STEEL BRIDGE MEMBER RESISTANCE – AASHTO COMPARED TO OTHER INTERNATIONAL CODES

Keywords

Steel design; FE Software; Codes of Practice; Eurocode; AASHTO; CSA S6-14; Buckling

Abstract

This paper contrasts the different approaches to member resistance calculations in AASHTO 8th edition, Eurocode EN1993-2:2006 and Canadian Bridge Design standard CSA S6-14.

An example steel truss footbridge is used to compare resistances and utilizations determined from each Code (on the basis of identical loading). AASHTO is found to be lacking two interaction checks, to be unconservative in one check and over-conservative in another by comparison to the Eurocode – and prohibits the use of some members based on slenderness alone. For its part, the Eurocode is found to be more opaque in expressing one interaction check and to be considerably more voluminous in the calculations required to obtain similar results.

The paper also examines the effects of some basic analysis assumptions on the load effects and therefore utilizations.

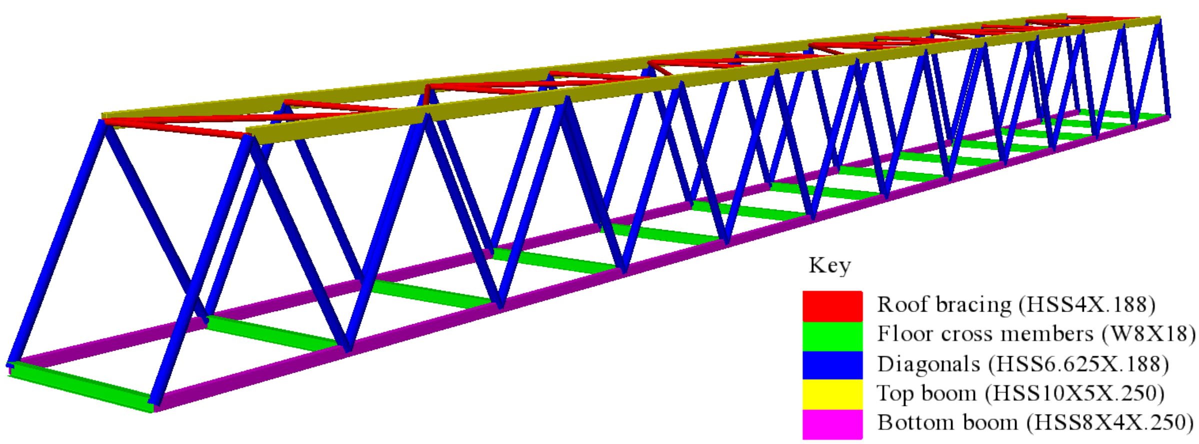

Figure 1: Example truss footbridge

Introduction

The ancestry of the AASHTO LRFD Bridge Design Specifications can be traced back – beyond the first edition published in 1994, through the Standard Specifications – more than 80 years.

In the UK, 27 other EU member states, and several non-member states as well, the Eurocodes are the current bridge design specifications. Their development can be traced to the formation of Comité Européen de Normalisation (CEN) in 1961, but work intensified in the 1980s and 90s, in parallel with the political desire to eliminate obstacles to trade. Bringing together engineers from nations – each having a long engineering heritage and well-established Codes of Practice (COPs) of their own – resulted in a far-reaching appraisal of the various methods used and available research, which the Eurocodes reflect. The objective might not have been to establish best practice, but it was a beneficial outcome. The final parts, completing the Eurocode Suite were published in 2007.

The Eurocode part applicable to Steel Bridges is EN1993-2 [1]. It is in fact a compendium of modifications and addenda to the 'steel building' part, EN1993-1-1 [2]. The clause numbering in the two documents corresponds without repetition and so clause references in this paper can be traced to either document.

Despite their quite independent development, we would expect that the underpinning engineering theory surrounding strengths of materials, buckling and the like would lead to a great deal of similarity between these COPs – and likewise for others, such as their Canadian counterpart. This paper explores the corresponding Articles for steel-only (non-composite) member resistance, comparing AASHTO 8th [3], Eurocode [1,2] and CSA S6-14 [4], using a steel truss bridge (Figure 1) to illustrate some of the differences.

For brevity, code references in this paper are prefixed with an ‘A’ for AASHTO, ‘E’ for EN1993-1-1 or EN1993-2, and ‘C’ for CSA S6-14; other code references are explicit.

The comparison of these COPs has been made possible by the development of detailed code-checking facilities for these – alongside other international standards – within the LUSAS software, with validation against many published examples [5,6,7,8,9], under the same lead engineer.

Contrasting Approaches

The articles for design of steel members are organized quite differently in the different COPs.

The sections in AASHTO: Tension members (A6.8), Compression members (A6.9) I-section flexural members (A6.10), Box-section flexural members (A6.11) and Miscellaneous flexural members (A6.12) reflect an expectation that the engineer will identify a member type – and then the relevant checks are set out. By contrast, the Eurocodes set out checks which are broadly intended to be applicable to any and all members, with the primary division being whether they concern ‘resistance of cross-sections’ (E6.2) or ‘buckling resistance of members’ (E6.3). CSA S6-14 is generally more similar to Eurocode in this respect.

The AASHTO approach to I-sections in flexure is stress-based (see extract from calculations in Figure 2 below), while Eurocodes and CSA S6-14 determine sectional resistances and buckling capacities separately, and use interaction formulae to consider biaxial bending, together with axial force – a single unified approach covering different section types. All three COPs include interaction formulae

LUSAS

Member 101: Floor cross members (W8X18) — ***Pass***
Element 23, Internal point 2
Units: kip,ft,kslug,s,F

Utilization of a section with discretely braced top flange in compression (lateral torsional buckling) — *6.10.8.1.1-1*

$$R_{ltb} = \frac{|f_{bu}| + \frac{1}{3}|f_l|}{\phi_f F_{nc,ltb}} = \frac{|(-2.58046E3)| + \frac{1}{3}|(-22.1194E-12)|}{(1.0)(5.2015E3)}$$

$= \mathbf{0.4961}$

Since $|f_l| \leq f_{l,lim}$ then f_l is acceptable — *6.10.1.6*

Since $L_b > L_{b,lim}$ amplified first-order values are used for f_l

Second-order compression-flange lateral bending stress (determined by amplifying first-order values) — *6.10.1.6-4*

$$f_l = \left(\frac{0.85}{1 - \frac{|f_{bu}|}{F_{cr}}}\right) f_{l1}$$

Figure 2: Extract from calculations – I-section floor beams – AASHTO Flexural check

for axial force and flexure including buckling, but only Eurocode and CSA S6-14 include combined shear and flexure.

AASHTO gives limiting slenderness ratios or proportion limits for tension members, compression members and flexural members (A6.8.4, A6.9.3, A6.10.2 etc); CSA S6-14 gives these only for tension and compression (C10.8.1.2, C10.9.1.3) while in the Eurocodes there are no such hard limits.

These sorts of root-and-branch differences make direct comparison of articles/clauses practically difficult and so examples help to explore the differences in approach and outcomes. Using examples from [6], [7] and [8], differences in ‘utilization’ up to 17% are observed for most checks, with the larger differences primarily due to the use of elastic section properties in AASHTO, rather than plastic section properties (for class 1 and 2 sections) in EN1993-1-1 [2] for the resistance of members in bending and differing approaches for lateral torsional buckling.

There were, however, some instances of differences greater than 17%, generally for checks on combined effects. Notably, AASHTO has no check for combined ‘bending and shear’ or combined ‘bending, shear and axial force’, unlike the Eurocode (E6.2.8 to E6.2.11 inc.).

Terminology

The terms and notation used in the three COPs are, of course, somewhat different. Of particular note, the Eurocode ‘section classes’ may be unfamiliar to some readers:

- Class 1 sections can mobilize plastic bending resistance and have sufficient rotational capacity to allow plastic global analysis;
- Class 2 sections can mobilize plastic bending resistance but have insufficient rotational capacity to allow plastic global analysis.

Class 1 and 2 cross sections would broadly correspond to ‘compact’ in the AASHTO commentary to A6.10.8.2.1 (see Figure 3 below). The two classes are treated differently in building design but alike for bridges as plastic global analysis is not generally permitted in EN1993-2 (E5.4.1).

- Class 3 sections have a bending resistance corresponding only to first yield (‘noncompact’ in Figure 3).
- Class 4 sections have a bending resistance corresponding to plate buckling in compression at a stress somewhat below yield (‘slender’ in Figure 3).

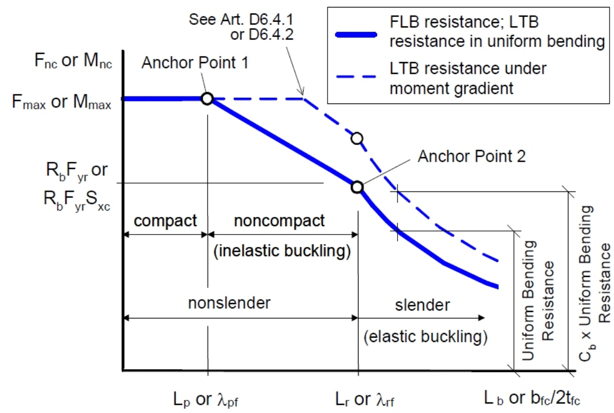

Figure 3: Basic form of all I-section compression flange flexural resistance equations, reproduced from AASHTO [3] Fig. C6.10.8.2.1-1

Example Truss Bridge

The following sub-sections consider an example truss footbridge, based loosely on a real project but modified to use three different section profiles in order to allow a more comprehensive comparison:

1. top and bottom booms are rectangular HSS, 10×5×0.25 and 8×4×0.25 respectively
2. floor beams are I-sections, W8×18
3. All other members are round HSS (diagonals 6.625×0.188 and roof members 4×0.188)

The bridge (Figure 1 above) is a simply supported single span of 144’, with two parallel 11’ deep Warren trusses of 12 equal bays, positioned at 10’ centers.

Steel grade is ASTM A500 Grade C to all hollow sections and AASHTO M270M/M270 Grade 50 to the W-beams. The equivalent yield and tensile strengths for these materials are used for the other COPs.

UDL applied to the floor beams is of magnitude 1.8kip/ft, representing 0.05ksf dead load and 0.1ksf live load. Load factors, which differ from code to code, are omitted in order to focus on the differences in member resistance articles.

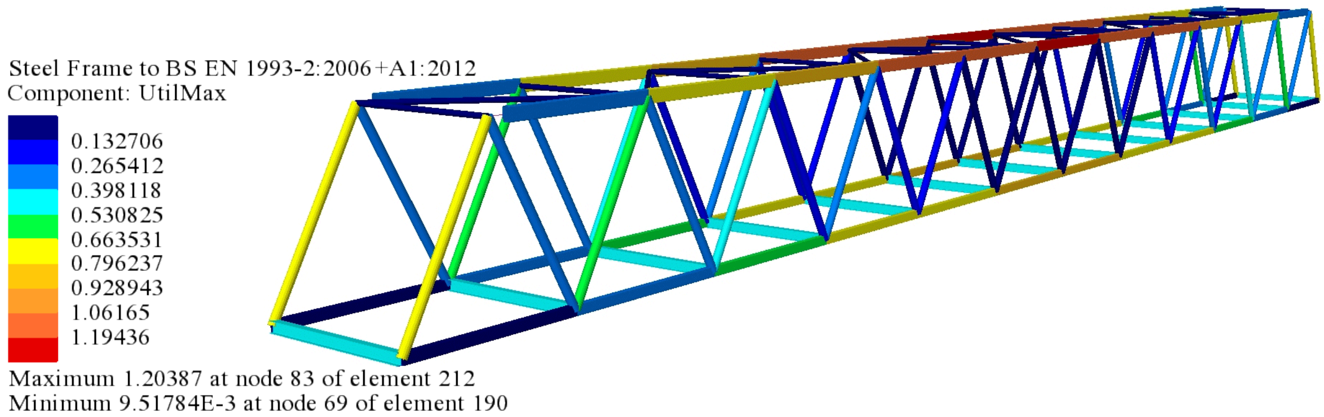

Figure 4: Example truss footbridge showing member utilizations

It has been assumed that the bridge deck does not provide any lateral restraint to the floor beams or bottom boom of the truss. While noting that in reality any decking system will provide some restraint in at least one horizontal direction, this assumption has been made in order to introduce lateral torsional buckling as a design criterion for comparison purposes.

The bridge was modelled in LUSAS, using thick (Timoshenko) beam elements. Fixed-ended connections were assumed for the continuous top and bottom booms only – pinned connections for all truss diagonals and bracing members. Every member was checked to AASHTO (7th and 8th editions), Eurocodes and CSA S6-14 using the optional steel design module – see, for example, the contour plot of maximum utilization in Figure 4. The resistances, utilizations and Article references are drawn from the rendered output which is provided by the LUSAS software (see extract in Figure 2 above).

A linear elastic (first-order) analysis was used for AASHTO and CSA S6-14, for which amplification factors are available (A4.5.3.2.2b, C10.9.4). The Eurocode offers no such approximations, leading the engineer towards a second-order (geometrically nonlinear or 'large displacement') analysis. This was the approach taken for the example truss bridge and carried out using LUSAS.

Comparison of Member Resistances

Tension

AASHTO Article 6.8 and EN 1993-1-1 clause 6.2.3 are very similar. Both consider both the plastic resistance of the gross cross-section and the ultimate (fracture) resistance of the net cross-section. Where plastic resistance dominates, the Eurocode is marginally less conservative, with a partial factor γ_{M0}=1.0 from clause E6.1(1) as opposed to the resistance factor ϕ_y=0.95 from A6.5.4.2 (identical to that in C10.5.7). For fracture, the resistance factor, ϕ_u=0.8, for AASHTO and CSA S6-14 corresponds with the Eurocode partial factor γ_{M0}=1.25.

For the rectangular HSS in the example bridge (bottom boom), utilization is governed by fracture resistance in both AASHTO and Eurocode. The resistances are identical (234kip) if bolt holes are assumed punched full size since AASHTO factor R_p=0.9 applies – matching a fixed 0.9 coefficient used in E6.2.1(2)(b)[1]. If holes are assumed drilled (or sub-punched and reamed), then R_p=1.0, matching the Canadian COP which has no 0.9 factor. AASHTO seems the most rational of the three COPs in this instance.

For the round HSS diagonals, again, fracture resistance governs but the AASHTO resistance (116kip) is 26% lower than Eurocodes (157kip) due to the shear lag factor, U=0.7364, in this case – determined from Table A6.8.2.2-1 (on the basis of connection using a gusset plate 8" long). In the Eurocode, shear lag may be neglected by reference to EN1993-1-5 [10] clause 3.1(1) and no provision seems to be made for shear lag occurring local to fasteners. Agreement between AASHTO and the

1 According to [6], this 0.9 coefficient was introduced "following a statistical evaluation of a large number of test results for net section failure of plates"; there is no specific reference to fastener holes.

Canadian code is closer, where C10.8.1.3.2.2 accounts for shear lag effects in bolted tension elements. Again, the Eurocode seems to be a little lacking by comparison to AASHTO.

For the round HSS roof members, the governing criterion is yielding in AASHTO 8th due to the aforementioned ϕ_y=0.95, giving a resistance of 91kip, whereas in the Eurocode calculation, the yield and fracture resistances coincidentally assume the same value (93kip). The Canadian COP gives a somewhat lower resistance on account of a different value for the shear lag factor.

Table 1: HSS tensile resistances (kip)

	AASHTO	Eurocode	CSA S6
Rect. HSS 8×4×0.25	234	234	249
Round HSS 6.625×0.188	116	157	131
Round HSS 4×0.188	91	93	78

Tension in the I-section members is negligible in the example structure.

Compression

Compressive resistance was governed by buckling in the example structure.

The three COPs all consider compressive resistance by way of both yielding and buckling strength. In the Eurocode, these are quite separate checks (E6.2.4, E6.3.1) whereas in AASHTO and the Canadian code, they appear in the same article (A6.9.4, C10.9.3 respectively).

Buckling is expressed very differently in Eurocode E6.3.1 as compared to AASHTO A6.9.4.1. The Eurocode approach hangs on a reduction factor (χ) applied to the yield strength, determined from slenderness ($\bar{\lambda}$) and an imperfection factor (α). α refers to a buckling curve, selected according to section shape, limiting thicknesses and grade of steel (see Figure 5 below). $\bar{\lambda}$ is based upon the critical elastic buckling resistance – for which no formulae are given to assist the engineer in the code itself.

AASHTO, on the contrary, moves directly from the yielding resistance (P_o) and the elastic critical buckling resistance (P_e), assuming the column strength curve 2P of Galambos [11] – giving a range of helpful formulae for the determination of P_e.

Imperfection factors for buckling curves

Buckling curve	a_0	a	b	c	d
Imperfection factor α	0,13	0,21	0,34	0,49	0,76

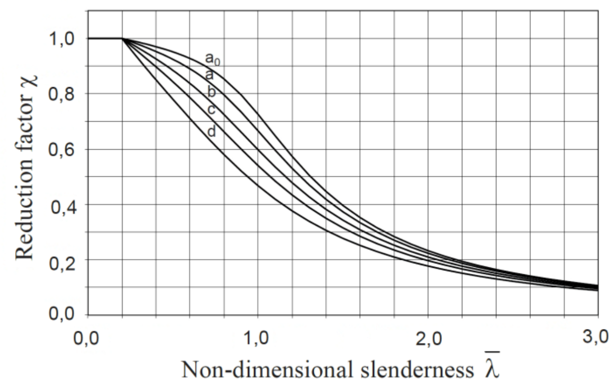

Figure 5: Buckling curves and imperfection factors, reproduced from EN1993-1-1 [2] Fig. 6.4

Perhaps surprisingly – since the Eurocode approach seems more refined in the area of buckling curves – the AASHTO approach gives considerably larger values for buckling resistance as seen in Table 2 below – these are unconservative by comparison to the Eurocode. The 8th Edition returns resistances 4% greater than those from the 7th Edition [12] due to the changes to A6.9.4.2.2.

Table 2: HSS compressive resistances (kip)

	AASHTO	Eurocode	CSA S6
Rect. HSS 10×5×0.25	223	180	†
Round HSS 6.625×0.188	115	94	102
Round HSS 4×0.188	‡	21	‡

† For the rectangular HSS 10×5×0.25 the section is class 4 to C10.9.2.1 and was not calculated.

‡ For the round HSS to the longer roof members the slenderness limits of A6.9.3 and C10.9.1.3 are exceeded. As previously described, there are no slenderness limits in EN1993-2 – and in the example structure, the maximum member utilization is less than 8%.

Compression in the I-section members is negligible in the example structure.

Flexure

Since the AASHTO approach to I-sections (A6.10.8) is stress-based rather than member-resistance based, utilizations must be compared, as in Table 3 below. These are only directly comparable because the example bridge floor beams are in uniaxial bending.

As for the compression checks, flexural resistance considers yielding and buckling strength – separately in the Eurocode (E6.2.5, E6.3.2), and together in a single article AASHTO and CSA S6-14 (A6.10.8, C10.10 respectively). Once again, the Eurocode approach is based upon a reduction factor (χ_{LT}), imperfection factor (α_{LT}) referring to a buckling curve, and requires the critical elastic buckling moment, for which – again – no formulae appear in the code. LTB is excluded as a concern when slenderness limits in E6.3.2.2(4) are met.

For all three COPs, LTB governs in Table 3 and AASHTO and the Eurocode utilizations practically agree. On the contrary, the Canadian COP gives a significantly lower utilization, based on a larger buckling resistance (58kip-ft compared to 45kip-ft from the Eurocode). This is in spite of identical elastic critical buckling strength and coefficient for moment distribution in both sets of calculations (taken from [13] and [14] respectively for the Eurocode, since it offers no formulae for these). The difference arises in C10.10.2.3(a), perhaps largely accounted for by a fixed 1.15 coefficient of unknown provenance.

Table 3: I-section flexural utilizations

	AASHTO	Eurocode	CSA S6
W8×18	0.50	0.51	0.39

Table 4: HSS flexural resistances (kip-ft)

	AASHTO	Eurocode	CSA S6
Rect. HSS 8×4×0.25	44	48	42
Rect. HSS 10×5×0.25	71	64	50
Round HSS 6.625×0.188	28	28	26
Round HSS 4×0.188	10	10	9

HSS resistances are calculated in A6.12.2.2 and are compared directly with the other COPs in Table 4 above.

The section capacities in flexure for round HSS show good agreement, but may differ somewhat between COPs for less slender members, since the limiting values for slenderness are different.

For the rectangular HSS, LTB governs for AASHTO and Eurocode, and the values also show good agreement. In CSA S6-14, the section resistance governs.

Shear Fz

Shear for loads parallel to the web in the floor members were checked for the example structure.

Table 5: I-section shear resistances (kip)

	AASHTO	Eurocode	CSA S6
W8×18	50	60	51

The checks in A6.10.9 and C10.10.5.1 are of a similar format; the plastic shear resistance is factored down to account for web buckling, based on slenderness considerations. In the Eurocodes, web buckling checks to EN1993-1-5 [10] are required when the slenderness limit in E6.2.6(6) is not met; else the plastic resistance from E6.2.6(2) governs.

For the example structure I-section floor beams, webs are not slender according to any of the three COPs, so the shear resistance for loads parallel to the web, shown in Table 5 above, is governed by the plastic resistance. The difference in values is primarily due to differences in shear areas: based on the overall depth in CSA S6-14, the web depth in AASHTO and a more nuanced approach in E6.2.6(3)(a), which gives the largest value of the three.

The resistance factors also have a bearing, with a more conservative $\phi_{s,shear}$=0.95 given in C10.5.7; ϕ_y=1.0 in A6.5.4.2 which corresponds with the Eurocode partial factor γ_{M0}=1.0 from clause E6.1(1).

Interaction Formulae

'Interaction formulae', which consider members subject to coexistent load effects, are included in all three COPs, with the Eurocode offering the most comprehensive options, as per Table 6.

Table 6: Articles concerning interaction

	AASHTO	Eurocode	CSA S6
Bending and shear	n/a	E6.2.9	C10.10.5.2
Bending and axial tension	A6.8.2.3	E6.2.10	C10.8.3
Bending, shear and axial force	n/a	E6.2.11	n/a
Bending and axial compression (buckling)	A6.9.2.2	E6.2.10, E6.3.3	C10.9.4.1

Bending and shear

The Eurocode and CSA S6-14 both allow consideration of coexistent bending and shear in members. No comparable check appears in AASHTO.

E6.2.9 (which refers to EN1993-1-1 [2] clause 6.2.8) uses a bending resistance which is lowered by considering the strength of material in the web to be reduced by a factor derived from the shear utilization. By contrast, the interaction formula of C10.10.5.2(c) is a simple summation of utilizations with the moment component reduced by a fixed coefficient of 0.727 and the shear component by 0.455. Of the two approaches, the Eurocode seems more rational.

For the example structure, bending and shear is not a considerable issue, since maximum bending occurs with zero shear and maximum shear with zero bending. However for other cases, such as continuous beams, this would not be the case.

Bending and axial tension

The treatment of 'bending with axial tension' is similar in AASHTO and the Canadian COP. A6.8.2.3 gives two alternative linear summations of utilization ratios where the axial component or moment components may be reduced in light of the significance of the axial contribution. C10.8.3 uses a simple summation of utilization ratios, curiously referring to a uniaxial bending condition only.

For tension-dominated members in the example structure, the 'bending and axial tension' interaction governs in AASHTO and CSA S6-14: in general, a little flexure also applies.

The Eurocode takes a different approach. In E6.2.10 (which refers to EN1993-1-1 [2] clause 6.2.9) a reduced plastic resistance for Class 1 and 2 cross sections (based on the section utilization under axial forces) is used. For Class 3 sections, a stress check using the net section properties is called for. For Class 4 sections, the stress check is performed using net *effective* section properties[2], along with a summation of utilization ratios that incorporates an eccentricity for axial load to reflect the location of the neutral axis of the effective section. Biaxial bending conditions are also accommodated.

For the example structure, the sections are all Class 1, 2 or 3 and the utilizations calculated as described above are not comparable to the other COPs, being really a 'modified moment utilization' for the tension-dominated member. E6.2.10 is not governing for any of the members in this structure.

Bending and axial compression (buckling)

In AASHTO 8th the interaction formulae for 'bending and axial compression' can follow one of two approaches. For nonslender sections in A6.9.4.2.1, a simple summation of utilization ratios is required – there are no reduction factors, and there is no requirement to take account of the magnification of moment which can arise due to axial loads and second-order effects. In other cases, A6.9.2.2 requires that moment magnification should be taken into account by way of a second-order elastic analysis or the approximate single step adjustment specified in A4.5.3.2.2b. With that in hand, two alternative summation of utilization ratio are offered, incorporating reduction factors (where the axial component or moment components may be reduced in light of the significance of the axial contribution). The critical members in this study broadly fell under A6.9.4.2.1.

C10.9.4 uses three alternative summations capturing cross-sectional strength, overall member strength and LTB strength. Second-order effects are incorporated by way of amplification factors (U_{1x}, U_{1y}) on the moment in each direction. Interaction

[2] Effective sections being those reduced to account for local buckling effects as per EN1993-1-5 [10] clause 4.2.

factors are introduced only for class 1 and 2 sections of I-shaped members (C10.9.4.4).

The Eurocode approach is once again more elaborate. E6.3.3 uses two alternative linear summation of utilization ratios, using the buckling reduction factors (χ and χ_{LT}) previously mentioned and interaction factors (k_{yy}, k_{yz}, k_{zy}, k_{zz}) calculated by one of two methods offered in EN1993-1-1 [2] Annex A and B. Second-order effects should be taken into account for all members according to E6.3.3(3), plus Table 6.7 for Class 4 members, however no amplification factor approach is given, leading the engineer towards a second-order (large displacements) analysis, as used in this example.

The Eurocode calculations are certainly more lengthy than those carried out using AASHTO or CSA S6-14. However, considering that this would be the governing case for most members in which there is axial compression, it might seem appropriate to spend the time trying to obtain a refined value.

In the example truss, the 'bending and axial compression' interaction broadly governs for members that are not tension-dominated. The top boom is over-utilized in the Eurocode (utilization 1.205) but passes in AASHTO (utilization 0.995).

Table 7: Utilizations for Tension-dominated members

Section / COP	Bottom boom	2nd diagonal	Roof (10')	Governed by	Article
	Rect HSS 8×4×0.25	**Round HSS 6.625×0.188**	**Round HSS 4×0.188**		
AASHTO	0.866	0.538	0.0222	Bending and axial tension	A6.8.2.3
Eurocode	0.824	0.394	0.0111	Tension check	E6.2.3
Canada	0.825	0.476	0.0229	Bending and axial tension	C10.8.3

Table 8: Utilizations for Compression-dominated members

Section / COP	Top boom	End diagonal	Roof diagonal	Governed by	Article
	Rect HSS 10×5×0.25	**Round HSS 6.625×0.188**	**Round HSS 4×0.188**		
AASHTO	0.995	0.547	a1	Bending and axial compression (buckling)	A6.9.2.2
Eurocode	1.205	0.669	0.097		E6.3.3
Canada	c1	0.617	c2		C10.9.4.1

Table 9: Utilizations for Flexure-dominated members

Section / COP	Floor beams	Governed by	Article
	W8×18 (I-section)		
AASHTO	0.496	Bending and axial compression (buckling)	A6.9.2.2
Eurocode	0.523		E6.3.3
Canada	0.393	Flexure (LTB)	C10.10

Utilization Ratios for Truss Bridge

The maximum utilization ratios for sample members from the Example bridge are given in Table 7, Table 8 and Table 9 above, with notes, ordered by Code of Practice, in the following sections.

AASHTO

a1. Slenderness limit for compression in 15’ 7.5” Round HSS 4×0.188 was not satisfied (A6.9.3).

The slenderness limits given in AASHTO are similar to those given in AISC-360-16 [15] sections D1 (tension) and E2 (compression), although they are suggested, rather than being mandatory. The AISC commentary gives some more information and notes, among other things relating the limits to construction economics, ease of handling and minimizing inadvertent damage.

Eurocode

The Eurocode was the only COP for which a full set of utilizations was obtained, because it does not disallow members on the basis of slenderness as per notes a1 and c2.

The British Standard which preceded the Eurocode – BS5400-3 [16] in the UK likewise gave no member slenderness limits.

CSA S6-14

c1. For the rectangular HSS 10×5×0.25 the section is class 4 to C10.9.2.1 and M_r was not calculated as part of this exercise.

c2. Slenderness limit for compression in 15’ 7.5” Round HSS 4×0.188 is not satisfied (C10.9.1.3).

Comparison of critical members

For tension-dominated members in the example truss, ‘bending and axial tension’ interaction checks govern in AASHTO and CSA S6-14, whereas in the Eurocode, this interaction is handled by a reduction in flexural resistance and so tension checks govern. The approach seems to result in a marginal under-estimate of the utilization of these members by the Eurocode. Otherwise there is reasonable agreement.

For compression-dominated members in the example truss, ‘bending and axial compression (buckling)’ interaction checks govern in all three COPs.

For the top boom, the utilization ratio from the EN1993-2 is 20% more onerous than AASHTO, primarily because values for compressive resistance in AASHTO were considerably larger, as per Table 2 above.

It is noted that the ‘bending and axial compression (buckling)’ interaction check was to A6.9.4.2.1. Use of A6.9.2.2 would have introduced some differences – the use of the approximate moment magnifier of A4.5.3.2.2b and a reduction factor of $^8/_9$ upon those moments. The net result would likely be not much changed.

The Eurocode seems to adopt a more sophisticated approach to the buckling resistance (the use of multiple buckling curves as per Figure 5 above); to the moment magnification – the use of geometrically nonlinear (second-order) analysis; and to interaction factors (values of k_{zy}=0.93 and k_{zz}=0.62 determined from EN1993-1-1 [2] Annex B rather than fixed factors). One might have expected the more refined method to lead to a more accurate, less conservative, assessment of utilization – surprisingly, the reverse is true.

Regarding flexural checks, despite very different approaches, the AASHTO and Eurocode checks gave very similar results for both I-sections and HSS in the example structure. CSA S6-14 on the other hand might be regarded as a touch over-conservative by comparison to both other COPs for HSS and unconservative for I-sections, possibly owing to the fixed 1.15 coefficient in C10.10.2.3(a).

Modeling assumptions

Support conditions

The example truss was modeled with simply supported articulation: pinned at one bearing and guided/free so as to allow free expansion and contraction.

Errors in support conditions are some of the most common modeling mistakes seen in bridge models. The bridge engineer is inclined to pay a great deal of attention to every detail of the superstructure, while being perfectly content with coarse assumptions (such as infinite rigidity) for support conditions, and sometimes can neglect support conditions to the extent of defining them incorrectly. A common error in a bridge such as this is to define all bearings as pinned (fixed in all translations).

Even considering just a first-order analysis, this alters behavior of the structure considerably, affecting the tension in the bottom chord greatly. Instead of utilizations around 80% near midspan and 10% near the supports; midspan utilizations drop to about 20% and end utilizations are pushed >90%.

Member end conditions

In the example model pinned connections were assumed in the members other than top and bottom chord. If these are modified to be fixed in rotations, utilizations are considerably affected.

Floor beams W8×18 (I-section) would see a decrease in utilization of the order 50% → 38%. Roof members would also see a decrease in utilization of the order ¼ to ½

End diagonals (Round HSS 6.625×0.188) would see an increase in utilization of the order 66% → 77%.

Conclusion

The preparation of a set of calculations in standard typeface and format for each code, giving comparable results, allows a comparison of the calculation effort by way of 'numbers of pages'. For every 10 pages of AASHTO calculations there was 12 pages of Canadian calculations and 31 pages of Eurocode calculations, also demanding use of a geometrically nonlinear analysis approach.

The Eurocodes might be regarded as lacking by comparison to AASHTO in:

1. Tension check.
   - (a) AASHTO has factor Rp=0.9 if bolt holes are assumed punched full size, 1.0 if drilled or sub-punched and reamed. Better than CSA or Eurocode (where fixed factors of 1.0 and 0.9 apply respectively).
   - (b) AASHTO and CSA S6-14 make provision for shear lag occurring local to fasteners, Eurocode makes no such provision.
2. Compression check.
   - (a) The buckling resistances examined were rather conservative by comparison to AASHTO.
   - (b) Limiting slenderness ratios or proportion limits appearing in AASHTO but not in the Eurocode may be of pragmatic use.
3. Interaction checks.
   - (a) Bending and axial tension checks are handled by way of reduced moment resistances dependent upon axial load. In AASHTO A6.8.2.3 the 'summation of utilization ratio' approach seems to better capture the addition of effects leading to, as expected, a higher utilization.
   - (b) No approximate method for moment magnifiers is offered.

On the other hand, AASHTO might be regarded as lacking by comparison to Eurocode in:

1. Compression check.
   - (a) Eurocode utilizes an imperfection factor selected according to section shape, limiting thicknesses and grade of steel, rather than a single buckling curve. The values examined from AASHTO were unconservative by comparison to the Eurocode.
   - (b) Limiting slenderness ratios or proportion limits precludes use of some sections which are allowed in the Eurocode and found to have low utilizations.
2. Shear check. The more nuanced approach to the determination of shear areas in the Eurocode may produce a more accurate value for shear resistance
3. Interaction checks.
   - (a) Eurocode provides rules for interaction of bending with shear (E6.2.8), and bending, shear and axial force (E6.2.10), for which there are no comparable articles in AASHTO.
   - (b) Bending and axial compression (buckling) checks are more straightforward, but may be more prone to conservatism as compared to CSA S6-14 and especially E6.3.3.

It is noted that the analytical model used, even for a simple truss such as the example should be examined to ensure good assumptions including

- Correct support conditions
- Suitable member end conditions
- Use of first- or second-order analysis options

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