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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:

Although North American mills do not add boron to carbon and low alloy steels, there have been instances of off-shore steels containing elevated levels of boron. Boron has been shown to increase hardenability (Devletian et al., Lee) which can lead to loss of ductility in the heat affected zone and weld cracking. For steel members that are to be welded, it is recommended that chemical analysis include testing for boron. Unless special welding procedures are developed, a limit of 0.0008% should be imposed. For members with bolted or riveted connections, testing for boron is not necessary. Boron is intentionally added to quenched and quenched and tempered steels. The limits specified in CSA G40.21 or ASTM A709, as applicable, should apply.

- The resistance factors used in Section 10 of this Standard have been obtained from reliability analysis using statistical strength data gathered from North American mills (Kennedy and Baker, 1984; Schmidt and Bartlett, 2002). For offshore steels, the mean strength and standard deviation may be different from the values typically found from North American mills. Kennedy and Gad Aly (1980) have found a mean bias coefficient (ratio of measured value to design value) for the static yield strength of 1.101 and a corresponding coefficient of variation of 0.0915. The corresponding values for yield strengths shown on mill test reports, which are showing dynamic yield strength values, are 1.18 and 0.085, respectively. Any mean bias coefficient from mill test reports significantly lower than 1.18 and coefficient of variation significantly larger than 0.085 should be investigated to verify that the steel meets the intent of the Code for the required level of safety.

C10.4.5 Bolts

Both ASTM F3125 Grades A325/A325M and A490/A490M heavy hex bolts may be used in bridge structures. Corresponding grades of twist-off bolts (also known as tension control bolts) satisfying the requirements for grades F1852 (tensile strength of 825 MPa) and F2280 (tensile strength of 1035 MPa) may also be used. Although metric size bolts are cited in the Code, the availability of such bolts in the quantities required should be checked with suppliers before ordering. The most common high-strength bolts are 1, 7/8, and 3/4 inch in diameter.

Galvanized A490/A490M and F2280 bolts are not permitted because of the possibility of delayed fracture in service as the result of hydrogen embrittlement that can take place either during the galvanizing process or in service during corrosion of the zinc coating (Kulak et al., 1987). In accordance with ASTM F3125, hot-dip galvanizing of grade F1852 bolts is not allowed due to the high resulting friction between the bolt and nut and the difficulty this would represent in controlling the level of pretension in twist-off style bolts.

When twist-off style bolts are used, the installation procedure outlined in the RCSC specification (2014) should be followed. The pretension reached in twist-off bolts depends on the lubricant used on the bolts as part of the manufacturing process. Both the depth of the annular groove and the lubricant are calibrated to reach the desired level of pretension in the bolt, which is approximately 5% greater than the specified pretension (70% of the tensile strength of the bolt). Therefore, bolts should not be relubricated before installation. It should be noted that delayed installation of tension-control bolts and installation at low temperature have been found to lead to a significant reduction in pretension (Maleev 2007). Installation of twist-off bolts should therefore be completed promptly after the sealed container in which the bolts are shipped has been opened.

C10.4.6 Welding electrodes

Welding electrodes are produced with varying levels of diffusible hydrogen control and this level of control must be considered as part of the selection of welding electrode to be used. The designation of electrodes is typically done through the designation "Hx" within an electrode classification, where x represents the number of milliliters of diffusible hydrogen in 100 grams of deposited weld metal (e.g. H8 = 8ml of diffusible hydrogen in 100 grams). A lower H number represents an electrode with better diffusible hydrogen properties. CSA W59 specifies minimum H numbers for a given material. CSA W48 specifies the testing requirements for each H number."

C10.4.9 High-strength bars

High-strength bars used as load-carrying elements in bridges must possess sufficient toughness to prevent brittle fracture. Although toughness requirements are not specified in ASTM A722/A722M, the same requirements as for primary tension members or fracture-critical members must be applied to high-strength bars when such bars are used as primary tension or fracture-critical members. These requirements ensure that high-strength bars can resist brittle fracture in the presence of small flaws or small fatigue cracks at low temperature and under impact loading.

C10.4.11 Identification

The values for yield and tensile strength reported on mill test reports are not to be used for design. Only the specified minimum values that are published in product standards and specifications may be used because the statistical variation in the strength has been assessed in developing resistance factors. For the same reason, it is not acceptable to re-classify a steel to a higher grade, unless it is done at the mill at the time of production. Furthermore, when steel is identified by tests, the specified minimum values of the steel, once classified, should be used as the basis for design.

C10.4.13 Pins and rollers

Forged and annealed and forged and normalized material may conform to ASTM A668 or be of similar shaft material. It is strongly recommended that material availability be investigated before specifying.

C10.5 Design theory and assumptions

C10.5.1 General

The limit states to be satisfied must be satisfied at all stages of the lifecycle of the bridge, including fabrication, transportation, erection, and service life of the structure.

C10.5.2 Ultimate limit states

The requirements at ultimate limit states generally relate to collapse of all or part of the structure. Load effects, increased to account for statistical variations in the loads, are compared to the resistances modified by a resistance factor to account for possible variations in strength.

C10.5.3 Serviceability limit states

See Clause C10.5.4.

C10.5.4 Fatigue limit state

At fatigue and serviceability limit states, satisfactory performance needs to be ensured. The prevention of deformation and slip in bolted connections needs to be considered at a load level likely to occur relatively few times during the life of a structure.

C10.5.7 Resistance factors

The resistance factor of 0.95 for flexure is based on Kennedy and Baker (1984) who, for a target reliability index of 3.5, recommended a value of 0.93 for both welded and rolled sections. For compression members, they obtained resistance factors ranging from 0.85 to 0.93, and the use of 0.90 represents a maximum under-design of 5%. See also Kennedy and Gad Aly (1980) and Bartlett et al. (2003). The resistance factors given in Clause 10.5.7 are virtually the same as used in AASHTO (1994), except for cables.

Resistance factors for cables

Traditional stress levels

Most long-span bridges supported by cables, such as suspension and cable-stayed bridges, have been designed using working stress design methods. Because the stress-strain curve for steel wire used for bridges does not have a well-defined yield plateau, it has been common to express the allowable stress as a fraction of the specified minimum tensile strength of the wire or strand, although some, e.g., Roberts (1967), have related it to the 0.2% offset stress.

Following the traditional allowable stress design approach, the dead load stress, f_D, plus the live load stress including dynamic load allowance, f_L is some fraction, C, of the specified minimum tensile strength, F_u, or

$$f_D + f_L = CF_u$$ **Equation C10.5.7(a)**

Typical values of C for main cables of suspension bridges have been between 0.4 and 0.5, as shown in Table C10.1, where the values of 0.36 for Tancarville and 0.37 for Ogdensburg-Prescott are considered more conservative than current practice.

Table C10.1
Allowable stress fraction, C, of specified minimum tensile strength, F_u, for main cables of suspension bridges
(See Clause C10.5.7.)

Bridge	C	Reference
Bosporus	0.44	Brown and Parsons, 1976
Forth	0.40	Roberts, 1967
Severn	0.46	Roberts, 1970
Tancarville	0.36	Carrière, 1961
Lions' Gate	0.46	Banks, 1942
A. Murray MacKay	0.46	Dorton, 1968
Ogdensburg-Prescott	0.37	Modjeski and Masters, 1961

Values of C for suspension bridge hanger ropes bent over the cable bands are generally lower, with values of 0.33, 0.35, and 0.29 for Lions' Gate, A. Murray MacKay, and Ogdensburg-Prescott, respectively, and are based on tests of a two-part rope over a sheave. For single part hangers, bridge strand is usually used with C values similar to those for main cables such as 0.44 for the Severn Bridge.

For cables in cable-stayed bridges, the allowable stress fraction specified is usually between 0.40 and 0.45 (Freyssinet International, 1987). The PTI (1990) recommended 0.45. A typical value is 0.42 as used for the Alex Fraser Bridge (Dorton et al., 1996).

Secondary effects

These values of the allowable stress fraction could be considered conservative for structures for which the dead load is well controlled and is typically 70 to 90% of the total load. However, Roberts (1967) explains the low allowable stress as follows:

> Considering that the direct cable tension in the design condition can be calculated accurately, that approximately nine-tenths of this is due to dead load which will never be increased, and that adjustment during the spinning process ensures that the tension is shared evenly between the wires, it might be thought that a higher working stress would be justified. The reason for the relatively low allowable direct stress is the presence of secondary stresses, about which quantitatively little is known, at the points of restraint afforded by the saddles and cable bands. These take the form of bending stresses in individual wires and redistribution of stress across the cable section, and they arise because of the large deflexions of the cable as the dead load builds up during the last stages of erection as well as under live loads and temperature. Further information on this may be found in a record of work carried out at the author's instigation by Wyatt (1963). If the secondary stresses could be calculated accurately or demonstrated to lie within certain limits, then an increase of the direct working stress might indeed by justified, and in the interests of economy ought to be adopted.

Calibration

Considering these unknown, but significant, effects as well as the possibility of corrosion, which tends to occur at points of maximum secondary stress, it is appropriate that a limit states design code should maintain essentially the same levels of safety as have been found satisfactory. These levels are inherent in the typical values of C given in Table C10.1. In limit states design terms, with factored load effects and factored resistances, we therefore have

$$\phi_{tc} T_u \geq 1.1D_1 + 1.2D_2 + 1.5D_3 + 1.8L$$ **Equation C10.5.7(b)**

where

ϕ_{tc} = the resistance factor for cables in tension which is to be determined

T_u = specified minimum tensile resistance

D_1 = dead load effect of factory produced components, excluding wood

D_2 = dead load effect of cast-in place concrete, wood, and nonstructural components

D_3 = dead load effect to wearing surface

L = live load effect

The values of the load factors in order in the LSD expression of 1.1, 1.2, 1.5, and 1.8, not known at the time of writing as the calibration exercise had not yet been carried out, are generally taken from the latest edition of the OHBDC except for the live load factor of 1.8. Some changes may be expected. The live load factor is based on Clause C3.8.3.1.3. Of the factors for dead load, the factor for overlay thickness of 1.5 is likely high for long span bridges where the thickness is more likely better controlled, but in any case, this is a small component of the total load for such bridges.

Ratios of live load to dead load

Equation C10.5.7(b) must be evaluated for different load fractions so that the effect of the different load factors can be assessed properly. Table C10.2 gives values of the live load as a fraction of the total load in the cables for a wide range of existing suspension bridges. The data presented in Table C10.2 indicates a trend between the main span length and the ratio of the live load to total load ratio: as the main span length increases, the live to total load ratio decreases.

Table C10.2
Ratios of live total load for suspension bridge cables
(See Clause C10.5.7.)

Bridge	Year	Main span, m	$L/[L + D]$
Verrazano Narrows	1964	1299	0.12
Golden Gate	1937	1280	0.16
Mackinac	1958	1158	0.18
Bosporus	1973	1074	0.15
George Washington	1931	1067	0.12
George Washington	1962	1067	0.17
Forth	1964	1006	0.11
Tacoma Narrows	1940	854	0.15
Tacoma Narrows	1950	854	0.19
San Francisco-Oakland	1936	704	0.30
Bronx Whitestone	1939	701	0.21
Delaware Memorial	1951	655	0.17
Tancarville	1959	608	0.21
Ambassador	1929	564	0.21
Williamsburg	1903	488	0.25
Lions' Gate	1938	473	0.21
Mid-Hudson	1930	457	0.25
Angus L. Macdonald	1955	441	0.24
A. Murray Mackay	1971	427	0.31
Ogdensburg-Prescott	1961	350	0.24

The live-to-dead-load ratios for cable-stayed bridges depends on the cable configuration and also varies for the cables within any one bridge. For the Alex Fraser Bridge, the ratio varies between 0.12 and 0.27.

The dead load portion of the total load, 1.0 minus the live load portion given in Table C10.2, is further split into the three categories for dead load. However, because D_3 is a relatively small component — perhaps 0.05 to 0.08 of the total — and because the load factors for categories D_1 and D_2 are about the same value, the proportions of dead load do not affect the calculations for the resistance factor appreciably.

Calculation of resistance factor, ϕ_{tc}

Table C10.3 gives typical values of the decimal fractions of the live load and the three different dead load categories of the total load for long span bridges. The live load is likely to be 0.10 to 0.30 of the total load for most cable-supported bridges. The range from Table C10.2 is between 0.11 to 0.31 for suspension bridges. However, future suspension bridges are unlikely to be built with spans less than 600 m and, with that limit, the range is 0.11 to 0.21, except for the San Francisco–Oakland Bridge with a ratio of 0.30. This live loading is unusual, with two levels and a provision for electric trains. For rail transit bridges and two-level bridges, special studies to establish appropriate resistance factors should be carried out.

Table C10.3
Ratios of live and dead loads to total load for various long-span cable-supported bridges, with corresponding resistance factors for three values of *C*
(See Clause C10.5.7.)

1	2	3	4	5	6	7
Load category	**Low L High D_1**	**High L Low D_1**	**Typical cable-stayed, c.i.p. deck**	**Alex Fraser Low L**	**Alex Fraser Typical**	**Alex Fraser High L**
L	0.10	0.30	0.22	0.12	0.22	0.27
D_1	0.84	0.29	0.23	0.71	0.63	0.59
D_2	0.02	0.35	0.49	0.08	0.07	0.07
D_3	0.04	0.06	0.06	0.09	0.08	0.07
Total	1.00	1.00	1.00	1.00	1.00	1.00
Value of C	**Value of resistance factor, ϕ**					
0.45	0.54	0.62	0.60	0.55	0.58	0.60
0.42	0.50	0.57	0.56	0.52	0.54	0.56
0.40	0.48	0.55	0.53	0.49	0.52	0.53

The last three rows of Table C10.3 give resistance factors that correspond to three values of the allowable stress ratio, C, in Table C10.1.

Column 2 is typical of a large suspension bridge. For the Forth Bridge, C = 0.40 and thus ϕ = 0.48. For the more recent Severn and Bosporus bridges, C = 0.46 and 0.44, respectively, yielding ϕ = 0.54 for the mean value of C of 0.45.

For the Alex Fraser cable-stayed bridge, designed with C = 0.42, from columns 5, 6, and 7, resistance factors of 0.52 to 0.56 and typically 0.54 are found.

The resistance factor for the "typical" cable-stayed bridge of column 4, with C = 0.42 is 0.56. This column also represents a shorter span suspension bridge.

Traditionally, the hangers on suspension bridges are designed more conservatively because of bending over the cable bands and the greater likelihood of corrosion, and usually values of C of 0.40 or less are used. On the other hand, the hangers have a greater ratio of live load than the main cables. On the Forth Bridge, for example, with a live-to-total-load ratio of 0.29, Column 3 gives ϕ = 0.55 for C = 0.40. However, the live load on hangers can be significantly higher than 0.30 of the total load and reaches 0.58 for the Lions' Gate Bridge and 0.62 for the A. Murray MacKay. For the latter, even with a low value of C = 0.35, the resistance factor, ϕ = 0.53, showing that the resistance factor calculated for hangers is not sensitive to changes in the live load ratios.

Overall, a value of ϕ in the range of 0.52 to 0.56 appears appropriate for cable-stays, suspension bridge cables, and suspension bridge hangers. For simplicity, a value of ϕ = 0.55 is recommended for all cables applied to the specified ultimate tensile strength of the wire for cables that are spun in place and to the strand or rope for cables or hangers that are preformed.

Values of ϕ are not reported in the literature as cable supported bridges have been designed by working stress methods. However, a limit states approach using the partial factors of BS5400 was used to determine the strengthening required for the Severn Bridge. The partial factor on the minimum strength used to assess the main cables was 1.65.

The reciprocal of this factor, 0.61, is the resistance factor. Because a slightly higher ϕ for evaluation is not unusual, this lends support to the recommended value of 0.55.

Resistance factors could, however, be developed for cables for specific long span bridges, when warranted by economic conditions, as they have been for the steel reinforcement, prestressing strands, and concrete components of the Confederation Bridge, even to the extent of assigning reliability indices related to the consequences of failure of the particular component (MacGregor et al., 1997).

C10.5.8 Analysis

Although the strength of a section is normally defined, according to section classification, as the ultimate strength or yield strength, only elastic analyses procedures are allowed. The consideration of plastic redistribution of bending moments along the length of the structure, for example, requires approval.

C10.5.9 Design lengths of members

C10.5.9.2 Compression members

C10.5.9.2.1 General

The unbraced length and the effective length factors may be different for different axes of buckling. Information about effective lengths is given in Ziemian (2010), Tall et al. (1974), and in Annex G of CSA S16.

The last sentence introduces the concept that effective length factors depend on the potential failure modes — how the member would fail if the forces (and moments) were increased sufficiently — as discussed in subsequent clauses.

C10.5.9.2.2 Failure modes involving in-plane bending

The effective length is taken as the actual length for members for both sway and braced frames, provided that the sway effects are considered in determining the stress resultants acting on the ends of the members. Clause 10.5.9.2.2 now extends the case to members of braced frames as well. Thus, the approach has been made more consistent.

When the end moments and forces acting on a beam-column have been determined for the displaced configuration of the structure, that is to say, the sway effects have been included in the design force effects, the in-plane bending strength of the beam-column can be determined by analyzing a free-body of the member isolated from the remainder of the structure. In-plane displacements between the ends that contribute to failure arise from the end-moments and forces acting on the actual length. When the actual member length and the actual (or at least approximate) deflected shape are used, the analysis of the free-body will give a result close to the correct member strength. Recourse to effective length factors is neither necessary nor appropriate.

When the actual member length is used together with the interaction expressions of Clause 10.9.4, the analysis is approximate and the in-plane member bending strength obtained will tend to be conservative. This simply arises because the value of the compressive resistance inherent in the interaction expression by using a length equal to the actual length (a K factor of 1.00) is that corresponding to single curvature buckling. For any other deflected shape, having accounted for sway effects, the compressive resistance is greater because the points of inflection of the deflected member shape are less than the member length apart. Under these circumstances, a better estimate of the strength can be obtained when the compressive resistance is based on the actual distance between points of inflection.

Therefore, the relatively simple but sometimes conservative approach given in the Code, which obviates the use of effective length factors, is presented as the usual procedure.

C10.5.9.2.3 Failure modes involving buckling

The compressive resistance of an axially loaded column depends on its end restraints, as does the out-of-plane resistance of a beam or beam-column under uniaxial strong axis bending. In both cases, the buckling resistance is a function of the “effective length” of the column or beam-column. The failure of an axially loaded column or a beam subjected to bending about strong axis bending is a bifurcation mechanism.

C10.5.9.2.4 Compression members in trusses

The potential failure modes of compression members in trusses fall into either in-plane bending or buckling modes. The effective length factors are, therefore, either taken to be equal to 1.0 or are based on the restraint at the ends. Thus, the following situations arise for in-plane and out-of-plane behaviour:

- In-plane behaviour:
  A compression member with bolted or welded end connections and with in-plane joint eccentricities acts in-plane as a beam-column with axial forces and end moments that can be established. It can be isolated from the structure and is designed as a beam-column based on its actual length, that is, with an effective length factor of 1.0.
  A compression member with bolted or welded end connections and without in-plane joint eccentricities, designed as an axially loaded member, has end restraints, provided that all members meeting at the two end joints do not reach their ultimate loads (yielding in tension or buckling in compression) simultaneously. The effective length factor depends on the degree of restraint. This is likely the case for highway bridge trusses designed for vehicle loads. It also occurs for trusses in which some members are oversize, for example, trusses with constant size chords. All members do not fail simultaneously, and the effective length factors may be less than one. If, however, all members reach their ultimate loads simultaneously and none restrain others, the effective length factor should be taken as 1.0.
- Out-of-plane behaviour:
  Unless cross-frames or members exist at the end joints under consideration, the restraint for out-of-plane buckling is small and should be neglected. Provided that no out-of-plane displacement of the member ends occurs, an effective length factor of 1.0 is appropriate. Cross-frames, adequately