CSA S6.1:19

Commentary on CSA S6:19, Canadian Highway Bridge Design Code

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Preface

This is the fifth edition of CSA S6.1, *Commentary on CSA S6:19, Canadian Highway Bridge Design Code*. It supersedes the previous editions, published in 2014, 2006, 2000, and 1990.

Throughout this Commentary, CSA S6:19 is referred to as the “Code”. Other Codes are always identified in a manner that allows them to be readily distinguished from the Code.

The purpose of this Commentary is to provide background on the design provisions of the Code and thereby to help designers deal with issues not explicitly addressed in the Code.

Each section and clause in this Commentary bears the number of its corresponding section or clause in the Code, with the addition of the prefix “C”. For example, Section C1 provides commentary on Section 1 of the Code, and within Section C1, Clause C1.1.1 provides commentary on Clause 1.1.1 of the Code. The same approach is used in the numbering of annexes. Tables and figures are numbered sequentially (for example, the first table in Section C3 is Table C3.1, which is followed by Table C3.2, etc.). However, they do not correspond to the tables and figures bearing the same numbers (minus the “C”) in the Code.

The Code contains many clauses dealing with “approval” by owner or regulatory authority (see the definitions in Clause 1.3.2 of the Code). Where possible, this Commentary provides guidance for owners or regulatory authorities consulting such clauses.

This Commentary was prepared by the Technical Committee on the Canadian Highway Bridge Design Code; however, it is not a consensus document.

Notes:

1) *Use of the singular does not exclude the plural (and vice versa) when the sense allows.*
2) *Although the intended primary application of this Special Publication is stated in its Preface, it is important to note that it remains the responsibility of the users of this Special Publication to judge its suitability for their particular purpose.*
3) *All enquiries regarding this Special Publication should be addressed to CSA Group, 178 Rexdale Boulevard, Toronto, Ontario, Canada M9W 1R3.*

Section C1
General

C1.1 Scope

C1.1.1 Scope of Code

The OHBDC (MTO 1991) was written for application within Ontario. CAN/CSA-S6-88 was generated with interprovincial co-operation for use in the other provinces of Canada and was largely derived from the preceding OHBDC edition. The provinces and CSA then agreed that the successor edition to both codes would be the Code, published by CSA.

The scope of the Code is a little broader than that of the third and last edition of the OHBDC (MTO 1991). Long span bridges and movable bridges are included. Over the years, new sections have been added to the Code, namely Section 16 on Fibre-reinforced Structures and Section 17 on Aluminum Structures. The 2019 edition of the Code includes a new annex in Section 8 on fibre-reinforced concrete. In addition to incorporating newer technology, more emphasis is placed on criteria related to seismic design, durability, sustainability, and access for inspection and maintenance. Also new in the 2019 edition of the Code are limited climate change requirements and guidance provided for the design of structures in impacted regions of Canada.

The scope statement lists types of structures to which the Code is not intended to apply. The list is not exhaustive. The application of the Code to the types of structures listed is not precluded where the owner of the structure has designated all or part of the Code as being applicable.

C1.1.2 Scope of this Section

Geometrical provisions have been minimized by referring to the *Geometric Design Guide for Canadian Roads* (TAC 2017).

Many catastrophic failures have been caused by scour at bridge piers and abutments. Good hydraulic design is a fundamental requirement for bridges. Basic hydraulic requirements are specified in the Code, and reference is made to the *Guide to Bridge Hydraulics* (TAC 2004) for guidance concerning good hydraulic design and detailing.

C1.2 Reference publications

All specifications, standards, manuals, and similar documentation referred to in the Code are listed.

C1.3 Definitions

C1.3.1 General

Definitions are divided into three groups: general administrative definitions, general technical definitions, and hydraulic definitions.

Section C2
Durability and sustainability

C2.1 Scope

The scope of this Section has been expanded to include design requirements for durability and sustainability as well as special considerations for climate and exposure considerations. It has been recognized that the ultimate and serviceability limit states requirements of the Code cannot be met without proper integration with design for service life, quality management of design and construction, and life cycle maintenance program.

The high cost of repairs and replacements, as well as the high costs to users due to traffic delays are compelling reasons to focus attention on durability and sustainability. On heavily travelled highways, it is increasingly difficult to obtain access to carry out the necessary repairs or replacement. The safety risk to workers and to the travelling public is another major consideration, as is the increasing cost of delays and detours.

The sustainability of highway bridges and other structures covered by the Code, has been given widespread attention, as the consideration of environmental impacts as well as the importance of social and economic well-being of the neighboring communities and society at large have been recognized. The focus on sustainability in making decisions regarding the design, construction, and maintenance of bridges is sought by stakeholders and the public.

Bridges and other structures covered by the Code might be subjected to a variety of natural and man-made hazards, including traffic overload, vehicle collision, aging and deterioration, and extreme events (including earthquake, extreme weather events, flood/scour, fire/heat, and blast). Regional and local climatic characteristics influencing durability and service life should be identified and included in the design and construction of new bridges and in the maintenance of existing bridges and structures covered by the Code.

C2.2 Definitions

Examples of “major repair” include a need to restore a bridge superstructure condition after a high load collision or to rectify an unplanned deficiency such as insufficient deck reinforcing cover.

C2.3 Design for durability

C2.3.1 General

The past inadequate performance of some types of bridges and the associated high costs for their rehabilitation and replacement indicate the need for the consideration of design for durability. Bridge structures and components must be designed such that, under foreseeable environmental conditions, they maintain their performance requirements at ultimate and serviceability limit states during the service life of components and structures. Many decisions taken during design and construction greatly influence the durability and life cycle costs of bridge structures and their components. There is a need to take into account at the design stage the foreseeable environmental conditions that exist or are likely to exist at the site and to identify the potential mechanisms of deterioration and the rate of

Section C3
Loads

C3.1 Scope

The Code is applicable to all highway bridges in Canada, including those with long spans.

C3.2 Definitions

The definitions include terms used in Annexes A3.1 to A3.4.

C3.3 Abbreviations and symbols

The symbols listed below are those used in this Section of the Commentary. These are additional to those used in the Code.

A_c = the cross-sectional area of concrete, mm^2

A_s = the cross-sectional area of steel, mm^2

a = truncated distance assumed equal to the radius of a circular pier, m; acceleration, m/s^2

a_i = modal coefficient of magnitude of the oscillatory displacement for mode of vibration, i, m

C = a non-dimensional coefficient

$D(x)$ = the diameter or frontal width of a member at location x, m

d = section depth, mm

E = Young’s modulus for ice, GPa

E_c = the modulus of elasticity of concrete, MPa

E_l = longitudinal flexural rigidity, MPa, mm^4

E_s = the modulus of elasticity of steel, MPa

f_1 = first flexural frequency, Hz

g = acceleration due to gravity, m/s^2

K = configuration from Table C3.1

L = span length, m

L_k = live load effect due to loads in kth lane

l = characteristic length calculated from the expression, m; dynamic load allowance; impact fraction not to exceed 0.30

$n_e(x)$ = frequency at which vortex shedding occurs at location x, Hz

R_n = unfactored resistance

S_j = the total of the load effects due to factored loads

t = time, s; maximum thickness of the ice, m

V = the mean wind speed at location x, m/s

w_s = maximum static superstructure deflection due to a vertical concentrated force of 700 N, m

x = coordinate describing length along the member

Section C4
Seismic design

C4.1 Scope

This Commentary explains the rationale behind the seismic design provisions in the 2019 edition of the Code, including performance-based design, seismic hazard, time-history analysis, geotechnical aspects, concrete and steel design, base isolation and damping, and seismic evaluation of existing bridges. This Commentary also explains the background and implementation of performance-based seismic design of highway bridges. Section 4 is currently unique within this Code to have a performance-based design methodology, as opposed to the load and resistance factor approach to other environmental and internal structural design effects.

The 2014 edition of the Code introduced seismic hazard, and structural and geotechnical aspects of the seismic design of bridges. For the 2019 edition of the Code, the geotechnical aspects of seismic design have been moved to Section 6. The reader must refer to both Sections of the Code and Commentary for a complete seismic design approach.

C4.3 Abbreviations and symbols

The two different design approaches, force-based design and performance-based design, are abbreviated as FBD and PBD, respectively.

C4.4 Earthquake effects

C4.4.1 General

Section 4 establishes two different design approaches, with performance-based design being the standard method of design and force-based design being permitted for special cases.

The performance-based design approach is aimed at meeting a series of performance criteria described in Clause 4.4.6.3. The performance levels to be investigated are described in Clause 4.4.6.2 and depend on the importance category of the bridge and on the return period of the earthquake that needs to be investigated.

The force-based design approach is permitted for some cases described in Clause 4.4.5.3. The performance of structures designed using the force-based design approach is expected to be consistent with performance-based design at the 2% in 50-year probability of exceedance (2475-year return period)..

A capacity-protected member is a member whose force level is limited by yielding of one or more connecting members.

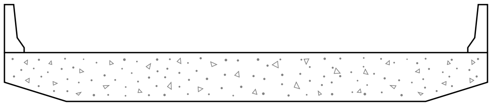
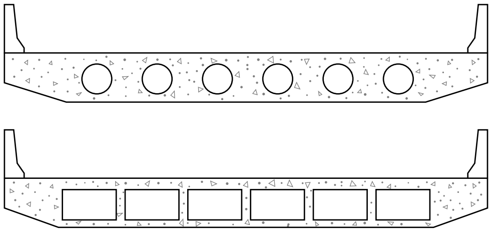
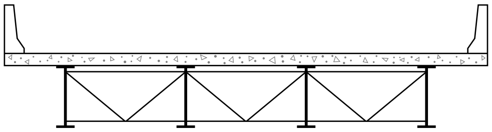
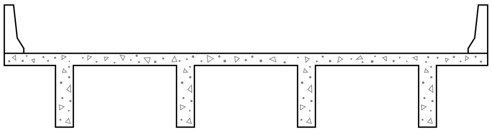
The capacity design procedures within Section 4 might not apply directly for special bridges, e.g., arches, cable-supported bridges, and large trusses. Special studies are necessary to account properly for the complex dynamic behaviour and non-linear performance of these bridges during earthquakes.

Section C5
Methods of analysis

C5.1 Scope

This Section presents the background for the methods of analysis for the design and evaluation of bridge superstructures. It includes general requirements for specific bridge types and specific requirements for the use of the simplified method of analysis. Table C5.1 illustrates representative cross-sections and elevations of bridge types and applicable Clauses covered by Section 5.

Table C5.1
Representative cross-sections and elevations of bridge types
(See Clause C5.1.)

Bridge type	**Supporting components**	**Type of deck**	**Typical cross-section**	**Applicable clauses**
Solid slab	Cast-in-place solid slab	Monolithic		5.5.2 5.6.2 5.6.3 5.6.4.2 5.6.5
Voided slab	Circular voided slab or rectangular voided slab	Monolithic		5.5.2 5.5.3 5.6.2 5.6.3 5.6.4.2 5.6.5
Deck-on-girder	Steel I-girders	Cast-in-place concrete slab, full-depth precast concrete deck panels or composite cast-in-place concrete slab on partial-depth precast concrete deck panels		Clauses: 5.5.4 5.6.2 5.6.4.3 5.6.6
Deck-on-girder	Cast-in-place concrete T-beams	Monolithic concrete		5.5.4 5.6.2 5.6.4.3 5.6.6

(Continued)

Section C6
Foundations and geotechnical systems

C6.1 Scope

Limit states design philosophy was accepted in structural design prior to its application to foundations and geotechnical systems.

Previously, substructure design was based on allowable stress, even though much of geotechnical engineering has been based on concepts of limiting equilibrium — an ultimate limit state. The use of two design philosophies — one, a limit states design philosophy for superstructures; the second, an allowable stress design philosophy for substructures — led to complications. It became evident that a limit state approach was required for foundation design.

This evolution has not been without difficulties. Initially (OHBDC, 1979, 1983), the partial factor format was adopted following Danish practice (DGI, 1985 and earlier). Soil strength factors were applied to soil cohesion and angle of internal friction to account for uncertainties in material properties. However, the partial strength factor approach did not lead to design consistency with the allowable stress approach and the use of partial strength factors was not readily accepted by many geotechnical engineers. When strength factors were applied to cohesion and friction, well-proven empirical relationships for geotechnical design no longer applied. For example, many of the empirical relationships for bearing resistance of shallow and deep foundations are based on limiting vertical deflection. Thus, it was difficult for the geotechnical engineer to utilize earlier predictive methods and sources of data.

To address these concerns, the approach was modified in the third edition of the OHBDC (1991). Resistance factors were applied to the various ultimate limit states of geotechnical resistance rather than using partial strength factors applied to the soil strength parameters of friction and cohesion. The resistance factors were chosen to give a level of safety generally comparable to that obtained by the earlier allowable stress design procedures. Thus, traditional design aids were applicable, with some modifications.

The total resistance factor approach has been carried on in the Code. The ultimate and serviceability geotechnical resistances are computed using traditional methods with characteristic geotechnical parameters. This Code now applies both a resistance factor and a consequence factor to the geotechnical resistance to obtain the factored resistance used in design. The two factors are designed to keep the failure probability of the geotechnical system below acceptable levels, the latter of which are related to failure consequence.

Some key elements of the Code are as follows:
- As in the 2014 edition of the Code, foundation design considers structural “loads” applied against geotechnical “resistances”.
- In the Code, “resistance” refers to the load that the ground or geotechnical system can support at each limit state. At ultimate limit states, this load generally corresponds to a limit equilibrium capacity. At serviceability limit states, this load typically corresponds to that which yields less settlement or displacement than a predefined amount.
- In response to a desire in the geotechnical community to provide differing levels of safety for differing failure consequences (e.g., failure of a major multi-lane highway bridge versus failure of a minor secondary bridge), a “consequence factor” has been introduced in the Code. Three failure

Section C7
Buried structures

C7.1 Scope

Projects using buried structures have grown in size and variety of use in recent years. Section 7 focuses on the design of structures which rely upon soil-structure interaction to achieve performance. In an effort to both bring consistency to how these structures are referenced and to emphasize the fact that these are structures in the same context as other bridge types in the Code, the term buried structures is used throughout in lieu of other commonly used terms such as culverts or pipes. Section 7 applies only to buried structures with a span greater than 3 m.

Unlike other sections of the Code, Section 7 specifies minimum standards of both design and construction. The performance of a buried metal or concrete structure is governed by the soil-structure interaction and depends as much on methods of construction as it does on design. Good construction practices for these structures are now established and it is the intent of this Commentary to emphasize the geotechnical requirements and recommended construction practice. Design provisions are also given for soil-metal structures with shallow, deep, and deeper corrugations. Concrete structures covered by Section 7 include round pipes, box structures, three-sided box structures, and arches with footings.

C7.3 Abbreviations and symbols

C7.3.2 Symbols

The symbols listed below are those used in this Section of the Commentary. These are additional to those used in the Code. The units given are those generally used; however, care should be taken to check the consistency of units in any equation.

k_E = a Young's modulus number that depends on soil type and compaction

m = a Young's modulus exponent, $0 < m < 1$

P_A = atmospheric pressure in the chosen units

σ_d = the current level of major principal stress difference

$(\sigma_d)_f$ = the major principal stress difference at failure

$(\sigma_d)_{ult}$ = the ultimate stress difference from the best fit hyperbola to the data

R_F = the failure ratio

C7.4 Hydraulic design

Although the design of buried structures is based primarily on the structural and geotechnical considerations specified in Section 7, failures of those structures, which convey water, are often caused by washouts and/or hydraulic uplift. Experience indicates that most hydraulic failures could be prevented by provision of one or more of the following:

- an adequate waterway to carry the design flood;

Section C8
Concrete structures

C8.1 Scope

Concrete structural components may be reinforced with non-prestressed or prestressed reinforcement or both, thus permitting partial prestressing. Although prestressing forces can be introduced in various ways, Section 8 is based on the use of high-strength steel prestressing tendons. Post-tensioning tendons may be internal or external but they must be protected by grouting. The use of low-density or semi-low-density concretes should be based on the availability of suitable low-density or semi-low-density aggregates.

This Section does not provide requirements for concrete poles. The requirements for concrete poles are given in CSA A14.

C8.3 Symbols

CSA A23.3 symbols have been used wherever possible. The following additional symbols have been used in this Section of the Commentary.

A_l = the longitudinal reinforcement in the flange, mm^2

A_t = the transverse reinforcement in the flange, mm

C = the force in the concrete strut, N

C_{c1} = the factored compressive force in the overhanging portion of the flange, as shown in Figure C8.8, N

C_{c2} = the factored compressive force in the web, as shown in Figure C8.8, N

C_c = the factored compressive force in the equivalent rectangular stress block, N

CR_t = the prestress loss due to creep up to t days after transfer, MPa

C'_s = the factored force in the compression reinforcement, N

d = the distance from the extreme compression fibre to the centroid of the non-prestressed compression reinforcement, mm

d_k = the depth of corrugated shear keys, mm

d_s = the distance from extreme compression fibre to the centroid of the non-prestressed tension reinforcement, mm

E_b = the modulus of elasticity of the bearing plate, MPa

f_b = the stress in anchor plate at a section taken at the edge of the wedge hole or holes, MPa

f_{ca} = the compressive stress in the strut, MPa

f_{min} = the algebraic minimum stress, with tension positive and compression negative, MPa

f_p = the stress at the extreme fibre in tension due to moment acting on the precast portion of a composite section, MPa, or the stress in prestressing steel, MPa

f_{pe} = the compressive stress in concrete due to effective prestress at the extreme fibre of a section at which tensile stresses can be caused by live load, MPa

f_{sr} = the stress range in reinforcing bars, MPa

Section C9
Wood structures

C9.1 Scope

Section 9 applies to the types of wood structures and components likely to be required for highways, including glued-laminated girders, timber stringers, transversely and longitudinally laminated decks, laminated wood-concrete composite decks, prestressed laminated decks, and trusses. Section 9 does not apply to falsework or formwork.

C9.4 Limit states

C9.4.1 General

Only SLS and ULS requirements are given for wood components. No explicit FLS requirements apply, but fatigue may be a consideration for steel components used in wood structures.

C9.4.2 Serviceability limit states

Under load, wood is subject to increasing deflection with time, the amount depending on the species, magnitude of stress, moisture content at the time of loading, and subsequent changes in moisture content that take place under load. Slip and rotation of mechanical joints also contribute. Care should be taken to avoid adverse effects arising from the deformations. A deflection limit is specified in order to limit the working of the elements and joints due to large repetitive deflections, which may lead to progressive deterioration of the elements and joints, and distress of any asphalt wearing surface.

C9.4.4 Resistance factor

Resistance factors for wood design are chosen to allow for inaccuracy in the prediction of structural behaviour, approximations in the determination of specified strengths, the variability of the material, and the consequences of exceeding the limit state. They have been chosen to provide a safety index value of 3.5, generally, but yield a value of about 3.0 for single member cases. This lower value is applicable only to single-load-path structures, such as single leaf trusses and beams, the failure of which can cause failure of the entire structure.

Resistance factors are higher than might be expected because specified strengths are based on lower 5th percentile strengths, which are significantly less than mean strengths because of the characteristically large variability of wood.

C9.5 General design

C9.5.1 Design assumption

Plastic redistribution is not to be assumed in the calculation of load effects.

Section C10
Steel structures

C10.4 Materials

C10.4.1 General

The design requirements specified in Section 10 have been developed on the assumption that the materials and products that will be used are those listed in Clause 10.4. These materials and products are covered by standards prepared by CSA or ASTM. With approval, the use of materials and products other than those listed is permitted, provided that the designer ensures that the materials and products have the characteristics required to perform satisfactorily in the structure. In particular, ductility, weldability, and fracture toughness may be as important as the material strength. Only steels exhibiting a yield plateau followed by a strain hardening range should be used in Class 1 and Class 2 sections.

C10.4.2 Structural steel

Atmospheric corrosion-resistant steel, commonly referred to as weathering steel, is available as Type A or Type AT, as designated in CSA G40.21. ASTM A588/A588M steel has similar chemical composition and mechanical properties as Type A steel and can therefore be used as a substitute for Type A steel. However, to use ASTM A588/A588M steel as a substitute for Type AT steel, the fracture toughness must be verified using Charpy impact testing. Weathering steel is a low-alloy high-strength steel containing a controlled content of chromium, nickel, and copper. When exposed to repeated wetting and drying cycles, the metal develops a tightly adherent protective oxide film. Initially, weathering steel appears to oxidize like mild steel and quickly assumes a fine, sandy appearance. However, unlike mild steel whose oxide continuously spalls off, the surface oxide layer of weathering steel stabilizes with time, provided that the exposure conditions allow the steel to dry out periodically. The rust then becomes darker, granular, and adheres tightly to the surface, thus protecting the underlying steel by reducing the permeability of the oxide layer to water and air. This film usually takes from 1.5 to 3 years to form.

Uncoated weathering steel has been used for bridge construction in Canada since 1967. While most of the weathering steel bridges have been performing as expected, some have experienced accelerated corrosion usually in local areas (e.g., areas beneath leaking expansion joints). The successful application of uncoated weathering steel depends primarily on proper design of details and avoidance of harsh environmental exposure. See Clause C10.6.2.

Because of the need for intermittent drying to stabilize the oxide film, weathering steel should not be used where it will be immersed in natural water or buried in soil.

Designers should be aware that the expected toughness can be assured only by specifying AT, WT, or QT steel and the appropriate category, although other steels may have the chemical composition required to attain the toughness characteristics listed in Tables 9A and 9B of CSA G40.21. Therefore, where such assurance is necessary, as for fracture-critical or primary tension members, Charpy V-Notch tests should be specified. Procedures for conducting the Charpy V-notch tests are specified in CSA G40.21 and CSA G40.20.

- The material specifications for structural steel are based on the production practices of North American steel mills. When off-shore steel is used, additional restrictions should be considered:

Section C11

Joints and bearings

C11.1 Scope

The emphasized preferences are based largely on experience developed during the long-term operation and performance of many past installations. The joints and bearings need to allow movements to occur due to temperature changes, creep and shrinkage, elastic shortening due to prestressing, traffic loading, construction tolerances, or other effects. If these movements are restrained, large horizontal forces may be induced. In a cast-in-place concrete bridge deck, it may be unwise to fix or guide all of the bearings at a single support, because such fixity would prevent this transverse expansion and contraction. Externally applied horizontal loads such as wind, earthquake, or traffic braking forces may be carried either on a small number of bearings near the centreline of the bridge or by an independent guide system. The latter is likely to be needed if the horizontal forces are large.

The distribution of vertical load among bearings may also cause problems with individual bearings. This is particularly critical when the girders are stiff in bending and torsion and bearings are stiff in compression, and the construction method does not allow minor misalignments to be corrected.

Background information on requirements for joints and bearings can be found in Manning and Bassi (1986), Manning and Witechi (1981), and Mihaljevic et al. (2010).

C11.3 Abbreviations and symbols

C11.3.2 Symbols

The symbols listed below are those used in this Section of the Commentary. These are additional to those used in the Code.

a = the overall width (short dimension) of the strip bearing, mm

d = the diameter of the hole or holes in the bearing, mm

d_d = ULS vertical deflection, mm

h_r = thickness of elastomeric pad, mm

h_w = height from top of rim to underside of piston, mm

h_p = vertical clearance between top of piston and top of pot wall, mm

I = the moment of inertia of the plan shape of bearing, mm

R = radial distance from center of pot to object in question (e.g., pot wall, anchor rod, etc.), mm

C11.4 Common requirements

C11.4.1 General

Bridge movements arise from a number of different causes and the direction and magnitude of these movements need to be accurately estimated to determine the design requirements for the joint or bearing. Simplified estimates of bridge movements, particularly on bridges with complex geometry, may

Section C12

Barriers and highway accessory supports

C12.1 Scope

Section 12 specifies requirements for the design of permanent bridge barriers, including their transitions to approach roadway barriers. Requirements for temporary or movable barriers, retrofitting of barriers on existing bridges, barriers protecting bridge substructure elements, or barriers protecting other roadside hazards are not included.

Information on highway temporary barriers and some guidance on the design of existing bridge barrier upgrades may be found in AASHTO (2011).

Section 12 also specifies requirements for the design of highway accessory supports. Requirements concerning the performance of highway accessories, such as illumination levels and sign message presentation and legibility, are not included. The design of supports supporting small signs and traffic signals on cables spanning between supports is not covered by the Code. Guidance on the design of these structures can be found in AASHTO (2015).

C12.4 Barriers

C12.4.1 General

Barriers on bridges receiving salt are exposed to a highly corrosive environment. To ensure long-term performance, these barriers need to be made from materials that can withstand this environment or protected by an adequate protective coating.

Damaged barriers need to be repaired quickly with minimal disruption to traffic. On high-traffic volume roadways, consideration should be given to using traffic barrier types that are not expected to require major repairs after an accident. Other traffic barrier types should be designed with features such as anchorages that are unlikely to be damaged or cause damage to the bridge deck or primary load carrying members during an accident and modular construction using prefabricated sections that allow damaged sections to be repaired quickly.

Snow drifts will accumulate on a bridge deck behind the upwind barrier, particularly if a solid barrier is used. The use of an open barrier will allow more snow to be blown off the deck, provided that the openings in the barrier are not blocked by snow removal operations. Also, traffic barrier geometry that makes it difficult for snow to be removed right to the face of a barrier, e.g., a barrier set back from the face of a curb, can lead to vehicles ramping on the snow in front of the barrier and either vaulting over the barrier or rolling over after striking the barrier (Desmarchais and Baass, 1988).

Barriers should not prevent roadway users from having an adequate view of their surroundings. For instance, in situations where there is an intersection near the end of a bridge, the use of a barrier with a low profile or that is readily seen through might be required to maintain adequate sight distances. Barriers should also be substantial enough to provide roadway users with a feeling of safety. This is of particular concern on highly elevated or narrow bridges.

Section C13
Movable bridges

C13.1 Scope

History

The history of movable bridge design specifications can be traced back at least as far back as 1908 to C.C. Schneider's Paper No. 1071 in the June 1908 *ASCE Transactions*, Volume 60, page 258.

The basic content of the Schneider specification appears to have been adopted by AREA, CESA, and AASHO Movable Bridge specifications in 1922, 1927, and 1938, respectively. Early movable bridges designed using the standards outlined in the Schneider paper have proved to be very durable. In contrast, early twentieth-century proprietary movable bridge designs using less stringent requirements have been more problematic. In 1970, the AASHO (now AASHTO) *Movable Bridge Specifications* were revised to more closely match the then current AREA specifications. Thus, it appears that the Schneider specification and the succeeding AREA and AASHO *Movable Bridge Specifications* have successfully defined adequate design standards for typical movable bridges.

Various changes and additions have been made over the years to these specifications.

The Canadian Engineering Standards Association (now CSA Group) published the first standard specification on movable bridges in 1927 as CESA A20-1927. An updated edition was prepared by CSA in 1960, designated CSA S20-1960, that incorporated the experience gained in the design of many movable bridges on the St. Lawrence Seaway. Updates to the Standard were issued up to 1969, after which it was withdrawn.

The void since 1969 was filled with the arrival of a new section in the the 2000 edition of the Code dedicated to movable bridges.

C13.5 General

C13.5.2 Type of deck

The use of open deck grating systems is discouraged. Open bridge decks allow salt-laden water onto the bridge-supporting elements, leading to severe deterioration and shorter service life. Open bridge decks also, in general, reduce bridge safety and ride quality.

C13.5.6 Protection of traffic and pedestrians

C13.5.6.3 Movable barriers

Movable resistance gate

The following considerations should be taken into account during a risk analysis for the implementation of movable resistance gates used as an energy absorbing system:

- maximum speed of the traffic;
- configuration of the road before the barrier;

Section C14
Evaluation

C14.1 Scope

The cost of upgrading or replacing a bridge may be great; Section 14 offers a method of evaluation by setting safety levels that are consistent and appropriate for the bridge or bridge component being evaluated. The intention is to avoid some of the conservatism that, in the interests of simplicity, may have been incorporated into the design provisions. Section 14 is not to be used for design.

The philosophy behind Section 14 is to determine a suitable safety level for each element of the bridge under evaluation, which varies with the type of element failure to be expected: more safety is required for an element that fails abruptly; less safety is required for an element that will retain its capacity after failure and may shed its load to other members without collapse.

The main parameters in setting the required safety level, defined by the reliability index, β, are the behaviour of the element being considered, the behaviour of the structural system of which the element is a part, and the degree of inspection of the bridge. Inspection is important to ensure that the bridge is indeed in the condition the evaluator is assuming, and to verify that it has carried previous loads without distress.

Need for evaluation

The need for evaluation may be created by any of the following:
- observed or suspected defects, deterioration, or damage that may affect load capacity;
- an anticipated increase in actual or legally permitted traffic loading or loading effect;
- a change in road classification;
- a review of an existing load limit posting;
- any alteration to a bridge that may affect its live load carrying capacity;
- an application for a permit to allow a vehicle not conforming with legal limits to cross a bridge;
- an unsatisfactory performance of the bridge in terms of serviceability or fatigue; or
- a bridge, in a moderate or severe earthquake zone, not constructed to modern seismic standards, including detailing.

Application

Section 14 applies to bridges where the level of load enforcement in place is typical for highways in Canada. It addresses the evaluation of an existing bridge for the purpose of
- establishing the legal load limit;
- establishing whether a bridge can safely carry trucks to the legal limit;
- establishing a restricted load limit for a bridge without sufficient capacity to carry legislated vehicle weights. The lack of sufficient capacity may occur because of deterioration or because the bridge was built for a lesser capacity or both;
- determining whether passage of an overload vehicle may be permitted;
- assisting in the development of programs for the repair, strengthening, and replacement of bridges; and
- assessing the adequacy of the bridge’s probable performance during an earthquake.

Section C15
Rehabilitation and repair

C15.1 Scope

Rehabilitation is the implementation of engineered design to address structural defects or deficiencies by modification, replacement, or repair of bridge elements with the intent of extending the service life of an existing structure, and often involves upgrading of elements to meet current standards. Repair, a subset of rehabilitation, is usually localized and may not involve upgrading of the element to meet the current standard.

Emphasis on remaining service life and assessment of ongoing deterioration in the rehabilitation design have been added in the 2019 edition of the Code.

Material specifications, rehabilitation procedures, and maintenance procedures are not covered in Section 15; however, they are important factors in ensuring the effectiveness of the rehabilitation design and achieving the service life objectives.

C15.3 General requirements

Clause 15.3 sets out a framework for the use of Section 15 for rehabilitation design. Relevant comments are given in the commentaries on the appropriate clauses. Further useful material can be found in *Structure Rehabilitation Manual* (MTO, 2007), *Structural Financial Analysis Manual* (MTO, 1990), and *Cathodic Protection Manual for Concrete Bridges* (MTO, 1993), *Manuel d’entretien des structures* (MTQ, 2016), *Bridge Assessment Guidelines* (Alberta Transportation, 2016), and other references listed under the appropriate specific clauses.

C15.3.2 Limit states

For a rehabilitation designed to allow passage of a controlled vehicle using load factors from Section 14, permanent deformations can occur. The consequences of such permanent deformations or other minor damage should be considered.

C15.3.3 Condition data

The intent of the condition survey report is to provide a comprehensive collection of data and an objective representation of the condition of the structure, based on which the rehabilitation design engineer will interpret and assess the condition of the structure prior to completing the rehabilitation design. The accuracy and completeness of the condition data is essential.

Due to construction tolerances and errors, and alterations made to components during and after original construction, actual dimensions may differ from those shown on available drawings. Overall bridge dimensions and types of components should therefore be verified in the field.

In the event that it is impossible or infeasible to collect certain pertinent condition data, the rehabilitation design engineer should make the owner aware of the risks and alternatives available for

Section C16
Fibre-reinforced structures

C16.1 Scope

The scope of Section 16 has been enhanced in the 2019edition of the Code, as summarized below:
- durability, especially with reference to the 2019 edition of CSA S807 and the 2014 edition of CSA S808; and
- limit on bend-radius-to-bar-diameter ratio of bent FRP bars to guard against sustained load.

New provisions have been added on
- the development of bundled bars, bent bars, spliced bars, and headed bars;
- combined shear and torsion;
- compression members;
- deck slabs with FRP stay-in-place structural forms;
- strut and tie model for deep beams, corbels and short walls;
- barrier walls; and
- retrofit for confinement and lap splice clamping.

An Annex on GFRP composite bridges has also been added in Section 16.

C16.2 Definitions

Fibre-reinforced composite

Examples of fibre-reinforced composites are numerous; however, the composites included in Section 16 are polymers reinforced with continuous synthetic fibres and concrete reinforced with randomly distributed steel or synthetic fibres.

Strand/Tendon

A distinction is made between these two prestressing components. Strands refer, for example, to pultruded FRP rods and tendons to bundled FRP rods stressed by a common anchor. Sometimes individual anchors are attached to single strands. In Section 16, these strands are referred to as tendons.

C16.3 Abbreviations and symbols

C16.3.1 Abbreviations

The abbreviations listed below are those used in this Section of the Commentary. These are additional to those used in the Code.

CFRP — carbon-fibre-reinforced polymers

FRC — fibre-reinforced concrete

FRP — fibre-reinforced polymers

HDPE — high-density polyethylene

RC — reinforced concrete

Section C17

Aluminum structures

C17.1 Scope

Section 17 specifies requirements for the design, fabrication, and erection of aluminum highway and pedestrian bridges. See Section 4 for seismic design and Section 7 for aluminum soil-metal structures. Where permitted in Section 12, the contents of Section 17 may also be applied to highway accessory structures. Additional information on the design of highway accessory structures is provided in Section 12. The requirements of this Section are largely based on CSA S157 and Section 10 of this Code.

C17.2 Definitions

Definitions common to metal bridges generally follow those given in Section 10. Definitions specific to aluminum follow those given in CSA S157.

C17.3 Abbreviations and symbols

C17.3.1 Abbreviations

Abbreviations common to metal bridges generally follow those given in Section 10. Abbreviations specific to aluminum follow those given in CSA S157.

C17.3.2 Symbols

Symbols common to metal bridges generally follow those given in Section 10. Symbols specific to aluminum follow those given in CSA S157. The symbols listed below are those used in this Section of the Commentary. These are additional to those used in the Code.

A = area of two flanges

b = width of the plate element; element width; overall flange width, mm

c = distance from the centroid to the extreme fibre in compression, mm

c_1 = coefficient which depends upon the ratio b/a

d_i = the distance from the centre of rotation to the centre of the i th element, mm

d_o = hole diameter

E = modulus of elasticity, MPa

F_c = stress to cause flange buckling, MPa

F_e = theoretical elastic stress to cause lateral-torsional buckling, MPa

F_f = external factored force applied to each bolt, N

F_k = bending stress to cause lateral-torsional buckling, MPa

F_{sc} = shear buckling stress, MPa

F_{so} = the maximum shear stress at the tension corner of the web, MPa

F_u = ultimate strength of aluminum