appendix E describes a method, known as the "Real Truck Rationalization Method", that can be used to load evaluate a bridge using the actual legal non-permit truck configurations once a load evaluation using the CS truck configurations has been completed. The load evaluation results from the "Real Truck Rationalization Method" shall be recorded on the Real Truck Rationalization Form (Figure E1) in Appendix E.

Commentary 6.3

> *As noted in Section 6.1, CS1, CS2 and CS3 truck configurations produce load effects that are a conservative representation of the load effects produced by actual legal non-permit truck configurations. Therefore, it is possible for a bridge to have adequate capacity to carry legal non-permit trucks even when it does not have adequate capacity to carry a 28 tonne CS1 truck, 49 tonne CS2 truck or 63.5 tonne (54 tonne of local roads) CS3 truck.*

6.4   **One Trucked Bridges**

"One Trucked Bridges" are bridges that are signed to carry only one truck at a time. The following conditions must be met before a bridge load evaluation can be based on the bridge being "One Trucked":

- Traffic volumes are below 100 AADT;
- The bridge clear roadway is 5.5 metres or less;
- The operating speed is a maximum of 50km/hr to enable trucks to safely stop and yield to oncoming traffic.
- The sightlines and grades at each end of the bridge allow adequate time for a truck to see an oncoming truck on the bridge and to then stop short of the bridge;
- Suitable signage shall be provided; and
- AT approves the bridge being one-trucked.

Lane Load and Multiple Lane Loading need not be considered for "One Trucked Bridges".

6.5   **Dynamic Load Allowance**

6.5.1 <u>Non-Permit Trucks</u>

6.5.1.1 <u>CS Trucks</u>

For the CS1, CS2 and CS3 trucks, the dynamic load allowance shall be determined in accordance with CSA S6-14, Section 3.8.4.5, except that in Section 3.8.4.5.3, the following dynamic load allowances shall be used:

- 0.40 when only one axle of the CS truck is used;
- 0.30 when two axles of the CS truck are used; and
- 0.25 when three or more axles of the CS truck are used.

6.5.1.2 <u>Alternative Loading Trucks</u>

For "Alternative Loading", as defined by Section 14.9.1.6 of CSA S6-14, the dynamic load allowance shall be determined in accordance with CSA S6-14, Section 3.8.4.5, except that in Section 3.8.4.5.3, the following dynamic load allowances shall be used:

- 0.40 when only one axle of the "Alternative Loading" truck is used;

- 0.30 when two axles of the "Alternative Loading" truck are used; and
- 0.25 when three or more axles of the "Alternative Loading" truck are used.

For "Alternative Loading" the term "axle" shall be interpreted as an "axle group" (i.e. single axle, tandem axle or tridem axle).

6.5.2 Permit Trucks

6.5.2.1 Permit Trucks with Conventional Suspensions

For permit trucks with conventional suspensions, the dynamic load allowance shall be determined in accordance with CSA S6-14, Section 3.8.4.5, except that in Section 3.8.4.5.3, the following dynamic load allowances shall be used:

- 0.40 when only one axle of the permit truck is used;
- 0.30 when two axles of the permit truck are used; and
- 0.25 when three or more axles of the permit truck are used.

For "Permit Trucks" with conventional suspensions, the term "axle" shall be interpreted as an "axle group" (i.e. single axle, tandem axle or tridem axle).

6.5.2.2 Permit Trucks with Hydraulic Suspensions

For permit trucks with hydraulic suspensions, the dynamic load allowance shall be determined in accordance with CSA S6-14, Section 3.8.4.5, except that in Section 3.8.4.5.3, the following dynamic load allowances shall be used:

- 0.40 when only one axle of the permit truck is used;
- 0.30 when two axles of the permit truck are used; and
- 0.25 when three or more axles of the permit truck are used.

For "Permit Trucks" with hydraulic suspensions, the term "axle" shall be interpreted as any group of axles sharing a common hydraulic pressure and within 4.5m of each other.

Commentary 6.5.2.2

Axles on permit trucks with hydraulic suspensions are allowed to be spaced at a tighter spacing (eg. 1.5m axle spacing) than axles on permit trucks with conventional suspensions. As a result, the axles cannot be divided up into axles groups such as tandems or tridem axle groups as is done for permit trucks with conventional suspensions. Instead, an axle group is defined as all axles that are within 4.5m of each other. This 4.5m spacing can include up to three axles which is the same number of axles included in a tridem axle group.

6.5.2.3 Reduced Dynamic Load Allowance for Permit Category PC Trucks

The dynamic load allowance may only be reduced in accordance with CSA S6-14, Section 14.9.3 for Permit Category PC when a reduced speed is a condition of the permit. A reduced dynamic load allowance shall not be used for Permit Categories PA and PS.

Commentary 6.5.2.3

Allowing a reduced dynamic load allowance by virtue of a "slow speed" condition is an exceptional practice and should be done only for Permit Category PC. This is because there is no guarantee that the speed limit will be adhered to unless the move is done under supervision. Having a vehicle stop prior to crossing a bridge and cross the bridge at slow speed can create a traffic hazard. Therefore, the driver of a Permit Category PA or PS truck would not normally stop the truck and travel across a bridge at slow speed if it would conflict with other traffic or create a safety hazard.

6.6 Multiple Truck Loading

Sections 6.7 and 6.8 of this Manual address the load evaluation of bridges when multiple trucks are on the bridge. Section 6.7 considers truck loading in multiple lanes on the bridge while Section 6.8 considers multiple truck loading (or truck lane loading) in a single lane on the bridge.

When permit trucks are combined with non-permit trucks the following load combinations should be considered.

1.   Permit Truck Load on the Bridge with Non-Permit Truck Load in an Adjacent Lane

   - Place permit truck load in its own lane;
   - Place non-permit truck load in an adjacent lane in accordance with Section 6.7.2 of this Manual; and
   - Determine the Live Load Capacity Factor in accordance with Sections 6.7.2 and 6.8 of this Manual where "L" includes the dynamic load allowance for both the permit truck load and the non-permit truck load.

2.   Permit Truck Load on the Bridge with Non-Permit Truck Lane Load in an Adjacent Lane

   - Place permit truck load in its own lane;
   - Place non-permit truck lane load in an adjacent lane in accordance with Sections 6.7.2 and 6.8 of this Manual; and
   - Determine the Live Load Capacity Factor in accordance with Sections 6.7.2 and 6.8 of this Manual where "L" includes the dynamic load allowance for the permit truck load but not for the non-permit truck lane load.

3.   Permit Truck Lane Load on the Bridge with Non-Permit Truck Lane Load in an Adjacent Lane

   - Place permit truck lane load in its own lane in accordance with Section 6.8 of this Manual;
   - Place non-permit truck lane load in an adjacent lane in accordance with Sections 6.7.2 and 6.8 of this Manual; and
   - Determine the Live Load Capacity factor in accordance with Section 6.7.2 and 6.8 of this Manual where "L" does not include the dynamic load allowance for either the permit truck lane load or the non-permit truck lane load.

6.7 Multiple Lane Loading

6.7.1 Non-Permit Trucks

Bridge load evaluations for non-permit trucks shall consider multiple lane loading on the bridge unless the bridge is "One Trucked" in accordance with Section 6.4 of this Manual or unless otherwise approved by AT on a bridge specific basis. The multiple lane reduction factors in Table 3.5 of CSA S6-14 may be used in lieu of the multiple lane reduction factors in Table 14.2 of CSA S6-14.

Commentary 6.7.1

Multiple lane loading needs to be considered unless bridge specific traffic or operational conditions can reasonably exclude the possibility of multiple lanes being concurrently loaded by heavy trucks. Note that even on low traffic volume bridges, there can be a significant possibility of multiple lane loading by heavy trucks if the distribution of trucks crossing the bridge with time is not random, e. g., if the bridge is on a haul route, such as a gravel haul. An example of an operational condition that reasonably excludes the possibility of multiple lanes being concurrently loaded by heavy trucks is the posting of a bridge to have only one truck on the bridge at a time.

6.7.2 <u>Permit Trucks</u>

Bridge load evaluations for permit trucks shall consider multiple lane loading on the bridge unless the bridge is "One Trucked", the conditions of the permit prohibit the presence of non-permit traffic on the bridge with the permit truck or otherwise approved by AT.

If there is no permit condition prohibiting the presence of non-permit traffic on the bridge with the permit truck, multiple lane loading shall be considered by lateral positioning a non-permit truck relative to the permit truck such that:

- The non-permit truck is travelling in its own lane with the centre of the nearest wheel being a minimum of 0.75 metres from the edge of the lane nearest to the permit truck; and
- The centre of the nearest wheel of the non-permit truck is a minimum of 1.25 metres away from the outside edge of the nearest wheel of the permit truck.

The non-permit truck load shall be the percentage of the CS3 truck load or lane load, whichever governs, specified in Table 14.4 of CSA S6-14 based on a CS3 truck gross vehicle weight of 63.5 tonnes (54 tonnes on local roads).

The live load factor, α_L, used for the non-permit truck load shall be the live load factor used for non-permit trucks.

When multiple lane loading is considered in the bridge load evaluation for a permit truck, the live load capacity factor "F" in CSA S6-14, Section 14.15.2.1, shall be determined using the equation in Section 6.8 of this Manual where "A" includes the load effects due to the non-permit truck as well as the load effects due to the uniformly distributed load portion of the permit truck lane load.

Commentary 6.7.2

Multiple lane loading needs to be considered unless bridge specific traffic or operational conditions can reasonably exclude the possibility of a non-permit truck being on the bridge concurrently with the permit truck. Examples of operational conditions that reasonably exclude the possibility of a non-permit truck being on the bridge concurrently with the permit truck are the posting of a bridge to have only one truck on the bridge at a time or the conditions of the permit not allowing other trucks to be on the bridge with the permit truck.

If the bridge is wider than two lanes AT should be contacted to provide additional direction on how the non-permit truck(s) should be positioned on the bridge relative to the permit truck.

A permit condition prohibiting the presence of non-permit traffic on the bridge with the permit truck will only be issued for:

- *Permit Category PC; and*

- *Permit Category PS and PA if the permit truck is being accompanied by escort vehicles and if high traffic volumes do not make it difficult for the carrier to properly control other traffic.*

6.8   Lane Load

When lane load is considered in the bridge load evaluation, the live load capacity factor "F" in CSA S6-14, Section 14.15.2.1, shall be determined using the following equation:

$$F = \frac{UR_r - \sum \alpha_D D - \sum \alpha_A A}{\alpha_L L}$$

Where the symbols used in the equation are as defined in CSA S6-14, Section 14, except as noted below:

- "L" shall be taken to be the load effects due to a percentage of the truck load as specified in Sections 6.8.1 and 6.8.2 of this Manual;
- "A" shall be taken to be the load effects due to the uniformly distributed load portion of the lane load as specified in CSA S6-14, Sections 14.9.1 and 14.9.2. For permit trucks "A" shall also include the load effects due to non-permit truck loads or truck lane loads in adjacent lanes as described in Section 6.7.2 of this Manual; and
- α_A shall be taken to be equal to α_L for non-permit trucks.

Commentary 6.8

In CSA S6-14, the lane load consists of a uniform load and a set of truck axle loads. As shown in Figure 6.8, the uniform load portion of the lane load represents both a percentage of the truck load (Portion A) and the additional traffic on the bridge with the truck load (Portion B). The truck axle load portion of the lane load is set equal to the remaining percentage of the truck axle loads so that the combined load effects of the Portion A uniform load and the reduced truck axle loads are approximately equivalent to the load effects of the fully loaded truck.

In CSA S6-14, Section 14.15.2.1 the equation for determining the live load capacity factor "F" assumes that the truck load "L" includes the entire uniform load portion of the lane load (Portion A and Portion B) as well as a percentage of the truck axle loads. The result of this is that the magnitudes of both Portion A (which represents a percentage of the truck load) and Portion B (which represents additional traffic on the bridge) increase or decrease with the magnitude of the truck load that can be carried by the bridge. In other words;

- *if the live load capacity factor determined for the truck load assumed in the load evaluation is greater than 1, the load evaluation indicates that the bridge can carry both a truck load and additional traffic on the bridge (Portion B of the uniform load) greater than those assumed in the load evaluation. This outcome is conservative as the magnitude of the additional traffic on the bridge (Portion B of the uniform load) that can be carried by the bridge is increased above that specified by CSA S6-14; and*
- *if the live load capacity factor determined for the truck load assumed in the load evaluation is smaller than 1, the load evaluation indicates that the bridge can not carry the truck load combined with the additional traffic on the bridge (Portion B of the uniform load) assumed in the load evaluation. This outcome is unconservative as not only the magnitude of the truck load but also the magnitude of the additional traffic on the bridge (Portion B of the uniform load) that can be carried by the bridge is decreased below that specified by CSA S6-14.*

This Manual adopts an alternate method for determining the live load capacity factor. For this method, Portion A and Portion B of the uniform load are included in "A" while only a percentage of the truck axle

load is included in "L". This is equivalent to including Portion A of the uniform load in "A" and Portion B of the uniform load in "L" as long as the magnitude of the truck load assumed in the load evaluation gives a live load capacity factor of 1.0. Therefore, in this method, the percentage of the truck axle loads represented by Portion A of the uniform load is always based on the magnitude of the truck load that gives a live load capacity factor of 1.0 rather than the magnitude of the truck load assumed in the load evaluation. The drawback of this method, as well as of the method in CSA S6-14, is that the percentage of truck axle load used in the evaluation should theoretically vary with the live load capacity factor. However, this would involve an iterative and cumbersome process. To simplify the process, the approach adopted by this Manual allows the percentages of truck axle loads to be kept constant (as stated in Sections 6.8.1 and 6.8.2 of this Manual).

6.8.1 <u>Non-Permit Trucks</u>

Bridge load evaluations for non-permit trucks shall consider lane load on the bridge unless the bridge is "One-Trucked" or unless otherwise approved by AT on a bridge specific basis.

For non-permit trucks, "L" as defined in Section 6.8 of this Manual, shall be taken to be the load effects due to 80% of the truck load.

Commentary 6.8.1

Lane load needs to be considered unless bridge specific traffic or operational conditions can reasonably exclude the possibility of a lane being concurrently loaded by more than one heavy truck. Note that even on low traffic volume bridges, there can be a significant possibility of a lane being loaded by more than one heavy truck if the distribution of trucks crossing the bridge with time is not random, e. g., if the trucks are travelling in a convoy. An example of an operational condition that reasonably excludes the possibility of a lane being concurrently loaded by more than one heavy truck is the posting of a bridge to have only one truck on the bridge at a time.

For non-permit trucks the percentage of the truck load assumed to be represented by the uniform load portion of the lane load is 20% of the truck load. This is consistent with Section 14.9.1.6 of CSA S6-14 which reduces the magnitude of the truck load portion of the lane load by 20% before combining it with the uniformly distributed load portion of the lane load.

6.8.2 <u>Permit Trucks</u>

Bridge load evaluations for permit trucks shall consider lane load on the bridge unless the bridge is "One-Trucked", the conditions of the permit prohibit the presence of non-permit traffic on the bridge with the permit truck or otherwise approved by AT.

Commentary 6.8.2

A permit condition prohibiting the presence of non-permit traffic on the bridge with the permit truck will only be issued for:

- *Permit Category PC; and*
- *Permit Category PA and PS if the permit truck is being accompanied by escort vehicles and if high traffic volumes do not make it difficult for the carrier to properly control other traffic.*

6.8.2.1 Permit Category PA

For Permit Category PA trucks, "L", as defined in Section 6.8 of this Manual, shall be taken to be the load effects due to 80% of the truck load for logging trucks and the load effects due to 85% of the truck load for trucks carrying permanently mounted equipment.

Commentary 6.8.2.1

The 20% reduction specified in this section for logging trucks is consistent with the reduction specified for non-permit trucks and takes into account that, with the exception of some logging truck configurations, AT restricts the gross vehicle weights of Permit Category PA trucks to those of non-permit trucks.

The 15% reduction specified in this section for trucks carrying permanently mounted equipment is consistent with the reduction specified for Permit Category PS trucks.

6.8.2.2 Permit Category PS

For Permit Category PS trucks, "L", as defined in Section 6.8 of this Manual, shall be taken to be the load effects due to 85% of the truck load.

Commentary 6.8.2.2

For Permit Category PS trucks the percentage of the truck load assumed to be represented by the uniform load portion of the lane load is 15% of the truck load. This is consistent with Section 14.9.2.5 of CSA S6-14 which reduces the magnitude of the permit truck load portion of the lane load by 15% before combining it with the uniformly distributed load portion of the lane load.

6.8.2.3 Permit Category PC

For Permit Category PC trucks, lane load need not be considered.

6.9 Lateral Distribution of Live Load

This Section 6.9 (Lateral Distribution of Live Load) provides information on methods for determining the lateral distribution of live load between primary load carrying members.

The lateral distribution of live load for non-permit trucks shall be determined based on:

- The Simplified Method (see Section 6.9.3 of this Manual);
- The Statically Determinate Method (see Section 6.9.1 of this Manual) for bridge superstructure types not covered by the Simplified Method; or
- The Sophisticated Method (see Section 6.9.2 of this Manual) if approved by AT.

The lateral distribution of live load for permit trucks shall be determined based on:

- Either the Statically Determinate Method (see Section 6.9.1 of this Manual) or the Sophisticated Method (see Section 6.9.2 of this Manual); or
- The Simplified Method (see Section 6.9.3 of this Manual) only if the Simplified Method results in a Live Load Capacity Factor greater than or equal to 1 for the permit truck under consideration.

6.9.1 <u>Statically Determinate Method</u>

6.9.1.1 <u>Bridges with Two Primary Load Carrying Members</u>

The Statically Determinate method shall be used for all bridges with two primary load carrying members, such as truss bridges and bridges with two girder lines unless otherwise approved by AT.

The effects of bracing shall not be used to improve the lateral distribution of live load between primary load carrying members at the ultimate limit state unless otherwise approved by AT.

Each primary load carrying member shall be assumed to carry a minimum of 60% of the live load.

Commentary 6.9.1.1

The effects of bracing are not to be used to improve the lateral distribution of live load between primary load carrying members as it is unlikely that the bracing has been designed for this purpose.

Primary load carrying members are to be considered to carry a minimum of 60% of the live load to account for the fact that it is unlikely that the live load will be perfectly centred on the bridge.

6.9.1.2 <u>Bridges with More Than Two Primary Load Carrying Members</u>

6.9.1.2.1 <u>Deck on Girder Bridges</u>

If the lateral distribution of live load to interior girders of deck on girder bridges is determined by assuming that the deck is pinned at the girders (see Figure 6.9.1.2.1a) the lateral distribution of live load may be considered to have been determined by the Statically Determinate method.

If the lateral distribution of live load to exterior girders of deck on girder bridges is determined by assuming that the deck is pinned at the interior girder and continuous across and pinned to the exterior girder (see Figure 6.9.1.2.1b) the lateral distribution of live load may be considered to have been determined by the Statically Determinate method.

Commentary 6.9.1.2.1

The requirements of this section provide for a system of live load distribution between girders of deck on girder bridges that conservatively satisfies equilibrium and is safe provided the bridge load carrying system has adequate ductility to allow for redistribution of load between girders. The deck on girder bridges in AT's bridge inventory are generally considered to have adequate ductility for this purpose, although, the reasonableness of this assumption should be assessed on a bridge specific basis. The bridge specific assessment should include an assessment of both the stiffness of the bearings supporting the girders and the magnitude of the girder deflections that will occur prior to girder failure. More flexible bearings, such as elastomeric bearings, will deflect more than stiffer bearings, such as steel bearings, and provide for a more uniform load distribution between girders. Also, girders that deflect more prior to failure will be better able to distribute load to adjacent girders resulting in a more uniform load distribution between girders.

Deck on girder bridges are typically Span Types "RG", "RB", "WG", "FR", "CT", "CX", "CXP", "PB", "PO", "OM" or "NU".

6.9.1.2.2 Unconnected Girder Bridges

If the lateral distribution of live load between the girders of unconnected girder bridges is determined by assuming that each girder supports the wheel loads and portions of wheel loads applied directly to it (based on the wheel widths shown in CSA S6-14, Figure 3.2 for non-permit trucks and the wheel widths provided by the carrier for permit trucks), the lateral distribution of live load between girders may be considered to have been determined by the Statically Determinate method.

Commentary 6.9.1.2.2

The requirements of this section provide for a conservative system of live load distribution between the girders of unconnected girder bridges. Unconnected girder bridges are typically precast reinforced concrete girder bridge Span Types "GR", "HH" and "PG".

6.9.1.2.3 Shear Connected Girder Bridges

If the lateral distribution of live load moment between shear connected girders is determined by assuming that the truck load is carried by the girders, or portions of girders, directly beneath the out-to-out width of the truck axles (based on the axle and wheel widths shown in CSA S6-14, Figure 3.2 for non-permit trucks and the axle and wheel widths provided by the carrier for permit trucks), i.e. the percentage of truck load carried by the most heavily loaded girder is equal to the girder width divided by the out-to-out width of the truck axles, the lateral distribution of live load moment between girders may be considered to have been determined by the Statically Determinate method.

If the lateral distribution of live load shear between shear connected girders is determined by assuming that each girder supports the wheel loads or portions of wheel loads applied directly to it (based on the wheel widths shown in CSA S6-14, Figure 3.2 for non-permit trucks and the wheel widths provided by the carrier for permit trucks), i.e. there is no lateral distribution of live load between girders, the lateral distribution of live load shear between girders may be considered to have been determined by the Statically Determinate method.

Commentary 6.9.1.2.3

The requirements of this section provide for a conservative system of live load distribution between girders connected together with either bolted steel shear connectors at discrete locations only or with continuous concrete shear keys. Girders with bolted steel shear connectors are typically precast reinforced concrete girder bridge Span Types "HC" and "VH" as well as precast prestressed concrete girder bridge Span Types "SM", "SC" and "SL". Girders with continuous concrete shear keys are typically precast reinforced concrete girder bridge Span Types "PA", "PE" and "PES" as well as precast prestressed concrete girder bridge Span Types "FC", "VF", "LF", "FM", "PM", "VM", "RD", "RM", "DBT" and "CBT".

6.9.1.2.4 Slab Bridges

If the lateral distribution of live load moment in a slab is determined by assuming that the truck load is uniformly carried by the width of slab directly beneath the out-to-out width of the truck axles (based on the axle and wheel widths shown in CSA S6-14, Figure 3.2 for non-permit trucks and the axle and wheel widths provided by the carrier for permit trucks), i.e. the percentage of truck load carried by a 1 m width of slab is equal to 1 divided by the out-to-out width of the truck axles, the lateral distribution of live load moment may be considered to have been determined by the Statically Determinate method.

If the lateral distribution of live load shear in a slab is determined by assuming that each 1 m width of slab supports the wheel loads or portions of wheel loads applied directly to it (based on the wheel widths shown in CSA S6-14, Figure 3.2 for non-permit trucks and the wheel widths provided by the carrier for permit trucks), i.e. there is no lateral distribution of live load between 1 m widths of slab, the lateral distribution of live load shear may be considered to have been determined by the Statically Determinate method.

Commentary 6.9.1.2.4

The requirements of this section provide for a conservative system of live load distribution in the slab. Slab bridges are typically Span Type "CS", "CV" or abutment roof slabs of other Span Type bridges.

The lateral distribution of live load shear presented in this section assumes that a 600 mm wide dual wheel will be distributed over a minimum width of 1 m, i.e. the wheel load will spread out a minimum distance of 200 mm on each side of the wheel. For slabs with a minimum thickness of 400 mm this means that the wheel load needs to spread out at a maximum angle of 45° between the top surface and mid-depth of the slab.

6.9.1.3 Determination of Skew Effects on Shear

The effects of skew on the lateral distribution of live load shear between the girders of deck-on-girder, unconnected girder and shear-connected girder bridges may be ignored if the lateral distribution is determined using the Statically Determinate Method and the bridge skew is not greater than 30°.

The effects of skew on the lateral distribution of live load shear in slab bridges may be ignored if the lateral distribution is determined using the Statically Determinate Method and the bridge skew is not greater than 20°.

Commentary 6.9.1.3

Skew effects reduce moments in primary load carrying members and can therefore be ignored for all bridge skews.

Skew effects increase shears in primary load carrying members particularly near the obtuse corners of the bridge superstructure. However, as the Statically Determinate method provides a conservative estimate of the lateral distribution of live load between primary load carrying members, these increased shear effects may be ignored as long as the primary load carrying members have adequate ductility to allow for redistribution of load between primary load carrying members. The bridges in AT's bridge inventory, with skews not greater than 30° for girder bridges and 20° for slab bridges, are generally considered to have adequate ductility for this purpose, although, the reasonableness of this assumption should be assessed on a bridge specific basis. This bridge specific assessment should include an assessment of both the stiffness of the bearings supporting the girders and the magnitude of the girder deflections that will occur prior to girder failure. More flexible bearings, such as elastomeric bearings, will deflect more than stiffer bearings, such as steel bearings, and allow for a greater redistribution of load away from the obtuse corners of the bridge superstructure. Also, girders that deflect more prior to failure will be better able to distribute load to adjacent girders and away from the obtuse corners of the bridge superstructure.

The requirement that skew effects be considered in slab bridges with skews greater than 20° is consistent with the requirements of the Ontario Highway Bridge Design Code, 2nd Edition (OHBDC, 1983).

6.9.2 <u>Sophisticated Method</u>

Acceptable Sophisticated methods are the grillage analogy method, the finite element method and the semi-continuum method. Other Sophisticated methods of analysis may be used if approved by AT on a bridge specific basis.

Commentary 6.9.2

The grillage analogy method has historically been the Sophisticated method most commonly used by AT. Each primary load carrying member is represented by one or more longitudinal members in the model and the bridge deck is modeled as a series of discretely spaced transverse members.

The finite element method is sometimes used in lieu of the grillage analogy method. In the finite element method, the bridge deck can be modeled using shell elements and the primary load carrying members can be modeled as longitudinal beam elements which can be offset vertically from the deck with rigid links.

The semi-continuum method models the bridge deck as a semi-infinite plate and the primary load carrying members as longitudinal beam elements (Jaeger and Bakht, 1989). It can only be used for square bridges or bridges that can be approximated as square. The semi-continuum method is used by the computer program SECAN4 to determine the lateral distribution of loads between girders.

For shear-connected girders, the transverse members in the model should be assumed to be pinned between girders. Also, if the grillage analogy method or the finite element method with the longitudinal members modeled as beam elements is used, each channel shaped shear-connected girder should be modeled by a longitudinal member at each girder leg location so that warping torsion effects are captured in the model.

Each of the above Sophisticated methods requires that the primary load carrying members in the model be given both a flexural and torsional stiffness. Since primary load carrying members are typically not designed for torsional effects, the members should be assessed to verify that they can resist the resulting torques if they are given a significant torsional stiffness. Alternatively, torsional effects can be ignored if the members are given a negligible torsional stiffness in the model.

6.9.2.1 <u>Non-Permit Trucks</u>

The Sophisticated method shall not be used to determine the lateral distribution of live load between primary load carrying members for non-permit trucks unless approved by AT on a bridge specific basis.

Commentary 6.9.2.1

To maintain consistency between the load evaluation results for the different bridges in AT's bridge inventory, the lateral distribution of live load between primary load carrying members should be determined using either the Statically Determinate or Simplified methods unless otherwise approved by AT. The Sophisticated method should generally only be used when the use of either the Statically Determinate or Simplified methods result in a bridge load capacity that is not adequate for carrying legal non-permit trucks.

6.9.2.2 <u>Permit Trucks</u>

The Sophisticated method may be used to determine the lateral distribution of live load between primary load carrying members for permit trucks.

6.9.2.3 Shear Force Effects

Sophisticated methods, such as the grillage analogy method, that model a continuous bridge deck as a series of discrete transverse members will not automatically give accurate girder shear load effects adjacent to concentrated truck wheel loads. The girder shear load effects obtained from these Sophisticated methods shall be adjusted in accordance with the requirements of Appendix F of this Manual, to correct for the inaccuracies introduced.

Commentary 6.9.2.3

If the bridge deck is modeled as a series of transverse members rather than as a continuum, the envelope of the shear load effects along the length of the primary load carrying member will be a step function rather than the correct uniformly varying function (with the exception of at wheel load locations). Therefore, corrections to the step function are required to obtain correct results. A method of correcting the step function is given in Appendix F.

6.9.2.4 Transverse Bracing

The effects of transverse bracing on the lateral distribution of live load between primary load carrying members shall not be considered unless approved by AT on a bridge specific basis.

Commentary 6.9.2.4

If bracing members between primary load carrying members are included in the model, the bracing members should be load evaluated to verify whether or not they have adequate capacity to resist the resulting loads.

6.9.2.5 Effects of Skew

The Sophisticated method shall take the skew of the bridge into account unless the effects of skew are accounted for separately. For deck on girder bridges the effects of skew on the lateral distribution of live load shear forces between girders may be determined in accordance with the requirements of CSA S6-14 Section 5.6.6.2.

6.9.3 Simplified Method

The Simplified method shall be based on CSA S6-14, Section 5 unless noted otherwise in this Section 6.9.3.

6.9.3.1 Non-Permit Trucks

The Simplified method may be used to determine the lateral distribution of live load between primary load carrying members for the non-permit CS truck configurations.

Commentary 6.9.3.1

The Simplified method is based on the CL truck configurations shown in CSA S6-14, Section 3.8.3.1.2. However, the CS and CL truck configurations have enough similarities that the Simplified method may also be assumed to be applicable for the CS truck configurations.

In accordance with Section 6.1.4 of this Manual, the truck wheels may be assumed to be 0.75 m rather than 0.6 m from the edges of their travel lanes. The results of the Simplified method will therefore be conservative for this assumption.

6.9.3.2 Permit Trucks

The Simplified method may be used to determine the lateral distribution of live load between primary load carrying members for permit trucks provided the Bridge Load Evaluator has determined, based on engineering judgment, that the Simplified method will give conservative results for lateral distribution of live load between primary load carrying members for the permit truck under consideration.

Commentary 6.9.3.2

For permit trucks, the axle widths and number of wheels per axle are generally greater than or equal to the axle widths and number of wheels per axle assumed by the Simplified method for non-permit trucks. Therefore, the Simplified method will generally give conservative results for lateral distribution of live load between primary load carrying members when used for permit trucks.

6.9.3.3 Shear Connected Girders

The Simplified Method may only be used to determine the lateral distribution of live load between shear connected girders with continuous concrete shear keys if the concrete shear keys have no damage or deterioration other than hairline cracking. If the concrete shear keys have damage or deterioration other than hairline cracking the lateral distribution of live load shall be determined in accordance with Section 6.9.1.2.3 of the Manual unless otherwise approved by AT.

When using the Simplified Method, the lateral distribution of live load between shear connected girders with continuous concrete shear keys shall be determined in accordance with the requirements of Appendix G, rather than in accordance with the requirements of CSA S6-14, Section 5.

Commentary 6.9.3.3

Shear connected girders with continuous concrete shear keys laterally transfer live load between the girders through the shear keys which connect the girders together. If the shear keys are damaged or deteriorated their ability to transfer live load between the girders may be compromised. In this case the lateral distribution of live load between the girders is conservatively determined by Section 6.9.1.2.3 of the Manual which assumes minimal transfer of live load moment between the girders and no transfer of live load shear between the girders.

The requirements of Appendix G are based on recommendations made by Ramsay (Ramsay, 2015a). This study determined the lateral distribution of live load between shear connected girders for three types of shear connected girders (Type SL voided box girders, Type FC channel girders and Type DBT bulb-tee girders) in AT's bridge inventory using four different current and historical bridge codes, including a modification of the CAN/CSA S6-88 code. The results were compared with the results from twelve grillage analyses. The grillage analyses were carried out on three different span lengths of two lane voided box and channel girders. Six of the grillage analyses placed the truck wheels a minimum of 0.6 m away from the edges of the travel lanes while the remaining six grillage analyses placed the truck wheels a minimum of 0.75 m away from the edges of the travel lanes.

The predictions of the modified CAN/CSA S6-88 method were generally lower than those of the other codes and were also generally found to be conservative relative to the results from the grillage analyses provided the truck wheels were placed a minimum of 0.75 m away from the edges of the travel lanes. The one exception to this conclusion was that the predicted lateral distribution of live load moment between shear connected girders with lower than typical span to depth ratios was up to 4% unconservative. These girders are quite stiff in the longitudinal direction and are therefore less efficient at distributing live load moment laterally between girders than more longitudinally flexible girders. However, girders with low span to depth ratios will often have excess strength and not govern the load capacity of a bridge although this should be verified on a bridge specific basis.

The provisions of Appendix G are therefore based on the lateral distribution of live load between shear connected girders provisions of the modified CAN/CSA S6-88 method.

6.9.3.4 Determination of Skew Effects on Moment

The effects of skew shall be ignored in the determination of the lateral distribution of live load moments between primary load carrying members.

Commentary 6.9.3.4

Skew effects reduce moments in primary load carrying members and can therefore be ignored.

6.9.3.5 Determination of Skew Effects on Shear

The effects of skew on the lateral distribution of live load shear between the girders of deck-on-girder, unconnected girder and shear connected girder bridges may be ignored if the lateral distribution is determined using the Simplified Method and the bridge skew is not greater than 30°.

The effects of skew on the lateral distribution of live load shear in slab bridges may be ignored if the lateral distribution is determined using the Statically Determinate Method and the bridge skew is not greater than 20°.

For deck on girder bridges the effects of skew on the lateral distribution of live load shear forces between girders may be determined in accordance with the requirements of CSA S6-14 Section 5.6.6.2.

Commentary 6.9.3.5

See Section 6.9.1.3 of this Manual for the rationale for ignoring skew effects in low skew bridges.

CS3 : VEHICLE - TRAINS

Axle Spacing (m)

Gross Vehicle Weight = 1.0 W

Axle Load (W)

	4.0	6.0	6.0
0.10W	0.30W	0.30W	0.30W
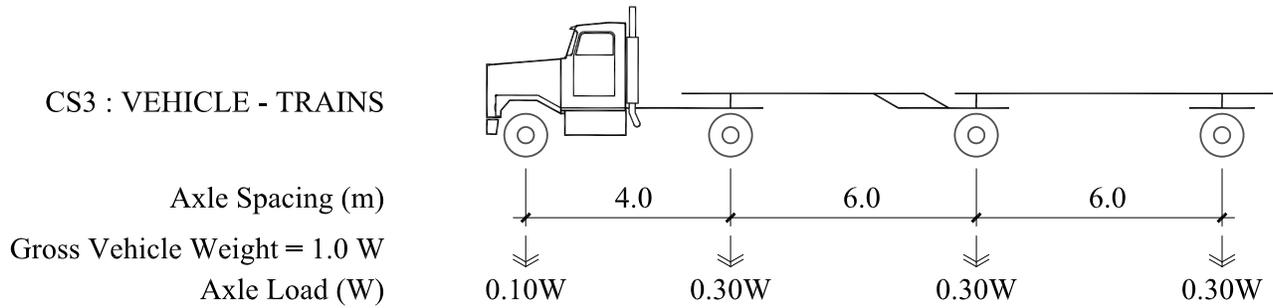
FIGURE 6.1.1 CS3 Load Evaluation Truck

CS2 : TWO-UNIT VEHICLES

Axle Spacing (m)

Gross Vehicle Weight = 1.0 W

Axle Load (W)

	4.0	6.0
0.14W	0.43W	0.43W
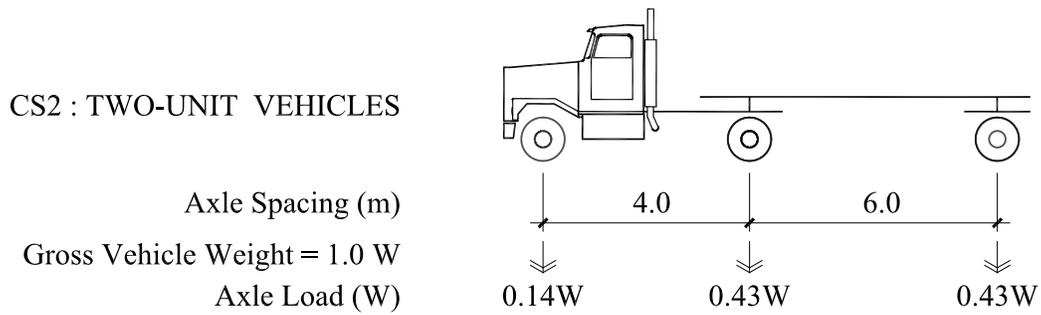
FIGURE 6.1.2 CS2 Load Evaluation Truck

CS1 : SINGLE UNIT VEHICLES

Axle Spacing (m)

Gross Vehicle Weight = 1.0 W

Axle Load (W)

	4.0
0.25W	0.75W
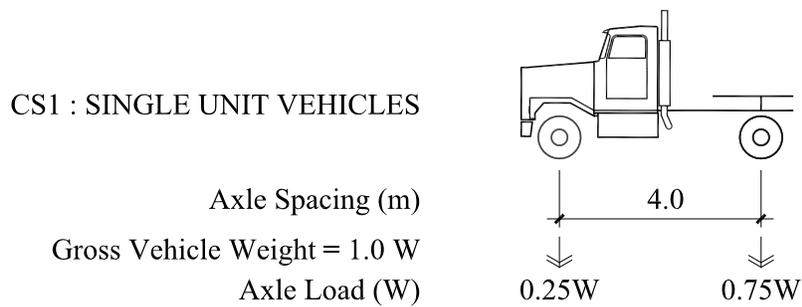
FIGURE 6.1.3 CS1 Load Evaluation Truck
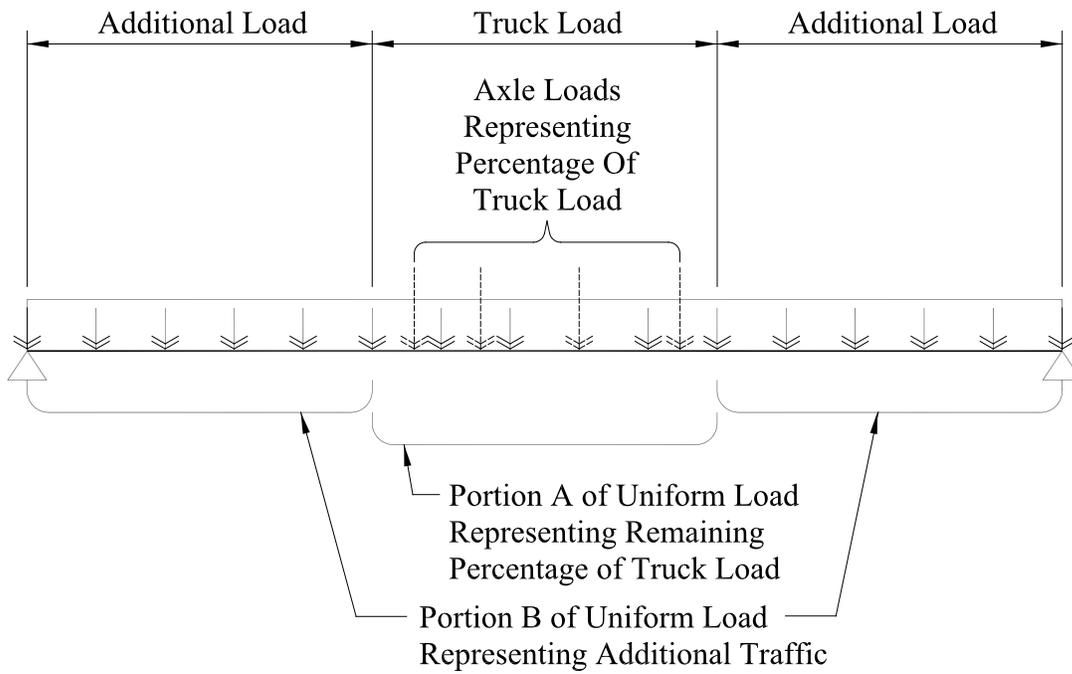
FIGURE 6.8 Treatment of Uniform Load Portion of Lane Load
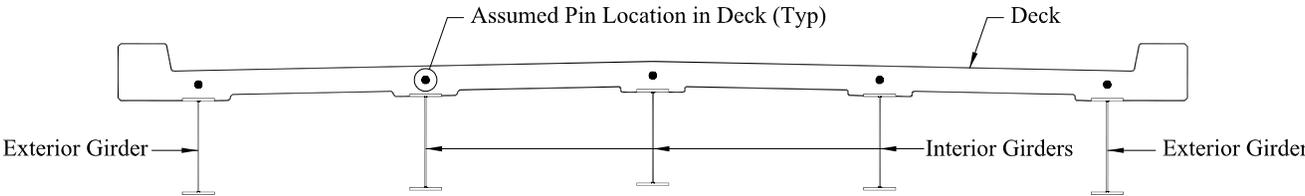
Assumed Pin Location in Deck (Typ)   Deck

Exterior Girder   Interior Girders   Exterior Girder

a) Determination of Lateral Distribution of Live Load to Interior Girders

Assumed Pin Location in Deck (Typ)   Deck

Exterior Girder   Interior Girders   Exterior Girder
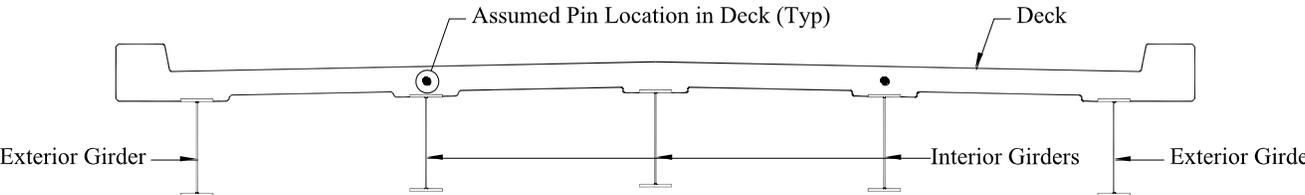
b) Determination of Lateral Distribution of Live Load to Exterior Girders

Figure 6.9.1.2.1 Simplified Lateral Distribution of Live Load Between Girders

7.   Target Reliability Indices

This Section 7 (Target Reliability Indices) supplements CSA S6-14, Section 14.12 (Target Reliability Index). It provides information on:

- The selection of System Behaviour categories for common AT bridge Span Types;
- The selection of Element Behaviour categories for common AT bridge Span Types; and
- The selection of Inspection Level categories.

7.1   System Behaviour

System Behaviour takes into account the consequences of failure if the bridge component being load evaluated fails. System Behaviour Category S1 is selected if failure of the bridge component is expected to lead to total collapse of the bridge while System Behaviour Category S3 is selected if failure of the bridge component is expected to lead to local failure only. System Behaviour Category S2 is selected if failure of the bridge component is expected to lead to more than local failure but less than total collapse of the bridge.

Recommended System Behaviour categories for common bridge components in AT's bridge inventory are provided in Sections 7.1.1 to 7.1.3.

7.1.1 <u>Category S1 Examples</u>

- Simple truss spans;
- Moment in simple girder spans with two girder lines, e.g. two "I" girders or a single box girder;
- Shear in continuous truss spans; and
- Shear in simple and continuous girder spans with two girder lines, e.g. two "I" girders or a single box girder.

7.1.2 <u>Category S2 Examples</u>

- Floorbeams;
- Moment in continuous truss spans;
- Moment in simple girder spans with three girder lines, e.g. three "I" girders;
- Moment in continuous girder spans with two girder lines, e.g. two "I" girders or a single box girder; and
- Shear in simple and continuous girder spans with three girder lines, e.g. three "I" girders;

7.1.3 <u>Category S3 Examples</u>

- Stringers;
- Moment in simple girder spans (including trellis girder spans) with four or more girder lines, e.g. four or more "I" girders or two or more box girders;
- Moment in continuous girder spans with three or more girder lines, e.g. more than two "I" girders or more than a single box girder;
- Shear in simple and continuous girder spans (including trellis girder spans) with four or more girder lines, e.g. four or more "I" girders or two or more box girders; and
- Decks and slabs (including decks and slabs on trellis girders).

7.2 **Element Behaviour**

Element Behaviour takes into account the warning of failure that will be provided before the bridge component being load evaluated fails. Element Behaviour Category E1 is selected if failure of the bridge component is expected to occur in a brittle manner with no warning of failure while Element Behaviour Category E3 is selected if failure of the bridge component is expected to occur in a gradual manner and only after large member deformations have occurred. Element Behaviour Category E2 is selected if failure of the bridge component is expected to occur in a gradual manner but without large member deformations occurring.

Recommended Element Behaviour categories for common bridge components in AT's bridge inventory are provided in Sections 7.2.1 to 7.2.3.

7.2.1 <u>Category E1 Examples</u>

- Elastic buckling of steel compression members;
- Elastic buckling of steel girder compression flanges;
- Steel tension members at a net section;
- Steel girders in bearing;
- Steel girders in shear that fail in elastic buckling with no tension field action;
- Concrete girders in moment not meeting the requirements of CSA S6-14, Section 8.8.4.5;
- Concrete girders in moment not meeting the requirements of CSA S6-14, Section 8.8.4.3;
- Concrete girders in shear if the transverse shear reinforcing steel is less than 50% effective in resisting shear based on the requirements of CSA S6-14, Section 14.14.1.6;
- Anchorage of reinforcing steel in concrete girders that does not develop the yield strength of the reinforcing steel;
- Connections between members or components;
- Timber members in moment;
- Timber members in shear; and
- Timber members in axial load.

7.2.2 <u>Category E2 Examples</u>

- Inelastic buckling of steel compression members;
- Inelastic buckling of steel girder compression flanges;
- Steel girders in shear that fail in inelastic buckling with no tension field action; and
- Concrete girders in shear if the transverse shear reinforcing steel is at least 50% effective in resisting shear based on the requirements of CSA S6-14, Section 14.14.1.6 and the transverse shear reinforcing steel and longitudinal reinforcing steel resisting the shear have adequate anchorage to develop their yield strengths.

7.2.3 <u>Category E3 Examples</u>

- Yielding of steel members at a gross section;
- Steel girders in shear that fail by yielding;
- Steel girders in shear that fail in elastic or inelastic buckling with subsequent tension field action;
- Concrete girders in moment meeting the requirements of CSA S6-14, Sections 8.8.4.3 and 8.8.4.5 provided the longitudinal reinforcing steel resisting the moment has adequate anchorage to develop its yield strength; and
- Timber members in bearing.

Unless better information on the mode of member failure is available, the Element Behaviour Category for members whose failure is the result of combined load effects (e.g. combined moment and shear, combined moment and axial load) shall be based on the least ductile member failure resulting from any of the individual load effects.

Commentary 7.2

A steel compression member can be assumed to fail in elastic buckling if the value of "λ" as determined in accordance with CSA S6-14, Section 10.9.3 is greater than 0.85. It can be assumed to fail in inelastic buckling if "λ" is less than or equal to 0.85 and its capacity is less than the yield capacity.

A steel girder compression flange can be assumed to fail in elastic buckling if the value of M_u as determined in accordance with CSA S6-14, Section 10.10.3.3 is less than 67% of the yield moment. It can be assumed to fail in inelastic buckling if M_u is greater than or equal to 67% of the yield moment and less than the yield moment.

A steel girder can be assumed to fail in elastic shear buckling if the value of V_r is governed by CSA S6-14, Section 10.10.5.1 (c) and in inelastic shear buckling if the value of V_r is governed by CSA S6-14, Section 10.10.5.1 (b). It can be assumed to fail by yielding if the value of V_r is governed by CSA S6-14, Section 10.10.5.1 (a). There will be no tension field action if the value of F_t in CSA S6-14, Section 10.10.5.1 is taken to be zero.

7.3   Inspection Level

Inspection Level takes into account the quality of the information known about the condition of the bridge component being load evaluated through inspection. Inspection Level INSP0 is selected for new bridge components which have never carried load or been inspected. Inspection Level Category INSP1 is selected if the bridge component being load evaluated can't be inspected because it was not visible during the inspection. Unless otherwise authorized by AT, Inspection Level Category INSP3 may only be selected if a recent detailed bridge inspection of the primary load carrying members being load evaluated has been carried out and the primary load carrying members are not damaged or deteriorated. The Inspection Level Category INSP3 load evaluation shall be valid for a maximum of six months from the time of the detailed bridge inspection. Inspection Level INSP2 is selected if none of the other Inspection Levels apply

INSP0 "β" values are not given in CSA S6-14. They shall be taken to be 0.25 greater than the INSP1 "β" values.

Recommended Inspection Level categories are provided in Sections 7.3.1 to 7.3.4.

7.3.1 Inspection Level INSP0 Examples

- Components of new bridges with no load carrying history.

7.3.2 Inspection Level INSP1 Examples

- Webs of concrete box girders placed side by side;
- Buried portions of foundations;
- Abutment roof slabs with no access from beneath;
- Girders covered with an asphalt wearing surface when the resistance of the top of the girder governs the resistance of the girder;
- Non-visible girder bearing areas; or

- Bridge components or portions of bridge components not clearly visible during an inspection, (e.g. interior spans and/or piers of Major River Bridges during a Level 1 BIM Inspection).

7.3.3 Inspection Level INSP2 Examples

- None of the other Inspection Levels apply and the bridge has been inspected in accordance with the requirements of Section 2.1 of this Manual; or
- Girders covered with an asphalt wearing surface when the resistance of the top of the girder does not govern the resistance of the girder.

7.3.4 Inspection Level INSP3 Examples

- A detailed bridge inspection has been carried out within the last six months in accordance with the requirements of Section 3.2.3 of this Manual and the primary load carrying members being load evaluated are clearly visible during the inspection and not damaged or deteriorated. Inspection Level INSP3 is only valid for loads that cross the bridge within six months of the detailed bridge inspection.

Commentary 7.3.4

Inspection Level INSP 3 is only to be used if it can be verified that the observations made during the inspection are still valid at the times that the loads, for which the load evaluation was carried out, are crossing the bridge. This Manual assumes that this will generally be the case if a maximum of six months has passed since the inspection. However, the Bridge Load Evaluator should verify with AT on an ongoing basis that the bridge has not been subjected to accident damage or maintenance/repair activities.

Inspection Level INSP3 is not allowed to be used for damaged or deteriorated primary load carrying members because of the increased uncertainty associated with determining the resistance of damaged or deteriorated members.

7.4 **Important Structures**

A bridge shall not be designated an important structure unless approved by AT.

Commentary 7.4

The designation of a structure as an important structure should be considered to be an unusual event. It should only be considered if the loss of service of a bridge would physically isolate a community from essential services.

8. Load Factors

This Section 8 (Load Factors) supplements CSA S6-14, Section 14.13 (Load Factors). It provides information on:

- The load factors to be used with CS1, CS2 and CS3 trucks;
- The Short Spans live load factors that are to be used in lieu of those provided in CSA S6-14; and
- The adjustments that may be made, subject to AT approval, to the CSA S6-14 load factors used for Permit Category PS trucks.

8.1 Non-Permit Trucks

Except as noted in Section 8.3.1.1 of this Manual, the "All Spans" Live Load Factors shown in CSA S6-14, Table 14.8 shall be used for the CS1, CS2 and CS3 trucks.

Commentary 8.1

See Section 6.1.3 of this Manual for the rationale for using the same live load factors for the CS trucks as for the CL trucks for spans other than short spans.

8.2 Permit Category PS Trucks

The Live Load Factors shown in CSA S6-14, Table 14.13 may be divided by 1.08 for Permit Category PS trucks whose weights have been measured and will be scaled during transport, if approved by AT.

Commentary 8.2

Generally Permit Category PS truck permits are issued based on the carrier providing estimated weights for the axle weights and gross vehicle weights. However, if the Permit Category PS weights are based on measured weights that will be scaled during transport rather than on estimated weights, the live load factors in CSA S6-14, Table 14.13 can sometimes be decreased by 8%. This is based on the live load factors for Permit Category PC trucks (whose weights are verified by weighing) being 12% lower than the live load factors for Permit Category PS trucks (whose weights are not verified by weighing) for the same target reliability index. The 8% decrease in live load factor for weighed Permit Category PS trucks is two-thirds of this difference. The other 4% accounts for any inaccuracies in the procedure for determining the measured weights or in the accuracy of the weigh scales. However, this approximately 8% increase in the allowable weight of a Permit Category PS truck is only allowed in specific situations at AT's discretion.

The weight of a Permit Category PS truck is to be verified at a Government of Alberta weigh scale after transport of the load has begun as a permit condition. If the scaled weights exceed those for which the permit has been issued, AT is to be advised so that they can determine if any additional actions will be required, prior to transport of the load resuming.

8.3   **Short Spans Live Load Factors**

8.3.1 Non-Permit Trucks

8.3.1.1 CS Truck Loading

8.3.1.1.1 CS1 Truck Loading

The "All Spans" Live Load Factors shown in CSA S6-14, Table 14.8 shall be used for all span lengths. "Short Spans" Live Load Factors shall not be used.

8.3.1.1.2 CS2 Truck Loading

The "All Spans" Live Load Factors shown in CSA S6-14, Table 14.8 shall be used for all span lengths. "Short Spans" Live Load Factors shall not be used.

8.3.1.1.3 CS3 Truck Loading

The "All Spans" Live Load Factors shown in CSA S6-14, Table 14.8 shall be used for span lengths greater than 15 metres. "Short Spans" Live Load Factors shall be used for spans up to 15 m and shall be based on the "All Spans" Live Load Factors shown in CSA S6-14, Table 14.8 modified as follows:

- For spans less than or equal to 10 m the "Short Spans" Live Load Factors shall be equal to the "All Spans" Live Load Factors multiplied by 1.10; and
- For spans varying between 10 m and 15 m the "Short Spans" Live Load Factors shall be equal to the "All Spans" Live Load Factors multiplied by a number varying linearly from 1.10 at a span length of 10 m to 1.00 at a span length of 15 m.

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Commentary 8.3.1.1

The provisions of Sections 8.3.1.1.1 to 8.3.1.1.3 of this Manual are intended to result in the 28 t CS1 truck, 49 t CS2 truck and 63.5 t CS3 truck having the same axle loads on "Short Spans".

8.3.1.2 Alternative Truck Loading

The "Short Spans" Live Load Factors shown in CSA S6-14, Table 14.9 for non-permit trucks (Alternative Loading) shall not be used. Instead, alternative "Short Spans" Live Load Factors shall be used for spans up to 26 m and shall be based on the "Other Spans" Live Load Factors shown in CSA S6-14, Table 14.9 modified as follows:

- For spans less than or equal to 6 m the "Short Spans" Live Load Factors shall be equal to the "Other Spans" Live Load Factors multiplied by 1.20; and
- For spans varying between 6 m and 26 m the "Short Spans" Live Load Factors shall be equal to the "Other Spans" Live Load Factors multiplied by a number varying linearly from 1.20 at a span length of 6 m to 1.00 at a span length of 26 m.

Commentary 8.3.1.2

Axle weights have a higher variation than gross vehicle weights. To account for this the "Short Spans" Live Load Factors given in CSA S6-14, Table 14.9 for non-permit trucks (Alternative Loading) are based on a bias coefficient that is 30% greater than that used to determine the "Other Spans" Live

Load Factors in Table 14.9. Note that the bias coefficient used to determine the "Short Spans" Live Load Factors is the ratio of the mean axle weights of actual trucks to the corresponding legal axle weights while the bias coefficient used to determine the "Other Spans" Live Load Factors is the ratio of the mean gross vehicle weights of actual trucks to the corresponding legal gross vehicle weights.

However, a truck weight survey carried out in Alberta in the early 1990s showed that for trucks in Alberta "Short Spans" Live Load Factors should be only 20% larger than "Other Spans" Live Load Factors" (Ramsay, 1997). This is consistent with the relationship between the "Short Spans" and "Other Spans" Live Load Factors shown in CAN/CSA S6-88 (CSA, 1988), Section 12, which was also based on Alberta truck weight data. Therefore, for non-permit trucks (Alternative Loading), this section of the Manual specifies "Short Spans" Live Load Factors that are a maximum of 20% larger than the "Other Spans" Live Load Factors shown in CSA S6-14, Table 14.9.

Also, the "Short Spans" Live Load Factors given in this section of the Manual provide a gradual transition between the "Short Spans" Live Load Factors and "Other Spans" Live Load Factors rather than a step transition as specified in CSA S6-14, Section 14.13.3.1.

The following methodology was used to calculate the "Short Spans" Live Load Factors given in this section of the Manual. If desired, this methodology can be adapted to determine "Short Spans" Live Load Factors for non-permit trucks (Alternative Loading) in lieu of using the "Short Spans" Live Load Factors provided in this section.

- *Increase one axle of a non-permit (Alternative Loading) truck by 20% and reduce an adjacent axle by the same amount;*

- *Determine the maximum shears and moments in simple span girders of varying lengths due to the above noted truck with the unbalanced axle loads;*

- *Determine the maximum shears and moments in simple span girders of varying lengths due to the same truck but without the unbalanced axle loads;*

- *Determine the ratios of the maximum shears and moments between the truck with unbalanced axle loads and the truck without unbalanced axle loads. Approximate the ratios with an equation that is either constant or varies linearly with span length. This equation is then used to convert the "Other Spans" Live Load Factors to "Short Spans" Live Load Factors.*

The "Short Spans" Live Load Factors are terminated at a span length where the difference between the "Short Spans" Live Load Factors and "Other Spans" Live Load Factors is less than approximately 5%.

8.3.2 <u>Permit Category PS Trucks</u>

The "Short Spans" Live Load Factors shown in CSA S6-14, Table 14.13 for Permit Category PS trucks shall not be used for Permit Category PS trucks. Instead, alternative "Short Spans" Live Load Factors shall be used for spans up to 18 m and shall be based on the "Other Spans" Live Load Factors shown in CSA S6-14, Table 14.13 modified as follows:

- For spans less than or equal to 6 m the "Short Spans" Live Load Factors shall be equal to the "Other Spans" Live Load Factors multiplied by 1.12; and
- For spans varying between 6 m and 18 m the "Short Spans" Live Load Factors shall be equal to the "Other Spans" Live Load Factors multiplied by a number varying linearly from 1.12 at a span length of 6 m to 1.00 at a span length of 18 m.

Commentary 8.3.2

Axle weights have a higher variation than gross vehicle weights. In the late 1980s, AT obtained scaled weights for a significant number of Permit Category PS trucks. These scaled weights included both axle and gross vehicle weights. Analysis of the scaled weights indicated that to achieve a consistent level of safety, live load factors for axle loads needed to be 12% higher than live load factors for truck loads. Therefore, for Permit Category PS trucks, this section of the Manual specifies "Short Spans" Live Load Factors that are a maximum of 12% larger than the "Other Spans" Live Load Factors shown in CSA S6-14, Table 14.13.

Also, the "Short Spans" Live Load Factors given in this section of the Manual provide a gradual transition between the "Short Spans" Live Load Factors and "Other Spans" Live Load Factors rather than a step transition as specified in CSA S6-14.

The following methodology was used to calculate the "Short Spans" Live Load Factors given in this section of the Manual. If desired, this methodology can be adapted to determine "Short Spans" Live Load Factors for Permit Category PS trucks in lieu of the "Short Spans" Live Load Factors provided in this section.

- *Increase one axle of a Permit Category PS truck by 12% and reduce an adjacent axle by the same amount;*

- *Determine the maximum shears and moments in simple span girders of varying lengths due to the above noted truck with the unbalanced axle loads;*

- *Determine the maximum shears and moments in simple span girders of varying lengths due to the same truck but without the unbalanced axle loads;*

- *Determine the ratios of the maximum shears and moments between the truck with unbalanced axle loads and the truck without unbalanced axle loads. Approximate the ratios with an equation that is either constant or varies linearly with span length. This equation is then used to convert the "Other Spans" Live Load Factors to "Short Spans" Live Load Factors.*

The "Short Spans" Live Load Factors are terminated at a span length where the difference between the "Short Spans" Live Load Factors and "Other Spans" Live Load Factors is less than approximately 5%.

8.3.3 Permit Category PA Trucks

The "Short Spans" Live Load Factors shown in CSA S6-14, Table 14.10 for Permit Category PA trucks shall not be used. Instead, the "Short Spans" Live Load Factors specified in Section 8.3.1.2 of this Manual shall be used for logging trucks and the "Short Span" Live Load Factors specified in Section 8.3.2 of this Manual shall be used for trucks carrying permanently mounted equipment.

Commentary 8.3.3

See Section 6.2.1 of this Manual for the rationale for the "Short Spans" Live Load Factors specified for Permit Category PA trucks.

8.3.4 Permit Category PC Trucks

The "Short Spans" Live Load Factors shown in CSA S6-14, Table 14.12 for Permit Category PC trucks shall not be used. Instead, alternative "Short Span" Live Load Factors shall be used for spans up to 18 m and shall be based on the "Other Spans" Live Load Factors shown in CSA S6-14, Table 14.12 multiplied by 1.02.

Commentary 8.3.4

"Short Spans" Live Load Factors given in this section of the Manual for Permit Category PC are specified as being 2% larger than the "Other Spans" Live Load Factors for spans up to 18 m based on the controls AT places on Permit Category PC trucks. These controls include verifying the weight and centre of gravity of the loads using load cells.

9.   Resistances

This Section 9 (Resistances) supplements CSA S6-14, Section 14.14 (Resistance). It provides information on the following for the most common bridge Span Types in AT's bridge inventory:

- The identification of bridge members that typically require load evaluation;
- The determination of the resistance of bridge member details not covered by CSA S6-14;
- The determination of the resistance of bridge members not meeting the requirements of CSA S6-14;
- The determination of the resistance of strengthened bridge members; and
- The determination of the resistance of damaged or deteriorated bridge members.

All resistances are determined at the ultimate limit state unless noted otherwise.

9.1   Steel Truss Bridges

9.1.1 General

The resistances of steel truss bridge members shall be determined in accordance with the requirements of CSA S6-14 and this Manual unless otherwise approved by AT. When combined effects occur simultaneously in the same member such that the resistance for one is affected by the magnitude of the other, the live load capacity factor shall be determined by successive iteration or other suitable method.

Commentary 9.1.1

Historically, AT has load evaluated steel truss bridges using Bridge Branch Inventory and Overload Stresses (see Appendix A). These load evaluation provisions are based on allowable stresses. The load evaluation results obtained from Bridge Branch Inventory Stresses and Bridge Branch Overload Stresses are generally consistent with the load evaluation results obtained from CSA S6-14, Section 14 for non-permit trucks and Permit – Single Trip (PS) trucks respectively, with the possible exception of the load evaluation results for the top chords of pony trusses.

While the existing posted load limits on AT's steel truss bridges are generally based on Bridge Branch Inventory Stresses, it is AT's intention that going forward steel truss bridges will be load evaluated on the basis of the ultimate limit state provisions of CSA S6-14.

9.1.2 Steel Truss Bridge Members Requiring Load Evaluation

Selection of the steel truss bridge members requiring load evaluation shall be based on engineering judgment.

The following steel truss bridge members will typically require load evaluation:

- Main truss members, e.g. top chords, bottom chords, batter posts, diagonals and verticals;
- Floorbeams;
- Stringers; and
- Timber decks where the stringer spacing is greater than 600 mm.

Commentary 9.1.2

The following steel truss bridge members will typically not require load evaluation:

- *Horizontal bracing members and their connections, provided that additional horizontal loads, such as increased wind load, do not need to be resisted by the bracing. This is typically the*

case when truck loads are increased, provided the bracing members are not being counted on to improve the lateral distribution of live load between trusses. If a truss is enclosed with hoarding to accommodate painting, or for any other reason, the truss will be subjected to increased wind loads due to the increased surface area exposed to the wind. In this case the horizontal bracing members and their connections need to be load evaluated to determine if they have sufficient resistance to resist the increased loads.

- *Connections between main truss members. Historically, truss member connections were designed to have resistances in excess of those of the truss members they connected. Therefore, it has traditionally been assumed that if the truss members have adequate resistances, the truss member connections also have adequate resistances. However, this assumption may not be appropriate if the truss member resistances as determined by CSA S6-14 and this Manual are significantly greater than those determined in the original design. An example of this is tension members whose resistances were originally based on the member's yield resistance at the net section but whose resistances can now be based on the member's ultimate resistance at the net section.*

- *Gusset plates. Note that even though truss member connection gusset plates will typically not require load evaluation, their capacities should be assessed qualitatively to confirm that they were adequately designed in the original design. In general, truss member connection gusset plates should be heavier at the ends of a truss than at midspan. This is because the larger shear forces at the ends of the truss place larger loads in the truss diagonals, verticals and batter posts at those locations. Guidance on the load evaluation of steel gusset plates can be found in FHWA documentation (Ibrahim, 2009).*

- *Concrete decks.*

- *Timber decks where the stringer spacing is less than 600 mm.*

9.1.3 <u>Section Properties of Steel Truss Bridge Rolled Sections No Longer Manufactured</u>

The properties of steel truss bridge rolled sections that are no longer manufactured can be obtained from the American Institute of Steel Construction Historic Shapes Database on the Internet (www.aisc.org).

9.1.4 <u>Resistance of Steel Truss Bridge Members</u>

9.1.4.1 <u>Resistance of Steel Stringers</u>

Steel stringers may be considered to be continuously supported along their compression flanges provided they support a concrete deck or are clipped to a timber deck. A timber deck may only be assumed to be clipped to its supporting steel stringers if the locations and good condition of the clips has been verified by inspection.

If the stringers support a timber deck but are not clipped to it, the bending resistance of the stringer shall be based on the actual lateral restraint provided between the stringer and the deck as determined by analysis.

Commentary 9.1.4.1

Steel stringers are loaded by truck wheel loads that are applied to the deck and then transferred in bearing to the top flanges of the stringers. The bearing force between a timber deck and steel stringer may create adequate friction between the deck and stringer to laterally support the stringer compression flange. Research on the adequacy of the friction connection between a timber deck and steel stringer to laterally support the stringer compression flange has been carried out at the University of Texas at Austin (Webb and Yura, 1992).

A concrete deck may be assumed to provide continuous lateral support to a stringer even if it was not designed to be composite with the stringer.

9.1.4.2 Resistance of Steel Truss Compression Members

Built-up steel compression members shall meet the detailing requirements of CSA S6-14, Sections 10.14.2 and 10.18.4. Lacing used to interconnect components of built-up steel compression members shall meet the requirements of CAN/CSA S16-14, Section 19.2 (CSA, 2014).

Except for the top chords of pony trusses, an effective length factor "K" of 0.75 may be used for determining the buckling resistances of steel truss compression members.

Commentary 9.1.4.2

The use of an effective length factor of 0.75 is based on historical AT practice and accounts for the rotational restraint provided by the truss gusset plates at the ends of the truss members.

9.1.4.3 Buckling Resistance of Pony Truss Top Chord

The elastic buckling resistance of a pony truss top chord shall be determined based on a buckling analysis of a three-dimensional model of the bridge. The model shall include the pony truss members and the floorbeams. All members of the three-dimensional model shall be rigidly connected together. The model shall not include the possible contribution of the bridge deck to the stiffness of the floorbeams. The position of the truck loading on the floorbeams shall be representative of the actual loading locations on the bridge and shall be positioned along the length of the bridge to minimize the elastic buckling resistance of the pony trusses.

The buckling resistance of the pony truss top chord shall be obtained using CSA S6-14, Section 10.9.3.1 where "λ" shall be taken to be equal to:

$$\lambda = \sqrt{\frac{P_y}{P_e}}$$

Where the symbols used in the equation are:

- P_y is the yield resistance of the pony truss top chord; and
- P_e is the elastic buckling resistance of the pony truss top chord.

Both P_y and P_e shall be obtained for the same top chord location. For a pony truss with a variable top chord member size, the section that results in the maximum value of "λ" shall be used for the calculation of the buckling resistance.

Commentary 9.1.4.3

The contribution of the bridge deck to the stiffness of the floorbeams is not accounted for in the determination of the buckling resistance of the pony truss top chords. This is because the additional restraint provided to the pony trusses by the composite floorbeams could overload the connections between the floorbeams and the trusses.

9.1.5 Resistance of Steel Truss Bridge Members Not Meeting the Requirements of CSA S6-14

9.1.5.1 Resistance of Steel Truss Compression Members

The resistances of steel truss compression members not meeting the width to thickness ratio requirements of CSA S6-14, Section 10.9.2, may be determined in accordance with CSA S6-14, Section 10.9.3.1 using one of the following modifications:

- Base the member cross-section area "A" on reduced flange and web widths that meet the width to thickness requirements of CAN/CSA S6-06, Section 10.9.2. The member radius of gyration "r" shall be based on the full flange and web widths; or
- Base the member yield stress "F_y" on a reduced yield stress that satisfies the flange and web width to thickness requirements of CSA S6-14, Section 10.9.2. The reduced member yield stress shall then be used in the calculation of both "λ" and "C_r".

If the yield stress is reduced for one component of a steel truss compression member it shall be reduced to the same level for all of the components of the truss member.

Commentary 9.1.5.1

The provisions of this section of the Manual are based on CAN/CSA S16-14, Section 13.3.5.

9.1.6 Resistance of Strengthened Steel Truss Bridge Members

9.1.6.1 Resistance of Strengthened Steel Truss Tension Members

The resistance of a strengthened steel truss tension member may be determined based on the assumption that the dead and live load stresses are uniformly distributed between the original and strengthening components of the member.

The adequacy of the connections at the ends of the strengthened member to transfer the strengthening component stresses back into the original component shall be verified.

Commentary 9.1.6.1

AT has historically strengthened truss bottom chord tension members by post-tensioning, although, strengthening by adding strengthening components to the bottom chord has also been carried out. Other truss tension members such as diagonals have historically been strengthened by adding strengthening components to the tension member.

Post-tensioning reduces the load effects applied to a member but does not increase the member's resistance. It, therefore, has no effect on the determination of the member's resistance.

Even though a strengthening component added to a truss tension member will not initially carry any of the load in the original component, it is assumed that the original component will be ductile enough to hold its yield stress until after the load in the original component has been uniformly distributed between the strengthening and original components.

Adding a strengthening component to a truss tension member can increase its resistance provided the strengthening component is extended far enough to connect to portions of the truss that do not require

strengthening. For a truss tension member, this typically requires that the strengthening component be adequately connected to the gusset plates at the ends of the strengthened member.

9.1.6.2 Resistance of Strengthened Steel Truss Compression Members

The determination of the buckling resistance of a strengthened steel truss compression member shall account for the dead and live load stresses not being uniformly distributed between the original and strengthening components of the member.

The compression resistance of a strengthened member may be conservatively determined by assuming that the initial stress in the strengthening component is equal to the stress in the original component just before strengthening and that subsequent stresses applied after strengthening are uniformly distributed between the original and strengthening components of the member.

Based on the above, the compression resistance "C_r" of the member available to resist the loads applied after the member is strengthened may be determined as follows:

$$C_r = C_{rs} - \frac{P_i \cdot A_{ss}}{A_s}$$

Where the symbols used in the equation are:

- C_{rs} is the calculated resistance of the member, determined in accordance with CSA S6-14, Section 10.9.3.1 and using the section properties of the strengthened member;
- P_i is the factored load on the member just before strengthening;
- A_{ss} is the cross-section area of the strengthened member; and
- A_s is the cross-section area of the original member.

If the strengthening component does not extend to the ends of a strengthened member, the member's buckling resistance shall be determined on the basis of the member being a stepped column.

The adequacy of the connections at the ends of the strengthening component to transfer the strengthening component stresses back into the original component shall be verified.

Commentary 9.1.6.2

AT has historically strengthened truss compression members such as top chords, batter posts and verticals by adding strengthening components to the compression member.

The steel compression member resistance determined in accordance with CSA S6-14, Section 10.9.3.1 is based on the assumption that the applied load creates a uniform stress in the member. However, a strengthening component added to a truss compression member does not initially carry load which results in a non-uniform stress distribution between the original and strengthening components of the member. Therefore, to satisfy the requirements of CSA S6-14, Section 10.9.3.1 it is conservative to assume that the strengthening component of the member is subjected to the same level of stress as the original component. The equation provided in this section of the Manual satisfies this assumption by assuming that the additional load carrying capacity of the compression member after strengthening is equal to the resistance of the strengthened compression member, determined in

accordance with CSA S6-14, Section 10.9.3.1, minus the stress in the original member just before strengthening (P_f/A_s) multiplied by the area of the strengthened member (A_{ss}).

Strengthening components need to be extended far enough to connect to portions of the truss that do not require strengthening. For a truss compression member, this typically requires that the strengthening component be extended at least to the gusset plates at the ends of the strengthened member.

9.1.7 Resistance of Damaged or Deteriorated Steel Truss Bridge Members

9.1.7.1 General

Steel truss bridge members are typically damaged by corrosion or by collision impact on the member.

Corrosion can result in a loss of member section and a reduction in ductility but will typically not result in a change in the member yield stress.

Collision impact can result in permanent member deformations as well as loss of member section due to tearing or cracking of the steel. It will typically not result in a change in the member yield stress provided that the member is not heat straightened more than twice at the same location. The extent of cracking should be established by non-destructive testing techniques such as dye penetrant or magnetic particle testing for the detection of surface cracks.

Commentary 9.1.7.1

Corrosion is primarily caused by de-icing salts and typically affects truss bridge members within the roadway splash zone such as stringers, floorbeams, bottom chord members and the lower portions of batter posts, verticals and diagonals.

There are three forms of corrosion that are commonly observed in steel bridges; uniform, pitting and crevice corrosion. Although uniform corrosion usually affects a large surface area it can also be localized around the splash zone. Uniform corrosion is usually found on flat vertical surfaces such as the webs of girders, gusset plates and truss members and can result in a significant loss of cross-section.

Pitting corrosion, which is common in the presence of chlorides, creates very localized damage and can even penetrate through the steel member, causing high stress raisers. Since pitting corrosion does not result in a significant loss of cross-section, it is not expected to have a significant effect on the ultimate strength of members.

Crevice corrosion is a form of corrosion that takes place in confined areas where the supply of oxygen is limited, such as in lap splices, along edge openings of built-up members or gusset plates, between back-to-back angles and at the contact surface between steel girders and concrete slabs. As corrosion products accumulate in crevices, the pressure builds up and can cause bulging of the plates and failure of the connecting rivets/bolts.

All forms of corrosion increase surface roughness and as a result cause a reduction in fatigue resistance. In addition, cyclic loading acting simultaneously with corrosion, results in corrosion fatigue which is worse than corrosion acting alone. Therefore, a bridge load evaluation may need to consider a bridge's fatigue capacity as well as its ultimate capacity if it is experiencing a large number of loading cycles combined with corrosion.

Collision impact damage is primarily caused by high or wide truck loads and can affect both primary load carrying and bracing members. It can also be caused by stream debris striking the bottom chord of a truss. Collision impact damage typically results in permanent deformation of the member but may also result in tearing/cracking of the member. While some cracks are easy to detect visually, others may be too small to detect and may propagate under load. Nicks and gouges in the damaged zone can create stress risers that can lead to cracking when the member is loaded or attempts are made to straighten the member. Such discontinuities should be ground smooth in accordance with AT requirements (Waheed et al., 2004) to avoid the initiation of cracks.

Permanent deformations in a truss member will increase the stresses in the member due to the eccentricities introduced by the deformations. Equations have been developed (Connor et al., 2008) to help predict the magnitude of these increased stresses in girder flanges based on the height and length of the deformation and the width and thickness of the girder flange. These equations should also be helpful in the prediction of the increased stresses in the deformed flanges of truss I section members.

If the permanent deformation consists of a localized bulge or crimp, the plastic strain can be sufficiently large enough to severely reduce the ductility of the member. Research (Avent and Mukai, 1998) indicates that strains up to 100 times the yield strain have no detrimental effect on the yield and tensile strength of the steel, but that some reduction in ductility can be expected.

However, if the deformation is caused by excessive heat, such as exposure to a fire, the above limit of plastic deformation is no longer applicable, In that case the material properties of the steel may have been affected and an assessment of the damage should be carried out based on metallurgical investigations, material testing and engineering judgment.

Research (Avent and Mukai, 1998 and Connor et al., 2008) indicates that a member can not be heat straightened more than twice at the same location without adversely affecting its strength and other properties. There is however a reduction in ductility of approximately 30% after the first heat straightening.

AT typically repairs or replaces damaged or deteriorated bridge members. However, damaged or deteriorated members may need to continue carrying reduced loads on a temporary basis until repairs or replacement can be carried out.

Steels used in trusses typically did not have minimum notch toughness requirements specified and could be susceptible to brittle fracture at high stresses and/or cold temperatures. Therefore, if these steels are to be highly stressed at low temperatures an investigation of the adequacy of their notch toughness should be considered.

9.1.7.2 Resistance of Damaged or Deteriorated Steel Truss Tension Members

The determination of the resistance of a damaged or deteriorated steel truss tension member shall account for loss of member section, the location of the damage or deterioration along the length of the member and any reduction in member ductility.

Element Behaviour Category E1 shall be used and the resistance of the steel truss tension member shall be limited to that of its yield resistance at the governing net section if the member has permanent deformations, section loss or corrosion (except for surface corrosion with negligible section loss).

The resistance of a steel truss tension member deformed by collision impact may be taken to be equal to the resistance of the undeformed member provided there are no localized bulges or crimps that induce plastic strains exceeding 100 times the yield strain of the steel.

Commentary 9.1.7.2

The resistance of a damaged or deteriorated steel truss tension member is affected by the location of the damage or deterioration. For example, damage or deterioration at or near the net section of a tension member can be expected to have a greater effect on member resistance than damage or deterioration at a gross section.

The eccentricity of the section loss in a steel tension member is assumed to not affect the capacity of the member. This is consistent with how the capacity of a tension member is determined at a net area location, i.e. the eccentricity of the member net area relative to the member gross area is ignored when determining the member capacity.

The resistance of a deformed steel truss tension member may be assumed to be equal to the resistance of the undeformed member based on the assumption that the member has adequate ductility to allow the member to deform back into its original position under load. However, if strains due to localized deformation induced curvatures exceed more than 100 times the yield strain, the yield strength of the member might not be attained prior to failure.

9.1.7.3 Resistance of Damaged or Deteriorated Steel Truss Compression Members

The determination of the resistance of a damaged or deteriorated steel truss compression member shall account for loss of member section, any eccentricity of the section loss relative to the neutral axis of the member, member deformations, the location of the damage or deterioration along the length of the member and any loss in member ductility.

Element Behaviour Category E1 shall be used if the member has permanent deformations, section loss or corrosion (except for surface corrosion with negligible section loss).

Commentary 9.1.7.3

The determination of the resistance of a damaged or deteriorated steel truss compression member is a complicated procedure and should only be undertaken with care and the use of conservative assumptions. As a minimum, the following should be considered:

- *The damaged or deteriorated portion of the member cross-section should be assumed to have been removed and to have no load carrying capacity;*

- *The yield stress of the member should be reduced or member component widths reduced in accordance with Section 9.1.5.1 of this Manual as required to meet the width to thickness ratio requirements of CSA S6-14, Section 10.9.2;*

- *The member should be treated as a beam-column with the determination of the eccentricity of the axial load on the member taking into account any member deformations, including any residual deformations remaining after heat straightening, as well as the shift in the neutral axis of the member resulting from the removal of the damaged or deteriorated portion of the member;*

- *The buckling resistance of the member should be determined assuming the member to be a stepped column with a reduced bending stiffness at the location of the damaged or deteriorated section; and*

- *The effects of member residual stresses, due to member deformations or subsequent straightening, on the member's buckling resistance can be assumed to be accounted for if the buckling resistance of the member is determined in accordance with CSA S6-14, Section 10.9.3.1 and n = 1.34.*

The resistance of a damaged or deteriorated steel truss compression member is affected by the location of the damage or deterioration. For example, damage or deterioration at mid-length of a compression member can be expected to have a greater effect on member buckling resistance than damage or deterioration at the ends of the member.

9.2   Steel Girder Bridges

9.2.1 General

The resistances of steel girder bridge members shall be determined in accordance with the requirements of CSA S6-14 and this Manual unless otherwise approved by AT. When combined effects occur simultaneously in the same member such that the resistance for one is affected by the magnitude of the other, the live load capacity factor shall be determined by successive iteration or other suitable method.

9.2.2 Steel Girder Bridge Members Requiring Load Evaluation

Selection of the steel girder bridge members requiring load evaluation shall be based on engineering judgment.

The following steel girder bridge members will typically require load evaluation:

- Main steel girders in moment and shear;
- Main steel girders in bearing if there are no bearing stiffeners;
- Field splices in steel girders when the applied loads at the field splices exceed 75% of the resistances of the main girders;
- Hangers in tension between main steel girder sections;
- Pins in shear between main steel girder sections;
- Bracing members at kinks in chorded main steel girders or in curved main steel girders;
- Floorbeams; and
- Stringers.

Commentary 9.2.2

The following steel girder bridge members will typically not require load evaluation:

- *Bracing members and their connections not at kinks in chorded main steel girders on in curved main steel girders, provided that additional horizontal loads (eg. increased wind load) do not need to be resisted by the bracing. This is typically the case when truck loads are increased provided the bracing members are not being counted on to improve the lateral distribution of live load between girders. They will however require load evaluation if the horizontal loads, e.g. wind loads, applied to the girders are increased. This can occur if the girders are enclosed with hoarding to accommodate painting, or for any other reason;*

- *Shear connection between the steel girders and concrete deck for composite girders. Historically, the shear connection has been designed to provide full composite action between the girder and deck at the ultimate limit state. Therefore, the shear connection may typically be assumed to be adequate in the load evaluation;*
- *Field splices in steel girder members when the applied loads at the field splices do not exceed 75% of the resistances of the main girder members. Historically, girder field splices have been designed to have resistances of at least 75% of the girder resistance at the location of the splice. Therefore, the field splices are typically assumed to have adequate resistances provided the applied loads at the splices are not greater than 75% of the girder resistances at the splice locations; and*
- *Main steel girders in bearing if there are bearing stiffeners. Steel girders with bearing stiffeners typically have bearing resistances well in excess of the applied loads and can, therefore, be assumed to be adequate in the load evaluation.*

9.2.3 Serviceability Limit State

Bending stresses in steel girders shall not exceed the yield stress for CSA S6-14 Serviceability Limit State Combination 1.

Commentary 9.2.3

Bending stresses in steel girders are to remain below the yield stress at the serviceability limit state to prevent permanent deformations in the girders.

9.2.4 Section Properties of Steel Girders

9.2.4.1 Analysis

The section properties used for the analysis of steel girders shall be based on composite or non-composite behaviour with the deck depending on whether or not the girders were originally designed to be composite or non-composite with the deck for the loading under consideration.

Commentary 9.2.4.1

If a load evaluation is carried out using girder stiffnesses different from those assumed in the original design, e.g. deck and girder assumed to be composite along the entire length of the girder, a different distribution of moments along the length of the girder will be obtained. This different distribution of moments will increase the likelihood that the girder will be found to have inadequate resistance particularly near the points of contraflexure.

Prior to the late 1950s, it was AT practice to analyze and design girders as non-composite girders. However starting in the late 1950s, AT began to analyze and design girders as composite in the positive moment regions and as non-composite in the negative moment regions. This practice was based on a reluctance to weld studs to the tension top flange in the negative moment regions. However, starting in approximately 1990, AT began to analyze and design girders as composite girders in both the positive and negative moment regions. In the negative moment regions, the girders were assumed to be composite with the deck reinforcement only.

9.2.4.2 <u>Design</u>

The resistances determined for steel girders shall be based on composite or non-composite behaviour with the deck depending on whether or not the girders were originally designed to be composite or non-composite with the deck.

9.2.5 <u>Resistance of Steel Girder Bridge Members Not Meeting the Requirements of CSA S6-14</u>

9.2.5.1 <u>Flanges and Webs</u>

The resistances of steel girder flanges and webs in compression and not meeting the width to thickness ratio requirements of CSA S6-14, Section 10.9.2 may be determined using one of the following procedures:

- Reducing the yield stress of the steel member to a level at which the width to thickness ratio requirements are met; or
- Reducing the widths of the girder flanges in accordance with the requirements of CSA S6-14, Section 10.10.3.4 if the flanges do not meet the requirements of CSA S6-14, Section 10.9.2 and reducing the moment capacity of the girder in accordance with CSA S6-14, Section 10.10.4.3 if the web does not meet the requirements of CSA S6-14, Section 10.9.2.

If the yield stress is reduced for one component of the girder it needs to be reduced to the same level for all of the girder components except as noted otherwise in Section 9.2.5.3 of this Manual.

9.2.5.2 <u>Longitudinal Stiffeners</u>

The required section properties of longitudinal stiffeners not meeting the requirements of CSA S6-14, Section 10.10.7.2 may be met by using one of the following procedures:

- Reducing the stiffener width to a level at which the width to thickness ratio requirements are met and checking to see if the remaining section property requirements are still met;
- Checking the stiffener width to thickness ratio requirement using the factored applied stress in the stiffener rather than its yield strength; or
- A combination of the above.

Commentary 9.2.5.2

Typically, the full width of the longitudinal stiffener (i.e. width unreduced due to width to thickness ratio requirements) will be required for it to meet its required geometric properties.

A longitudinal stiffener will typically be subjected to a girder bending stress that is considerably less than its yield stress. Therefore, the stiffener width to thickness requirements may be checked using the factored applied stress in the stiffener at the ultimate limit state rather than the yield stress of the stiffener.

9.2.5.3 <u>Transverse Stiffeners</u>

9.2.5.3.1 <u>Width to Thickness Requirements</u>

The width to thickness ratio requirements of single-sided transverse stiffeners shall be determined in accordance with CSA S6-14, Section 14.14.1.8. If the width to thickness ratio requirements are not met a reduced yield stress, of not less than 248 MPa, may be used to satisfy the width to thickness ratio requirements.

Reducing the yield stress of the transverse stiffener does not require that the yield stress of the other girder components also be reduced.

Commentary 9.2.5.3.1

A minimum stress is specified for the transverse stiffener yield stress as transverse stiffeners need to have an undefined minimum strength in order for them to provide the assumed pinned support boundary conditions at the edges of the web panel. A minimum yield stress of 248 MPa is specified as historically transverse stiffeners with yield stresses as low as 248 MPa (A36 steel) have been used.

9.2.5.3.2 Stiffness Requirements

The required section properties of transverse stiffeners not meeting the geometric requirements of CSA S6-14, Sections 10.10.6.2 (a) and 10.10.7.3 may be determined based on an increased stiffener spacing that satisfies the stiffener section geometric property requirements. The shear resistance of the girder shall then be based on the increased stiffener spacing.

Commentary 9.2.5.3.2

In accordance with CSA S6-14 transverse stiffeners are to be designed to have adequate stiffness to maintain the required boundary conditions around the girder web panels, i. e. the stiffener must maintain a vertical line of near zero lateral deflection along the line of the stiffener, until after the web panels have achieved their ultimate shear resistance. In plate girders with web panel aspect ratios, a/h, less than 1.0 much stiffer transverse stiffeners are required to develop the increased shear buckling resistance of the panel. If the geometric properties of the transverse stiffener do not meet the requirements for the actual girder transverse stiffener spacing they may still be adequate for a greater transverse stiffener spacing. This is because the girder's shear resistance and, therefore, the demands on the transverse stiffener decreases with larger transverse stiffener spacings. Therefore, the girder shear resistance based on the assumed greater transverse stiffener spacing, for which the stiffness of the transverse stiffeners is adequate, represents a conservative estimate of the girder's shear resistance based on the actual transverse stiffener spacing and geometric properties.

9.2.5.3.3 Area Requirements

The required area of a transverse stiffener shall be determined in accordance with the requirements of CSA S6-14, Section 10.10.6.2 (b) with V_f/V_r equal to 1.0. If a reduced yield stress and/or increased stiffener spacing was used to meet the transverse stiffener section geometric property requirements, in accordance with Sections 9.2.5.3.1 and 9.2.5.3.2 of this Manual, the reduced yield stress and/or increased stiffener spacing shall be used in the determination of the required stiffener area. If the required stiffener area is greater than the provided area, the tension field component of the shear resistance F_t, determined in accordance with CSA S6-14, Section 10.10.5.1 shall be reduced by the ratio of:

$$Ratio = \frac{\left(A_p/Y + 18w^2\right)}{\left(A_s/Y + 18w^2\right)}$$

Where the symbols used in the equation are:

- A_p is the provided area of the transverse stiffener;
- A_s is the required area of the transverse stiffener;

- w is the girder web thickness; and
- Y is the ratio of the yield stress of the web steel to the yield stress of the transverse stiffener steel.

Commentary 9.2.5.3.3

In the above equation the term "$A_p/Y + 18w^2$" is the area provided by the actual transverse stiffener and associated web while the term "$A_s/Y + 18w^2$" is the required area of the transverse stiffener and associated web. The second term is calculated from the equation for "A_s" in CSA S6-14, Section 10.10.6.2 (b) if the terms "Y" and "$18w^2$" are transferred to the left side of the equation. The ratio of the two terms gives the percentage by which the transverse stiffener is unable to fully develop the shear resistance of the web and therefore the shear resistance of the web needs to be reduced by the ratio of the two terms.

9.2.6 <u>Resistance of Strengthened Steel Girder Bridge Members</u>

9.2.6.1 <u>Resistance of Strengthened Steel Girder Tension Flanges</u>

Unless otherwise approved by AT, the resistance of a strengthened steel girder tension flange shall be determined based on the assumption that the loads applied prior to strengthening are resisted by the original flange only and that the loads applied after strengthening are resisted by both the original flange and the strengthening component.

The adequacy of the connections at the ends of the strengthening component to transfer the strengthening component stresses back into the original girder flange shall be verified.

Commentary 9.2.6.1

AT has historically strengthened steel girder tension flanges by post-tensioning. Post-tensioning reduces the load effects applied to the girder tension flange but does not increase its resistance. Post-tensioning can also increase the load effects on the girder compression flange and this needs to be taken into account when determining the adequacy of the compression flange.

Adding a strengthening component to a tension flange can increase its resistance provided the strengthening component is extended far enough to connect to portions of the flange that do not require strengthening.

The requirements of this section assume that the steel girders in AT's bridge inventory do not have adequate ductility to allow for redistribution of stress between the tension flange and strengthening component.

9.2.6.2 <u>Resistance of Strengthened Steel Girder Compression Flanges</u>

The determination of the lateral torsional buckling resistance of a strengthened steel girder compression flange shall account for the dead and live load stresses not being uniformly distributed between the original flange and strengthening component.

The buckling resistance of a strengthened steel girder compression flange may be conservatively determined by assuming that the initial stress in the strengthening component is equal to the stress in the original flange just before strengthening and that subsequent stresses applied after strengthening are uniformly distributed between the original flange and strengthening component.

Based on the above the moment resistance "M_r" of the girder available to resist the loads applied after the girder is strengthened may be determined as follows:

$$M_r = M_{rs} - \frac{M_i \cdot S_{ss}}{S_s}$$

Where the symbols used in the equation are:

- M_{rs} is the calculated resistance of the girder, determined in accordance with CSA S6-14, Sections 10.10.3 or 10.10.4 and using the section properties of the strengthened girder;
- M_i is the factored moment on the member just before strengthening;
- S_{ss} is the section modulus of the strengthened girder; and
- S_s is the section modulus of the original girder.

The adequacy of the connections at the ends of the strengthening component to transfer the strengthening component stresses back into the original flange shall be verified.

Commentary 9.2.6.2

AT has historically strengthened steel girder compression flanges by adding strengthening components to the compression flange or by reducing the unsupported length of the flange. The strengthening component added to the compression flange typically affects both the flange's buckling resistance and its load carrying cross-section.

The steel girder compression flange resistance determined in accordance with CSA S6-14, Sections 10.10.3 or 10.10.4 is based on the assumption that the applied load results in a linear varying stress in the girder cross-section. However, a strengthening component added to a girder compression flange does not initially carry load which results in a non-uniform stress distribution between the original flange and strengthening component. Therefore, to satisfy the requirements of CSA S6-14 it is conservative to assume that the strengthening component is subjected to the same level of stress as the original compression flange. The equation provided in this section of the Manual satisfies this assumption by assuming that the additional load carrying capacity of the girder after strengthening is equal to the resistance of the strengthened girder, determined in accordance with CSA S6-14, Sections 10.10.3 or 10.10.4, minus the stress in the original girder compression flange just before strengthening (M_i/S_s) multiplied by the section modulus of the strengthened girder (S_{ss}).

Strengthening components need to be extended far enough to connect to portions of the compression flange that do not require strengthening. Also, if the strengthening component does not extend to locations where restraint against lateral-torsional buckling is provided, the compression flange's buckling resistance will need to be determined on the basis of the girder being a non-prismatic or stepped member.

9.2.6.3 Resistance of Strengthened Steel Girder Webs in Shear

Transverse stiffeners added to strengthen a steel girder web in shear may be assumed to carry full dead and live loads.

Commentary 9.2.6.3

Transverse stiffeners added to strengthen a steel girder web in shear may be assumed to carry full dead and live load as their function is to provide the proper boundary conditions to develop the web buckling resistance and anchor the tension field that develops after web buckling. Both of these functions can typically be assumed to occur at loads higher than the dead loads applied to the girder prior to addition of the transverse stiffeners.

9.2.7 Resistance of Damaged or Deteriorated Steel Girder Bridge Members

9.2.7.1 General

Steel girder bridge members are typically damaged by corrosion or by collision impact on the member.

Corrosion can result in a loss of member section and a reduction in ductility but will typically not result in a change in the member yield stress.

Collision impact can result in permanent member deformations as well as loss of member section due to tearing or cracking of the steel. It will typically not result in a change to the yield stress provided that the member is not heat straightened more than twice at the same location. The extent of cracking should be established by non-destructive testing techniques such as dye penetrant or magnetic particle testing for the detection of surface cracks.

Commentary 9.2.7.1

Corrosion is primarily caused by de-icing salts and typically affects girders adjacent to deck joints. See Section 9.1.7.1 of this Manual for a further discussion on the types of corrosion that can occur in steel bridges.

All forms of corrosion increase surface roughness and as a result cause a reduction in fatigue resistance. In addition, cyclic loading acting simultaneously with corrosion results in corrosion fatigue which is worse than corrosion acting alone. Therefore, a bridge load evaluation may need to consider a bridge's fatigue capacity as well as its ultimate capacity if it is experiencing a large number of loading cycles combined with corrosion.

Collision impact damage is primarily caused by high or wide truck loads and can affect both primary load carrying and bracing members. It can also be caused by stream debris striking the bottom flange of a girder. Collision impact damage typically results in permanent deformation of the member but may also result in tearing/cracking of the member. While some cracks are easy to detect visually, others may be too small to detect and may propagate under load. Nicks and gouges in the damaged zone can create stress risers that can lead to cracking when the member is loaded or attempts are made to straighten the member. Such discontinuities should be ground smooth in accordance with AT requirements (Waheed et al., 2004) to avoid the initiation of cracks.

Permanent deformations in a girder member will increase the stresses in the member due to the eccentricities introduced by the deformations. Equations have been developed (Connor et al., 2008) to help predict the magnitude of these increased stresses in girder flanges based on the height and length of the deformation and the width and thickness of the girder flange.

If the permanent deformation consists of a localized bulge or crimp, the plastic strain can be sufficiently large enough to severely reduce the ductility of the member. Research (Avent and Mukai, 1998)

indicates that strains up to 100 times the yield strain have no detrimental effect on the yield and tensile strength of the steel, but that some reduction in ductility can be expected.

However, if the deformation is caused by excessive heat, such as exposure to a fire, the above limit of plastic deformation is no longer applicable, In that case the material properties of the steel may have been affected and an assessment of the damage should be carried out based on metallurgical investigations, material testing and engineering judgment.

Research (Avent and Mukai, 1998 and Connor et al., 2008) indicates that a member can not be heat straightened more than twice at the same location without adversely affecting its strength and other properties. There is however a reduction in ductility of approximately 30% after the first heat straightening.

AT typically repairs or replaces damaged or deteriorated bridge members. However, damaged or deteriorated members may need to continue carrying reduced loads on a temporary basis until repairs or replacement can be carried out.

Girder steels used by AT prior to the 1970s typically did not have minimum notch toughness requirements specified and could be susceptible to brittle fracture at high stresses and/or cold temperatures. Therefore, if these steel girders are to be highly stressed at low temperatures an investigation of the adequacy of their notch toughness should be considered.

9.2.7.2 <u>Resistance of Damaged or Deteriorated Steel Girder Tension Flanges</u>

The determination of the resistance of a damaged or deteriorated steel girder tension flange and adjacent web shall account for loss of member section, any eccentricity of the section loss relative to the neutral axis of the girder, the location of the damage or deterioration along the length of the girder and any reduction in member ductility.

Element Behaviour Category E1 shall be used and the resistance of the girder limited to that of a Class 3 section, i.e. girder resistance shall be limited to the yield resistance of the girder, if the girder has permanent deformations, section loss or corrosion (except for surface corrosion with negligible section loss).

The resistance of a steel girder tension flange and adjacent web deformed by collision impact but without section loss may be taken to be equal to the resistance of the undeformed girder provided there are no localized bulges or crimps that induce plastic strains exceeding 100 times the yield strain of the steel.

Commentary 9.2.7.2

> *The resistance of a damaged or deteriorated steel girder tension flange and adjacent web is affected by the location of the damage or deterioration. For example, damage or deterioration of the tension flange at mid-span can be expected to have a greater effect on girder resistance than damage or deterioration near an abutment.*

> *The resistance of a deformed steel girder tension flange and adjacent web without section loss may be assumed to be equal to the resistance of the undeformed girder based on the assumption that the girder has adequate ductility to allow the member to deform back into its original position under load. However, if strains due to localized deformation induced curvatures exceed 100 times the yield strain, the yield strength of the girder might not be attained prior to failure.*

9.2.7.3 Resistance of Damaged or Deteriorated Steel Girder Compression Flanges

The determination of the resistance of a damaged or deteriorated steel girder compression flange and adjacent web shall account for loss of member section, any eccentricity of the section loss relative to the neutral axis of the girder, girder deformations, the location of the damage or deterioration along the length of the girder and any reduction in member ductility.

Element Behaviour Category E1 shall be used and the resistance of the girder limited to that of a Class 3 section, i.e. girder resistance shall be limited to the yield resistance of the girder, if the girder has permanent deformations, section loss or corrosion (except for surface corrosion with negligible section loss).

Commentary 9.2.7.3

The determination of the resistance of a damaged or deteriorated steel girder compression flange and adjacent web is a complicated procedure and should only be undertaken with care and the use of conservative assumptions. As a minimum the following should be considered:

- *The damaged or deteriorated portion of the compression flange and adjacent web cross-section should be assumed to have been removed and to have no load carrying capacity;*

- *The yield stress of the girder should be reduced or member component widths reduced in accordance with Section 9.2.5 of this Manual as required to meet the width to thickness ratio requirements of CSA S6-14, Section 10.9.2;*

- *The eccentricity of the compression load on the flange and adjacent web should take into account any girder deformations, including any residual deformations after heat straightening, as well as the shift in the neutral axis of the girder resulting from the removal of the damaged or deteriorated portion of the girder;*

- *The lateral-torsional buckling resistance of the girder should be determined assuming the girder to have a non-uniform stiffness along its length with reduced stiffness at the location of the damaged or deteriorated section; and*

- *The effects of girder residual stresses due to girder deformations, or subsequent straightening, on the girder's lateral-torsional buckling resistance can be assumed to be accounted for if the buckling resistance is determined in accordance with CSA S6-14, Sections 10.10.3 or 10.10.4.*

The resistance of a damaged or deteriorated steel girder compression flange and adjacent web is affected by the location of the damage or deterioration. For example, damage or deterioration of the compression flange at mid-span of a girder or at a pier can be expected to have a greater effect on girder buckling resistance than damage or deterioration near an abutment.

9.2.7.4 Resistance of Damaged or Deteriorated Steel Girder Webs and Stiffeners

The determination of the resistance of a damaged or deteriorated steel girder web and stiffeners shall account for loss of member section, any eccentricity of the section loss relative to the neutral axis of the girder, member deformations, and a reduction in member ductility.

Element Behaviour Category E1 shall be used if the girder has permanent deformations, section loss or corrosion (except for surface corrosion with negligible section loss).

If the web is permanently deformed by collision impact, the classical buckling component of the web shear resistance F_{cr}, determined in accordance with CSA S6-14, Section 10.10.5.1, shall be taken equal to zero.

Tension field shear resistance shall account for section loss in the web, stiffeners and flanges. It shall also account for the effects of flange/stiffener section loss eccentricity and out of plane deformations on tension field shear resistance. In-plane deformations need not be considered but could lead to increased girder deflections.

The adequacy of a damaged or deteriorated girder web to resist combined moment and shear shall be checked in accordance with CSA S6-14, Section 10.10.5.2 using reduced shear and moment resistances that account for girder damage and deterioration.

Commentary 9.2.7.4

> *Collision impacts severe enough to deform the girder web will likely deform the girder bottom flange as well. In this case the deformations are assumed to be severe enough to have compromised the classical buckling component "F_{cr}" of the web shear resistance.*
>
> *The tension field component "F_t" of the shear resistance can be reduced by web section loss, but will typically not be reduced by eccentricity of the web section loss or by web deformations.*
>
> *The tension field component "F_t" of the shear resistance can be reduced if the girder stiffeners or girder flanges, anchoring the web panel under consideration, undergo section loss or out of plane deformations. The section loss, including any eccentricity of the section loss, and/or deformations shall be considered when determining the ability of the girder stiffeners and girder flanges, acting as steel compression members (see Section 9.1.7.3 of this Manual), to anchor the vertical and horizontal components of the tension field respectively.*
>
> *The tension field component "F_t" of the shear resistance will typically not be reduced if the girder stiffeners or girder flanges, anchoring the web panel under consideration, undergo in plane deformations only. However, girder deflections may increase prior to development of the ultimate shear resistance due to increased shear deformations in the web panel.*

9.3   Reinforced Concrete Bridges

9.3.1 General

The resistances of reinforced concrete bridge members shall be determined in accordance with the requirements of CSA S6-14 and this Manual unless otherwise approved by AT. When combined effects occur simultaneously in the same member such that the resistance for one is affected by the magnitude of the other, the live load capacity factor shall be determined by successive iteration or other suitable method.

9.3.2 Reinforced Concrete Bridge Members Requiring Load Evaluation

Selection of the reinforced concrete bridge members requiring load evaluation shall be based on engineering judgment.

The following reinforced concrete bridge members will typically require load evaluation:

- Main reinforced concrete girders in moment and shear;
- Reinforced concrete slabs in moment and shear in the primary load carrying direction(s);

- End anchorage of longitudinal reinforcement at supports;
- Bridge deck cantilevers in moment; and
- Bridge pier cap cantilevers in moment and shear.

Commentary 9.3.2

The following reinforced concrete bridge members will typically not require load evaluation:

- *Bracing members and connections not at kinks in chorded main reinforced concrete girders or in main reinforced concrete curved girders. These bracing members typically do not require load evaluation provided they are not being asked to carry increased loads. This is typically the case when truck loads are increased provided the bracing members are not being counted on to improve the lateral distribution of live load between girders;*
- *Transverse reinforcement perpendicular to the primary load carrying direction in reinforced concrete slabs acting in one-way moment and shear. The design of transverse reinforcement in one-way reinforced concrete slabs has historically been based on the requirement that the transverse reinforcement be a minimum percentage of the primary load carrying reinforcement. Therefore, the transverse reinforcement can typically be assumed to be adequate provided that the primary load carrying reinforcement is adequate;*
- *Vertical substructure members not susceptible to buckling. These members (e.g. concrete pier shafts and columns) typically have higher load carrying capacities and higher dead to live load ratios than the primary load carrying superstructure members; and*
- *Horizontal substructure members except for cantilevers. These members (e.g. concrete pier caps, with span to depth ratios of less than 3) typically have higher load carrying capacities and higher dead to live load ratios than the primary load carrying superstructure members.*

9.3.3 <u>Serviceability Limit State</u>

Tension stresses in the deck reinforcing steel due to negative moments shall be checked at CSA S6-14 Serviceability Limit State Combination 1. The results shall be reported in the bridge load evaluation report. However, they need not be considered in the determination of the bridge's load carrying capacity.

Commentary 9.3.3

The tension stresses in the deck reinforcing steel due to negative moments should remain well below the yield stress of the reinforcing steel at the serviceability limit state to control deck crack widths. However, while excessive crack widths can reduce the service life of a bridge deck they will not reduce the bridge's load carrying capacity provided the bridge deck remains in good condition. Therefore, they are not considered in the determination of the bridge's load carrying capacity.

9.3.4 <u>Resistance of Reinforced Concrete Bridge Girder and Slab Members</u>

9.3.4.1 <u>Longitudinal Reinforcing Steel in Girders and Slabs</u>

The moment resistances determined for reinforced concrete girders and slabs shall be based on the assumption that the end of any longitudinal reinforcing steel without a standard hook and contributing to the moment resistance of the girder is terminated "$d_v \cot\theta$" short of its actual termination points.

Commentary 9.3.4.1

Since the cut-off points for longitudinal steel reinforcement need to be extended past the points at which they are no longer required in new design in accordance with CSA S6-14, Section 8.9.3.10, these extensions should be ignored when determining the adequacy of the longitudinal reinforcing steel for moment during a load evaluation.

This requirement is similar to the recommendation made by MacGregor (MacGregor, 1987) for determining the shear resistance of reinforced concrete girder bridges.

9.3.4.2 Transverse Shear Reinforcing Steel in Girders and Slabs

The transverse shear reinforcing steel available to resist an applied shear force at a section may be taken to be the transverse shear reinforcing steel located over a girder length of "$d_v \cot\theta$" and centred on the section being considered.

Commentary 9.3.4.2

This requirement is a refinement of CSA S6-14, Section 8.9.3.9.

9.3.4.3 Variable Depth Girders and Slabs

In accordance with CSA S6-14, Section 8.9.1.7, for variable depth girders and slabs, the effects of the inclined bending forces on the magnitudes of the applied shears shall be considered in the bridge load evaluation.

Commentary 9.3.4.3

Variations in the depth of girders and slabs will either increase or decrease the applied shears on the girders/slabs.

9.3.4.4 Slabs Without Transverse Reinforcing Steel

For slabs without transverse reinforcing steel, the live load beam shear applied to a 1 metre width of slab shall be determined in accordance with Section 6.9 of this Manual rather than in accordance with CSA S6-14, Section 8.9.4.1, unless otherwise approved by AT on a bridge specific basis.

Commentary 9.3.4.4

CSA S6-14, Section 8.9.4.1 specifies that the entire width of a slab may be assumed to resist beam shear. However, if shear failure occurs in a brittle manner the slab may not have adequate ductility to redistribute the live loads across the entire width of the slab prior to failure of the most highly stressed portions of the slab. While thinner slabs without transverse reinforcing steel are expected to fail in a more ductile manner than thicker slabs without transverse reinforcing steel approval from AT is required before assuming that the entire width of a slab is effective in resisting beam shear.

9.3.5 Resistance of Reinforced Concrete Bridge Members Not Meeting the Requirements of CSA S6-14

9.3.5.1 Transverse Shear Reinforcing Steel in Girders and Slabs

If the transverse shear reinforcing steel ratio is less than that allowed by CSA S6-14, Section 8.9.1.3 or if the spacing of the transverse shear reinforcing steel is greater than that allowed by CSA S6-14, Section 8.14.6 the shear resistance of the member shall be determined in accordance with CSA S6-14, Section 14.14.1.6.

9.3.6 Resistance of Strengthened Reinforced Concrete Bridge Girder and Slab Members

9.3.6.1 Resistance of Strengthened Reinforced Concrete Bridge Girder and Slab Members in Moment

The resistance of a strengthened reinforced concrete bridge girder or slab member in moment may be determined based on the assumption that the dead and live load stresses are uniformly distributed between the original and strengthening components provided that the longitudinal steel reinforcement ratio after strengthening meets the requirements of CSA S6-14, Sections 8.8.4.3 and 8.8.4.5.

The adequacy of the connections at the ends of the strengthening component to transfer the strengthening component stresses back into the original component shall be confirmed.

Commentary 9.3.6.1

AT has historically strengthened reinforced concrete bridge girder and slab members in moment by adding strengthening components, such as steel plates, to the members. The strengthening component needs to be extended far enough to connect to portions of the member that do not require strengthening. The strengthening component may be assumed to carry both dead load and live load as the member is assumed to have adequate ductility to allow stresses to be redistributed between the original and strengthening components.

9.3.6.2 Resistance of Strengthened Reinforced Concrete Bridge Girder Members in Shear

External stirrups added to strengthen a reinforced concrete bridge girder in shear may be assumed to carry full dead and live loads provided that the external stirrup spacing is less than "$0.5 \, d_v \cot\theta$" at the location of the stirrups, where "d_v" is the effective shear depth of the girder as defined by CSA S6-14 and "θ" is the angle of inclination of the shear crack relative to the horizontal as determined in accordance with CSA S6-14, Section 8.9.3.7, If the external stirrup spacing exceeds "$0.5 \, d_v \cot\theta$" any contribution of the external stirrups to the girder's shear resistance shall be ignored.

Commentary 9.3.6.2

AT has historically strengthened reinforced concrete bridge girders in shear by adding external stirrups to the girders. The maximum external stirrup spacing has historically been limited to 75% of the girder effective depth based on recommendations made by MacGregor (MacGregor, 1987) for reinforced concrete girders. This recommendation was based on observations that the critical shear crack generally extended a distance of at least 1.5 times the girder depth along the length of the girder. This section of the Manual is a refinement of MacGregor's recommendation. If the external stirrup spacing

exceeds "0.5 d_v cotθ" there is an increased possibility that the critical girder shear failure mechanism can occur between the external stirrups.

Since shear cracks are typically not expected to occur in a girder web under service loads and since the internal transverse shear reinforcing steel is not highly stressed until after shear cracks have formed, the external stirrups may be assumed to resist both dead load and live load shears along with the internal transverse shear reinforcing steel.

9.3.7 Resistance of Damaged or Deteriorated Reinforced Concrete Bridge Girder and Slab Members

9.3.7.1 General

Reinforced concrete bridge girder and slab members are typically damaged by corrosion of the reinforcing steel or by cracking and spalling of the concrete due to either corrosion of the reinforcing steel or collision impact.

Corrosion of the reinforcing steel can result in section loss but will typically not result in a change in the reinforcing steel yield stress. It can, however, result in a reduction in reinforcing steel ductility.

Collision impact on the reinforcing steel can result in section loss due to nicking or rupture of the steel.

Cracking of the concrete will result in a loss of tension resistance across the cracks and can cause a loss of interface shear transfer across the cracks and compression resistance parallel to the cracks. It can also result in a loss of reinforcing steel anchorage.

Spalling of the concrete can result in a loss of concrete compression and tension resistance at the spall location. It can also result in a loss of reinforcing steel anchorage. Spalled concrete includes loose or fractured concrete adjacent to the spall that has not yet fallen away.

Commentary 9.3.7.1

Corrosion of the reinforcing steel is primarily caused by de-icing salts. This corrosion not only results in section loss and a reduction in ductility of the reinforcing steel but also results in cracking and spalling of the concrete which affects the anchorage of the reinforcing steel.

Collision impacts on reinforced concrete bridge members are typically caused by high loads. These impacts can damage or completely sever the reinforcing steel as well as crack and spall the concrete around the reinforcing steel.

9.3.7.2 Resistance of Damaged or Deteriorated Reinforced Concrete Bridge Girder and Slab Members in Moment

The determination of the moment resistance of a damaged or deteriorated reinforced concrete bridge girder or slab member shall account for section loss, a reduction in reinforcing steel ductility and loss of anchorage of the reinforcing steel. It shall also account for loss of concrete area due to spalling.

If the main longitudinal reinforcing steel of a reinforced concrete bridge girder or slab member is corroded and the surrounding concrete cracked and/or spalled the Element Behaviour Category shall be taken as being either Category E1 or E2 in the determination of the moment resistance of the member. The Element Behaviour Category used shall depend on the location and severity of the corrosion, cracking and/or spalling and its effect on the anchorage of the reinforcing steel.

Commentary 9.3.7.2

Section loss of the main longitudinal reinforcing steel due to corrosion is typically not great enough to cause a significant decrease in member resistance, but this should be verified on a bridge specific basis. More significant is the reduction in ductility and reduced anchorage of the longitudinal reinforcing steel that results from the concrete cracking and spalling. Unless otherwise verified on a bridge specific basis, reinforcing steel lengths where the concrete surrounding the reinforcing steel bar is cracked and/or spalled should be assumed to be incapable of providing anchorage to the bar unless the bar is directly above a bearing support and the only damage/deterioration is hairline cracking.

Concrete spalling can reduce the compression resistance of a reinforced concrete bridge girder or slab member in moment although this loss in resistance is seldom critical.

Loss of anchorage of the reinforcing steel will be most critical if the loss of anchorage occurs in its development length near the end of a reinforcing steel bar. Therefore, Element Behaviour Category E1 should be used when the loss of anchorage occurs near the end of a reinforcing steel bar.

9.3.7.3 Resistance of Damaged or Deteriorated Reinforced Concrete Bridge Girder and Slab Members in Shear

The determination of the shear resistance of a damaged or deteriorated reinforced concrete bridge girder or slab member shall account for section loss, loss of anchorage and a reduction in ductility of the reinforcing steel. It shall also account for loss of concrete shear strength due to cracking and spalling.

If the transverse shear reinforcing steel of a reinforced concrete bridge girder or slab member is corroded and the surrounding concrete cracked and/or spalled the Element Behaviour Category shall be taken as being Category E1 in the determination of the shear resistance of the member.

If the concrete surrounding the longitudinal reinforcing steel contributing to the shear resistance of a reinforced concrete bridge girder or slab member is cracked and/or spalled, the Element Behaviour Category shall be taken as being Category E1 in the determination of the shear resistance of the member.

Commentary 9.3.7.3

Section loss of the transverse shear reinforcing steel typically occurs where the reinforcing steel is bent around the longitudinal reinforcing steel and can result in a loss of section, loss of anchorage and/or a reduction in ductility of the transverse shear reinforcing steel. However, as noted in Section 9.4.5.3, tests on precast concrete channel girders showed that a reduction of 50% in the cross-sectional area of the transverse shear reinforcing steel where it was bent around the longitudinal reinforcing steel did not reduce the shear resistances of the girders to below the calculated shear resistances although the ductility of the failure modes was reduced. These results are also expected to be applicable to reinforced concrete members as they typically have deeper cross-sections than precast reinforced concrete girders.

Section loss of the main longitudinal reinforcing steel shall be considered in accordance with Section 9.3.7.2 of this Manual.

Cracking and spalling of the concrete around the main longitudinal reinforcing steel can result in a reduction in the anchorage of the longitudinal reinforcing steel. Unless otherwise verified on a bridge specific basis, reinforcing steel lengths where the concrete surrounding the reinforcing steel bar is

cracked and/or spalled should be assumed to be incapable of providing anchorage to the bar unless the bar is directly above a bearing support and the only damage/deterioration is hairline cracking.

Excessive cracking of the web of a reinforced concrete bridge member can result in a reduction in the member's shear resistance. However, a reinforced concrete bridge girder's shear resistance is not expected to be reduced by concrete cracking provided the following requirements are met:

- *the longitudinal reinforcing steel and transverse shear reinforcing steel meet the requirements of CSA S6-14 and this Manual including anchorage requirements;*
- *concrete cracking and spalling have not reduced the anchorage of the longitudinal reinforcing steel and transverse shear reinforcing steel to less than the requirements of CSA S6-14 and this Manual; and*
- *the shear cracks are less than 0.3 mm wide, are not at an angle between 25° and 60° to the horizontal, and do not extend over a vertical or horizontal projection greater than one-half of the girder depth (Habel and Quinton, 2010).*

Note that although these guidelines were developed for Type NU precast, prestressed concrete bridge girders they are also expected to be applicable to reinforced concrete bridge girders with properly anchored longitudinal and transverse shear reinforcing steel. If concrete cracking and spalling have reduced the anchorage of the longitudinal and/or transverse shear reinforcing steel to less than design requirements, shear failure could occur with little warning due to anchorage failure (Mao et al., 1997). In this case cracks of any length or width that are inclined at an angle between 25° and 60° to the horizontal could be providing warning of shear failure. The above guidance does not apply to shear cracks in slabs without transverse shear reinforcing steel as transverse shear reinforcing steel is not available to control the growth of the shear crack widths.

The above guidance regarding shear cracks may not apply to reinforced concrete bridge girders that have been strengthened in shear with external stirrups. This is because the external stirrups are not bonded to the girder concrete. The external stirrup strains resulting from increased crack widths are therefore distributed over the entire length of the external stirrups rather than locally in the vicinity of the cracks as is the case for internal stirrups. As a result, increases in stirrup stresses due to increasing crack widths are smaller for external stirrups than for internal stirrups. External stirrups are therefore less effective in controlling the growth of the shear crack widths. Therefore, if a girder has been strengthened with external stirrups, the possibility of shear cracks of any width reducing the shear resistance should be considered. This concern also applies to shear cracks present in a girder prior to shear strengthening. However, provided the external stirrups have been adequately stressed so that they will pick up load with any increase in shear crack width, it is reasonable to expect the shear crack width to remain stable over time, provided the loading on the bridge at the time of strengthening is not increased,

Concrete spalling from the girder web above the level of the bottom row of longitudinal reinforcing steel can reduce the shear resistance of a reinforced concrete girder by decreasing the effective web width, "b_v" of the member.

9.4 Precast Reinforced Concrete Bridges

9.4.1 General

The resistances of precast reinforced concrete bridge members shall be determined in accordance with the requirements of CSA S6-14 and this Manual unless otherwise approved by AT. When combined effects occur

simultaneously in the same member such that the resistance for one is affected by the magnitude of the other, the live load capacity factor shall be determined by successive iteration or other suitable method.

9.4.2 Precast Reinforced Concrete Bridge Girder Members Requiring Load Evaluation

Selection of the precast reinforced concrete bridge girder members requiring load evaluation shall be based on engineering judgment. The following precast reinforced concrete bridge girder members will typically require load evaluation:

- Main precast reinforced concrete girders in moment and shear; and
- End anchorage of longitudinal reinforcement at supports.

Commentary 9.4.2

The following precast reinforced concrete bridge girder members will typically not require load evaluation:

- *Top flanges of girders in transverse moment and shear. The clear spans of the girder top flanges are typically less than 600 mm and can be assumed to distribute wheel loads between the girder webs by arching action.*

Note that many precast reinforced concrete bridge superstructures are supported on timber bridge substructures that may also require load evaluation:

9.4.3 Resistance of Precast Reinforced Concrete Bridge Girders

9.4.3.1 Lifting Hook and Shear Connector Pockets

Lifting hook and shear connector pockets shall be considered to be ungrouted when determining girder moment and shear resistances.

The moment and shear resistances of a girder at the location of a lifting hook or shear connector pocket may be determined by;

- Reducing the effective depth of the girder by the depth of the pockets at lifting hook pocket locations when determining shear resistance;
- Reducing the area of the compression flange by the area of the pockets at lifting hook pocket locations when determining moment resistance; and
- Reducing the effective width of the girder flange and webs by the widths of the shear connector pockets at shear connector pocket locations.

Commentary 9.4.3.1

Lifting hooks and shear connector pockets may not have been filled with grout or may have been filled with grout under adverse environmental conditions, resulting in the grout being of a poor quality. Also, the grout in these pockets is subject to deterioration due to exposure to de-icing salts and/or water getting into shrinkage cracks between the grout and surrounding concrete. As a result, the girder's resistance may be reduced at the lifting hook and shear connector pocket locations even if the grout appears to be in good condition. This has been confirmed by load tests carried out on precast reinforced concrete girders in AT's bridge inventory (Ghali, 1986 and Mao et al., 1997).

The above recommendations will reduce the shear resistances of HC girders by approximately 18% at lifting hook/shear connector pocket locations.

9.4.3.2 Longitudinal Reinforcing Steel

The moment resistances determined for precast reinforced concrete bridge girders shall be based on the assumption that the end of any longitudinal reinforcing steel without a standard hook and contributing to the moment resistance of the girder is terminated "$d_v \cot\theta$" short of its actual termination point.

Commentary 9.4.3.2

Since the cut-off points for the longitudinal reinforcing steel need to be extended past the points at which they are no longer required in new design in accordance with CSA S6-14, Section 8.9.3.10, these extensions should be ignored when determining the adequacy of the longitudinal reinforcing steel for moment during a load evaluation.

This requirement is similar to the recommendation made by MacGregor (MacGregor, 1987) for determining the shear resistance of reinforced concrete girder bridges.

9.4.3.3 Transverse Shear Reinforcing Steel

The transverse shear reinforcing steel available to resist an applied shear force at a section may be taken to be the transverse shear reinforcing steel located over a girder length of "$d_v \cot\theta$" and centred on the section being considered.

Commentary 9.4.3.3

This requirement is a refinement of CSA S6-14, Section 8.9.3.9.

9.4.4 Resistance of Precast Reinforced Concrete Bridge Girders Not Meeting the Requirements of CSA S6-14

9.4.4.1 Transverse Shear Reinforcing Steel

The inside legs of shear transverse reinforcing steel in precast reinforced concrete channel girder legs are exempt from meeting the anchorage requirements of CSA S6-14, Section 8.15.1.5.

If the transverse shear reinforcing steel ratio is less than that allowed by CSA S6-14, Section 8.9.1.3, or if the spacing of the transverse shear reinforcing steel is greater than that allowed by CSA S6-14, Section 8.14.6, the shear resistance of the member shall be determined in accordance with CSA S6-14, Section 14.14.1.6.

Commentary 9.4.4.1

Tests carried out on precast reinforced concrete channel girders in AT's bridge inventory have shown that combined torsion and shear effects in the most heavily loaded girder web of an eccentrically loaded girder are additive on the outside web surface and subtractive on the inside web surface (Ghali, 1986 and Mao et al., 1997) (see Figure 9.4.4.1). As a result, full anchorage of the stirrup legs adjacent to the inside surfaces of the girder webs is not required for the stirrups to be fully effective in contributing to the girder's shear resistance.

9.4.5 Resistance of Damaged or Deteriorated Precast Reinforced Concrete Bridge Girders

9.4.5.1 General

Precast reinforced concrete bridge girders are typically damaged by corrosion of the reinforcing steel or by cracking and spalling of the concrete due to either corrosion of the reinforcing steel or collision impact.

Corrosion of the reinforcing steel can result in section loss but will typically not result in a change in the reinforcing steel yield stress. It can, however, result in a reduction in reinforcing steel ductility.

Collision impact on the reinforcing steel can result in section loss due to nicking or rupture of the steel.

Cracking of the concrete will result in a loss of tension resistance across the cracks and can cause a loss of interface shear transfer across the cracks and compression resistance parallel to the cracks. It can also result in a loss of reinforcing steel anchorage.

Spalling of the concrete will result in a loss of concrete compression and tension resistance at the spall location. It can also result in a loss of reinforcing steel anchorage. Spalled concrete includes loose or fractured concrete adjacent to the spall that has not yet fallen away.

Commentary 9.4.5.1

Corrosion of the reinforcing steel is primarily caused by de-icing salts which penetrate down through the gaps between the girders. This corrosion not only results in section loss and reduced ductility of the reinforcing steel but also results in cracking and spalling of the concrete which affects the anchorage of the reinforcing steel. It typically affects both the reinforcing steel in the top flange of the member and the bottom reinforcing steel and stirrups in the webs.

Collision impacts on precast reinforced concrete bridge girders are typically caused by high loads. These impacts can damage or completely sever the reinforcing steel as well as crack and spall the concrete around the reinforcing steel.

9.4.5.2 Resistance of Damaged or Deteriorated Precast Reinforced Concrete Bridge Girders in Moment

The determination of the moment resistance of a damaged or deteriorated precast reinforced concrete bridge girder shall account for section loss, a reduction in reinforcing steel ductility and loss of anchorage of the reinforcing steel. It shall also account for loss of concrete area due to spalling.

If the main longitudinal reinforcement of a precast reinforced concrete bridge girder is corroded and the surrounding concrete cracked and/or spalled the Element Behaviour Category shall be taken as being either Category E1 or E2 in the determination of the moment resistance of the girder. The Element Behaviour Category used shall depend on the location and severity of the corrosion, cracking and/or spalling and its effect on the anchorage of the reinforcing steel.

Commentary 9.4.5.2

Section loss of the main longitudinal reinforcing steel due to corrosion is typically not great enough to cause a significant decrease in girder resistance but this should be verified on a bridge specific basis. More significant is the reduction in ductility and reduced anchorage of the longitudinal reinforcing steel

that results from the concrete cracking and spalling. Note, however, that even if the concrete has spalled off of a reinforcing steel bar, partial anchorage of the reinforcing steel bar will still be provided by the transverse shear reinforcement holding the bar against the remaining concrete unless the transverse steel is heavily corroded or broken.

Tests carried out on precast concrete channel girders showed that main longitudinal reinforcing steel corrosion, and the accompanying cracking and spalling of the surrounding concrete, did not reduce the moment resistances of the girders to below the calculated moment resistances despite two of the three main longitudinal reinforcing steel bars in each web being terminated short of the girder support (Mao et al., 1997).

Loss of anchorage of the reinforcing steel will be most critical if the loss of anchorage occurs in the development length near the end of a reinforcing steel bar. Therefore, Element Behaviour Category E1 should be used when the loss of anchorage occurs near the end of a reinforcing steel bar.

Concrete spalling can reduce the compression resistance of a precast reinforced concrete girder in moment although this loss in resistance is seldom critical.

9.4.5.3 Resistance of Damaged or Deteriorated Precast Reinforced Concrete Bridge Girders in Shear

The determination of the shear resistance of a damaged or deteriorated precast reinforced concrete bridge girder shall account for section loss, loss of anchorage and a reduction in the ductility of the reinforcing steel. It shall also account for loss of concrete shear strength due to cracking and spalling.

If the transverse shear reinforcing steel of a precast reinforced concrete bridge girder is corroded and the surrounding concrete cracked and/or spalled the Element Behaviour Category shall be taken as being Category E1 in the determination of the shear resistance of the girder.

If the concrete surrounding the longitudinal reinforcing steel contributing to the shear resistance of a precast concrete bridge girder is cracked and/or spalled, the Element Behaviour Category shall be taken as being Category E1 in the determination of the shear resistance of the girder.

Commentary 9.4.5.3

Section loss of the transverse shear reinforcing steel typically occurs where the reinforcing steel is bent around the longitudinal reinforcing steel and can result in a loss of section, loss of anchorage and/or a reduction in ductility of the transverse shear reinforcing steel. However, as noted below, tests on precast concrete channel girders showed that a reduction of 50% in the cross-sectional area of the transverse shear reinforcing steel where it was bent around the longitudinal reinforcing steel did not reduce the shear resistances of the girders to below the calculated shear resistances although the ductility of the failure modes was reduced.

Section loss of the main longitudinal reinforcing steel shall be considered in accordance with Section 9.4.5.2 of this Manual.

Cracking and spalling of the concrete around the main longitudinal reinforcing steel can result in a reduction in the anchorage of the longitudinal reinforcing steel although transverse shear reinforcement which is not heavily corroded or broken can provide partial anchorage to the longitudinal reinforcing steel bar. Tests carried out on precast concrete channel girders showed that longitudinal and transverse shear reinforcing steel corrosion, and the accompanying cracking and spalling of the

concrete surrounding the longitudinal reinforcing steel, did not reduce the shear resistances of the girders to below the calculated shear resistances despite two of the three main longitudinal steel bars in each web being terminated short of the girder end support and/or the cross-sectional area of the transverse shear reinforcement being reduced by 50% where it was bent around the longitudinal reinforcing steel. The ductility of the failure mode was also not reduced provided there was adequate anchorage of the longitudinal reinforcing steel (bond failures prevented) and the transverse shear reinforcement (Mao et al., 1997).

The tests also showed that there was a reduction in the ductility of the failure mode when the longitudinal reinforcing steel was not adequately anchored (bond failures) or when the cross-sectional area of the transverse shear reinforcement was reduced by 50% where it was bent around the longitudinal reinforcing steel. These test results demonstrate that a reduction in the ductility of the failure mode occurs when the reinforcing steel is not adequately anchored.

Excessive cracking of the web of a precast reinforced concrete bridge girder can result in a reduction in the girder's shear resistance. However, a precast reinforced concrete bridge girder's shear resistance is not expected to be reduced by concrete cracking provided the following requirements are met:

- *the longitudinal reinforcing steel, and transverse shear reinforcing steel meet the requirements of CSA S6-14 and this Manual including anchorage requirements;*
- *concrete cracking and spalling have not reduced the anchorage of the longitudinal reinforcing steel and transverse shear reinforcing steel to less than the requirements of CSA S6-14 and this Manual; and*
- *the shear cracks are less than 0.3 mm wide, are not at an angle between 25° and 60° to the horizontal, and do not extend over a vertical or horizontal projection greater than one-half of the girder depth (Habel and Quinton, 2010).*

Note that although these guidelines were developed for Type NU precast, prestressed concrete bridge girders they are also expected to be applicable to precast, reinforced concrete bridge girders with properly anchored longitudinal and transverse shear reinforcing steel. If concrete cracking and spalling have reduced the anchorage of the longitudinal and/or transverse shear reinforcing steel to less than design requirements, shear failure could occur with little warning due to anchorage failure (Mao et al., 1997). In this case cracks of any length or width that are inclined at an angle between 25° and 60° to the horizontal could be providing warning of shear failure.

Concrete spalling from the girder web above the level of the bottom row of longitudinal reinforcing steel can reduce the shear resistance of a precast reinforced concrete girder by decreasing the effective web width, "b_v" of the girder.

9.5   Precast Prestressed Concrete Bridges

9.5.1 General

The resistances of precast prestressed concrete bridge girders shall be determined in accordance with the requirements of CSA S6-14 and this Manual unless otherwise approved by AT. When combined effects occur simultaneously in the same member such that the resistance for one is affected by the magnitude of the other, the live load capacity factor shall be determined by successive iteration or other suitable method.

9.5.2 Precast Prestressed Concrete Bridge Girder Members Requiring Load Evaluation

Selection of the precast prestressed concrete bridge girder members requiring load evaluation shall be based on engineering judgment.

The following precast prestressed concrete bridge girder members will typically require load evaluation:

- Main precast prestressed concrete girders in moment and shear;
- End anchorage of longitudinal reinforcement and prestressing steel at supports; and
- Girder top flanges in transverse moment and shear having clear spans greater than or equal to 1000 mm or cantilever spans greater than or equal to 600 mm.

Commentary 9.5.2

The following precast prestressed concrete bridge girder members will typically not require load evaluation:

- *Top flanges of girders in transverse moment and shear, having clear spans less than 1000 mm or cantilever spans less than 600 mm, can typically be assumed to be adequate to distribute wheel loads between the girder webs; and*
- *Bracing members and connections not at kinks in chorded precast, prestressed concrete girders. Bracing members do not require load evaluation provided they are not being asked to carry increased loads. This is typically the case when truck loads are increased provided the bracing members are not being counted on to improve the lateral distribution of live load between girders.*

9.5.3 Serviceability Limit State

Tension stresses in the prestressing steel shall be checked to determine if they exceed 90% of the yield stress for CSA S6-14 Serviceability Limit State Combination 1. The results shall be reported in the bridge load evaluation report. However, they need not be considered in the determination of the bridge's load carrying capacity.

Tension stresses in the deck reinforcing steel due to negative moments shall be checked at CSA S6-14 Serviceability Limit State Combination 1. The results shall be reported in the bridge load evaluation report. However, they need not be considered in the determination of the bridge's load carrying capacity.

Commentary 9.5.3

High tension stresses in the prestressing steel at the serviceability limit state can reduce the service life of a bridge due to premature deterioration of the girders caused by excessive stresses and cracking. However, they will not reduce the bridge's load carrying capacity provided the bridge girders remain in good condition. Therefore, they are not considered in the determination of the bridge's load carrying capacity.

The tension stress in the prestressing steel will be equal to:

- *The stress due to prestressing after all prestressing losses have occurred; plus*

- *The increase in stress due to loads applied to the uncracked girder section. The maximum load to be applied to the uncracked girder section is the load that places zero stress in the outer fibre of the tension flange of the girder; plus*

- *The increase in stress due to loads applied to the cracked girder section. The loads applied to the cracked girder section are the loads that exceed the load that places zero stress in the outer fibre of the tension flange of the girder.*

If a load applied to a bridge girder exceeds the girder's cracking moment, the stress range in the girder's prestressing steel under load increases dramatically. If the load is applied to the bridge on a regular basis, the bridge's service life could be shortened by the fatigue life of the prestressing steel. This is of particular concern at strand deflection points. Therefore, if a bridge is being load evaluated for a truck load that will both cross the bridge on a regular basis and cause the bridge girder's cracking moment to be exceeded, consideration should be given to whether or not the fatigue life of the prestressing steel will shorten the service life of the bridge.

The tension stresses in the deck reinforcing steel due to negative moments should remain well below the yield stress of the reinforcing steel at the serviceability limit state to control deck crack widths. However, while excessive crack widths can reduce the service life of a bridge deck they will not reduce the bridge's load carrying capacity provided the bridge deck remains in good condition. Therefore, they are not considered in the determination of the bridge's load carrying capacity.

9.5.4 Section Properties of Precast Prestressed Concrete Bridge Girders

9.5.4.1 Precast Prestressed Concrete Bridge Girders Made Continuous for Live Load with a Reinforced Concrete Deck

Precast prestressed concrete bridge girders made continuous for live load with a reinforced concrete deck over the piers shall be analyzed based on composite gross section properties in the positive moment regions and on composite cracked section properties in the negative moment regions. The negative moment regions may be considered to extend for 20% of the span lengths on each side of the piers with the remaining span lengths considered to be positive moment regions.

If the girder transverse reinforcing steel extending into the reinforced concrete deck to provide for composite action between the deck and the girders does not meet the requirements of CSA S6-14, Section 8.15.1.5 for anchorage into the deck, the girders shall be analyzed in accordance with the requirements of Section 9.5.6.1 of this Manual.

Commentary 9.5.4.1

The assumed length of the negative moment region is a simplification based on the recommendations of CAN/CSA S6-88 (CSA, 1988). Large cambers in precast prestessed concrete girders can result in large deck haunches being required between the girders and deck near the ends of the girders. As a result the transverse reinforcing steel may not extend sufficiently into the deck to ensure composite action between the deck and girders as required by CSA S6-14, Section 8.15.1.5 unless additional hat reinforcing steel is present in the deck haunches (Ramsay, 2009).

In the past, camber in AT's precast, prestressed concrete girders has been accommodated by either:

- *Constructing the girders with a sag profile in their top surface by increasing the girder depth at their ends. After release of the prestressing the girder camber and top surface sag profile theoretically result in a non-cambered top girder surface;*
- *Using a form that allows the girders to be constructed in a sagged position. After release of the prestressing the girder camber and sagged girder profile theoretically result in a non-cambered top girder surface;*

- *Ignoring the camber. This has been the practice for shorter span precast, prestressed concrete girders where girder camber can be considered to be negligible; or*
- *Allowing the girders to camber freely. This been the practice for longer span precast, prestressed concrete Span Type "NU" girders.*

Therefore, with the exception of Span Type "NU" girders, it can be assumed that the transverse reinforcing steel in precast, prestressed concrete girders extends sufficiently into the deck to ensure composite action between the deck and girders. When load evaluating Span Type "NU" girders the load evaluator should review the as-constructed/Record drawings and/or construction Final Details Report to determine if the transverse reinforcing steel extends sufficiently into the deck to ensure composite action between the deck and girders.

9.5.4.2 Precast Prestressed Concrete Bridge Girders Made Continuous for Live Load with Post-Tensioning

For bridges with precast prestressed concrete bridge girders made continuous with post-tensioning, the section properties for analysis shall be based on the following section properties:

- Gross section properties of the girders if, for the loads being considered, the girders were not designed to be composite with the deck;
- Composite gross section properties of the girders and deck if, for the loads being considered, the girders were designed to be composite with the deck.

9.5.5 Resistance of Precast Prestressed Concrete Bridge Girders

9.5.5.1 Longitudinal Reinforcing Steel

The moment resistances determined for precast prestressed concrete bridge girders shall be based on the assumption that the end of any longitudinal reinforcing steel without a standard hook and contributing to the moment resistance of the girder is terminated "$d_v \cot\theta$" short of its actual termination point.

Commentary 9.5.5.1

Since the cut-off points for longitudinal reinforcing steel need to be extended past the points at which they are no longer required in new design in accordance with CSA S6-14, Section 8.9.3.10 these extensions should be ignored when determining the adequacy of the longitudinal reinforcing steel for moment during a load evaluation.

This requirement is similar to the recommendation made by MacGregor (MacGregor, 1987) for determining the shear resistance of reinforced concrete girder bridges.

9.5.5.2 Transverse Shear Reinforcing Steel

The transverse shear reinforcing steel available to resist an applied shear force at a section may be taken to be the transverse shear reinforcing steel located over a girder length of "dv cotθ" and centred on the section being considered.

Commentary 9.5.5.2

This requirement is a refinement of CSA S6-14, Section 8.9.3.9.

9.5.5.3 Transverse Shear Reinforcing Steel With Re-Entrant Corners

The shear resistance of prestressed concrete girders that have transverse shear reinforcing steel with re-entrant corners shall be based on the lower of the yield stress of the transverse shear reinforcing steel and the stress required in the transverse shear reinforcing steel to spall off the transverse shear reinforcing steel concrete cover at the re-entrant corner. The resistance of the concrete cover at the re-entrant corner to spalling may be determined in accordance with the requirements of CSA S6-14, Section 8.16.7.2.2 where the effective stress area "A_{eff}" is taken to be equal to:

$A_{eff} = L_{eff} \bullet d_{eff}$

Where the symbols used in the equation are:

d_{eff} = 2 times the concrete cover to the centre of the transverse shear reinforcing steel; and

L_{eff} = the effective length of the concrete resisting spalling as shown in Figure 9.5.5.3.

The concrete cover assumed at the re-entrant corner shall take into account the nominal design concrete cover shown on the drawings as well as the tolerances typically used for placing reinforcing steel in precast concrete fabrication plants.

If the stress required in the transverse shear reinforcing steel to spall off the concrete cover at the re-entrant corner is less than the yield stress the Element Behaviour Category shall be taken as being Category E1 in the determination of the shear resistance of the girder.

Commentary 9.5.5.3

> *Transverse shear reinforcing steel in Span Type "PO" girders in AT's bridge inventory may have re-entrant corners. Re-entrant corners in transverse shear reinforcing steel can result in the reinforcing steel spalling off its concrete cover at the re-entrant corner prior to its reaching its yield stress. If this occurs, the resistance of the transverse shear reinforcing steel is assumed to be limited to the force in the reinforcing steel at the time the concrete cover spalls off.*

> *The force in the transverse shear reinforcing steel at which spalling of the concrete cover occurs is a function of the concrete strength, concrete cover depth and bend radius of the reinforcing steel at the re-entrant corner. In the absence of better information the placing tolerance in the concrete cover can be assumed to be 10 mm and the bend radius of the reinforcing steel can be assumed to be three times the transverse shear reinforcing steel bar diameter.*

9.5.5.4 Determination of Girder Prestress Losses

Concrete elastic shortening, concrete creep, concrete shrinkage and steel relaxation prestress losses in precast prestressed concrete girders with 28 day concrete design strengths of 40 MPa or less, shall be determined in accordance with the requirements of Appendix H, rather than in accordance with the requirements of CSA S6-14. For precast prestressed concrete girders with 28 day concrete design strengths greater than 40 MPa, the provisions in CSA S6-14 shall be followed.

Commentary 9.5.5.4

> *The requirements of Appendix H are not recommended for determining prestress losses in precast prestressed concrete girders with 28 day concrete design strengths greater than 40 MPa. This is because the "PCILOSS" (PCI Committee on Prestress Losses, 1975) recommendations were not developed for precast prestressed concrete girders with 28 day concrete design strengths greater than 40 MPa.*

> *The requirements of Appendix H are based on recommendations made by Ramsay (Ramsay, 2015b). This study considered prestress losses in precast, prestressed concrete girders for five types of shear connected girders (Type PM, RD and SL voided box girders, Type FC channel girders and Type DBT bulb-tee girders) in AT's bridge inventory. The prestress loss predictions of eight current and historical bridge codes, including a Lump Sum method, were compared. The predictions from either the "Lump Sum" or "PCILOSS" (PCI Committee on Prestress Losses, 1975) prestress loss methods were recommended for determining prestress losses. The provisions of these methods are given in Appendix H.*

> *The "Lump Sum" and "PCILOSS" methods were selected because they predict prestress losses that are accepted by the precast concrete industry in North America. They were also selected because they predict prestress losses that are consistent with those historically used by AT for designing the precast prestressed girders in their bridge inventory. These girders have had a history of good performance in the field with no evidence of cracking other than in anchorage areas.*

> *The "Lump Sum" and "PCILOSS" methods predict similar prestress losses for girders with typical levels of prestressing. However the "PCILOSS" method takes into account the level of prestressing when determining prestress losses. It therefore predicts lower prestress losses for girders that are lightly prestressed and higher prestress losses for girders that are highly prestressed.*

9.5.6 <u>Resistance of Precast Prestressed Concrete Bridge Girders not Meeting the Requirements of CSA S6-14</u>

9.5.6.1 <u>Precast Prestressed Concrete Bridge Girders Made Continuous for Live Load</u>

Bridges with precast prestressed concrete girders made continuous for live load with a reinforced concrete deck over the piers, but with the girder transverse shear reinforcing steel not extending into the deck or not extending sufficiently into the deck to provide for composite action between the deck and the girders as required by CSA S6-14, Section 8.15.1.5, shall be load evaluated at the ultimate limit state in accordance with the following:

- The positive moment and shear resistances of the girders shall be determined based on the girders being simple spans and non-composite with the deck; and
- The shear resistances of the girders shall also be determined based on the girders being continuous spans and composite with the deck.

The lowest shear resistances determined using the above two procedures shall govern.

Bridges with precast, prestressed concrete girders made continuous for live load with a reinforced concrete deck shall have the stresses in the deck reinforcing steel determined in the negative moment regions in accordance with the requirements of Section 9.5.3 of this Manual. In determining the deck reinforcing steel stresses the girders shall be considered to be continuous spans and composite with the deck even if the girder transverse

shear reinforcing steel does not extend into the deck or sufficiently into the deck to meet the requirements of CSA S6-14, Section 8.15.1.5.

For composite and continuous spans, the negative moment regions may be considered to extend for 20% of the span lengths on each side of the piers with the remaining span lengths considered to be positive moment regions. Also, the analysis shall be based on composite gross section properties in the positive moment regions and on composite cracked section properties in the negative moment regions.

Commentary 9.5.6.1

If the transverse shear reinforcing steel does not extend into the deck or does not extend sufficiently into the deck to ensure composite action between the girders and the deck the girders may debond from the deck at the ultimate limit state. If the girders debond from the deck they will act in a non-composite manner and as simple spans when carrying the applied loads. In this case the girder's resistances in positive moment and shear need to be checked based on the girders being non-composite with the deck. See Section 9.5.4.1 of this Manual for situations where girder camber may result in the transverse shear reinforcing steel not extending sufficiently into the deck to ensure composite action between the girders and deck.

If the girders do not debond from the deck they will act in a composite manner with the deck and as continuous spans. In this case the girder's resistance in positive moment does not need to be checked as it will be greater than its resistance when it is non-composite with the deck. However, the girder's shear resistance needs to be checked based on its being composite with the deck as its shear resistance may be reduced by the tension bending stresses induced into the girder by the moments in the negative moment regions. Also, the stresses in the deck reinforcing steel need to be assessed in the negative moment regions at the serviceability limit state as a check on the crack widths in the deck (see Section 9.5.3 of this Manual).

9.5.6.2 Transverse Shear Reinforcing Steel

If the transverse shear reinforcing steel ratio is less than that allowed by CSA S6-14, Section 8.9.1.3, or if the spacing of the transverse shear reinforcing steel is greater than that allowed by CSA S6-14, Section 8.14.6, the shear resistance of the girder shall be determined in accordance with CSA S6-14, Section 14.14.1.6.

9.5.7 Resistance of Strengthened Precast Prestressed Concrete Bridge Members

9.5.7.1 Resistance of Strengthened Precast Prestressed Concrete Bridge Girders in Moment

The resistance of a strengthened precast prestressed concrete bridge girder in moment may be determined based on the assumption that the dead and live load stresses are uniformly distributed between the original and strengthening components provided that the longitudinal steel reinforcement ratio after strengthening meets the requirements of CSA S6-14, Sections 8.8.4.3 and 8.8.4.5.

The adequacy of the connections at the ends of the strengthening component to transfer the strengthening component stresses back into the original component shall be confirmed.

Commentary 9.5.7.1

AT has historically strengthened precast prestressed concrete bridge girders in moment by either adding strengthening plates to the girder flanges or by external post-tensioning. The strengthening

plates and/or external post-tensioning need to be extended far enough to connect to portions of the girder that do not require strengthening. The strengthening plates and/or external post-tensioning may be assumed to carry both dead load and live load as the girder is assumed to have adequate ductility to allow stresses to be redistributed between the original and strengthening components.

9.5.7.2 Resistance of Strengthened Precast Prestressed Concrete Bridge Girders in Shear

External stirrups added to strengthen a precast prestressed concrete bridge girder in shear may be assumed to carry full dead and live loads provided that the external stirrup spacing is less than "$0.5 \, d_v \cot\theta$" at the location of the stirrups, where "d_v" is the effective shear depth of the girder as defined by CSA S6-14 and θ is the angle of inclination of the shear crack relative to the horizontal as determined in accordance with CSA S6-14, Section 8.9.3.7, If the external stirrup spacing exceeds "$0.5 \, d_v \cot\theta$" any contribution of the external stirrups to the girder's shear resistance shall be ignored.

Commentary 9.5.7.2

AT has historically strengthened precast prestressed concrete bridge girders in shear by adding external stirrups to the girders. The maximum external stirrup spacing has historically been limited to 75% of the girder effective shear depth based on recommendations made by MacGregor (MacGregor, 1987) for reinforced concrete girders. This recommendation was based on observations that the critical shear crack generally extended a distance of at least 1.5 times the girder depth along the length of the girder. These observations are also expected to be applicable for precast, prestressed concrete girders as the prestressing force reduces the angle of the critical shear crack relative to the horizontal. This section of the Manual is a refinement of MacGregor's recommendation. If the external stirrup spacing exceeds "$0.5 \, d_v \cot\theta$" there is an increased possibility that the critical girder shear failure mechanism will occur between the external stirrups.

Since shear cracks are typically not expected to occur in a girder web under service loads and since the internal transverse shear reinforcing steel is not highly stressed until after shear cracks have formed the external stirrups may be assumed to carry both dead load and live load shears along with the internal transverse shear reinforcement.

9.5.8 Resistance of Damaged or Deteriorated Precast Prestressed Concrete Bridge Girders

9.5.8.1 General

Precast prestressed concrete bridge girders are typically damaged by corrosion of the reinforcing steel and/or prestressing steel, by collision impact on the reinforcing steel and/or prestressing steel, and by cracking and spalling of the concrete due to either corrosion of the reinforcing steel and/or prestressing steel or collision impact.

Corrosion of the reinforcing steel can result in section loss but will typically not result in a change in the reinforcing steel yield stress. It can, however, result in a reduction in reinforcing steel ductility.

Corrosion of the prestressing steel can result in section loss, a reduction in the prestressing steel ultimate tensile stress and a reduction in ductility. The remaining fatigue life of the prestressing steel can also be reduced by corrosion.

Collision impact on the reinforcing steel and/or prestressing steel can result in section loss due to nicking or rupture of the steel. The remaining fatigue life of the prestressing steel can also be reduced as a result of the damage.

Cracking of the concrete will result in a loss of tension resistance across the cracks and can also cause a loss of interface shear transfer across the cracks and compression resistance parallel to the cracks. It can also result in a loss of reinforcing steel abd/or prestressing steel anchorage.

Spalling of the concrete will result in a loss of concrete compression and tension resistance at the spall location. It can also result in a loss of reinforcing steel and/or prestressing steel anchorage. Spalled concrete includes loose or fractured concrete adjacent to the spall that has not yet fallen away.

Commentary 9.5.8.1

> *Corrosion of the reinforcing steel is primarily caused by de-icing salts which penetrate down through any gaps between the girders. This corrosion not only results in section loss and a reduction in ductility of the reinforcing steel but also results in cracking and spalling of the concrete which affects the anchorage of the reinforcing steel.*

> *Corrosion of the prestressing steel is also primarily caused by de-icing salts. The de-icing salts typically gain access to the prestressing steel through any gaps between the girders or through the deck joints at the girder ends and then travel along the prestressing steel strand surfaces. Prestressing steel corrosion can cause a loss of section, a reduction in the ultimate tensile stress, and a reduction in ductility in the prestressing steel. The reduction in the prestressing steel ultimate tensile stress is caused by local pitting corrosion which reduces the stress at which brittle fracture occurs in the steel.*

> *To this point in time, although there has been corrosion observed in the bottom row of prestressing steel strands in some girders, there is no evidence that prestressing steel corrosion is a major concern with prestressed concrete girders in AT's bridge inventory. However, this may be due to the corrosion not being sufficiently advanced to crack and/or spall the surrounding concrete and, therefore, not being detected.*

> *Collision impacts on precast prestressed concrete bridge girders are typically caused by high loads. These impacts can damage or completely sever the reinforcing steel and/or prestressing steel as well as crack and spall the concrete around the reinforcing steel and/or prestressing steel.*

9.5.8.2 Resistance of Damaged or Deteriorated Precast Prestressed Concrete Bridge Girders in Moment

The determination of the moment resistance of a damaged or deteriorated precast prestressed concrete bridge girder shall account for section loss, reduction in ductility, a reduction in the ultimate tensile stress, and loss of anchorage of any reinforcing steel and/or prestressing steel contributing to the moment resistance. It shall also account for loss of concrete area due to spalling.

Prestressing steel wires, including individual wires of seven wire strands, which have suffered damage due to corrosion (except for surface corrosion with negligible section loss and no pitting) or due to collision impact, shall be ignored in determining the moment resistance.

If the concrete, surrounding the reinforcing steel and/or prestressing steel contributing to the moment resistance, of a precast prestressed concrete bridge girder is cracked and/or spalled, the Element Behaviour Category shall

be taken as being either Category E1 or E2 in the determination of the moment resistance of the girder. The Element Behaviour Category used shall depend on the location and severity of the corrosion, cracking and/or spalling and its effect on the anchorage of the reinforcing steel and/or prestressing steel.

Commentary 9.5.8.2

Section loss or a reduction in the ultimate tensile stress of the main prestressing steel can result in a decrease in girder moment resistance. It can also significantly reduce the future fatigue life of the prestressing steel. Therefore, prestressing steel wires, including individual wires of seven wire strands, which have suffered any significant level of damage due to corrosion or collision impact, should not be relied on in determining the moment resistance of a precast prestressed concrete bridge girder. Significant damage includes any damage to the prestressing steel caused by high load damage or by corrosion other than surface corrosion with negligible section loss and no pitting. Section loss of the main longitudinal reinforcing steel due to corrosion is typically not great enough to cause a significant decrease in girder moment resistance, but this should be verified on a bridge specific basis.

Concrete cracking and spalling, caused by prestressing steel and/or reinforcing steel corrosion or collision impact, can reduce girder moment resistance by reducing the anchorage of the prestressing steel and/or reinforcing steel. Loss of anchorage of the reinforcing steel and/or prestressing steel will be most critical if the loss of anchorage occurs in the development length near the end of a reinforcing steel bar or prestressing steel strand. Therefore, Element Behaviour Category E1 should be used when the loss of anchorage occurs near the end of a reinforcing steel bar or prestressing steel strand.

See Section 9.3.7.2 of this Manual for a discussion of the loss of anchorage of longitudinal reinforcing steel bars in cracked and/or spalled concrete.

Prestressing steel strand lengths, where the concrete surrounding the strand is cracked and/or spalled, should be assumed to be incapable of providing anchorage to the strand unless the strand is directly above a bearing support and the only damage/deterioration to the concrete is hairline cracking.

Concrete spalling can reduce the compression resistance of a precast prestressed concrete bridge girder in moment although this loss in resistance is seldom critical.

9.5.8.3 Resistance of Damaged or Deteriorated Precast Prestressed Concrete Bridge Girders in Shear

The determination of the shear resistance of a damaged or deteriorated precast prestressed concrete bridge girder shall account for section loss, loss of anchorage, a reduction in the ultimate tensile stress, and a reduction in ductility of the reinforcing steel and/or prestressing steel. It shall also account for loss of concrete shear strength due to cracking and spalling.

Prestressing steel wires, including individual wires of seven wire strands, which have suffered significant damage due to corrosion (except for surface corrosion with negligible section loss and no pitting) or due to collision impact shall be ignored in determining the shear resistance.

If the transverse shear reinforcing steel of a precast prestressed concrete bridge girder is corroded and the surrounding concrete cracked and/or spalled the Element Behaviour Category shall be taken as being Category E1 in the determination of the shear resistance of the girder.

If the concrete, surrounding the prestressing steel and/or longitudinal reinforcing steel contributing to the shear resistance, of a precast prestressed concrete bridge girder is cracked and/or spalled, the Element Behaviour Category shall be taken as being Category E1 in the determination of the shear resistance of the girder.

Commentary 9.5.8.3

Section loss of the transverse shear reinforcing steel typically occurs where the reinforcing steel is bent around the longitudinal reinforcing or prestressing steel and can result in a loss of section, loss of anchorage, and/or a reduction in ductility of the transverse shear reinforcing steel. However, as noted in Section 9.4.5.3, tests on precast concrete channel girders showed that a reduction of 50% in the cross-sectional area of the transverse shear reinforcing steel, where it was bent around the longitudinal reinforcing steel, did not reduce the shear resistances of the girders to below the calculated shear resistances, although the ductility of the failure modes was reduced. These results are expected to be applicable to precast, prestressed concrete girders as they typically have the same or deeper cross-sections than precast reinforced concrete girders.

Section loss of the main longitudinal reinforcing steel and prestressing steel shall be considered in accordance with Section 9.5.8.2 of this Manual.

Cracking and spalling of the concrete around the prestressing steel strands and/or longitudinal reinforcing steel bars can result in a reduction in the anchorage of the prestressing steel strands and/or reinforcing steel bars. See Section 9.3.7.2 of this Manual for a discussion on the loss of anchorage of longitudinal reinforcing steel bars in cracked and/or spalled concrete. Prestressing steel strand lengths where the concrete surrounding the strand is cracked and/or spalled should be assumed to be incapable of providing anchorage to the strand unless the strand is directly above a bearing support and the only damage/deterioration to the concrete is hairline cracking.

Excessive cracking of the web of a precast prestressed concrete bridge girder can result in a reduction in the girder's shear resistance. However, a precast prestressed concrete bridge girder's shear resistance is not expected to be reduced by concrete cracking provided the following requirements are met:

- *the prestressing steel, longitudinal reinforcing steel, and transverse shear reinforcing steel meet the requirements of CSA S6-14 and this Manual including anchorage requirements;*
- *concrete cracking and spalling have not reduced the anchorage of the prestressing steel, longitudinal reinforcing steel and transverse shear reinforcing steel to less than the requirements of CSA S6-14 and this Manual; and*
- *the shear cracks are less than 0.3 mm wide, are not at an angle between 25° and 60° to the horizontal, and do not extend over a vertical or horizontal projection greater than one-half of the girder depth (Habel and Quinton, 2010).*

If concrete cracking and spalling have reduced the anchorage of the prestressing steel, longitudinal reinforcing steel and/or transverse shear reinforcing steel to less than design requirements, shear failure could occur with little warning due to anchorage failure (Mao et al., 1997). In this case cracks of any length or width that are inclined at an angle between 25° and 60° to the horizontal could be providing warning of shear failure.

The above guidance regarding shear cracks may not apply to precast prestressed concrete bridge girders that have been strengthened in shear with external stirrups. This is because the external stirrups are not bonded to the girder. The external stirrup strains resulting from increased crack widths are therefore distributed over the entire length of the external stirrups rather than locally in the vicinity

of the cracks as is the case for internal stirrups. As a result, increases in stirrup stresses due to increasing crack widths are smaller for external stirrups than for internal stirrups. External stirrups are therefore less effective in controlling the growth of the shear crack widths. Therefore, if a girder has been strengthened with external stirrups, the possibility of shear cracks of any width reducing the shear resistance of the girder should be considered. This concern also applies to shear cracks present in a girder prior to shear strengthening. However, provided the external stirrups have been adequately stressed so that they will pick up load with any increase in shear crack width, it is reasonable to expect the shear crack width to remain stable over time, provided the loading on the bridge at the time of strengthening is not increased,

Concrete spalling from the girder web, above the level of the bottom row of longitudinal reinforcing steel or prestressing steel, can reduce the shear resistance of a precast prestressed concrete girder by decreasing the effective web width, "b_v" of the girder or by decreasing the anchorage of the inclined prestressing steel strands.

9.6 **Other Bridges**

9.6.1 Timber Bridges

The resistances of Timber Bridges shall be determined in accordance with the requirements of CSA S6-14 and this Manual unless otherwise approved by AT.

For permit vehicles, the value of "k_d" as specified in CSA S6-14, Section 9.5.3 may be assumed to be 1.1.

9.6.1.1 Timber Bridge Superstructures

Selection of the timber bridge superstructure members requiring load evaluation shall be based on engineering judgment. The following timber superstructure members will typically require load evaluation:

- Timber stringers in moment and shear; and
- Timber decks where the spacing between stringers is greater than 600 mm.

Commentary 9.6.1.1

The following timber superstructure members will typically not require load evaluation:

- *Timber decks where the spacing between stringers is less than 600 mm.*

9.6.1.2 Timber Bridge Substructures

Selection of the timber bridge substructure members requiring load evaluation shall be based on engineering judgment. The following timber substructure members will typically require load evaluation:

- Timber caps in moment, shear and bearing; and
- Timber piles.

Commentary 9.6.1.2

The following timber substructure members will typically not require load evaluation:

- *Timber bracing. These members typically do not require load evaluation as they are not generally subjected to increased loads under increased traffic loading.*

Timber substructures often support other bridge Span Type superstructures such as precast reinforced concrete and precast, prestressed concrete bridge superstructures. The timber substructures should be load evaluated at the same time as the precast reinforced concrete bridge superstructures are load evaluated.

9.6.1.3 Resistance of Damaged or Deteriorated Timber Bridge Members

Any portion of a timber bridge member that has rot shall be ignored when determining the resistance of the member.

Commentary 9.6.1.3

Timber bridge members typically rot from the inside out and the presence of rot will often need to be identified by coring. Results from coring timber members can be found in the AT bridge correspondence file.

Portions of timber members that are not described as being "sound" should be ignored when determining the resistance of timber members.

9.6.2 Steel Rigid Frame Bridges

The guidance provided in this Manual for determining the resistances of Steel Girder Bridges may also be used for Steel Rigid Frame Bridges. In addition, the adequacy of the resistances of the frame legs and their connections to the girders to carry the proposed loads shall be assessed. Axial loads, as well as moments and shears, shall be considered when determining the resistances of the girders and frame legs and their connections.

Commentary 9.6.2

For steel rigid frame bridges, special attention should be given to the connections between the frame girders and legs and how they transfer moment between the girders and legs.

Axial loads in the girders and frame legs will decrease their moment resistance and need to be considered.

9.6.3 Steel Tied Arch Bridges

The guidance provided in this Manual for determining the resistances of Steel Truss Bridges may also be used for Steel Tied Arch Bridges, including for the bridge floorbeams and stringers. Moments, as well as axial loads, shall be considered when determining the resistances of the arch and bottom tie members and their connections. In addition, the adequacy of the buckling resistance of the arches to carry the proposed loads shall be assessed.

Commentary 9.6.3

For steel tied arch bridge's special attention should be given to the determination of the buckling resistances of the tied arches. This may require that the arch analysis consider non-linear geometric effects in accordance with CSA S6-14, Sections 5.12 and 10.15.1.

Moments in the arch and bottom tie beam members will decrease their axial load resistances and need to be considered.

9.6.4 <u>Steel Suspension Bridges</u>

Requirements for the load evaluation of a suspension bridge will be provided by AT on a bridge specific basis.

9.6.5 <u>Reinforced Concrete Arch Bridges</u>

The guidance provided in this Manual for determining the resistances of Reinforced Concrete Bridges may also be used for Reinforced Concrete Arch Bridges, including for the resistances of the bridge floorbeams and stringers. Axial loads, as well as moments and shears, shall be considered when determining the resistances of the arch and bottom tie beam members and their connections. In addition, the adequacy of the buckling resistance of the arches to carry the proposed loads shall be assessed.

Commentary 9.6.5

For reinforced concrete arch bridges, special attention should be given to the determination of the buckling resistances of the tied arches. This may require that the arch analysis consider non-linear geometric effects in accordance with CSA S6-14, Section 5.12.

Compression axial loads in the arch members may increase or decrease the moment resistances of the arch members and need to be considered. Tension axial loads will decrease the moment and shear resistances of the bottom tie beam members and also need to be considered.

9.6.6 <u>Reinforced Concrete Frame Bridges</u>

The guidance provided in this Manual for determining the resistances of Reinforced Concrete Bridges may also be used for Reinforced Concrete Frame Bridges. In accordance with CSA S6-14, Section 3.5.2.2, minimum load factors shall be used with any earth pressures that counteract the live loads. In addition, the adequacy of the resistances of the frame legs and their connections to the girders to carry the proposed loads shall be assessed. Axial loads, as well as moments and shears, shall be considered when determining the resistances of the frame members and their connections.

Commentary 9.6.6

For reinforced concrete frame bridges, special attention should be given to the connections between the frame girders and legs and how they transfer moment between the girders and legs.

Compression axial loads in the frame members may increase or decrease the moment resistances of the frame members and need to be considered.

9.6.7 <u>Cast-in-Place Prestressed Concrete Box Girder Bridges</u>

The guidance provided in this Manual for determining the resistances of Precast Prestressed Concrete Bridges may also be used for Cast-in-Place Prestressed Concrete Box Girder Bridges.

9.6.8 <u>Precast Prestressed Concrete Frame Bridges</u>

The guidance provided in this Manual for determining the resistances of Precast Prestressed Concrete Bridges may also be used for Precast Prestressed Concrete Frame Bridges. In addition, the adequacy of the resistances of the frame legs, the connections of the frame legs to the girders and any hanger-corbel details in the girders to carry the proposed loads shall be assessed. Axial loads, as well as moments and shears, shall be considered when determining the resistances of the girders and frame legs and their connections.

Commentary 9.6.8

> *For precast prestressed concrete frame bridges, special attention should be given to the connections between the frame girders and legs and how they transfer moment between the girders and legs. Also, special attention should be given to the hanger-corbel details used in these bridge types as they may be susceptible to deterioration and/or brittle failure.*

> *Compression axial loads in the girder and frame leg members may increase or decrease the moment resistances of the members and need to be considered. Tension axial loads in the girder and frame leg members will decrease the moment and shear resistances of the members and also need to be considered.*

9.6.9 **Precast Prestressed Concrete Trellis Bridges**

The guidance in this Manual for determining the resistances of Precast Prestressed Concrete Bridges may be used for determining the resistances of the girders of Precast Prestressed Concrete Trellis Bridges. The guidance in this Manual for determining the resistances of Reinforced Concrete Bridges may be used for determining the resistances of the slabs of Precast Prestressed Concrete Trellis Bridges.
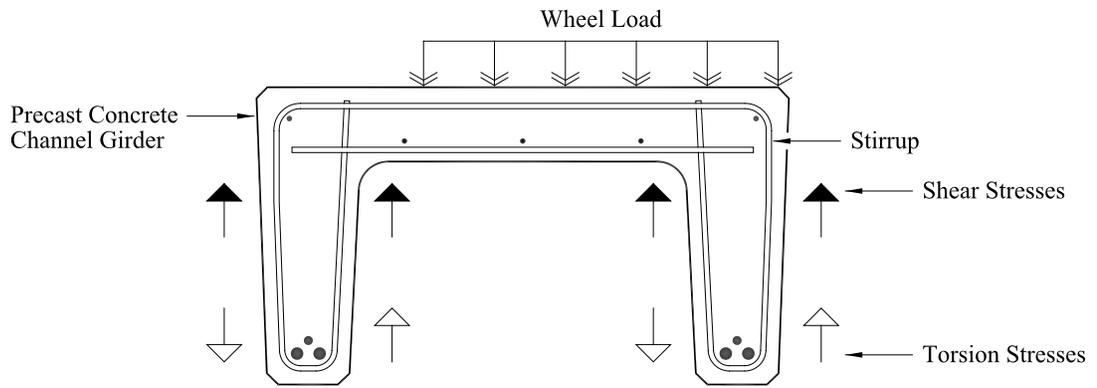
FIGURE 9.4.4.1 Stirrup Anchorage Requirements in Precast Concrete Channel Girders
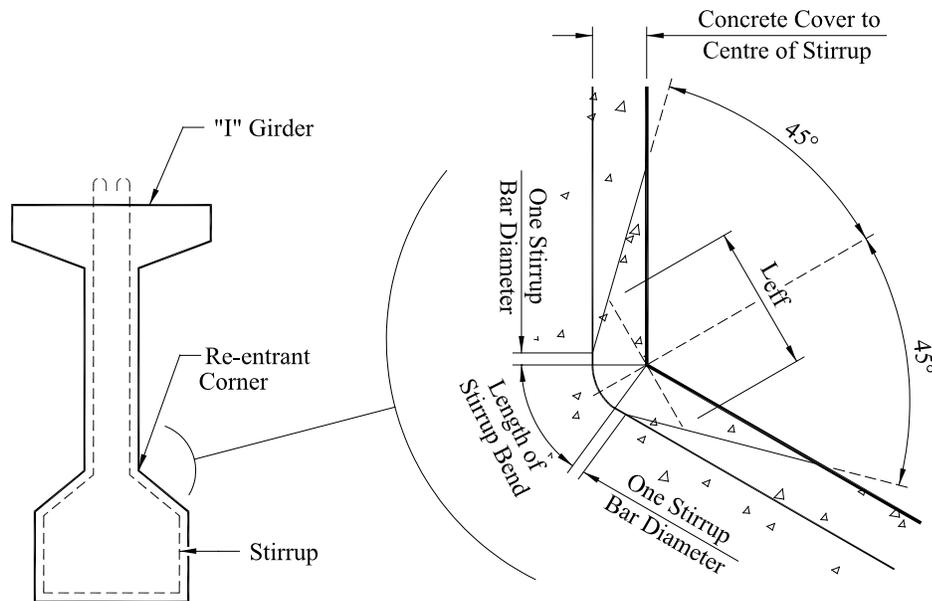
FIGURE 9.5.5.3 Determination of L$_{EFF}$ at Stirrup Re-entrant Corner

10. Bridge Posting

This Section 10 (Bridge Posting) supplements CSA S6-14, Section 14.15 (Live Load Capacity Factor) and Section 14.17 (Bridge Posting). It provides information on:

- The determination of bridge posting requirements for AT bridges based on the live load capacity factors determined for Evaluation Levels 1, 2 and 3.

10.1 Bridge Posting

When the governing live load capacity factor "F" for a bridge, calculated in accordance with Section 14.15 of CSA S6-14 or Section 6.8 of this Manual, is less than 1.0 for any of the following three load evaluation trucks, the bridge shall be posted using a triple posted sign with separate posting limits for Evaluations Levels 1, 2 and 3 (see Figure 10):

- Evaluation Level 3 - 28 tonne CS1 truck (see Section 6.1.3);
- Evaluation Level 2 - 49 tonne CS2 truck (see Section 6.1.2); and/or
- Evaluation Level 1 – 63.5 tonne CS3 truck except 54 tonne CS3 truck for local roads (see Section 6.1.1).

The load posting numbers used for the three Evaluation Levels shall be as follows:

- The load posting number beside the top symbol shall represent the bridge's maximum load carrying capacity for Evaluation Level 3 (CS1 load evaluation truck). This posting number shall be left blank if the bridge's load carrying capacity for the CS1 truck is 28 tonnes or more;
- The load posting number beside the middle symbol shall represent the bridge's maximum load carrying capacity for Evaluation Level 2 (CS2 load evaluation truck). This posting number shall be left blank if the bridge's load carrying capacity for the CS2 truck is 49 tonnes or more; and
- The load posting number beside the bottom symbol shall represent the bridge's maximum load carrying capacity for Evaluation Level 3 (CS3 load evaluation truck). This posting number shall be left blank if the bridge's load carrying capacity for the CS3 truck is 63.5 tonnes or more except for local roads where the posting number shall be left blank if the bridge's load carrying capacity is 54 tonnes or more.

If the Real Truck Rationalization Method described in Section 6.3 of this Manual is used, the three load posting numbers described above shall represent the bridge's maximum load carrying capacities for the most severe actual legal non-permit single unit truck (or single-unit vehicle), semi-trailer truck (or two-unit vehicle), and truck train (or vehicle train) (as shown in *Appendix E* of this Manual) respectively rather than for the CS1, CS2 and CS3 load evaluation trucks.

CSA S6-14, Section 14.17 shall not apply.

Commentary 10.1

A bridge requires load posting whenever its load carrying capacity as determined by a bridge load evaluation or other approved means is insufficient to safely carry a legal non-permit truck. Other possible means that might adequately demonstrate that a bridge has adequate load carrying capacity include a review of its load carrying history (see CSA S6-14, Section 14.17.1) or load testing (see CSA S6-14, Section 14.16).

On the triple posted sign;

- *The top symbol represents a single unit truck (or single-unit vehicle) as defined in Section 6.1 of this Manual;*

- *The middle symbol represents a semi-trailer truck (or two-unit vehicle) as defined in Section 6.1 of this Manual; and*

- *The bottom symbol represents a truck train (or vehicle train) as defined in Section 6.1 of this Manual.*

The operation of trucks in the Province of Alberta is governed by the Alberta Traffic Safety Act and truck sizes and weights are governed by the Commercial Vehicle Dimension and Weight Regulation (Alberta Regulation 315/2002). This regulation requires that:

- *The triple posted sign (as shown in Figure 10) be used to restrict the allowable truck gross vehicle weights on bridges. This is the only sign that is approved in the regulation; and*
- *That the reduction in the truck gross vehicle weight be divided equally among all of the carrying axles (i.e. non-steering axles) of the truck.*

  For example, for a truck with an allowable gross vehicle weight of 49 tonnes and four carrying axles, if the posted CS2 value is 41 tonnes, each truck carrying axle would need to be reduced by (49-41)/4 = 2 tonnes.
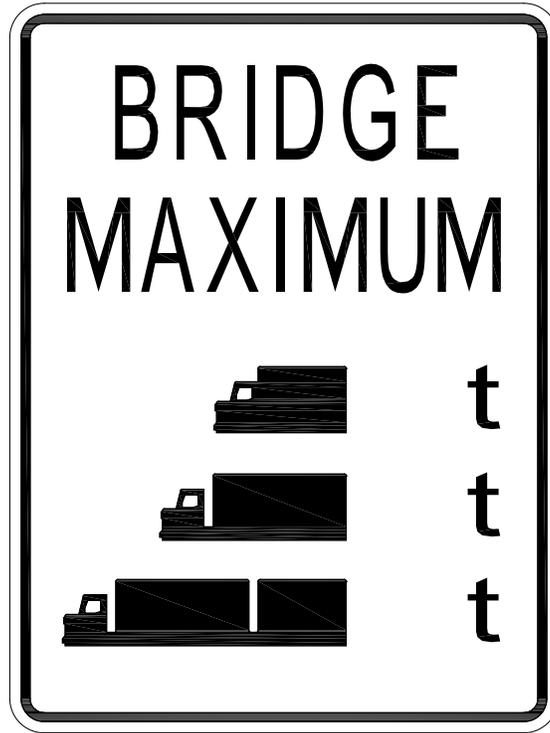
FIGURE 10 Bridge Load Posting Sign

11. Bridge Load Evaluation Report

11.1 Non-Permit Trucks

The results of a bridge load evaluation carried out by a Consultant for a non-permit truck shall be documented in a Bridge Load Evaluation Report. The report shall be stamped by the Bridge Load Evaluator and Bridge Load Evaluation Checker (if required) in accordance with the requirements of Section 2.1 of this Manual.

The following information shall be included in the report:

- A description of the bridge, including a description of the soils the bridge is founded on;
- Historical overview of the bridge, including any modifications, strengthening, overlays, chip seals, new barriers, etc;
- Identification of the sources for the bridge information used in the bridge load evaluation, e.g. bridge drawings used, information obtained from AT's bridge correspondence file, etc;
- Identification of the type of bridge condition inspection carried out, including the date of the inspection, and a summary description of the bridge condition inspection results;
- Identification of the bridge members and bridge foundations that were load evaluated:
- The rationale for not load evaluating the remaining bridge members and bridge foundations;
- Permanent loads, truck loads, including dynamic load allowance, and any other loads used;
- The lateral distributions of the permanent and truck loads used and a description of how they were determined;
- The reliability indices used and how they were determined;
- The permanent and live load factors used;
- The material properties used and how they were obtained;
- The bridge member resistances calculated including any assumptions made concerning the resistances of members not meeting the requirements of CSA S6-14 and this Manual;
- Any assumptions made concerning the resistances of the damaged or deteriorated members (if member damage or deterioration was observed during the bridge inspection) and a discussion of how the calculated member resistances correlate with the observed damage or deterioration;
- The truck load carrying capacities of each load evaluated bridge member in terms of LLRF and tonnes and presented in a tabular format. Note that the LLRF is the same as the live load capacity factor "F" referred to in CSA S6-14, Section 14.15.2.1; and
- The governing bridge truck load carrying capacity in terms of LLRF and tonnes for each truck load evaluated.

The following Appendices shall be included with the report:

- Appendix A identifying the applicable bridge drawings;
- Appendix B containing the bridge condition inspection reports, including any applicable Level 1 BIM Inspection reports, detailed bridge inspection reports and Level 2 BIM inspection reports used in the bridge load evaluation;
- Appendix C containing sketches of the load evaluation vehicles;
- Appendix D containing the Bridge Load Evaluator's design notes and, if applicable, the Bridge Load Evaluation Checker's design notes.

The identification of the applicable bridge drawings in Appendix A shall include a list of all the drawings (including shop drawings, etc.) used in the bridge load evaluation complete with the following:

- Full drawing name for each drawing;
- Full drawing number with the appropriate suffix "P" or "C" or prefix "A" or "C" for each drawing; and
- For standard drawings, the appropriate revision number of each standard drawing.

If a drawing used in the bridge load evaluation is not available in AT's bridge drawing system a copy of the drawing shall be included in Appendix A.

The design notes in Appendix D shall include the following:

- Calculations determining the bridge section properties and dead loads;
- Calculations determining the permanent and truck load effects on the bridge;
- Calculations determining the bridge member resistances; and
- Calculations determining the bridge member truck load carrying capacities in terms of LLRF and tonnes for each truck load evaluated. For members subjected to moment, the member capacities shall be determined at span tenth points as well as at other critical points along the span.

Calculations carried out by structural analysis computer programs shall be documented in a clear and concise manner. The documentation shall include:

- Identification of the program used;
- Identification, in a clear and easily understandable manner, of the inputs entered into the program to obtain the outputs; and
- Identification, in a clear and easily understandable manner, of the relevant outputs from the program.

The design notes may be documented in a spreadsheet provided that the spreadsheet is detailed enough for the calculations to be understood based on a hard copy of the spreadsheet.

The Bridge Load Evaluator's and Bridge Load Evaluation Checker's design notes, including spreadsheets and structural analysis computer program calculations, shall be independent of each other.

Commentary 11.1

The purpose of the Bridge Load Evaluation Report is to document the results of the bridge load evaluation and to demonstrate that the assumptions and analyses that the load evaluation is based on are in accordance with AT's standard practices and reasonable for the bridge's current condition. The Bridge Load Evaluation Report is to be detailed enough for the logic of the load evaluation to be followed without resorting to detailed calculations.

It is assumed that AT will have access to all relevant drawings used for load evaluating bridges in their bridge inventory. As a result, it is not necessary to include these drawings in the Bridge Load Evaluation Report and a list of the drawings is sufficient. However, if a situation arises where the bridge load evaluation uses a drawing that AT does not have access to in their bridge drawing system, the actual drawing shall be included in Appendix A.

11.2 **Permit Trucks**

The results of a bridge load evaluation carried out by a Consultant for a permit truck shall be documented in accordance with the requirements of Section 11.1 of this Manual. In addition, the Bridge Load Evaluation Report shall include the following:

- The rationale for using the Simplified Method of Section 6.9.3 of this Manual if it was used to determine the lateral distribution of the permit truck loads; and
- The conditions under which the permit truck can cross the bridge.

Commentary 11.2

The conditions under which a permit truck can cross a bridge may include requirements to weigh the truck and payload prior to transport, to verify the location of the payload centre of gravity on the transporter, to verify the geometry of the transporter, and to provide independent monitoring of the load when it crosses the bridge. It may also include restrictions on the transverse position of the truck on the bridge, the truck's speed when crossing the bridge and on any other vehicles being on the bridge during the crossing.

11.2.1 <u>Bridges Previously Load Evaluated for a Similar Permit Truck</u>

A Bridge Load Evaluation Report is not required if the Consultant load evaluating the bridge has previously load evaluated a bridge for AT using a permit truck load configuration geometrically similar to the load configuration under consideration. In this case the bridge load evaluation may be based on the results of the previous bridge load evaluation and documented in a Bridge Load Evaluation Letter Report. The Bridge Load Evaluation Letter Report shall be double stamped by the Bridge Load Evaluator and Bridge Load Evaluation Checker.

The following information shall be included in the letter report:

- A reference to the previous Bridge Load Evaluation Report by title and date;
- A confirmation that the bridge condition inspection reports used in the previous bridge load evaluation are still valid and that the details and condition of the bridge have not changed since the issuing of the referenced Bridge Load Evaluation Report. A copy of the bridge condition inspection report that the previous bridge load evaluation was based on shall be included with the letter report;
- A confirmation that the bridge load evaluation requirements for the bridge have not changed since the issuing of the referenced Bridge Load Evaluation Report;
- A comparison of the load configuration under consideration, including a sketch of the load configuration, with the previously load evaluated load configuration, including a sketch of the previously load evaluated load configuration, and the rationale for why the load evaluation results for the previously load evaluated load configuration apply to the load configuration under consideration: and
- The conditions under which the permit truck can cross the bridge.

Commentary 11.2.1

For two load configurations to be considered geometrically similar they must be identical other than for minor dimensional differences in axle spacing between the transporters or between the tractors and transporters.

12. Symbols and Abbreviations

12.1 Symbols

Unless defined otherwise in this Manual, all symbols shall be as defined in CSA S6-14.

12.2 Abbreviations

Unless defined otherwise in this Manual, all abbreviations shall be as defined in CSA S6-14. The following are common abbreviations used in this Manual:

AADT – Average Annual Daily Traffic

AASHTO – American Association of State and Highway Transportation Officials

APEGA – Association of Professional Engineers and Geoscientists of Alberta

AT – Alberta Transportation

BIM – Bridge Inspection & Maintenance System

BIS – Bridge Information System

CSA – Canadian Standards Association

LLRF – Live Load Rating Factor

PCI – Precast/Prestressed Concrete Institute

13. Definitions

Unless defined otherwise in this Manual, all terms shall be as defined in CSA S6-14. The following are common terms used in this Manual:

A-train - A combination of vehicles consisting of a truck tractor, a semi-trailer attached to the rear end of the truck tractor and either:

- a full trailer attached to the rear end of the lead semitrailer by an A-hitch,
- a pony trailer attached to the rear end of the lead semi-trailer,
- a pole trailer attached to the rear end of the lead semi-trailer, or
- a trailer converter dolly attached to the rear end of the lead semi-trailer.

B-train – A combination of vehicles consisting of a truck tractor, a semi-trailer attached to the truck tractor and a semi-trailer attached to the lead semi-trailer by means of a fifth wheel mounted no more than 0.3 metres behind the centre of the last axle on the lead semi-trailer.

Bridge Branch Inventory Stresses – allowable stresses historically used by AT to determine the load carrying capacities of bridges to carry non-permit trucks. Bridge Branch Inventory Stresses exceeded allowable design stresses by up to 50%.

Bridge Branch Overload Stresses – allowable stresses historically used by AT to determine the load carrying capacity of bridges to carry permit trucks. Bridge Branch Overload Stresses exceeded allowable design stresses by up to 65%.

Bridge Load Evaluator – the Engineer responsible for carrying out the bridge load evaluation.

Bridge Load Evaluation Checker – the Engineer responsible for independently checking the bridge load evaluation carried out by the Bridge Load Evaluator.

C-train – A combination of vehicles consisting of a truck tractor, a semi-trailer attached to the truck tractor and a semi-trailer attached to the rear end of the lead semi-trailer by means of a C-hitch;

Class A Inspector – a person certified by AT, as a Class A Inspector, to carry out BIM inspections on AT major bridges, standard bridges and culverts.

Class B Inspector – a person certified by AT, as a Class B Inspector, to carry out BIM inspections on AT standard bridges and culverts only.

Consultant – A person or company that has entered into a consulting services contract with AT for the provision of bridge load evaluation services to AT.

Design Trucks – Trucks which are a conservative representation of the non-permit trucks that a bridge needs to be designed to carry. If the bridge is designed to carry the design truck it will also be able to carry the non-permit trucks the design truck represents.

> *Additional information on the design trucks historically and currently used by AT can be found in Appendix C of this Manual.*

Engineer – A Professional Engineer registered with APEGA.

Interaxle Spacing – The longitudinal distance separating two axles or axle groups, or a steering axle and an axle group, as calculated from the centres of the two adjacent axles.

Legal Trucks – Trucks that can legally travel on Alberta highways in accordance with the Alberta Traffic Safety Act and Regulations, either as non-perrnit trucks or as permit trucks.

Additional information on the history of legal truck weights and configurations in Alberta can be found in Appendix D of this Manual.

Level I BIM Inspection – A general visual bridge condition inspection carried out from the ground without the use of access equipment.

Level II BIM Inspection – A specialized hands-on inspection of a bridge component(s) carried out using specialized tools and/or access equipment and requiring specialized knowledge or training.

Major River Bridge – A bridge that spans over a river and is built from site-specific drawings (ie. not using standard design drawings).

Load Evaluation Trucks - Trucks which are a conservative representation of the non-permit trucks that an existing bridge is required to carry. AT uses the CS1, CS2 and CS3 load evaluation trucks to represent single unit trucks, semi-trailer trucks, and truck trains, respectively.

Non-Permit Trucks – Trucks that have axle weights, axle spacings, gross vehicle weights and overall lengths within the limits prescribed by the Alberta Traffic Safety Act and Regulations and that are therefore allowed to travel on Alberta highways without a permit.

Examples of actual legal non-permit trucks can be found in Appendix E of this Manual.

Permit Trucks - Trucks that have axle weights, axle spacings, gross vehicle weights and/or overall lengths that exceed the limits allowed for non-permit trucks by the Alberta Traffic Safety Act and Regulations and that are, therefore, only allowed to travel on Alberta highways if they are granted a permit by AT.

Pony Trailer – A trailer that is:

- equipped with a drawbar that is rigidly attached to the structure of the trailer, and
- designed and used so that most of its weight and load is carried on its own axles, and includes a trailer that is commonly known as a stiff pole pup trailer.

Scaled Weights – Scaled weights are weights that have been obtained from a Government of Alberta weigh scale.

Semi-trailer – A trailer that:

- has axles only at or near its rear end;
- while being towed, is supported at its front end by the truck tractor or the immediately preceding trailer; and
- when connected to the truck tractor or preceding trailer, is connected by means of a kingpin and a fifth wheel.

Semi-Trailer Truck – as defined in Section 6.1 of this Manual. Also referred to as a "two-unit vehicle".

Single Axle – Means:

- any individual axle, or
- any combination of 2 axles whose centres are less than one metre apart.

Single Unit Truck – as defined in Section 6.1 of this Manual. Also referred to as a "single-unit vehicle".

Span Types – Bridge superstructure types as defined by AT's Bridge Information System.

Standard Permit Truck – A permit truck configuration for which most of the bridges on the provincial highway network have pre-determined bridge load evaluation results. Examples of standard permit truck configurations can be found in Appendix A of this Manual.

Steering Axle – The articulated axle of a commercial vehicle that can be controlled by the operator of the vehicle for the purpose of steering the vehicle.

Tandem Axle Group – An axle group consisting of any 2 consecutive axles on a vehicle where:

- the axles have an axle spread of not less than 1.2 metres and not greater than 1.85 metres, or
- in the case of a trailer manufactured before November 15, 1988, the axles have an axle spread of not less than 1.0 metre and not more than 2.4 metres, but does not include a lift axle in the down position or a single steer axle.

Trailer – A vehicle without motive power that is designed to be towed by another vehicle.

Tridem Axle Group – An axle group, on a trailer, consisting of any 3 consecutive axles of a vehicle where the axles are evenly spaced over a distance of not less than 2.4 metres and not greater than 3.7 metres, but does not include a lift axle in the down position or a single steer axle.

Truck – A motor vehicle designed and intended for the transport of goods or carrying of loads.

Truck Tractor – A truck that may be coupled to a semi-trailer by means of a fifth wheel, but does not include a bed truck, picker truck or winch truck.

Truck Train – as defined in Section 6.1 of this Manual. Also referred to as a "vehicle train".

14. References

1. Avent, R.R. and Mukai, D.J., 1998. Heat Straightening Repairs of Damaged Steel Bridges – A Manual of Practice and Technical Guide, Report No. FHWA-IF-99-004, U.S. Department of Transportation, Federal Highway Administration, Washington, D.C.

2. BIM, 2007. Bridge Inspection and Maintenance System – Level 2 Inspection (Version 1.1). Alberta Transportation.

3. BIM, 2008. Bridge Inspection and Maintenance System – Inspection Manual (Version 3.1). Alberta Transportation.

4. BIS, 1992. *Bridge Information System*. Alberta Transportation.

5. BSDC, 2012. Bridge Structures Design Criteria (Version 7.0). Alberta Transportation

6. CSA, 1988. *CAN/CSA S6-88 (Design of Highway Bridges)*. Canadian Standards Association, Missassauga, Ontario.

7. CSA, 2000. *CAN/CSA S6-00 (Canadian Highway Bridge Design Code)*. Canadian Standards Association, Missassauga, Ontario.

8. CSA, 2006. *CAN/CSA S6-06 (Canadian Highway Bridge Design Code)*. Canadian Standards Association, Missassauga, Ontario.

9. CSA, 2014. *CSA S6-14 (Canadian Highway Bridge Design Code)*. Canadian Standards Association, Missassauga, Ontario.

10. CSA, 2014. *CAN/CSA S16-14 (Design of Steel Structures)*. Canadian Standards Association, Missassauga, Ontario.

11. Connor, R. J., Urban, M. J. and Kaufmann, E. J., 2008. NCHRP 604 (Heat-Straightening Repair of Damaged Steel Bridge Girders: Fatigue and Fracture Performance), Transportation Research Board, Washington, D.C.

12. Ghali, A., 1986. *Strength of Concrete Bridges – Precast Stringers Type G*. University of Calgary, Calgary, Alberta.

13. Habel, K. and Quinton, R., 2010. *NU Girders BIM Rating Recommendations*. AECOM, Edmonton, Alberta.

14. Ibrahim, F., 2009. Load Rating Guidance and Examples for Bolted and Riveted Gusset Plates in Truss Bridges, Federal Highway Administration, Washington, DC.

15. Jaeger, L. G., and Bakht, B., 1989. *Bridge Analysis by Microcomputer*. McGraw-Hill Book Company, New York, New York.

16. Loo, T. and Kornelsen, R. W., 1997. Bridge Rating – New Rating Truck Models – Version 1.00. Alberta Transportation.

17. MacGregor, J. G., 1987. *Load Evaluation of T-Girder Bridges*. MKM Engineering, Edmonton, Alberta.

18. Mao, S., Alexander, S. D. B. and Rogowsky, D. M., 1997, *Tests of Alberta HC Type Stringers*. University of Alberta, Edmonton, Alberta.

19. OHBDC, 1983. Ontario Highway Bridge Design Guide, Ontario Ministry of Transportation, Downsview, Ontario.

20. PCI Committee on Prestress Losses, 1975, *Recommendations for Estimating Prestress Losses,* Precast Concrete Institute, Chicago, Illinois.

21. Ramsay, B., 1997. *Truck Weights in Alberta*. UMA Engineering Ltd, Edmonton, Alberta.

22. Ramsay, B., 2001. Proposed Design and Load Evaluation Trucks for Use with the Canadian Highway Bridge Design Code (CHBDC). UMA Engineering Ltd, Edmonton, Alberta.

23. Ramsay, B., 2009. *Typical Details Final Report*. AECOM, Edmonton, Alberta.

24. Ramsay, B., 2015a, Bridge Load Evaluation Distribution Factors for Shear Connected Precast Prestressed Concrete Girders, AECOM, Edmonton, Alberta.

25.   Ramsay, B., 2015b. Prestress Loss for Shear Connected Precast Prestressed Concrete Girders, AECOM, Edmonton, Alberta.

26.   Waheed, A, Kowal, E. and Loo, T., 2004, *Repair of Bridge Structural Steel Elements Manual,* Alberta Transportation, Edmonton, Alberta.

27.   Web, S.T. and Yura, J.A., 1992. *Evaluation of Bridge Decks as Lateral Bracing for Supporting Steel Stringers, Research Report No. 1239-3.* Centre for Transportation Research, Bureau of Engineering Research, The University of Texas at Austin.

Appendix A
Historical and Current Bridge Load Evaluation Practices in Alberta

1. Historical Bridge Load Evaluation Practices

AT first started carrying out bridge load evaluations in the 1950s on steel truss bridges. This was in response to the trusses having been designed for a maximum truck weight of 18.1 tonne (40 k) or less and the maximum allowable gross vehicle weight for non-permit semi-trailer trucks having been increased from 18.1 tonne (40 k) to 32.6 tonne (72 k) between 1953 and 1959. In 1957 AT adopted Bridge Branch Inventory and Overload Stresses for the load evaluation of steel trusses. Bridge Branch Inventory Stresses increased the allowable stresses in truss members by up to 50% over allowable design stresses for non-permit trucks while Bridge Branch Overload Stresses increased the allowable stresses by up to 65% over allowable design stresses for permit trucks. The maximum non-permit truck weights that the trusses were deemed to be capable of carrying for single unit and semi-trailer trucks respectively were the weights of the H and HS design truck configurations (see Appendix C) that could be carried at Bridge Branch Inventory Stresses. Subsequently in 1988 the CS design truck configuration (see Appendix C) was also used in load evaluations to determine the maximum non-permit truck train weights that the trusses were deemed capable of carrying at Bridge Branch Inventory Stresses.

Increased stresses above allowable design stresses were subsequently used for the load evaluations of steel girder, reinforced concrete and timber bridges. Prestressed concrete girder bridges were load evaluated at the ultimate limit state using reduced load factors for dead and live loads. In all cases the bridges were load evaluated using one or more of the H, HS and CS design trucks.

In 1980, CSA published a supplement to CAN/CSA S6-M78 entitled Clause 12 (Existing Bridge Evaluation). This supplement provided a methodology for load evaluating bridges at the ultimate limit state. The methodology could account for the variability in the magnitudes of the applied loads and member resistances, the levels of bridge redundancy and bridge member ductility, the past behaviour of the bridge and the acceptable probability of failure based on the consequences of failure.

The 1980 Clause 12 supplement to CAN/CSA S6-M78 gave conservative bridge load evaluation results and therefore was not used by AT except for the load evaluation of bridges for some permit trucks. In 1987 AT carried out an internal study to modify the load and resistance factors used in Clause 12 based on statistical information in the literature and correlation with past AT bridge load evaluation practices (Bridge Branch Inventory Stresses). This work was later incorporated into the 1990 version of Clause 12 which was published as a supplement to CAN/CSA S6-88. This 1990 version of Clause 12 was adopted by AT for bridge load evaluation except for the load evaluation of trusses which continued to be load evaluated using Bridge Branch Inventory and Overload Stresses. The maximum allowable truck weights that bridges were deemed to be capable of carrying for single unit, semi-trailer and truck train trucks respectively were the weights of the CS1, CS2 and CS3 design truck configurations that could be carried in accordance with the provisions of Clause 12. The CS1, CS2 and CS3 load evaluation trucks were developed internally by AT based on the CS design truck.

In 2001, AT adopted CAN/CSA S6-00 which replaced the 1990 Clause 12 supplement with Section 14 (Evaluation) of CAN/CSA S6-00. However, AT continued to use the internally developed CS1, CS2 and CS3 load evaluation trucks rather than the CL1, CL2 and CL3 load evaluation trucks that were specified by Section 14. They also continued to load evaluate trusses using Bridge Branch Inventory and Overload Stresses. The primary difference between the CS and CL trucks was that the drive axle was modeled as a tandem axle in the CL trucks and as a single axle in the CS trucks resulting in the CL trucks having heavier drive axles. As a result the CL trucks generally produced higher load effects in short spans than the CS trucks. The live load factors used with the CS trucks for short spans were therefore increased by 10% above those specified by Section 14.

2. AT Current Load Evaluation Practices

2.1 <u>Non-Permit Trucks</u>

Currently AT's bridge load evaluation procedures for non-permit trucks are based on CSA S6-14, Canadian Highway Bridge Design Code, and the CS1, CS2 and CS3 bridge load evaluation trucks. Bridges are posted if they have capacities of less than 28 tonnes for the CS1 truck, 49 tonnes for the CS2 truck and 63.5 tonnes (except 54 tonnes on local roads) for the CS3 truck.

It should be noted that many trusses in Alberta are still posted based on Bridge Branch Inventory Stresses and H, HS and CS load evaluation trucks. Also, other bridge Span Types in Alberta are still posted based on the requirements of CAN/CSA S6-88, Section 12 and CS1, CS2 and CS3 load evaluation trucks.

Currently AT deems bridges designed using a HS25 or larger design truck to be capable of carrying legal non-permit trucks without a bridge load evaluation being carried out.

For Non-Permit trucks bridge load evaluations are typically carried out by a Consultant.

2.2 <u>Permit Trucks</u>

For permit trucks, other than Permit - Controlled (PC) trucks, bridge load evaluations are typically carried out by Transport Engineering Section of AT. Transport Engineering Section has pre-determined bridge load evaluation results for a short list of standard permit trucks for most of the bridges on the provincial highway network. These standard permit trucks have various axle weights and spacings as well as various axle types. Some of these standard permit trucks are shown in Figure A1.

When a carrier applies for a permit the gross vehicle weight of the proposed permit truck configuration is compared with the bridge's load carrying capacity for a similar standard permit truck configuration. If the weight of the proposed permit truck configuration is less than the load carrying capacity of the bridge for the similar standard permit truck a permit is issued.

If the weight of the permit truck exceeds the load capacity of the bridge for the similar standard permit truck, a more detailed load evaluation can be performed by Transport Engineering Section to determine if the permit truck can cross the bridge. This load evaluation can typically justify an increase in the bridge's load carrying capacity through one or more of the following:

- Using the actual truck dimensions;
- Obtaining scaled weights; and/or

- Applying permit conditions.

A bridge load evaluation is also required if the proposed permit truck does not have a configuration similar to that of a standard permit truck.

For Permit – Controlled (PC) trucks, bridge load evaluations are typically carried out by a Consultant retained by the carrier. The carrier is also typically required to provide independent monitoring of the bridge crossings to verify that the conditions of the bridge load evaluation have been met.

TANDEM SEMI

Number of Wheels per Axle	2	4	4		4	4
Axle Spacing (m)		4.8	1.2	7.2	1.2	
Axle Load	0.092W	0.227W	0.227W		0.227w	0.227W

TRIDEM SEMI

Number of Wheels per Axle	2	4	4		4	4	4
Axle Spacing (m)		4.8	1.2	7.2	1.5	1.5	
Axle Load	0.086W	0.22W	0.22W		0.158W	0.158W	0.158W

16 WHEEL TRAILER

Number of Wheels per Axle	2	4	4		8	8
Axle Spacing (m)		4.8	1.2	8.5	1.2	
Axle Load	0.074W	0.185W	0.185W		0.278W	0.278W

8 WHEEL JEEP + 16 WHEEL TRAILER

Number of Wheels per Axle	2	4	4	4	4	8	8
Axle Spacing (m)		4.8	1.2	3.6	1.2	11.0	1.2	
Axle Load	0.024W	0.137W	0.137W	0.137W	0.137W	0.214W	0.214W

8 WHEEL JEEP + 16 WHEEL TRAILER + 8 WHEEL BOOSTER

Number of Wheels per Axle	2	4	4	4	4	8	8	4	4
Axle Spacing (m)		4.8	1.2	3.6	1.2	11.0	1.2	3.6	1.2	
Axle Load	0.05W	0.106W	0.106W	0.106W	0.106W	0.157W	0.157W	0.106W	0.106W

32 WHEELS

Number of Wheels per Axle	2	4	4	8	8	8	8
Axle Spacing (m)		4.8	1.2	3.6	1.2	11.0	1.2	
Axle Load	0.052W	0.118W	0.118W	0.178W	0.178W	0.178W	0.178W

Figure A1 Standard Permit Trucks

32 WHEELS + 8 WHEEL BOOSTER

Number of Wheels per Axle									
2	4	4	8	8		8	8	4	4

Axle Spacing (m): 4.8 | 1.2 | 3.6 | 1.2 | 11.0 | 1.2 | 3.6 | 1.2

Axle Load: 0.048W | 0.096W | 0.096W | 0.142W | 0.142W | 0.142W | 0.142W | 0.096W | 0.096W

48 WHEELS

Number of Wheels per Axle									
2	4	4	8	8		8	8	8	8

Axle Spacing (m): 4.8 | 1.2 | 3.6 | 1.2 | 14.6 | 1.2 | 3.6 | 1.2

Axle Load: 0.034W | 0.087W | 0.087W | 0.132W | 0.132W | 0.132W | 0.132W | 0.132W | 0.132W

64 WHEELS

Number of Wheels per Axle											
2	4	4	8	8	8	8		8	8	8	8

Axle Spacing (m): 4.8 | 1.2 | 3.6 | 1.2 | 3.6 | 1.2 | 17.0 | 1.2 | 3.6 | 1.2

Axle Load: 0.030W | 0.069W | 0.069W | 0.104W | 0.104W | 0.104W | 0.104W | 0.104W | 0.104W | 0.104W | 0.104W

WIDE 64 WHEELS

Number of Wheels per Axle											
2	4	4	8w	8w	8w	8w		8w	8w	8w	8w

Axle Spacing (m): 4.8 | 1.2 | 3.6 | 1.2 | 3.6 | 1.2 | 17.0 | 1.2 | 3.6 | 1.2

Axle Load: 0.030W | 0.069W | 0.069W | 0.104W | 0.104W | 0.104W | 0.104W | 0.104W | 0.104W | 0.104W | 0.104W

96 WHEELS

Number of Wheels per Axle											
2	4	4	12	12	12	12		12	12	12	12

Axle Spacing (m): 4.8 | 1.2 | 3.6 | 1.2 | 3.6 | 1.2 | 17.0 | 1.2 | 3.6 | 1.2

Axle Load: 0.030W | 0.069W | 0.069W | 0.104W | 0.104W | 0.104W | 0.104W | 0.104W | 0.104W | 0.104W | 0.104W

3x16 ROAD SCHEURLE

Number of Wheels per Axle									
2	4	4	8	8	8	8	8	8

Axle Spacing (m): 4.8 | 1.2 | 4.2 | 1.2 | 4.2 | 1.2 | 4.2 | 1.2

Axle Load: 0.042W | 0.104W | 0.104W | 0.125W | 0.125W | 0.125W | 0.125W | 0.125W | 0.125W

Figure A1 (cont) Standard Permit Trucks

4x16 ROAD SCHEURLE

Number of Wheels per Axle	2		4	4		8	8		8	8		8	8		8	8		8	8
Axle Spacing (m)		4.8		1.2	4.2		1.2	4.2		1.2	4.2		1.2	4.2		1.2			

Axle Load	0.030W		0.069W	0.069W	0.104W	0.104W	0.104W	0.104W	0.104W	0.104W	0.104W	0.104W

5x16 ROAD SCHEURLE

Number of Wheels per Axle	2		4	4		8	8		8	8		8	8		8	8		8	8
Axle Spacing (m)		4.8		1.2	4.2		1.2	4.2		1.2	4.2		1.2	4.2		1.2	4.2	1.2

Axle Load	0.030W		0.070W	0.070W	0.083W	0.083W	0.083W	0.083W	0.083W	0.083W	0.083W	0.083W	0.083W	0.083W

MOBILE CRANE M6

Number of Wheels per Axle	2		2	2	2	2	2
Axle Spacing (m)		2.4	1.8	1.8	1.8	1.8	

Axle Load	0.146W	0.146W	0.177W	0.177W	0.177W	0.177W

MOBILE CRANE M52

Number of Wheels per Axle	2		2	2	2	2	4	4
Axle Spacing (m)		2.8	1.6	2.2	1.6	4.2	1.5	

Axle Load	0.150W	0.150W	0.135W	0.135W	0.135W	0.148W	0.148W

MOBILE CRANE M522

Number of Wheels per Axle	2		2	2	2	2	4	4	4	4
Axle Spacing (m)		2.8	1.6	2.2	1.6	4.2	1.5	4.2	1.5	

Axle Load	0.116W	0.116W	0.111W	0.111W	0.112W	0.111W	0.111W	0.106W	0.106W

MOBILE CRANE M523

Number of Wheels per Axle	2		2	2	2	2	4	4	4	4	4
Axle Spacing (m)		2.8	1.6	2.2	1.6	4.2	1.5	4.2	1.5	1.5

Axle Load	0.110W	0.110W	0.094W	0.094W	0.094W	0.105W	0.105W	0.096W	0.096W	0.096W

Figure A1 (cont) Standard Permit Trucks

Two Wheel Axle

Four Wheel Axle

C.   Eight Wheel Axle

Eight Wide Wheel Axle

Twelve Wheel Axle

Figure A1 (cont) Standard Permit Trucks

Appendix B
Historical Bridge Span Types in Alberta

1. General

Table B1 shows the most common bridge Span Types that AT has in its bridge inventory and the years in which these Span Types were primarily constructed. It also shows the years in which each design truck was used.

Table B1 – AT Historical Design Trucks and Bridge Span Types

Bridge Span Type	Year Of Construction							
	< 1950s	1950s	1960s	1970s	1980s	1990s	2000s	2010s
Steel Truss Bridges								
Steel Girder Bridges								
Reinforced Concrete Bridges								
Precast Reinforced Concrete Bridges								
Precast Prestressed Concrete Bridges								
Treated Timber Bridges[1]								
Other Bridges								
Design Truck[2]	HS20			HS25[3]	MS300		CS750	CL800

Notes:
1. While the construction of treated timber superstructures was discontinued in the early 1980s treated timber substructures continued to be constructed with precast prestressed concrete bridge superstructures up to the 1990s.
2. Prior to 1953 a variety of design trucks were used. Trusses were typically designed for either a 12 ton or 20 ton single unit truck as defined by the Engineering Institute of Canada. In the early 1950s a number of bridges were designed for an H15, HS15 or H20 truck rather than for an HS20 truck.
3. During this time period the design truck changed from an HS25 truck to an MS23 truck and then to an MS230 truck. The MS23 and MS230 trucks were metric equivalents of the HS25 truck.

1.1 Steel Truss Bridges

Steel truss bridges were constructed in Alberta starting in the late 1890s and continuing into the 1960s. Most of these bridges were designed using a design truck lighter than the HS20 design truck.

Details of steel truss bridges can typically be found on supplier shop drawings identified by "A" identification numbers in AT's bridge drawing system.

1.2 Steel Girder Bridges

Riveted steel girder bridges were constructed in Alberta starting in the mid-1950s. Starting in 1961 they were replaced with welded steel girder bridges. Welded steel girders are typically "I" girders but can also be box girders. Rolled beam bridges were constructed in Alberta for shorter spans starting in the late 1940s and

continuing into the early 1970s. At that time they were replaced with Welded Wide Flange girders which are classified as welded girders. Most steel girder bridges have been designed for a HS20 or larger design truck.

Steel girder bridges typically have a minimum of 4 girder lines but in the 1960s and early 1970s a number of steel girder bridges with 2 or 3 girder lines were constructed.

Up until the late 1950s steel girder bridges were designed to be non-composite with their decks. Starting in the late 1950s they were designed to be composite with their decks in positive moment regions. Starting in approximately 1990 they were also designed to be composite with their decks in negative moment regions.

Details of steel girder bridges can be found on AT bridge specific drawings.

1.3 Reinforced Concrete Bridges

Reinforced concrete bridges were constructed in Alberta starting in the mid-1950s and continuing until the end of the 1960s. Although the construction of reinforced concrete bridges is no longer common in Alberta, reinforced concrete abutment roof slabs are still being constructed.

Reinforced concrete bridges typically have a minimum of 4 girder lines.

Details of reinforced concrete bridges can be found on AT bridge specific drawings.

1.4 Precast Reinforced Concrete Bridges

Precast reinforced concrete bridges were constructed in Alberta starting in 1950 and continuing until the mid-1970s. These bridges consist of standard weight or light weight precast reinforced concrete channel girders placed side by side. For some Span Types the girders are connected together with steel shear connectors or concrete shear keys, while for other Span Types the girders are unconnected. Most precast reinforced concrete bridges were designed for a HS20 truck but some "PA" girders were designed for a H15 truck and "VH" girders were designed for a HS25 truck.

Table B2 summarizes the properties of the different precast reinforced concrete girder types.

Table B2 - Precast Reinforced Concrete Girder Properties

Girder Type	Span Length (m)	Girder Shape	Girder Depth (mm)	Girder Spacing (mm)	Concrete Type	Girder Connection	Design Loading
PA	6.1 & 8.5	Channel	406	910	Standard Weight	Flange Bolts & Shear Keys	H15 or HS20
PES	6.1 & 8.5	Channel	406	910	Standard Weight	Shear Keys	HS20
PG/GR	6.1 & 8.5	Channel	406	910	Standard or Light Weight	None	HS20
PE	9.1 to 12.8	Channel	610	910	Standard Weight	Shear Keys	HS20
HC	6.1 to 11.6	Channel	508	910	Standard or Light Weight	Steel Shear Connectors	HS20
HH	6.1 & 8.5	Channel	508	910	Standard or Light Weight	None	HS20
VH	6.1 to 11.6	Channel	533 or 559	910	Standard Weight	Steel Shear Connectors	HS25

1.4.1 "PA" Girders

"PA" girders were constructed in Alberta from approximately 1950 to 1952. They are made from standard weight concrete and are connected to each other with flange bolts and shear keys. They have span lengths of 6.1 and 8.5 metres and are 406 mm deep. Details of "PA" girders can be found on AT Standard Drawings S-512, S-512-A, S-525, S-534, S-535, S-726, S-816 and S-817.

1.4.2 "PES" Girders

"PES" girders were constructed in Alberta in approximately 1952. They are made from standard weight concrete and are connected to each other with shear keys. They have span lengths of 6.1 and 8.5 metres and are 406 mm deep. Details of "PES" girders can be found on AT Standard Drawings S-534, S-535, S-814 and S-815.

1.4.3 "PG" Girders

"PG" girders were constructed in Alberta from approximately 1952 to 1961. They are often referred to as "G" girders. They are made from both light weight and standard weight concrete and are not connected to each other. They have span lengths of 6.1 and 8.5 m and are 406 mm deep. "PG" girders that have received a concrete overlay are designated as "PGO" girders. They replaced the "PA" and "PES" girders. Details of "PG" girders can be found on AT Standard Drawings S-534, S-535, S-591, S-591-A, S-665, S-665-A, S-666, S-667, S-667-A and S-668. "GR" Girders were also constructed in Alberta in the early 1960s and are an improved "PG" girder. They are indistinguishable from "PG" girders on the surface and it is possible that in some bridges constructed from salvage girders, "GR" girders have been misidentified as "PG" girders and vice versa. Details of "GR" girders can be found on AT Standard Drawings S-740 to S-743.

1.4.4 <u>"PE" Girders</u>

"PE" girders were constructed in Alberta from approximately 1959 to 1965. They are made from standard weight concrete and are connected to each other with shear keys. They have span lengths of 9.1, 10.7, 12.2 and 12.8 metres and are 610 mm deep. Their longer span lengths supplemented the "PG, "HC" and "HH" girder span lengths. Details of "PE" girders can be found on AT Standard Drawings S-692 to S-699, S-696-A to S-699-A, S-724, S-725 and S-806 to S-811.

1.4.5 <u>"HC" Girders</u>

"HC" girders were constructed in Alberta from approximately 1961 to 1975. They have span lengths of 6.1, 8.5, 10.1 and 11.6 metres and are 508 mm deep. They are made from standard weight concrete except for the 8.5 metre spans which are made from light weight concrete. The girders are connected to each other with steel shear connectors. "HC" girders that have received a concrete overlay are designated as "HCO" girders. They replaced the "PG" girders. Details of "HC" girders can be found on AT Standard Drawings S-780 to S-787, S-785-A, S-791, S-791-70, S-792, S-818, S-819, S-832-69, S-833-69, S-837 to S-839, S-1046 and S-1047.

1.4.6 <u>"HH" Girders</u>

"HH" girders were constructed in Alberta from approximately 1961 to 1966. They have span lengths of 6.1 and 8.5 metres. They are similar to "HC" girders except that they are not connected to each other. "HH" girders that have received a concrete overlay are designated as "HHO" girders. Details of "HH" girders can be found on AT Standard Drawings S-736 to S-739.

1.4.7 <u>"VH" Girders</u>

"VH" girders were constructed in Alberta from approximately 1973 to 1974. They are similar to "HC" girders except that they are 533 or 559 mm deep and were designed for HS25 loading. Details of "VH" girders can be found on AT Standard Drawings S-1087, S-1088 and S-1092 to S-1097.

1.5 <u>Precast Prestressed Concrete Bridges</u>

Precast prestressed concrete bridges have been constructed in Alberta since approximately 1955. They have been designed for a HS20 or larger design truck.

Tables B3, B4 and B5 summarize the properties of the different precast prestressed concrete girder types.

Table B3 - Long Span Precast Prestressed Concrete Girder Properties

Girder Type	Span Length (m)	Girder Shape	Girder Depth (mm)	Girder Spacing (mm)	Concrete Type	Prestressing Type	Girder Connection	Design Loading
PO	18 to 45	"I"	1118 to 1524	Varies	Standard Weight	Wire	Deck & Diaphragms	HS20
FC	10.7 to 36.6	Channel	1041 to 1270	1620	Standard or Semi-Light Weight	7 Wire Strands	Shear Keys & Connector Bolts	HS20
VF	12.2 to 38.7	Channel	1168 to 1371	1625	Semi-Light Weight	7 Wire Strands	Shear Keys & Connector Bolts	HS25
LF	13.7 to 38.1	Channel	1270 & 1371	1625	Semi-Light Weight	7 Wire Strands	Shear Keys & Connector Bolts	HS25
FM	10.0 to 38.1	Channel	1170 to 1370	1625	Semi-Light Weight	7 Wire Strands	Shear Keys & Connector Bolts	MS23
PB	38 to 50	Box	Varies	Varies	Standard Weight	7 Wire Strands	Deck	Varies
DBT	10 to 42	"T"	800 to 1800	1200 to 2000	Standard and/or Semi-Light Weight	7 Wire Strands	Shear Keys or Transverse Post-Tensioning	MS300
DBC	26 to 42	"T"	800 to 1800	1200 to 2000	Standard and/or Semi-Light Weight	7 Wire Strands	Shear Keys or Transverse Post-Tensioning	CS750
CBT	12.7 to 42	"T"	800 to 1800	1200 to 2000	Standard and/or Semi-Light Weight	7 Wire Strands	Deck	MS300
CBC	16 to 40	"T"	800 to 1800	1200 to 2000	Standard and/or Semi-Light Weight	7 Wire Strands	Deck	CS750
NU	20 to 85	"I"	1600 to 2800	Varies	Standard Weight	7 Wire Strands	Deck & Diaphragms	CS 750 or CL800

Table B4 - Intermediate Span Precast Prestressed Concrete Girder Properties

Girder Type	Span Length (m)	Girder Shape	Girder Depth (mm)	Girder Spacing (mm)	Concrete Type	Prestressing Type	Girder Connection	Design Loading
PM	9.1 to 20.1	Box	610 to 737	910	Standard Weight	7 Wire Strands	Shear Keys	HS20
VM	15.2 to 18.3	Box	711 to 813	910	Standard or Semi-Light Weight	7 Wire Strands	Shear Keys	HS25
RD	9.1 to 24.4	Box	610 to 914	1210	Semi-Light Weight	7 Wire Strands	Shear Keys	HS25
RM	12 to 28	Box	610 to 914	1210	Semi-Light Weight	7 Wire Strands	Shear Keys	MS23

Table B5 - Short Span Precast Prestressed Concrete Girder Properties

Girder Type	Span Length (m)	Girder Shape	Girder Depth (mm)	Girder Spacing (mm)	Concrete Type	Prestressing Type	Girder Connection	Design Loading
VS	6.1 to 10.7	Box	508	1210	Semi-Light Weight	7 Wire Strands	Shear Connectors	HS25
SM	6 to 11	Box	510	1210	Semi-Light Weight	7 Wire Strands	Shear Connectors	MS23
SMC	6 to 11	Box	510	1210	Semi-Light Weight	7 Wire Strands	Deck	MS230
SC	6 to 12.8	Box	510	1210	Standard or Semi-Light Weight	7 Wire Strands	Shear Connectors	CS750
SCC	6 to 12	Box	510	1210	Standard or Semi-Light Weight	7 Wire Strands	Deck	CS750
SL	6 to 14	Box	510	1210	Standard Weight	7 Wire Strands	Shear Connectors	CL800
SLC	6 to 20	Box	510 & 700	1210	Standard Weight	7 Wire Strands	Deck	CL800
SLW	6 to 20	Box	510	1210	Standard Weight	7 Wire Strands	Shear Connectors	CL800

1.6 Long Span Precast Prestressed Concrete Bridges

1.6.1 "PO" and "OM" Girders

"PO" girders were constructed in Alberta from approximately 1955 to 1968. They are post-tensioned "I" girders with a cast-in-place concrete deck and steel diaphragms between the girders. They were constructed in span

lengths varying from 18 to 45 metres. They are typically simple span girders although some girders were made continuous for live load with a reinforced concrete deck over the piers. They are made from standard weight concrete and come in depths of 1118, 1321 and 1524 mm. They were designed for a HS20 truck. Details of "PO" girders can be found on AT bridge specific drawings. Prestressing calculations (which contain additional information on the prestressing) can sometimes be found in AT's bridge specific correspondence file. Standard "PO" girder cross-sections are shown in Figure B1.

One "OM" girder bridge was constructed in 1983. An "OM" girder is the metric equivalent of a "PO" girder. Details of "OM" girders can be found on AT bridge specific drawings.

1.6.2 "FC" Girders

"FC" girders were constructed in Alberta from approximately 1963 to 1975. They are pre-tensioned channel girders placed side by side and connected together with connector bolts and shear keys. They were constructed in span lengths varying from 10.7 (35') to 36.6 (120') metres. They are typically simple span girders although some girders were made continuous for live load with a reinforced concrete deck over the piers. They were made from either standard weight or semi-light weight concrete and came in depths of 1041, 1143, 1168 and 1270 mm. They were designed for a HS20 truck. Details of "FC" girders can be found on AT Standard Drawings S-848 to S-857, S-859, S-864, S-874 to S-884, S-874-69, S-876-69, S-874-73 to S-876-73, S-877-72, S-878-72, S-879-73 to S-883-73, S-883-69, S-884-72, S-916, S-916-73, S-921, S-922, S-922-73, S-932, S-963 to S-968, S-971, S-971-73, S-976, S-977, S-977-73, S-979, S-979-71, S-979-73, S-981 to S-984, S-989, S-992, S-1006-70, S-1007-73, S-1012-70, S-1012-73, S-1042 to S-1045, S-1074, S-1076, S-1078 and S-1079.

1.6.3 "VF" Girders

"VF" girders replaced the "FC" girders. They are similar to "FC" girders except that they were designed for a HS25 truck, made from semi-light weight concrete and came in depths of 1168, 1270, 1295 and 1371 mm. They were constructed in Alberta from approximately 1973 to 1976 and in span lengths varying from 12.2 to 38.7 m. Details of "VF" girders can be found on AT Standard Drawings S-1082, S-1090, S-1091, S-1091-74, S-1113, S-1116, S-1117, S-1122. S-1123, S-1126 and S-1128 to S-1130.

1.6.4 "LF" and "FM" Girders

"LF" girders replaced the "VF" girders. They are similar to "VF" girders except that they came in depths of 1270 and 1371 mm. They were constructed in Alberta from approximately 1977 to 1978 and in span lengths varying from 13.7 to 38.1 m. They were designed for a HS25 truck. Details of "LF" girders can be found on AT Standard Drawings S-1275 and S-1279.

"FM" girders are the metric equivalent of "LF" girders and came in depths of 1170, 1270 and 1370 mm. They were constructed in Alberta from 1978 to 1983 and in span lengths varying from 10.0 to 38.1 m. They were designed for a MS23 truck. Details of "FM" girders can be found on AT Standard Drawings S-1361, S-1363, S-1364, S-1376, S-1377, S-1388 to S-1390 and S-1399.

1.6.5 "PB" Girders

"PB" girders have been constructed in Alberta since approximately 1980. They are pre-tensioned box girders fabricated in segments and joined together with cast-in-place concrete joints in the field. The segments are subsequently post-tensioned together to complete one or more continuous bridge spans. They were constructed in span lengths varying from 38 to 50 m. Details of "PB" girders can be found on bridge specific drawings.

1.6.6 <u>"DBT", "DBC", "CBT" and "CBC" Girders</u>

"DBT" girders were constructed in Alberta from approximately 1982 to 1990. They are pre-tensioned "T" shaped girders placed side by side so that their top flanges form the deck surface. They are connected together with shear keys and steel connectors or with shear keys and lateral post-tensioning. They were constructed in top flange widths of 1200 mm, 1500 mm and 2000 mm and depths of 800 mm, 1100 mm, 1500 mm and 1800 mm. They were constructed in span lengths varying from 10 to 42 metres as simple span girders. They are made from standard weight concrete except for the web and bottom flange of the 1800 mm deep girders which were made from semi-light weight concrete. They were designed for a MS300 truck. Details of "DBT" girders can be found on AT Standard Drawings S-1500, S-1502, S-1511, S-1514, S-1521, S-1526, S-1530, S-1533, S-1534, S-1550 to S-1558, S-1580 and S-1588.

"DBC" girders are similar to "DBT" girders except that they were designed for a CS750 truck. They were constructed in Alberta from approximately 1990 to 1996 and in span lengths varying from 26 to 42 m. Details of "DBC" girders can be found on AT Standard Drawings S-1589 and S-1597.

"CBT" girders were constructed in Alberta from approximately 1982 to 1990 and in span lengths varying from 12.7 to 42 m. They are "DBT" girders with a cast-in-place concrete deck. They are typically made continuous either by post-tensioning or by making the reinforced cast-in-place concrete deck continuous over the piers. They were designed for a MS300 truck. Details of "CBT" girders can be found on AT bridge specific drawings.

"CBC" girders are similar to "CBT" except that they were designed for a CS750 truck. They were constructed in Alberta from approximately 1990 to 1996 and in span lengths varying from 16 to 40 m. Details of "CBC" girders can be found on AT bridge specific drawings.

1.6.7 <u>"NU" Girders</u>

"NU" girders have been constructed in Alberta since approximately 1996. They are pre-tensioned "I" girders with a cast-in-place concrete deck and steel bracing between the girders. They are constructed in span lengths varying from 20 to 65 metres. They are typically continuous span girders although some girders are simple spans. The continuous spans are made continuous for live load with either a reinforced concrete deck over the piers or by post-tensioning. They are made from standard weight concrete and come in depths of 1200, 1600, 2000, 2400 and 2800 mm. They were designed for CS750 loading prior to 2002 and for CL800 loading subsequent to 2002. Details of "NU" girders can be found on AT bridge specific drawings.

1.7 <u>Intermediate Span Precast Prestressed Concrete Girder Bridges</u>

1.7.1 <u>"PM" and "VM" Girders</u>

"PM" girders were constructed in Alberta from approximately 1963 to 1973. They are pre-tensioned box girders placed side by side and connected together with shear keys. They were constructed in span lengths varying from 9.1 (30') to 20.1 (66') metres. They were constructed as simple span girders although some girders have subsequently been made continuous for live load through the addition of a reinforced concrete deck over the piers. They are made from standard weight concrete and come in depths of 610, 635, 660, 686 and 737 mm. "PM" girders that have received a concrete overlay are designated as "PMO" girders. They were designed for a HS20 truck. Details of "PM" girders can be found on AT Standard Drawings S-794 to S-801, S-794-69 to S-799-69, S-794-70 to S-801-70, S-794-70-A to S-798-70-A, S-794-73 to S-801-73, S-869, S-869-69, S-869-70, S-869-70-A, S-869-73, S-870, S-870-69, S-870-70, S-870-70-A, S-870-73, S-885, S-885-70, S-885-70-A, S-886,

S-886-70, S-886-70-A, S-907, S-907-69, S-907-70, S-907-70-A, S-908, S-908-69, S-908-70, S-908-70-A, S-926 to S-931, S-1003-69, S-1003-71, S-1003-73, S-1004-69, S-1004-71, S-1004-73, S-1008 and S-1009.

"VM" girders are similar to "PM" girders except that they were designed for a HS25 truck. They were made from either standard weight or semi-light weight concrete and came in depths of 711 and 813 mm. They were constructed in Alberta in 1974. Details of "VM" girders can be found on AT Standard Drawings S-1102 to S-1107, S-1109 and S-1110.

1.7.2 <u>"RD" and "RM" Girders</u>

"RD" girders were constructed in Alberta from approximately 1974 to 1980. They are pre-tensioned box girders placed side by side and connected together with shear keys. They replaced the "PM" and "VM" girders and were constructed in span lengths of 9.1 (30') to 24.4 (80') metres. They were constructed as simple span girders although some girders have subsequently been made continuous for live load through the addition of a reinforced concrete deck over the piers. They are made from semi-light weight concrete and come in depths of 610, 762 and 914 mm. They were designed for a HS25 truck. Details of "RD" Girders can be found on AT Standard Drawings S-1213 to S-1220, S-1223 to S-1230, S-1231-74, S-1232, S-1237 to S-1244 and S-1257 to S-1263.

"RM" girders are the metric equivalent of "RD" girders and were designed for a MS23 truck. They were constructed from 1979 to 1983. Details of "RM" girders can be found on AT bridge specific drawings.

1.8   <u>Short Span Precast Prestressed Concrete Bridges</u>

1.8.1 <u>"VS", "SM" and "SMC" Girders</u>

"VS" girders were constructed in Alberta from approximately 1974 to 1979. They are pre-tensioned box girders placed side by side and connected together with steel shear connectors. They were constructed in span lengths of 6.1, 7.6, 9.1 and 10.7 metres. They were constructed as simple span girders although some girders have subsequently been made continuous for live load through the addition of a reinforced concrete deck over the piers. They are made from semi-light weight concrete and come in a depth of 508 mm. They were designed for a HS25 truck. "VS" girders that have received a concrete overlay are designated as "VSO" girders. Details of "VS" girders can be found on AT Standard Drawings S-1201 to S-1208, S-1201-74-A to S-1208-74-A, S-1205-74 and S-1206-74.

"SM" girders are the metric equivalent of "VS" girders except that they were designed for MS23 loading and came in span lengths of 6, 8, 10 and 11 metres. They were constructed in Alberta from 1978 to 1990. "SM" girders that have received a concrete overlay are designated as "SMO" girders. Details of "SM" girders can be found on AT Standard Drawings S-1301 to S-1312 and S1301-88 to S-1312-88.

"SMC" girders were constructed in Alberta from 1983 to 1990. They are "SM" girders made continuous for live load through the addition of a reinforced concrete deck over the piers. They were designed for MS230 loading. Details of "SMC" girders can be found on AT bridge specific drawings.

1.8.2 <u>"SC" and "SCC" Girders</u>

"SC" girders were constructed in Alberta from approximately 1990 to 2008. They replaced the "SM" girders and were constructed in span lengths of 6, 8, 10 and 12 metres. They were also constructed in span lengths of 6.1, 8.5, 9.1, 10.1, 10.7, 11.6, 12.2 and 12.8 metres so that they could be used as replacement girders on existing

bridges with "PG", "HC", "HH" and "PE" girders. They were constructed as simple span girders. They were made from semi-light weight concrete up to 2007 and then from standard weight concrete. They came in a depth of 510 mm. They were designed for a CS750 truck. Details of "SC" girders can be found on AT Standard Drawings S-1535 to S-1547, S-1559 to S-1579, S-1573-A, S-1581 to S-1584, S-1584-A and S-1656-01 to S-1679-01.

"SCC" girders were constructed in Alberta from approximately 1990 to 2008. They are "SC" girders made continuous for live load through the addition of a reinforced concrete deck over the piers. They replaced the "SMC" girders. They were constructed in span lengths of 10, 12 and 14 metres. They were designed for a CS750 truck. Details of "SCC" girders can be found on AT Standard Drawings S-1619 and S-1630 to S-1636.

1.8.3 "SL", "SLC" and "SLW" Girders

 "SL" girders have been constructed in Alberta since approximately 2008. They replaced the "SC" girders and are intended for use on low volume and lower speed local roads with a gravel wearing surface and where de-icing salts are not applied. They are typically constructed in span lengths of 6, 8, 10, 12 and 14 metres. However, they are also constructed in span lengths of 6.1, 8.5, 9.1, 10.1, 10.7, 11.6, 12.2 and 12.8 metres so that they can be used as replacement girders on existing bridges with "PG", "HC", "HH" and "PE" girders. They are constructed as simple span girders. They are made from standard weight concrete and come in a depth of 510 mm. They are designed for a CL800 truck. Details of "SL" girders can be found on AT Standard Drawings S-1725-07, S-1727-07, S-1729-07, S-1731-07, S-1733-07, S-1735-07, S-1737-07, S-1739-07, S-1741-07, S-1743-07, S-1745-07, S-1747-07 and S-1749-07.

"SLC" girders have been constructed in Alberta since approximately 2008. They are intended for use on higher volume and higher speed highways with an ACP wearing surface. They are SL girders made continuous for live load through the addition of a reinforced concrete deck over the piers. They replaced the "SCC" girders. They come in depths of 510 mm and 700 mm. The 510 mm deep girders are constructed in span lengths of 8, 10, 12, 14 and 16 metres and the 700 mm deep girders are constructed in span lengths of 14, 16, 18 and 20 metres. They are designed for a CL800 truck. Details of "SLC" girders can be found on AT Standard Drawings S-1772-08 to S-1779-08, S-1783-08, S-1789-08 and S-1790-08.

"SLW" girders have been constructed in Alberta since 2012. They are an alternative to "SL" girders and are intended for use on low volume and lower speed local roads with an ACP wearing surface and where de-icing salts are expected. Although they are connected together with shear connectors, the gaps between the girders are also filled with concrete to provide a continuous deck support for the ACP wearing surface. They are simple span girders and are constructed in span lengths of 6, 8, 10, 12 and 14 metres. They are made from standard weight concrete and come in a depth of 510 mm. They are designed for a CL800 truck. Details of "SLW" girders can be found on AT Standard Drawings S-1816-12 to S-1837-12.

1.9 Treated Timber Bridges

Treated timber bridges consist of timber stringer and deck superstructures and timber pile bent substructures and were constructed from the 1920s to the early 1980s. Timber pile bent substructures were also commonly used with standard precast or prestressed concrete girder bridges up until the 1990s. These bridges were typically designed for a HS20 or lighter design truck although a metric standard timber bridge was designed for a MS23 truck. Details of standard timber bridges can be found on AT Standard Drawings S-500 to S-504, S-502-A, S-503-A, S-504-A, S-509, S-512-A, S-519, S-519-71, S-546, S-553, S-626, S-673, S-813, S-860,

S-860-69, S-860-70, S-860-73, S-860-A, S-862, S-862-70, S-1058, S-1111, S-1112, S-1119, S-1120, S-1419, S-1420, S-1421, S-1446, S-1605 and S-1606. However bridge specific details should be confirmed in the field.

1.10 Other Bridges

Other bridges are bridges that have Span Types that are unusual in AT's bridge inventory. They include steel rigid frame bridges, steel tied arch bridges, steel suspension bridges, reinforced concrete arch bridges, reinforced concrete frame bridges, cast-in-place prestressed concrete box girder bridges, prestressed concrete frame bridges and prestressed concrete trellis bridges. Details of "Other Bridges" can be found on AT bridge specific drawings.

a) Type O-44 Girder Dimensions

b) Type O-52 Girder Dimensions

c) Type O-60 Girder Dimensions
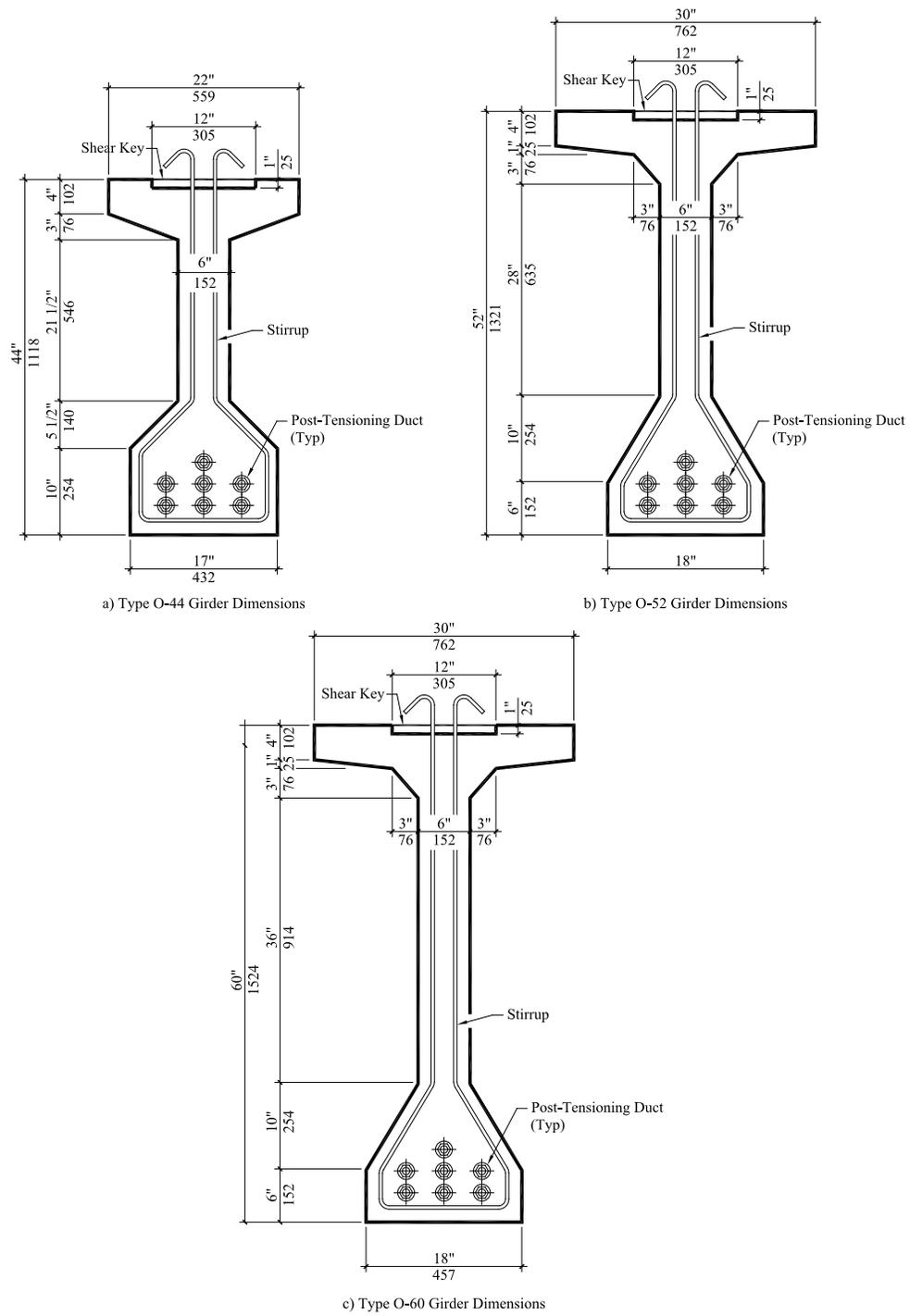
Figure B1 Type O Girder Properties

Appendix C
Historical Bridge Design Trucks in Alberta

Bridge design trucks historically used by AT are described below and shown in Figure C1.

Prior to 1953 a variety of bridge design trucks were used. Trusses were typically designed for either a 12 ton or 20 ton single unit truck as defined by the Engineering Institute of Canada. In the early 1950s a number of bridges were designed for a H15, HS15 or H20 truck rather than for a HS20 truck.

In 1954 AT adopted the HS20 design truck in response to an increase in the legal gross vehicle weight of non-permit semi-trailer trucks to 25.4 tonnes (56 kips).

In 1973 AT adopted the HS25 design truck in response to an increase in the legal gross vehicle weight of non-permit semi-trailer trucks to 36.3 tonnes (80 kips) and the legalizing of non-permit truck trains with a legal gross vehicle weight of 49.9 tonnes (110 kips). The adoption of the HS25 semi-trailer design truck configuration to model legal non-permit truck trains resulted in conservative designs for short span bridges.

In 1978 AT adopted first the MS23 design truck and then the MS230 design truck. These design trucks were the metric equivalent of the HS25 design truck.

In 1982 AT adopted the MS300 design truck in response to an increase in the legal gross vehicle weight of non-permit truck trains to 53.5 tonnes and a decrease in the minimum allowable spacing between tandem axle groups to 5.0 m.

In 1988 AT adopted the CS750 design truck in response to the legal gross vehicle weights of non-permit single unit trucks, semi-trailer trucks and truck trains being increased to 24.3 tonnes, 46.5 tonnes and 62.5 tonnes respectively. The CS750 truck train design truck more consistently modeled legal non-permit trucks than the previous MS300 semi-trailer design truck over all bridge span lengths.

In 2001 AT adopted the CL800 design truck. The CL800 design truck was intended to produce bridge designs similar to those produced by the CS750 design truck.

H15

Axle Spacing (m)

Gross Vehicle Weight = 30 k

Axle Load (k)

4.27

6
(2.7t)

24
(10.8t)

HS15

Axle Spacing (m)

Gross Vehicle Weight = 54 k

Axle Load (k)

4.27   4.27

6
(2.7t)

24
(10.8t)

24
(10.8t)

H20

Axle Spacing (m)

Gross Vehicle Weight = 40 k

Axle Load (k)

4.27

8
(3.6t)

32
(14.5t)

HS20

Axle Spacing (m)

Gross Vehicle Weight = 72 k

Axle Load (k)

4.27   4.27

8
(3.6t)

32
(14.5t)

32
(14.5t)

HS25

Axle Spacing (m)

Gross Vehicle Weight = 90 k

Axle Load (k)

4.27   4.27

10
(4.5t)

40
(18.15t)

40
(18.15t)

MS23

Axle Spacing (m)

Gross Vehicle Weight = 41.4 t

Axle Load (t)

4.25   4.25

4.6

18.4

18.4

Figure C1 Historical Alberta Transportation Bridge Design Trucks

MS230

Axle Spacing (m)   4.25   4.25

Gross Vehicle Weight = 414 kN

Axle Load (kN)   46 (4.7t)   184 (18.75t)   184 (18.75t)

MS300

Axle Spacing (m)   4.25   4.25

Gross Vehicle Weight = 540 kN

Axle Load (k)   60 (6.1t)   240 (24.5t)   240 (24.5t)

CS750

Axle Spacing (m)   4.0   6.0   6.0

Gross Vehicle Weight = 750 kN

Axle Load (kN)   75 (7.7t)   225 (22.9t)   225 (22.9t)   225 (22.9t)

CL800

Axle Spacing (m)   3.6   1.2   6.6   6.6

Gross Vehicle Weight = 800 kN

Axle Load (kN)   64   160 160   224   192
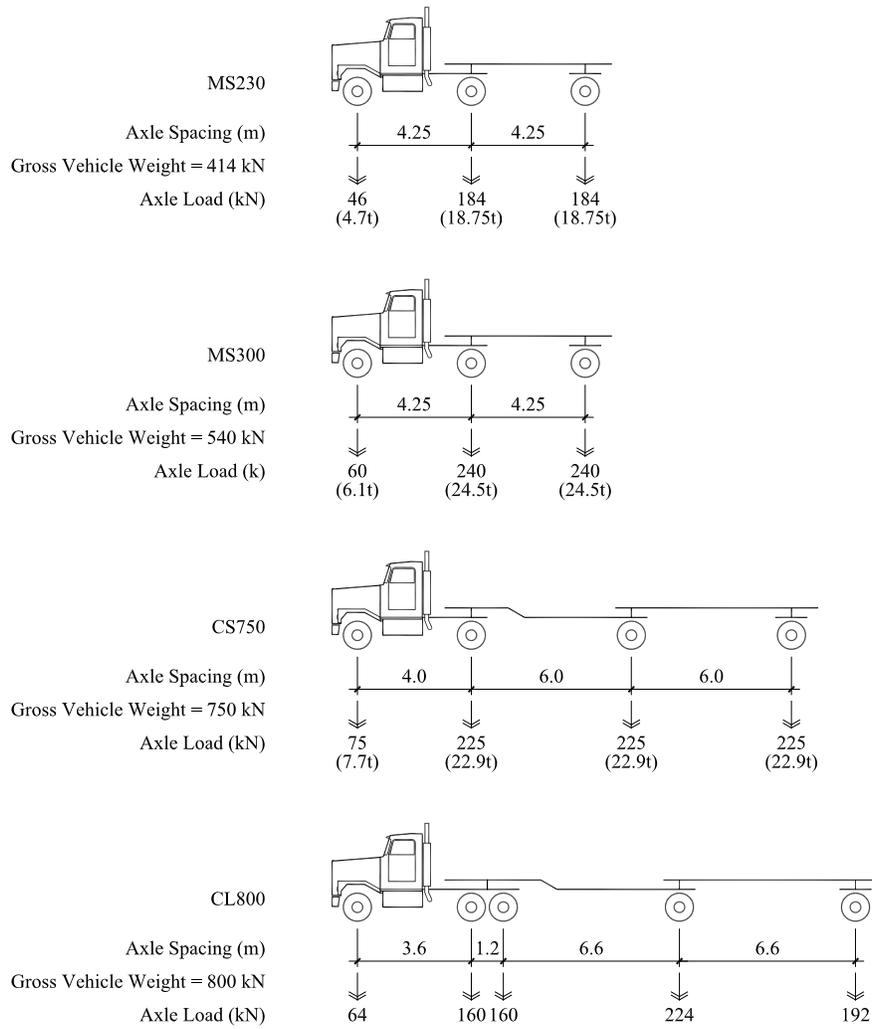
Figure C1 (cont) Historical Alberta Transportation Bridge Design Trucks

Appendix D
Historical Legal Truck Loads in Alberta

Historical legal truck loads in Alberta are described below and shown in Figures D1 to D3.

1. Pre-1950s to 1960s

1.1 Non-Permit Trucks

Prior to World War II and in the immediate post war period, equipment was light and could be transported on a regular flat-bed semi-trailer. The maximum legal gross truck weight for a non-permit single unit or semi-trailer truck was 18.1 tonnes (40 kips). In response to the need to transport heavier equipment the maximum legal gross truck weight for a semi-trailer truck was increased to 25.4 tonnes (56 kips) in 1954 and 32.6 tonnes (72 kips) in 1959. The maximum tandem axle weight was 14.5 t (32 kips).

1.2 Permit Trucks

Permit truck gross vehicle weights were allowed to exceed the non-permit truck gross vehicle weight limits. However, permit axle weights were not allowed to exceed the limits for non-permit trucks unless the road bed was frozen to a safe depth. In response to the need to transport heavier equipment, specialized transportation equipment was developed to keep axle weights within acceptable limits. These early developments included the development of permit trucks with jeeps and boosters as well as with specialized axle groups such as the 16 wheel tandem.

As permit truck loads became heavier in the 1960s there was increased demand for new innovations. The 24 wheel tandem was developed in western Canada for permit trucks on highways in the Prairie Provinces as it was critical to reduce the axle weights to prevent pavement damage. At the same time, several carriers acquired first generation hydraulic trailers (Scheuerle) to move very heavy loads.   Due to the high tare weight of these trailers, and with the heavy concentration of load on the closely spaced axles, they were not conducive for crossing bridges and were used mainly for local moves.

2. 1970s to Present

2.1 Non-Permit Trucks

In 1974 the maximum legal gross vehicle weight for non-permit semi-trailer trucks was increased to 36.3 tonnes and the maximum legal gross vehicle weight for non-permit truck trains was increased to 49.9 tonnes (110 kips). By 1982 the maximum legal gross vehicle weight for non-permit semi-trailer trucks had been increased from 36.3 tonnes to 37.5 tonnes and the maximum gross vehicle weight for non-permit truck trains from 49.9 tonnes to 53.5 tonnes. Also the minimum allowable spacing between tandem axle groups was reduced to 5.0 m.

In 1988, as part of the Memorandum of Understanding on Vehicle Weights and Dimensions signed by all the Canadian Provinces, the maximum legal gross vehicle weights for non-permit single unit trucks, semi-trailer trucks and truck trains were increased to 24.3 tonnes, 46.5 tonnes and 62.5 tonnes respectively.

In 2001 the maximum legal gross vehicle weight for non-permit truck trains was increased to 63.5 tonnes. Also, 23 tonne tridem drive axles were first allowed. This increased the maximum legal gross vehicle weights for non-permit single unit trucks and semi-trailer trucks to 30.3 tonnes and 54.3 tonnes respectively.

In 2012 the maximum legal axle weight for non-permit tractor steering axles was increased from 5.5 tonnes to 6.0 tonnes.

2.2 Permit Trucks

Up until the late 1970's most industrial plants were built on site. This required only the movement of construction equipment and the odd super heavy pressure vessel. The pressure vessels were typically built out of province and shipped by rail to a siding near the construction site, resulting in a short move on trucks. This was typical of the early oil sands developments such as Syncrude and Suncor.

Beginning in the early 1980's there was a shift to the construction of modules near Edmonton and Calgary where there was closer access to labour. However, this involved the transportation of large and heavy modules from the fabrication plants to the construction sites. This led to the construction of the high load corridor and to the designation of heavy haul routes to the Fort McMurray area as well as to the west and north-west parts of the province. This also contributed to the development of lighter and more efficient trucks such as the second generation hydraulic trailers and the 24 wheel tridems.

2.3 Log Haul Industry

In the late 1960's, the log haul industry received a concession from AT to allow winter axle weights at the same magnitude as were allowed for the heavy haul industry. There was little control of the loads and the logging trucks could travel on any road subject to load restricted bridges.

In the late 1980's following the collapse of two bridges in the Peace River area, the winter log haul program was revised. The bridges which had been recently load evaluated for an increase in legal weight to 62.5 tonnes for truck trains were also load evaluated for typical logging trucks. As logs are a divisible load, the PA loading category was used and where there were bridge load capacity concerns, the winter weights were reduced. Color coded maps were produced for each mill showing the maximum allowable logging truck gross vehicle weights for the different routes, with the color of the route being determined by the lowest capacity bridge on that route.

GROSS
VEHICLE WEIGHT
(GVW)

1953 - 1954

40k
(18.1t)

Axle Spacing (ft)
Axle Spacing (m)

10'*
(3.05)

3'-6"
(1.07)

8k
(3.6)

32k
(14.5)

Axle Load (t)

1954 - 1974
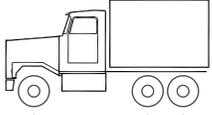
41k
(18.6t)

Axle Spacing (ft)
Axle Spacing (m)

10'*
(3.05)

3'-6"
(1.07)

9k
(4.1)

32k
(14.5)

Axle Load (t)

1974 - 1978
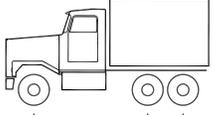
45k
(20.4t)

Axle Spacing (ft)
Axle Spacing (m)

10'*
(3.05)

3'-6"
(1.07)

10k
(4.5)

35k
(15.9)

Axle Load (t)

1978 - 1979

20.5t

Axle Spacing (m)

3.0

1.0

4.5

16

Axle Load (t)

1979 - 1982

23.3t

Axle Spacing (m)

3.6

1.0

7.3

16

Axle Load (t)

* Minimum spacing between steering wheel and adjacent drive axle
assumed to be 10' (3.05m) although not given in the regulations.

Figure D1 Historical Non-Permit Legal Truck Weights and Dimensions - Single Unit Trucks

GROSS
VEHICLE WEIGHT
(GVW)

1982 - 1988   23.3t

Axle Spacing (m)   3.0   1.0

Axle Load (t)   7.3   16

1988 - Present   24.3t

Axle Spacing (m)   3.0   1.2

Axle Load (t)   7.3   17

2001 - Present   30.3t

Axle Spacing (m)   5.4   1.2 1.2

Axle Load (t)   7.3   23
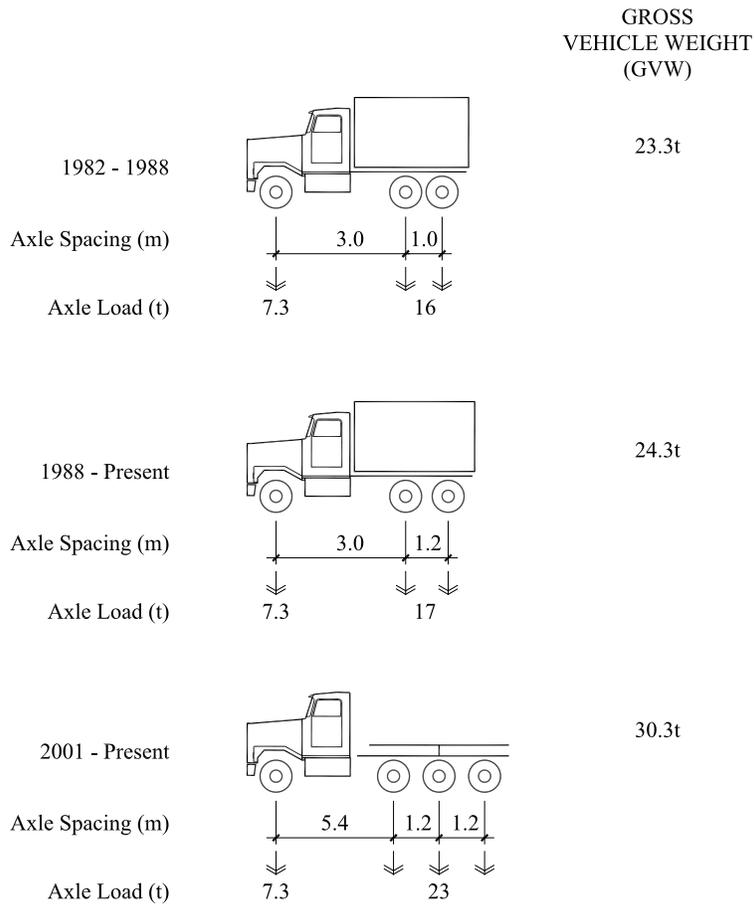
Figure D1 (cont) Historical Non-Permit Legal Truck Weights and Dimensions - Single Unit Trucks

GROSS
VEHICLE WEIGHT
(GVW)
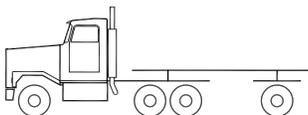
1954 ** - 1957   56k
                                                                                        (25.4t)

Axle Spacing (ft)   10'*   3'-6"   10'*
Axle Spacing (m)   (3.05)   (1.07)   (3.05)

                                8k   32k   16k
Axle Load (t)   (3.6)   (14.5)   (7.3)
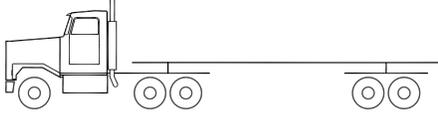
1957 - 1959   62k
                                                                                       (28.1t)

Axle Spacing (ft)   10'*   3'-6"   20'   3'-6"
Axle Spacing (m)   (3.05)   (1.07)   (6.10)   (1.07)

                                8k   32k   22k
Axle Load (t)   (3.6)   (14.5)   (10.0)

1959 - 1972   72k
                                                                                       (32.6t)
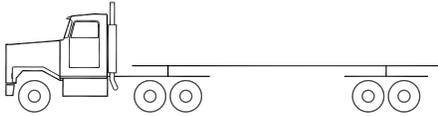
Axle Spacing (ft)   10'*   3'-6"   20'   3'-6"
Axle Spacing (m)   (3.05) (1.07)   (6.10)   (1.07)

                                8k   32k   32k
Axle Load (t)   (3.6)   (14.5)   (14.5)

1972 - 1974   72k
                                                                                       (32.6t)
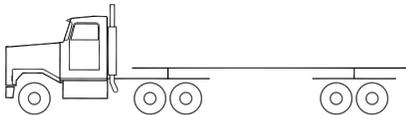
Axle Spacing (ft)   10'*   3'-6"   16'-6"   3'-6"
Axle Spacing (m)   (3.05) (1.07)   (5.03)   (1.07)

                                8k   32k   32k
Axle Load (t)   (3.6)   (14.5)   (14.5)

1974 - 1978   80k
                                                                                       (36.3t)

Axle Spacing (ft)   10'*   3'-6"   19'   3'-6"
Axle Spacing (m)   (3.05) (1.07)   (5.79)   (1.07)

                               10k   35k   35k
Axle Load (t)   (4.5)   (15.9)   (15.9)

* Minimum spacing between steering wheel and adjacent drive axle
   assumed to be 10' (3.05m) although not given in the regulations.

** Prior to 1954, semi-trailers were held to the same maximum gross vehicle
    weight as single unit trucks.

Figure D2 Historical Non-Permit Legal Truck Weights and Dimensions – Semi-Trailer Trucks

GROSS
VEHICLE WEIGHT
(GVW)

1978 - 1979　　　　　　　　　　　　　　　　　　　　　　　　36.5t

Axle Spacing (m)　　　3.0　　1.0　　5.8　　1.0

Axle Load (t)　　4.5t　　　16t　　　　16t

1979 - 1982　　　　　　　　　　　　　　　　　　　　　　　　37.5t

Axle Spacing (m)　　　3.0　　1.0　　5.7　　1.0

Axle Load (t)　　5.5　　　16　　　　16

1982 - 1988　　　　　　　　　　　　　　　　　　　　　　　　37.5t

Axle Spacing (m)　　　3.0　　1.0　　5.0　　1.0

Axle Load (t)　　5.5　　　16　　　　16

1988 - 2001　　　　　　　　　　　　　　　　　　　　　　　　46.5t

Axle Spacing (m)　　　3.0　　1.2　　5.5　　1.8　　1.8

Axle Load (t)　　5.5　　　17　　　　　24

2001 - 2012　　　　　　　　　　　　　　　　　　　　　　　　46.5t

Axle Spacing (m)　　　3.0　　1.2　　5.5　　1.5　　1.5

Axle Load (t)　　5.5　　　17　　　　　24

2001 - Present　　　　　　　　　　　　　　　　　　　　　　54.3t

Axle Spacing (m)　　　5.4　　1.2　1.2　　5.5　　1.5　　1.5

Axle Load (t)　　7.3　　　　23　　　　　　24

2012 - Present　　　　　　　　　　　　　　　　　　　　　　47.0t

Axle Spacing (m)　　　3.0　　1.2　　5.5　　1.5　　1.5

Axle Load (t)　　6.0　　　17　　　　　24

Figure D2 (cont) Historical Non-Permit Legal Truck Weights and Dimensions – Semi-Trailer Trucks
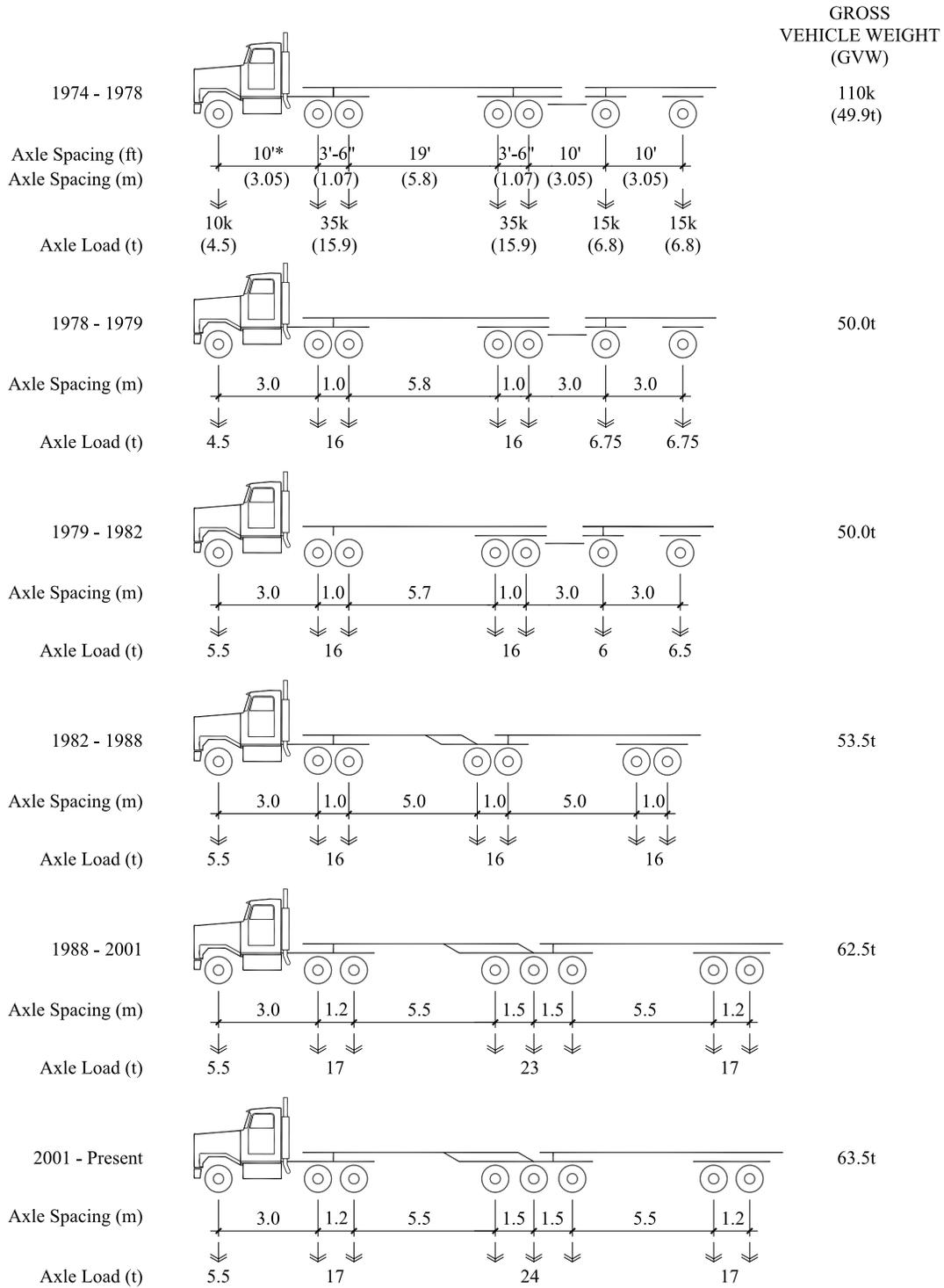
GROSS
VEHICLE WEIGHT
(GVW)

1974 - 1978

Axle Spacing (ft)   10'*   3'-6"   19'   3'-6"   10'   10'
Axle Spacing (m)   (3.05)   (1.07)   (5.8)   (1.07)   (3.05)   (3.05)

            10k   35k   35k   15k   15k
Axle Load (t)   (4.5)   (15.9)   (15.9)   (6.8)   (6.8)

110k
(49.9t)

1978 - 1979

Axle Spacing (m)   3.0   1.0   5.8   1.0   3.0   3.0

Axle Load (t)   4.5   16   16   6.75   6.75

50.0t

1979 - 1982

Axle Spacing (m)   3.0   1.0   5.7   1.0   3.0   3.0

Axle Load (t)   5.5   16   16   6   6.5

50.0t

1982 - 1988

Axle Spacing (m)   3.0   1.0   5.0   1.0   5.0   1.0

Axle Load (t)   5.5   16   16   16

53.5t

1988 - 2001

Axle Spacing (m)   3.0   1.2   5.5   1.5   1.5   5.5   1.2

Axle Load (t)   5.5   17   23   17

62.5t

2001 - Present

Axle Spacing (m)   3.0   1.2   5.5   1.5   1.5   5.5   1.2

Axle Load (t)   5.5   17   24   17

63.5t

* Minimum spacing between steering wheel and adjacent drive axle
assumed to be 10' (3.05m) although not given in the regulations.

Figure D3 Historical Non-Permit Legal Truck Weights and Dimensions – Truck Trains

Appendix E
Real Truck Rationalization Method

The Real Truck Rationalization Method allows the load capacity of a bridge to be based on real legal non-permit trucks rather than on the CS1, CS2 and CS3 trucks. The use of this Method can result in an increase in the calculated load carrying capacity of a bridge as the CS1, CS2 and CS3 trucks are conservative representations of real legal non-permit trucks. The real legal non-permit single unit truck, semi-trailer truck and truck train configurations that need to be considered in the bridge load evaluation are shown in Figures E2 to E4 respectively. Figure E1 (Real Truck Rationalization Form) can be used to assist in the carrying out of the Real Truck Rationalization Method.

The Real Truck Rationalization Method shall be carried out as follows:

- Load evaluate the bridge for the CS1, CS2 and CS3 trucks. The live load factors used in the load evaluation shall be based on Sections 8.1 and 8.3.1.1 of this Manual. The dynamic load allowances shall be based on Section 6.5.1.1 of this Manual;
- Record the Calculated Load Capacities on Form E1 along with the Live Load Factors "α_L" and Dynamic Load Allowances "DLA" used;
- Load evaluate the bridge for Real Trucks 101 to 104, 201 to 207, 301 to 307 and 401. The live load factors used in the load evaluation shall be based on CSA S6-14, Table 14.9 for "Other Spans" and on Section 8.3.1.2 of this Manual for "Short Spans". The dynamic load allowances shall be based on Section 6.5.1.2 of this Manual;
- Record the Live Load Factors "α_L" and Dynamic Load Allowances "DLA" used and the Calculated Load Capacities for the Real Trucks in Columns "B" to "D" of Form E1 respectively;
- Determine the "LLRF" for the Real Trucks by dividing the Calculated Load Capacities in Column "D" of Form E1 by the Real Truck Weights in Column "A" and record the results in Column "E";
- If all of the "LLRF" values in Column "E" of Form E1 are greater than 1.00 the bridge has adequate capacity to carry legal non-permit loads and does not require posting; and
- If all of the "LLRF" values in Column "E" of Form E1 are not greater than 1.00 the bridge shall be posted in accordance with the requirements of Section 10.1 of this Manual and the following;

  - The load posting number beside the top symbol of the triple posting sign shall be the lowest Calculated Load Capacity for those of Real Trucks 101 to 104 that have a "LLRF" less than 1.00. If none of Real Trucks 101 to 104 have a "LLRF" less than 1.00 the top posting number shall be left blank;
  - The load posting number beside the middle symbol of the triple posting sign shall be the lowest Calculated Load Capacity for those of Real Trucks 201 to 207 and 401 that have a "LLRF" less than 1.00. If none of Real Trucks 201 to 207 and 401 have a "LLRF" less than 1.00 the middle posting number shall be left blank; and
  - The load posting number beside the bottom symbol of the triple posting sign shall be the lowest Calculated Load Capacity for those of Real Trucks 301 to 307 that have a "LLRF" less than 1.00. If none of Real Trucks 301 to 307 have a "LLRF" less than 1.00 the bottom posting number shall be left blank.

Note that for Real Trucks 101 to 104, the only trucks that need to be load evaluated are those where:

$$\begin{pmatrix} Real\ Truck \\ Weight \end{pmatrix} x \begin{pmatrix} Real\ Truck \\ \alpha_L \end{pmatrix} x \begin{pmatrix} Real\ Truck \\ DLA \end{pmatrix} > \begin{pmatrix} CS1 \\ Calculated\ Load \\ Capacity \end{pmatrix} x \begin{pmatrix} CS1\ Truck \\ \alpha_L \end{pmatrix} x \begin{pmatrix} CS1\ Truck \\ DLA \end{pmatrix}$$

Similarly, for Real Trucks 201 to 207 and 401, the only trucks that need to be load evaluated are those where:

$$\begin{pmatrix} Real\ Truck \\ Weight \end{pmatrix} x \begin{pmatrix} Real\ Truck \\ \alpha_L \end{pmatrix} x \begin{pmatrix} Real\ Truck \\ DLA \end{pmatrix} > \begin{pmatrix} CS2 \\ Calculated\ Load \\ Capacity \end{pmatrix} x \begin{pmatrix} CS2\ Truck \\ \alpha_L \end{pmatrix} x \begin{pmatrix} CS2\ Truck \\ DLA \end{pmatrix}$$

Finally, for Real Trucks 301 to 307, the only trucks that need to be load evaluated are those where:

$$\begin{pmatrix} Real\ Truck \\ Weight \end{pmatrix} x \begin{pmatrix} Real\ Truck \\ \alpha_L \end{pmatrix} x \begin{pmatrix} Real\ Truck \\ DLA \end{pmatrix} > \begin{pmatrix} CS3 \\ Calculated\ Load \\ Capacity \end{pmatrix} x \begin{pmatrix} CS3\ Truck \\ \alpha_L \end{pmatrix} x \begin{pmatrix} CS3\ Truck \\ DLA \end{pmatrix}$$

ADJUSTMENTS TO BRIDGE LOAD CAPACITIES
USING REAL TRUCK RATIONALIZATION

File _____
Date _____
By _____
Hwy _____

Bridge Name_____
Span Lengths and Types _____

Calculated Load Capacities	Load Effect Type	Location	Live Load Factor	Dynamic Load Allowance
CS1_____	_____	_____	_____	_____
CS2_____	_____	_____	_____	_____
CS3_____	_____	_____	_____	_____

Ⓔ

Ⓐ   Ⓑ   Ⓒ   Ⓓ   Ⓓ ÷ Ⓐ

Real Truck Type	Real Truck Weight	Real Truck Live Load Factor	Real Truck Dynamic Load Allowance	Calculated Load Capacity for Real Truck	LLRF	Remarks
CS1	28.0					
101	16.4					
102	24.3					
103	30.6					
104	30.3					
CS2	49.0					
201	44.0					
202	47.0					
203	40.0					
204	24.2					
205	32.1					
206	54.3					
207	51.3					
401	50.4					
CS3	63.5					
301	61.0					
302	63.5					
303	57.0					
304	60.5					
305	60.5					
306	58.2					
307	63.5					

NOTES:

_____
_____
_____
_____
_____
_____
_____

Figure E1   Real Truck Rationalization Form

TRUCK 101

Axle Spacing (m)   3.0

Gross Vehicle Weight = 16.4 t
Axle Load (t)   7.3   9.1

TRUCK 102

Axle Spacing (m)   3.0   1.2

Gross Vehicle Weight = 24.3 t
Axle Load (t)   7.3   8.5 8.5

TRUCK 103

Axle Spacing (m)   1.0   5.0   1.2

Gross Vehicle Weight = 30.6 t
Axle Load (t)   6.8 6.8   8.5 8.5

TRUCK 104

Axle Spacing (m)   5.4   1.2   1.2

Gross Vehicle Weight = 30.3 t
Axle Load (t)   7.3   7.67 7.67 7.67
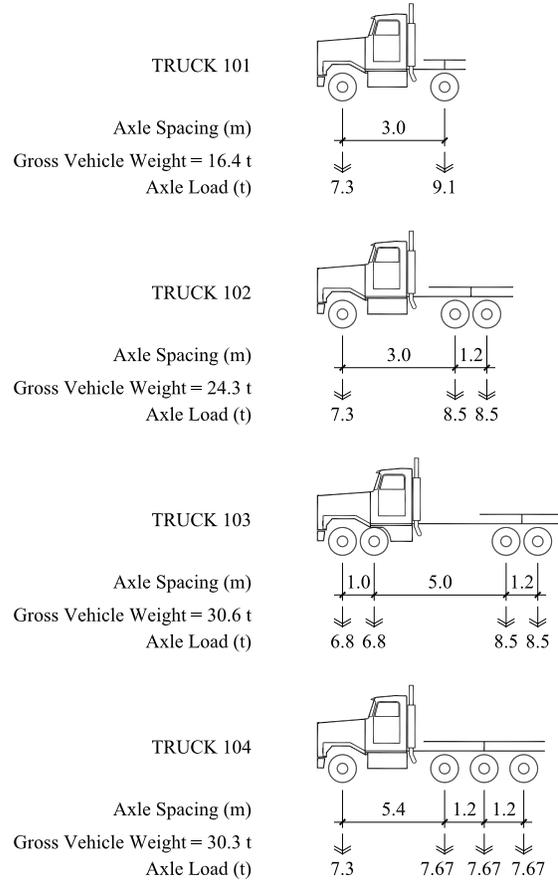
Figure E2 Current Non-Permit Legal Single Unit Truck Loads

TRUCK 201

Axle Spacing (m)   3.0   1.2   5.5   1.2   1.2

Gross Vehicle Weight = 44.0 t
Axle Load (t)   6.0   8.5   8.5   7.0 7.0 7.0

TRUCK 202

Axle Spacing (m)   3.0   1.2   5.5   1.5   1.5

Gross Vehicle Weight = 47.0 t
Axle Load (t)   6.0   8.5   8.5   8.0   8.0   8.0

TRUCK 203

Axle Spacing (m)   3.0   1.2   5.0   1.2

Gross Vehicle Weight = 40.0 t
Axle Load (t)   6.0   8.5   8.5   8.5   8.5

TRUCK 204

Axle Spacing (m)   3.0   5.25

Gross Vehicle Weight = 24.2 t
Axle Load (t)   6.0   9.1   9.1

TRUCK 205

Axle Spacing (m)   3.0   1.2   5.0

Gross Vehicle Weight = 32.1 t
Axle Load (t)   6.0   8.5   8.5   9.1

TRUCK 206

Axle Spacing (m)   5.4   1.2   1.2   5.5   1.5   1.5

Gross Vehicle Weight = 54.3 t
Axle Load (t)   7.3   7.67 7.67 7.67   8.0 8.0 8.0

TRUCK 207

Axle Spacing (m)   5.4   1.2   1.2   5.5   1.2   1.2

Gross Vehicle Weight = 51.3 t
Axle Load (t)   7.3   7.67 7.67 7.67   7.0 7.0 7.0

TRUCK 401

Axle Spacing (m)   3.0   1.2   3.0   5.65   1.2

Gross Vehicle Weight = 50.4 t
Axle Load (t)   7.3   8.5   8.5   9.1   8.5   8.5

Figure E3 Current Non-Permit Legal Semi-Trailer Truck Loads

TRUCK 301

Axle Spacing (m)		3.0	1.2	5.5	1.2	1.2	5.5	1.2
Gross Vehicle Weight = 61.0 t								
Axle Load (t)	6.0		8.5 8.5		7.0 7.0 7.0		8.5 8.5

TRUCK 302

Axle Spacing (m)		3.0	1.2	5.5	1.5	1.5	5.5	1.2
Gross Vehicle Weight = 63.5 t								
Axle Load (t)	5.5		8.5 8.5		8.0 8.0 8.0		8.5 8.5

TRUCK 303

Axle Spacing (m)		3.0	1.2	5.0	1.5	5.0	1.2
Gross Vehicle Weight = 57.0 t							
Axle Load (t)	6.0		8.5 8.5		8.5 8.5		8.5 8.5

TRUCK 304

Axle Spacing (m)		3.0	1.2	5.0	1.2	3.0	5.65	1.2
Gross Vehicle Weight = 60.5 t								
Axle Load (t)	6.0		8.5 8.5		8.5 8.5	9.1		5.7 5.7

TRUCK 305

Axle Spacing (m)		3.0	1.2	5.0	1.2	3.0	5.65	1.2
Gross Vehicle Weight = 60.5 t								
Axle Load (t)	6.0		5.7 5.7		8.5 8.5	9.1		8.5 8.5

TRUCK 306

Axle Spacing (m)		3.0	1.2	5.0	1.2	3.0	5.65
Gross Vehicle Weight = 58.2 t							
Axle Load (t)	6.0		8.5 8.5		8.5 8.5	9.1	9.1

TRUCK 307

| Axle Spacing (m) | | 5.4 | 1.2 | 1.2 | 5.5 | 1.2 | 5.0 | 1.2 |
| Gross Vehicle Weight = 63.5 t | | | | | | | | |
| Axle Load (t) | 7.3 | | 7.67 7.67 7.67 | | 8.5 8.5 | | 8.1 8.1 |

**Figure E4 Current Non-Permit Legal Truck-Train Loads**

Appendix F
Sophisticated Method – Correction of Shear Load Effects

The grillage analogy method typically models a bridge superstructure with longitudinal members that represent the longitudinal stiffnesses and locations of the girders (and associated portions of the deck) and with transverse members that represent the transverse stiffness of the deck (see Figure F1). The transverse members of necessity represent the continuous bridge deck as a series of discrete transverse members. This results in the load distribution between girders occurring at discrete points rather than as a continuum along the length of the deck and in girder shears whose magnitude is stepped at every transverse member (see Figure F2). Note in Figure F2 that the shape of the girder shear diagram varies significantly depending on whether or not a girder axle load is placed to the right or left of a transverse member.

To remove the steps in the girder shear diagram and obtain a more accurately shaped diagram the concentrated shears applied to the girder by the transverse members are assumed to be uniformly applied to the girder over a length equal to the transverse member spacing. If this is done the girder shear diagram becomes a uniformly varying diagram (away from wheel load locations) and the maximum shears at the transverse member locations are reduced (see Figure F3). Note that if these corrections are made the shape of the girder shear diagram is no longer sensitive to small changes in the wheel load locations.
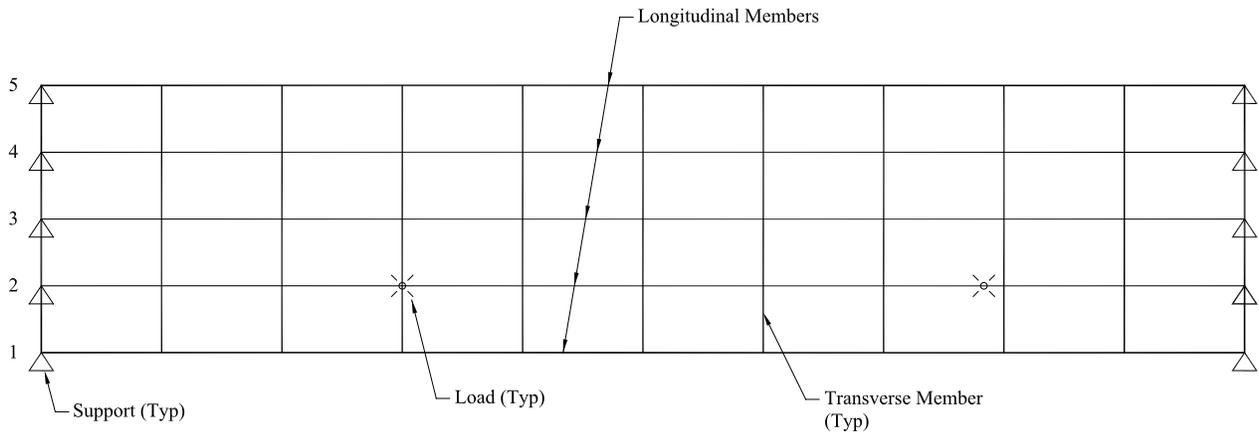
Figure F1 Grillage Analogy Model of Bridge Superstructure
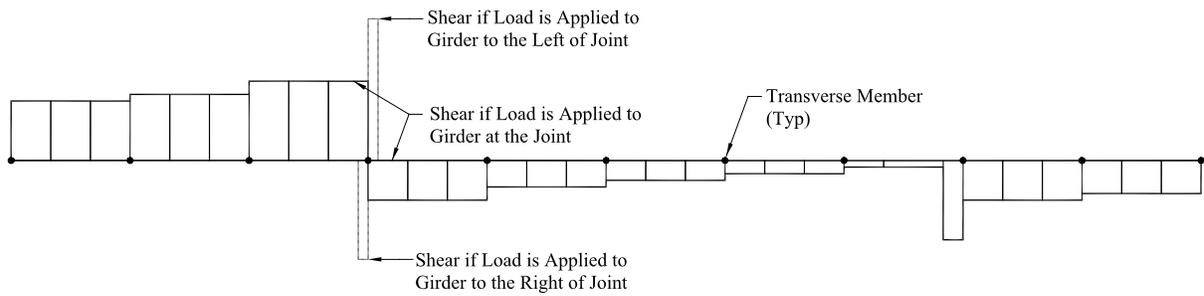
Figure F2   Shears in Loaded Girder with Discrete Transverse Members
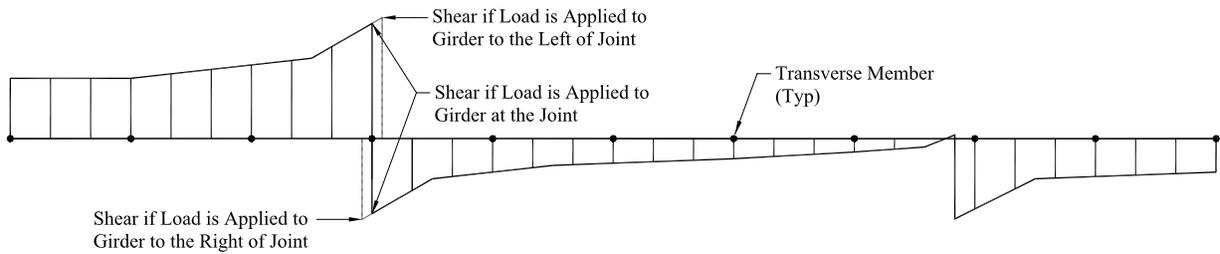
Figure F3   Shears in Loaded Girder with Continuous Transverse Members

Appendix G
Determination of Lateral Distribution of Live Load Between Shear-Connected Girders

The lateral distribution of live load between shear connected girders shall be based on the following, but shall in no case be less than the following:

$$\geq \frac{1.05 \text{ n } R_L}{N}$$

Where:

n = the number of design lanes specified in CSA S6-14 Section 3.8.2;

R_L = the multi-lane reduction factor specified in CSA S6-14 Section 3.8.4.2.1; and

N = the number of girders in the bridge.

1. Lateral Distribution of Live Load Moment Between Shear Connected Girders

The live load truck moment carried per shear connected girder shall be taken to be:

$$= \frac{S}{D} \text{x live load moment due to a single truck}$$

Where:

S = the girder width in metres;

$D = 3.5 + 1.65 \left(1 - \frac{C}{3}\right)^2$;

$C = K \left(\frac{10}{L}\right)$ but shall not be taken to be greater than 3;

L = the bridge span length in metres;

$K = \sqrt{\frac{I}{J}}$;

I = the shear connected girder moment of inertia in mm^4; and

J = the shear connected girder St. Venant's torsional constant in mm^4.

CAN/CSA S6-88 calculates "S" by dividing the assumed bridge width (based on the number of 12 foot wide lanes on the bridge and two 4.5 foot wide shoulders) by the number of girder lines on the bridge. In the above method "S" is set equal to the actual girder width so that it does not vary with bridge width.

The above equation for "D" is modified from CAN/CSA S6-88. It assumes that the critical load case will be for a two lane bridge (NL =2) with a bridge width of 10 m and a multi-lane reduction factor of 0.9. It has also been converted from imperial to metric units (feet to metres).

2.   Lateral Distribution of Live Load Shear Between Shear Connected Girders

The live load truck shear carried per shear connected girder shall be taken to be:

$$= \frac{S}{D} \text{x live load shear due to a single truck}$$

Where:

       (a)   For the truck axle at the girder location where the shear is being determined:

$$\frac{S}{D} = 0.5 \ \text{ if } S \le 1.2\text{m} ;$$

$$\frac{S}{D} = \text{the greater of } 0.5 \text{ and } 0.9 \left(1 - \frac{0.6}{S} \right) \ \text{ if } 1.2\text{m} < S \le 1.8\text{m} ;$$

$$\frac{S}{D} = 0.9 \left(1.5 - \frac{1.5}{S} \right) \ \text{ if } 1.8\text{m} < S \le 3.0\text{m} ;$$

       (b)   For all other truck axles:

$$\frac{S}{D} \text{ is calculated in the same manner as for live load moment in Section 1 of this Appendix G.}$$

The determination of "S/D" for the truck axle at the girder location where the shear is being determined is based on the AASHTO LRFD Bridge Design Specifications. The equations have been converted from imperial to metric units (feet to metres) and a multi-lane reduction factor of 0.9 for two lanes of loading has been added.

Appendix H
Determination of Prestress Losses in Precast Prestressed Concrete Girders

The final prestress stress after all losses (f_{se}) in precast, prestressed concrete girders shall be determined using either the Lump Sum Method or the PCILOSS Method as described below.

1. Lump Sum Method

The Lump Sum Method shall only be used to determine the final prestress stress after all losses (f_{se}) in precast, prestressed pre-tensioned girders. The final prestress stress after all losses (f_{se}) shall be taken from Table H1 where f_{pu} is the specified ultimate tensile strength of the prestressing steel.

Table H1 Final Prestress Stress after all Losses

	Normal Weight Concrete	Semi-Light Weight Concrete
Stress Relieved Strand	0.50 f_{pu}	0.48 f_{pu}
Low Relaxation Strand	0.55 f_{pu}	0.53 f_{pu}

The final prestress stresses after all losses (fse) shown in Table H1 are based on an initial prestress stress just prior to transfer of the prestressing force to the girder (fsi) of 0.7 x fpu. This assumption can be taken to be valid for all of the pre-tensioned precast, prestressed concrete girders in AT's bridge inventory.

The Lump Sum Method predicts final prestress stresses (fse) similar to those predicted by the PCILOSS Method for typical levels of prestressing. However, for low levels of prestressing the Lump Sum Method will predict final prestress stresses (fse) that are lower than those predicted by the PCILOSS Method.

2. PCILOSS Method

2.1 General

The PCILOSS Method may be used to determine the final prestress stress after all losses (f_{se}) in either precast, prestressed pre-tensioned or post-tensioned girders. The final prestress stress after all losses shall be taken to be equal to the initial prestress stress just prior to transfer of the prestressing force to the girder (f_{si}) minus the total prestress losses.

2.2   Total Prestress Losses

In precast, prestressed concrete girders the total prestress losses shall be taken to be equal to:

$$TL = FR + ANC + ES - DL + \Sigma\,(CR + SH + RET)$$

Where:

TL = total prestress losses;

FR = prestress loss due to friction in post-tensioned construction (FR = 0 for pre-tensioned construction);

ANC = prestress loss due to anchorage slip for post-tensioned construction (ANC = 0 for pre-tensioned construction if f_{si} is taken to be the initial stress in the prestressing steel after anchorage slip has occurred);

ES = prestress loss due to elastic shortening of the concrete;

DL = change in the prestressing stress due to additional dead load applied to the girder after transfer of the prestressing force to the concrete (e.g. wearing surface);

CR = prestress loss due to creep of the concrete;

SH = prestress loss due to shrinkage of the concrete; and

RET = prestress loss due to relaxation of the prestressing steel.

Prestress losses due to elastic shortening, concrete creep, concrete shrinkage and steel relaxation as well as any increase in the prestressing stress due to additional dead load being applied to the girder shall be determined in accordance with the following method.

Prestress losses due to anchorage slip and friction loss shall be determined in accordance with the requirements of CSA S6-14, Sections 8.7.4.2.2 and 8.7.4.2.3 respectively.

2.3 Time Stages

Prestress losses due to elastic shortening, concrete creep, concrete shrinkage and steel relaxation as well as any increase in the prestressing stress due to additional dead load being applied to the girder shall be the sum of the prestress losses determined over the following time stages for pre-tensioned girders;

1) From Pre-Tensioning to Transfer of Prestressing to Girder   t = 0 days to 0.75 days
2) From Transfer of Prestressing to Girder to Girder Erection   t = 0.75 days to 30 days
3) From Girder Erection to 1 Year   t = 30 days to 1 year
4) From 1 Year to 50 Years   t = 1 year to 50 years

The time stages for post-tensioned girders shall be the same as for pre-tensioned girders except that the first time stage shall extend "From Completion of Girder Concrete Curing to Transfer of Prestressing to Girder".

2.4 Elastic Shortening Prestress Loss

The elastic shortening prestress loss (ES) is determined at the end of the first time stage using the following equation:

$ES = \frac{E_p}{E_{ci}} f_{cir}$ for pre-tensioned girders; and

$ES = \left(\frac{N-1}{2N}\right) \frac{E_p}{E_{ci}} f_{cir}$ for post-tensioned girders.

Where:

E_p = the elastic modulus of the prestressing steel determined in accordance with CSA S6-14, Section 8.4.3.3 (MPa);

E_{ci} = the modulus of elasticity of the concrete at the time of transfer of the prestressing force to the girder determined in accordance with CSA S6-14, Section 8.4.1.7 (MPa);

f_{cir} = initial concrete stress at the level of the centre of gravity of the prestressing steel at the time of transfer of the prestressing force to the girder (MPa). The value of f_{cir} is based on the force in the prestressing steel after the prestressing force has been transferred to the girder and elastic shortening prestress losses and any steel relaxation prestress losses prior to transfer have occurred; and

N = the total number of post-tensioning tendons individually stressed.

2.5 Increase in Prestressing Stress Due to Additional Dead Load

The increase in the prestressing stress due to additional dead load being applied to the girder (DL) is determined at the end of the second time stage using the following equation:

$$DL = f_{cds} \frac{E_p}{E_c}$$

Where:

f_{cds} = the change in concrete stress at the level of the centre of gravity of the prestressing steel due to the additional dead load being applied to the girder (MPa);

E_p = the elastic modulus of the prestressing steel determined in accordance with CSA S6-14, Section 8.4.3.3 (MPa); and

E_c = the modulus of elasticity of the concrete at an age of 28 days determined in accordance with CSA S6-14, Section 8.4.1.7 (MPa).

2.6 Concrete Creep Prestress Loss

The concrete creep prestress loss (CR) is determined for each of the second to fourth time stages using the following equation:

$$CR = UCR \times SCF \times (PCR_e - PCR_i) \times f_c$$

Where:

UCR = the ultimate creep coefficient as shown in Table H2;

SCF = the creep factor as shown in Table H3 that adjusts the ultimate creep coefficient to account for the Volume/Surface (V/S) Ratio of the girder (mm);

PCR_e = the percentage of the ultimate creep that occurs up to the end of the time stage under consideration as shown in Table H4;

PCR_i = the percentage of the ultimate creep that has occurred prior to the beginning of the time stage under consideration as shown in Table H4;

f_c = the concrete stress at the level of the centre of gravity of the prestressing steel based on the force in the prestressing steel at the beginning of the time stage under consideration (MPa); and

V/S = cross-sectional area of the girder divided by the outside perimeter of the girder cross-section (mm).

In Table H2, Light Weight Concrete refers to concrete with a unit weight between 1420 kg/m^3 (90 pcf) and 1960 kg/m^3 (125 pcf) while Steam Cure refers to curing at an elevated temperature not greater than 70°C for a period of approximately 18 hours when steps are taken to retain moisture.

Precast, prestressed concrete girders in AT's bridge inventory may be assumed to have been Steam Cured.

Table H2 Ultimate Creep Coefficient (UCR) (MPa/MPa)

Concrete Type	Moist Cure Formulae	Steam Cure Formulae
Normal Weight Concrete	$95 - \dfrac{2{,}900E_c}{10^6} \geq 11$	$63 - \dfrac{2{,}900E_c}{10^6} \geq 11$
Light Weight Concrete	$76 - \dfrac{2{,}900E_c}{10^6} \geq 11$	$63 - \dfrac{2{,}900E_c}{10^6} \geq 11$

Table H3 Shrinkage and Creep Factors

Volume/Surface Ratio (mm)	Shrinkage Factor (SSF)	Creep Factor (SCF)
25	1.04	1.05
50	0.96	0.96
75	0.86	0.87
100	0.77	0.77
125	0.69	0.68
150	0.60	0.68

Table H4 Shrinkage and Creep Time Factors

Time after Prestress Transfer[1] (days)	Amount of Ultimate Shrinkage Factor (PSH)	Amount of Ultimate Creep Factor (PCR)
1	0.08	0.08
2	NA	0.15
3	0.15	NA
5	0.20	0.18
7	0.22	0.23
10	0.27	0.24
20	0.36	0.30
30	0.42	0.35
60	0.55	0.45
90	0.62	0.51
180	0.68	0.61
365	0.86	0.74
End of Service Life	1.00	1.00

Note 1: "Time after the End of Girder Concrete Curing" when determining PSH for post-tensioned girders.

2.7   Concrete Shrinkage Prestress Loss

The concrete shrinkage prestress loss (SH) is determined for each of the second to fourth time stages using the following equation. For post-tensioned girders concrete shrinkage occurring between the end of girder concrete curing and the transfer of the prestressing force to the girder shall be ignored:

$$SH = USH \times SSF \times (PSH_e - PSH_i)$$

Where:

USH = the ultimate shrinkage coefficient as shown in Table H5 (MPa);

SSF = the shrinkage factor as shown in Table H3 that adjusts the ultimate shrinkage coefficient to account for the Volume/Surface (V/S) Ratio of the girder (mm);

PSH_e = the percentage of the ultimate shrinkage that occurs up to the end of the time stage under consideration as shown in Table H4;

PSH_i = the percentage of the ultimate shrinkage that has occurred prior to the beginning of the time stage under consideration as shown in Table H4; and

V/S = cross-sectional area of the girder divided by the outside perimeter of the girder cross-section (mm).

Table H5 Ultimate Shrinkage Coefficient (USH) (MPa)

Concrete Type	Ultimate Shrinkage (USH)
Normal Weight Concrete	$186\ MPa - \dfrac{3{,}000 E_c}{10^6} \geq 83\ MPa$
Light Weight Concrete	$283\ MPa - \dfrac{10{,}000 E_c}{10^6} \geq 83\ MPa$

2.8 Steel Relaxation Prestress Loss

The steel relaxation prestress loss (RET) is determined for each of the four time stages for pre-tensioned girders and for each of the second to fourth time stages for post-tensioned girders. It is determined using the following equations:

$$\text{RET} = f_{st}\left(\frac{\log(24t_e) - \log(24t_i)}{10}\right)\left(\frac{f_{st}}{f_{py}} - 0.55\right) \text{ for stress-relieved prestressing strands;}$$

$$\text{RET} = f_{st}\left(\frac{\log(24t_e) - \log(24t_i)}{45}\right)\left(\frac{f_{st}}{f_{py}} - 0.55\right) \text{ for low relaxation prestressing strands; and}$$

$\left(\dfrac{f_{st}}{f_{py}} - 0.55\right) \geq 0.05.$

Where:

f_{st} = the stress in the prestressing steel at the beginning of the time stage under consideration (MPa);

f_{py} = the yield stress of the prestressing steel (MPa); equal to 0.85 x f_{pu} for stress-relieved steel and 0.9 x f_{pu} for low relaxation steel;

f_{pu} = the ultimate tensile stress of the prestressing steel (MPa);

t_e = the time in days up to the end of the time stage under consideration; and

t_i = the time in days that has occurred prior to the beginning of the time stage under consideration. For the first time stage considered t_i shall be taken to be 1/24th of a day.