# CHAPTER J

# Strength Requirements Comparison of 1999 AISC LRFD Specification and Eurocode 4, Part 1.1

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**ABSTRACT**

Although basic analysis theories between the Eurocode 4 (CEN 1992) and 1999 AISC Specification (AISC 1999) are similar, the requirements for determining the strength of composite beams differ in many respects. This is particularly true when considering the design of shear connectors. This paper explores those differences through a comparative step-by-step discussion of major design aspects, and an accompanying numerical example. Several shortcomings of 1999 AISC Specification are identified and adjustments proposed.

## J.1 INTRODUCTION

1999 AISC Specification and Eurocode 4, referred to herein as 1999 AISCS and EC4-1994-1.1, respectively, have similar approaches with respect to the design of composite beams. The nominal flexural strength of a composite section is determined from a plastic cross sectional analysis. Both base their requirements for calculating the strength of shear connectors on the model originally proposed by Grant et al. (1977), although subsequent adjustments and revisions to this model have made the present methods used by the two codes significantly different.

Solutions of most typical beam configurations using both codes will generally have two common threads. First, the EC4-1994-1.1 will typically require more design checks. Examples of this include checking the resistance of slab and connectors to longitudinal shear, shear connector and member ductility checks, and checks for member combined shear and bending. None of the noted checks are required by 1999 AISCS. Secondly, design strengths

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will typically be lower when computed by EC4-1994-1.1. There are two sources of the conservative nature of EC4-1994-1.1. The first is related to the safety provisions. While both codes are reliability based, safety coefficients are applied in different manners, and the end result is a lower design strength obtained by EC4-1994-1.1. The other aspect lies in the requirement for additional checks. For instance, the checks in EC4-1994-1.1 for slab and connector resistances to longitudinal shear will in many cases result in requirements for increased reinforcement, slab depth or the number of studs and their arrangement.

The following sections provide a theoretical and practical review, focusing on the strength requirements, of the most important differences between the two codes. A numerical example illustrating the differences is presented. Ideas are suggested on how the AISC Specification could be improved by taking advantage of existing experimental results and specification language reflected in EC4-1994-1.1.

## J.2 DESIGN STRENGH OF SHEAR CONNECTORS

This is the area in which the two specifications differ the most, as EC4-1994-1.1 is significantly more conservative then 1999 AISCS. The example given in this paper illustrates an application in which the results from 1999 AISCS are the closest to the results from EC4-1994-1.1 in terms of calculated strength of shear connectors, however the difference between the two is still substantial. As will be seen in the example, the same number of studs results in a 19% smaller flexural strength in EC4-1994-1.1. The difference in this particular case is largely due to application of safety factors.

EC4-1994-1.1 stipulates the application of partial safety factors to both load and resistance. The former are similar in application to the load factors used in Load Resistance Factor Design (LRFD). The latter are similar in application to LRFD strength reduction factors, $\phi$. EC4-1994-1.1 stipulates application of a material based partial factor for shear conectors, ?$_v$. The EC4-1994-1.1 Clause 2.3.3.2 specifies ?$_v$ = 1.25. The reduced strength is then used in the calculations of nominal moment strength, which is then reduced by its own partial factors of safety for concrete and steel, ?$_c$ and ?$_a$, respectively. In contrast, 1999 AISCS does not require application of resistance factors to the strength of shear connectors. An overall flexural strength reduction factor, $\phi_b$, of 0.85 is applied to the nominal flexural strength, $M_n$.

While others have calculated resistance factors for shear studs in LRFD format (Zeitoun 1984), the methods presented in this paper and work by Galambos and Ravindra (1976) and Mujagic and Easterling (2004) represent a rational manner of accounting for the statistical characteristics of shear studs in the overall margin of safety.

Partial factors of safety for loading are specified by Eurocode 1 (CEN 1992), while load factors to be used in 1999 AISCS are specified by applicable building codes. However, in typical applications, the overall effects of partial safety factors for load in Eurocode 1, and load factors in U.S. practice are approximately the same.

It should be noted that the "unreduced" nominal shear connection strength in EC4-1994-1.1 calculated in the numerical example differs only 4% from that calculated by 1999 AISCS, 1581 kN vs. 1510 kN. However, in some cases in which multiple studs per ribs are used, the EC4-1994-1.1 calculated strength can yield a value half that calculated by the 1999 AISCS. Based on a study by Stark and Hove (1991), $N_r$ was limited to 2 in computations (3 in 1999 AISCS), Eq. J.8 is limited to 0.75 when more than one stud per rib is present, and coefficient of 0.7 was reduced to its present value from the original 0.85. The goal was to eliminate, or at the very least reduce, unsafe and inconsistent outcomes of Eqs. J.5 and J.6, originally parts of the European design practice. The focus, however, was clearly on ribs with multiple studs. The 1999 AISC recognized and addressed the unconservative nature of Eq. J.6, but only for configurations with one stud per rib, indicating that more research is needed to draw conclusions when multiple studs are used (1999 AISC). As demonstrated by this study, further revisions are needed to improve the 1999 AISCS model. A suggested alternative may be the model proposed by Rambo-Roddenberry et al. (2002), which addresses the unconservative nature of the equations proposed by Grant et al. (1977).

## J.3 EFFECTIVE SLAB WIDTH

The effective slab width for simply supported composite beams is calculated in the same way by both specifications.

## J.4 DESIGN MOMENT STRENGTH of the COMPOSITE SECTION

With the exception of the application of safety provisions, the determination of the cross-section moment strength based on plastic stress distribution is identical. The same is true for the strength determined by superposition of elastic stresses. The difference occurs with respect to the determination of which moment strength is to be used. The EC4-1994-1.1 contains a complex set of rules to determine which analysis is appropriate. These rules evaluate both web and flange buckling criteria and take into consideration the transverse position of the stud and its ability to prevent the buckling of the top flange (ECCS 2000).

The 1999 AISCS criteria are adequate with regard to the assessment of the web stability. The 1999 AISCS criteria for local flange buckling should apply and a mandated 457 mm (18 in.) stud spacing is adequate to prevent lateral buckling for virtually all I-shaped beams.

The biggest difference pertains to the consideration of the ductility of shear connectors. Both 1999 AISCS and EC4-1994-1.1 recognize the occurrence of limited slippage along the slab-beam interface. Due to this, shear connectors are required to have certain ductility, also known as slip capacity. This property is essential to allow the full inelastic redistribution of longitudinal shear among shear connectors to occur. Lack thereof usually results in premature and sudden member failures. Further, significant slip may make it harder to predict and control the deflection of a composite member. The 1999 AISCS Commentary considers headed studs to be “ductile” without further quantifying the property. By the EC4-1994-1.1 criteria, a connector can be considered ductile if its slip capacity is at least 6 mm (0.24 in.). Headed studs meet this criterion.

Both issues are well controlled by establishing an adequate minimum degree of shear connection for a given member and configuration of shear connection. There are no requirements in 1999 AISCS specifying the minimum degree of composite action. Some authors have argued that this minimum limit should be set to 50% in order to adequately control excessive slip (McGarraugh et al. 1971). The EC4 sets this limit to 40% or more, depending on the type of shear connection, slab, cross-section, and beam length. Intuitively, the most critical component is the member length. To control the ductility and prevent excessive slip, the condensed EC4-1994-1.1 criteria (Eqs. J.1 and J.2), valid for both solid and ribbed slabs, can be used in AISC Specification as a guideline for minimum degree of shear connection:

$$\%\,\text{Composite} = \frac{N}{N_f} = \min\begin{Bmatrix} 0.4 \\ 0.25 + 0.03L \end{Bmatrix} \quad \text{(SI Units)} \qquad \text{(Eq. J.1)}$$

$$\% \text{ Composite } = \frac{N}{N_f} = \min \begin{Bmatrix} 0.4 \\ 0.25 + 0.01L \end{Bmatrix} \quad \text{(U.S. Units)} \qquad \text{(Eq. J.2)}$$

The other requirements given in EC4-1994-1.1 are overly restrictive and their significance should be further studied. Mathematical models to account for the effect of slip have been derived (Yam 1966, Dall'Asta 2000), however, they are computationally intensive, and not practical for day to day design application.

## J.5 COMPOSITE BEAM CHECK for SHEAR

In both specifications, any contribution from the concrete slab is disregarded and the shear strength is calculated based on the resistance of the bare steel section. In both methods, the check for hot rolled members is based on the Von Mises yield criteria.

## J.6 RESISTANCE of SLAB and CONNECTORS to LONGITRUDINAL SHEAR

While this issue is very well addressed by EC4-1994-1.1 rules, it is almost entirely ignored by 1999 AISCS. The only provision in 1999 AISCS regarding this subject is contained in the commentary, which is also contained in EC4-1994-1.1, and it is stated that the amount of transverse reinforcement contained in a slab should be equal to no less than 0.002 times the slab area in longitudinal direction. As will be seen in the example, the procedure stipulated by EC-4-1994-1.1 takes into account the contributions of concrete slab, transverse reinforcement, and formed steel deck in resisting the longitudinal shear. In the opinion of the authors, the EC4-1994-1.1 procedure should be replicated and included in either the main body or the commentary of the 1999 AISCS. Such a check would particularly be of benefit in beams with high beam depth to slab thickness ratios. Such beams are likely to experience high longitudinal shear and too little of contribution from the concrete slab to resist it.

## J.7 CHECK for COMBINED BENDING and SHEAR

Traditionally, the AISC Specifications have not required the check for combined bending and shear, as the combined effect of the two was deemed insignificant. The exception to this was the requirement for such a check for plate girders. However, the check for combined bending and shear can be of significance, especially at composite moment connections and continuous members, where framing members are generally subject to both maximum shear and bending. In the example given in this paper, where a simply supported composite beam was considered, the load carrying capacity was reduced 11% as a result of this check, which is significant. Introduction of this check into AISC Specification is not realistic. However, it is in the opinion of the authors that such a check constitutes a good design practice.

## J. 8 CHECK of APPLICABLE SPACING and DETAILING REQUIREMENTS

While more detailed criteria are given by EC4-1994-1.1, the authors believe the simple requirements given by 1999 AISCS in this regard are adequate. These include the minimum and maximum stud spacing, minimum height of stud above rib, stud placement with respect to the flange edge, etc. Possible improvements can include a requirement to place the studs on the strong side of the rib, which is an issue accounted for in the previously mentioned model proposed by Rambo-Roddenberry et al. (2002). Also, whenever possible, when single stud per rib is used, subsequent stud should be placed in staggered fashion to promote stability of both sides of the top flange. In this regard, it is interesting to note the EC4-1994-1.1 allows for the possibility that an otherwise non-compact flange could be classified as compact in a composite beam if the longitudinal stud spacing and the distance from the stud center to the edge of the flange fall within specified limits. In this particular example, this provision was of no consequence, because all studs are welded directly above the beam web and the beam flange satisfies the compactness criteria. This provision may be of value in cases where built-up beams are used, as they are often designed with non-compact flanges for economical considerations. The 1999 AISCS does not contain a similar provision, and external restraints, such as provided by shear studs, cannot be considered in the evaluation of flange compactness.

## J.9 DESIGN EXAMPLE

The following example of a simply supported partially composite beam is used to illustrate the differences in strength requirements of 1999 AISCS and EC4-1994-1.1. In some cases common notation and variables were changed to allow easier comparison and avoid confusion. For easier comparison, the final result is expressed in terms of maximum factored line load the beam can carry, since the effects of load factors in both specifications are practically the same for this application. Accompanying discussion evaluates provisions for each limit state presented relative to each other and identifies potential deficiencies, especially as they relate to 1999 AISCS. In the following example, $F_{yw} = F_{yf}$, thus both will be denoted with either $F_y$ or $f_y$ in the example, depending on whether a EC4-1994-1.1, or 1999 AISCS computation is being carried out. English units are provided in parenthesis.

**Strength and geometry parameters:**

| | |
|---|---|
| $L$ | = 10.67 m (35 ft) |
| $S_b$ | = 2.44 m (8 ft) |
| $d_s$ | = 127 mm (5 in.) |
| $f'_c \approx f_{ck}$ | = 27.58 N/mm$^2$ (4.0 ksi) |
| $w_c$ | = 2322.9 kg/m$^3$ (145 lb/ft$^3$) |

**Concrete modulus of elasticity:**

Equation J.3 reflects U.S. practice, and Eq. J.4 is given in EC4-1994-1.1.

$$E_c = w_c^{1.5}(0.043)\sqrt{f'_c} = 2322.9^{1.5}(0.043)\sqrt{27.58} = 25281\ \text{N/mm}^2\ \text{(3667 ksi)} \qquad \text{(Eq. J.3)}$$

$$E_{cm} = 30500\left(w_c/2400\right)^2 = 30500(2322.9/2400)^2 = 28571\ \text{N/mm}^2\ \text{(4144 ksi)} \qquad \text{(Eq. J.4)}$$

**Steel beam properties:** W410x53 A572M Gr. 345 (W16x36 A572 Gr. 50)

| | |
|---|---|
| $w_b$ | = 0.52 kN/m (36 lbs/ft) |
| $A_s$ | = 6810 mm$^2$ (2342 in.$^2$) |
| $d$, $h_a$ | = 403 mm (15.9 in.) |
| $h$ | = 346 mm (13.6 in.) |
| $t_w$ | = 7.50 mm (0.295 in.) |
| $b_f$ | = 177 mm (6.99 in.) |

| | |
|---|---|
| $t_f$ | = 10.9 mm (0.430 in.) |
| $F_y$, $f_y$ | = 345 N/mm$^2$ (50 ksi) |

**Stud parameters:** Studs will be placed in favorable (strong) side of each rib, directly above the beam web.

| | |
|---|---|
| D | = 19.0 mm (0.75 in.) |
| $A_{sc}$ | = 283.5 mm$^2$ (0.4418 in.$^2$) |
| $H_s$ | = 88.9 mm (3.5 in.) |
| $F_u$, $f_u$ | = 448.18 N/mm$^2$ (65.0 ksi) |
| N | = 17 per half-span |
| $N_r$ | = 1 |

**Deck parameters:** *Vulcraft* 2VLI ga. 20

| | |
|---|---|
| $h_r = h_p$ | = 50.8 mm (2 in.) |
| $w_r$, $b_0$ | = 152.4 mm (6 in.) |
| t | = 0.91 mm (0.0358 in.) |
| $F_{yp}$, $f_{yp}$ | = 276 N/mm$^2$ (40.0 ksi) |

**Reinforcement:** Welded wire fabric 6x6-W1.4xW1.4

| | |
|---|---|
| $F_s$, $f_{sk}$ | = 414 N/mm$^2$ (60 ksi) |
| $A_{wire}$ | = 9.03 mm$^2$ ( 0.0140 in.$^2$) |
| Square size | = 152 mm (6 in.) |
| x | = 68.1 mm (2.68 in.) |

## 1. Design strength of shear connection:

1999 AISCS: $Q_n = (SRF)(0.5)(A_{sc})\sqrt{f_c' E_c} \le A_{sc} F_u$ (Eq. J.5)

$$SRF = \frac{0.85}{\sqrt{N_r}}\left(\frac{w_r}{h_r}\right)\left[\left(\frac{H_s}{h_r}\right) - 1.0\right] \le 0.75 \quad \text{(Eq. J.6)}$$

$$SRF = \frac{0.85}{\sqrt{1}}\left(\frac{152.4}{50.8}\right)\left[\left(\frac{88.9}{50.8}\right) - 1.0\right] = 1.91 \rightarrow \text{use } 0.75$$

$$Q_n = (0.75)(0.5)(283.5)(1/1000)\sqrt{(27.58)(25281)} \le (283.5)(448.18/1000)$$

$$Q_n = 88.8\ \text{kN}\ (20.0\ \text{kips})$$

$$\Sigma Q_n = (17)88.8 = 1510\ \text{kN}\ (339\ \text{kips})$$

EC4-1994-1.1: $$P_{Rd} = k_1 \cdot \min\left\{0.8\frac{f_u}{\gamma_v}\left(\frac{\pi(D)^2}{4}\right); 0.29\alpha D^2 \frac{\sqrt{f_{ck}E_{cm}}}{\gamma_v}\right\} \qquad \text{(Eq. J.7)}$$

$$k_1 = \frac{0.70}{\sqrt{N_r}}\frac{b_0}{h_p}\left(\frac{H_s}{h_p} - 1.0\right) \le 1.0 \qquad \text{(Eq. J.8)}$$

$$k_1 = \frac{0.70}{\sqrt{1}}\frac{152.4}{50.8}\left(\frac{88.9}{50.8} - 1.0\right) = 1.58 \rightarrow \text{use } 1.0$$

$$\gamma_v = 1.25; \frac{H_s}{D} = \frac{88.9}{19.0} = 4.7 > 4.0 \rightarrow \alpha = 1.0$$

$$P_{Rd} = 1.0 \cdot \min\left\{0.8\frac{448.18}{1.25}\left(\frac{\pi(19.0)^2}{4}\right); 0.29(1.0)(19)^2 \frac{\sqrt{(27.58)(28571)}}{1.25}\right\}$$

$$P_{Rd} = \{81326; 74346\} = 74.4\ \text{kN}\ (16.7\ \text{kips})$$

$$\Sigma P_{Rd} = 17(74.4) = 1265\ \text{kN}\ (284\ \text{kips})$$

**2. Effective slab width:**

1999 AISCS: $$b_{eff} = \{L/4 = 10.67/4 = 2.67\ \text{m}; S_b = 2.44 = 2.44\ \text{m}\}$$

$$b_{eff} = 2440\ \text{mm}\ (96.06\ \text{in.})$$

EC4-1994-1.1: $$b_{eff} = b_{e1} + b_{e2}$$

$$b_{e1} = b_{e2} = \min\{L/8 = 10.67/8 = 1.33\ \text{m}; S_b/2 = 2.44/2 = 1.22\text{m}\}$$

$$b_{eff} = 2(1220) = 2440\ \text{mm}\ (96.06\ \text{mm})$$

**3. Design moment strength of the section:**

1999 AISCS: The following check determines whether the nominal moment strength, $M_n$, may be determined from plastic stress distribution on the composite section (Fig. J.1). The requirement shown below is contained in Section I3.2 of 1999 AISCS:

$$\left\{\frac{h}{t_w}=\frac{361}{7.50}=48.1\right\}<\left\{3.76\sqrt{\frac{E}{F_y}}=3.76\sqrt{\frac{200000}{345}}=90.6\right\}$$

Thus, plastic analysis may be used, and design moment is equal to $\phi_b M_n$.

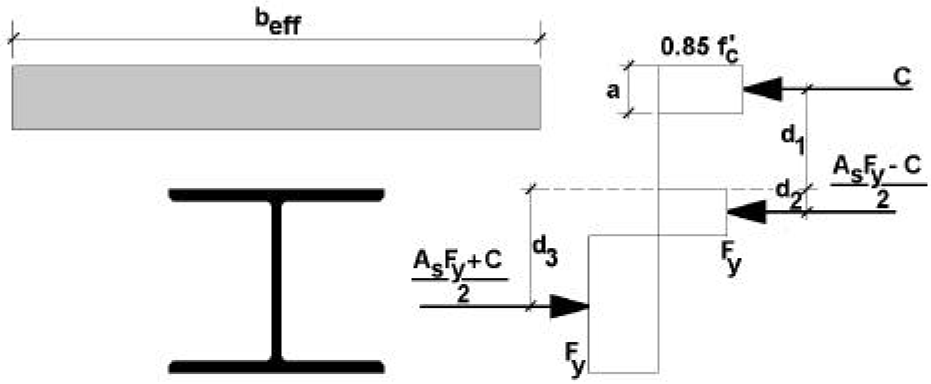


Fig. J.1 AISC Flexural Model of the Composite Beam

$$C=\min\left\{0.85f_c^{'}A_c; A_sF_y; \Sigma Q_n\right\} \qquad \text{(Eq. J.9)}$$

$$C=\min\left\{0.85(27.58)(2440)(127-50.8)/1000; (6810)(345)/1000; 1510\right\}$$

$$C=\min\left\{\begin{matrix}0.85(27.58)(2440)(127-50.8)/1000=4359\text{kN}\\ 6810(345)/1000=2349\text{kN}\\ 1510\text{kN}\end{matrix}\right\}$$

$C = 1510$ kN (339 kips)

Since $\Sigma Q_n < A_sF_y$, P.N.A. is in the steel. Let us assume it is in the flange.

$$a=\frac{C}{0.85(f_c^{'})(b_{eff})}=\frac{1510}{0.85(27.58/1000)(2440)}=26.40\text{ mm (1.04 in.)} \qquad \text{(Eq. J.10)}$$

Distance from top of upper flange to centroid of concrete force:

$$d_s-\frac{a}{2}=127-\frac{26.4}{2}=113.8\text{ mm (4.48 in.)}$$

Distance from top of upper flange to centroid of steel compression, assuming P.N.A. is in the top flange:

$$\frac{A_sF_y-C}{2}=\frac{6810(345/1000)-1510}{2}=420\text{ kips (94.4 kips)}$$

$$\frac{420}{(345/1000)(177)}=6.88\text{ mm (0.27 in.)} < t_f = 10.9\text{ mm}$$

Thus, P.N.A. is in the steel flange, and it is located $6.88/2 = 3.44$ mm below the top of upper flange.

Distance of from top of upper flange to the centroid of steel tension force:

$$\frac{d}{2} = \frac{403}{2} = 202 \text{ mm (7.93 in.)}$$

Minimum degree of shear connection check:

$$\textit{Degree} \text{ of Shear Connection } = \frac{1510}{2349} 100 = 64\,\%$$

Design moment capacity is computed:

$$\phi_b M_n = 0.9\left[C\left(d_1 + d_2\right) + A_s F_y \left(d_3 - d_2\right)\right] \qquad \text{(Eq. J.11)}$$

$$\phi_b M_n = 0.9\left[1510\left(\frac{113.8}{1000} + \frac{6.88}{1000}\right) + \left(6810\right)\left(\frac{345}{1000}\right)\left(\frac{202 - 6.88}{1000}\right)\right]$$

$$\phi_b M_n = 577 \text{ kNm (425 kip} \cdot \text{ft)}$$

EC4-1994-1.1: Classification of composite cross-section:

Spacing of the studs in lateral and longitudinal, $e_T$ and $e_L$, are defined by detailing requirements in EC4 Clause 6.41.5, and are used as primary parameters in determining whether the flange can be considered fully restrained. The outcome of this check determines the type of analysis to be used (EC4 Clause 4.3.2).

$$\left\{e_T = \frac{b_f}{2} = \frac{177}{2} = 89 \text{ mm}\right\} > \left\{9 t_f \sqrt{\frac{235}{f_y}} = 9\left(10.9\right)\sqrt{\frac{235}{245}} = 81 \text{ mm}\right\} \rightarrow \text{n.g.}$$

$$\left\{e_L = 368 \text{ mm}\right\} > \left\{ \left| \begin{array}{c} 15 t_f \sqrt{\frac{235}{f_y}} = 15\left(10.9\right)\sqrt{\frac{235}{345}} \\ 6\left(h_c + h_p\right) = 6\left(76.2 + 50.8\right) \\ 800 \text{ mm} \end{array} \right|_{\min} = 135 \text{ mm} \right\} \rightarrow \text{n.g.}$$

Thus, compression flange cannot be considered restrained by the studs.

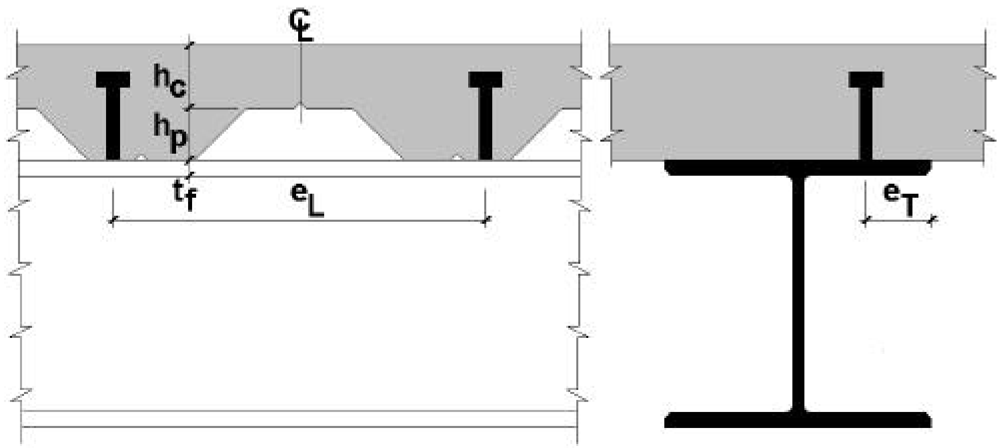


Fig. J.2 Stud Spacing Parameters in EC4

$$\left\{\frac{b_f}{2t_f} = \frac{177}{2(10.9)} = 8.11\right\} < \left\{10\sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{345}} = 8.35\right\} \rightarrow \text{Flange is of Class 1.}$$

As demonstrated below, the P.N.A. is in flange, web is fully in tension and the entire section may be classified Class 1. Therefore, plastic analysis may be used. Also, since $b_{eff} \geq 2h_a$, lateral-torsional buckling will not occur.

$$\text{Degree of shear connection } = \frac{\Sigma P_{Rd}}{\min\{F_a, F_c\}} = \frac{1265}{2136} = 0.592 \qquad \text{(Eq. J.12)}$$

In this example, $A_b = A_t$, and slab is ribbed, so either case A or case B are applicable to determine the minimum degree of shear connection. We apply case B, so equilibrium method can be used:

Case B: $N/N_f \geq 0.25 + 0.03L = 0.25 + 0.03 \cdot (10.67) = 0.570 \geq 0.40$

Thus, minimum degree of shear connection is satisfied.

For the case B, these ductility requirements also must be satisfied:

$\{H_s = 88.9\text{ mm}\} \geq \{4D = 4(19) = 76\text{ mm}\} \rightarrow$ o.k.

$16\text{ mm} \leq \{D = 19\text{ mm}\} \leq 22\text{ mm} \rightarrow$ o.k.

The computed moment strength with the plastic flexural theory (Eq. J.17) is similar to that used by AISC, except with differently defined force components (Eqs. J.13, J.14, and J.15). These force components are illustrated in Fig. J.3.

$$F_c = \frac{h_c b_{eff}\, 0.85 f_{ck}}{\gamma_c} = \frac{(76.2)(2440)(0.85)(27.58/1000)}{1.50} = 2906\ \text{kNm}\ (653\ \text{kips})$$

(Eq. J.13)

But, $F_c \le \Sigma P_{Rd}$, so $F_c = 1265\ \text{kN}\ (284\ \text{kip})$

$$F_a = \frac{A_a f_y}{\gamma_a} = \frac{(6810)(345/1000)}{1.10} = 2136\ \text{kNm}\ (480\ \text{kips}) \qquad \text{(Eq. J.14)}$$

$$F_w = \frac{(h_a - 2t_f)t_w f_y}{\gamma_a} = \frac{(403 - 2\cdot 10.9)(7.50)(345/1000)}{1.10} = 897\ \text{kNm}\ (202\ \text{kips})$$

(Eq. J.15)

Since $F_a > F_c > F_w$, P.N.A. lies in the upper steel flange.

$$z_c = (F_a - F_c)\frac{\gamma_a}{2bf_y} + h_c + h_p \qquad \text{(Eq. J.16)}$$

$$z_c = (2136 - 1265)\frac{1.10}{2(177)(345/1000)} + 76.2 + 50.8 = 134.8\ \text{mm}\ (5.31\ \text{in.})$$

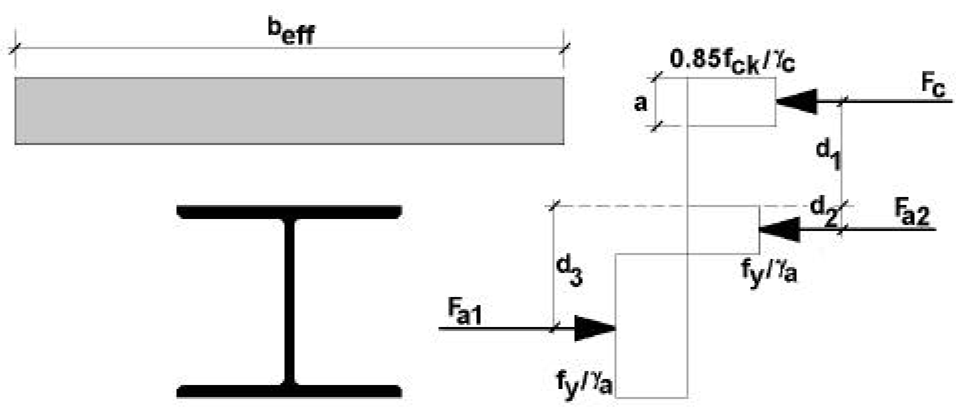


Fig. J.3 Eurocode Flexural Model of the Composite Beam

$$M_{pl,y,Rd} = F_a\left(\frac{h_a}{2} + \frac{h_c}{2} + h_p\right) - (F_a - F_c)\left(\frac{z_c + h_p}{2}\right) \qquad \text{(Eq. J.17)}$$

$$M_{pl,y,Rd} = 2136\left(\frac{403}{2\cdot 1000} + \frac{76.2}{2\cdot 1000} + \frac{50.8}{1000}\right) - (2136 - 1256)\left(\frac{134.8 + 50.8}{2\cdot 1000}\right)$$

$$M_{pl,y,Rd} = 539\ \text{kNm}\ (397\ \text{kip}\cdot\text{ft})$$

## 4. Composite beam check for shear and shear buckling:

1999 AISCS: Maximum shear occurs at member support:

$$\phi_b M_n = M_u = 577 = \frac{w_u L^2}{8} = \frac{w_u (10.67)^2}{8} \qquad \text{(Eq. J.18)}$$

$$w_u = 40.54 \text{ kN/m} \rightarrow V_u = 40.54(10.67/2) = 216 \text{ kN (48.6 kips)}$$

The composite section shear strength is based on shear strength of the bare beam as given by 1999 AISCS F2.2:

$$\left\{\frac{h}{t_w} = \frac{361}{7.50} = 48.1\right\} < \left\{2.45\sqrt{\frac{E}{F_y}} = 2.45\sqrt{\frac{200000}{345}} = 59.0\right\}$$

Thus, $\phi_v V_n = 0.9(0.6 F_y A_w)$ (Eq. J.19)

$$\phi_v V_n = 0.9\left(0.6 \frac{345}{1000} 403(7.50)\right)$$

$$\phi_v V_n = 563 \text{ kN (127 kips)} > 216 \text{ kN (48.6 kips)} \rightarrow \text{o.k.}$$

EC4-1994-1.1: Maximum shear occurs at member support:

$$M_{pl,y,Rd} = M_{y,Sd} = 539 = \frac{w_{z,Sd} L^2}{8} = \frac{w_{z,Sd} (10.67)^2}{8} \qquad \text{(Eq. J.20)}$$

$$w_{z,Sd} = 37.9 \text{ kN/m} \rightarrow V_{z,Sd} = 37.9(10.67/2) = 202 \text{ kN (45.3 kips)}$$

Shear strength in EC4 is also based on the bare beam strength prescribed by EC3 Clause 5.4.6.

$$V_{pl,z,Rd} = A_{v,z} \frac{f_y}{\sqrt{3}\gamma_a} \qquad \text{(Eq. J.21)}$$

$$A_{v,z} = A - 2b_f t_f + (t_w + 2r) t_f \qquad \text{(Eq. J.22)}$$

$$A_{v,z} = 6810 - 2(177)(10.9) + [7.50 + 2(29 - 10.9)]10.9 = 3428 \text{ mm}^2$$

$$V_{pl,z,Rd} = (3428)\frac{345}{1000\sqrt{3}(1.10)}$$

$$V_{pl,z,Rd} = 621 \text{ kN (140 kips)} > 202 \text{ kN (45.3 kips)} \rightarrow \text{o.k.}$$

Shear buckling check:

$$\left\{\frac{h}{t_w} = \frac{346}{7.50} = 46.1\right\} < \left\{69\sqrt{\frac{235}{345}} = 56.9\right\} \rightarrow \text{o.k.} \qquad \text{(Eq. J.23)}$$

**5. Resistance of slab and connectors to longitudinal shear:**

1999 AISCS: No check is specified or required. In this example, the commentary recommendation for minimum transverse reinforcement (0.002 x transverse area of slab) is satisfied.

EC4-1994-1.1: Shear connectors are the governing horizontal force component in this partially composite beam. Thus, the check limits the concrete force to the capacity of connectors.

First step is to determine load for which the check is performed, $v_{Sd}$:

$$v_{Sd} = \frac{1265}{10.67/2} = 237 \text{ kN/m (16.2 kips/ft)}$$

$$v_{Rd} = \left. \begin{vmatrix} 2.5A_{cv}\eta\tau_{Rd} + A_e \dfrac{f_{sk}}{\gamma_s} + v_{pd} \\ 0.2A_{cv}\eta\dfrac{f_{ck}}{\gamma_c} + \dfrac{v_{pd}}{\sqrt{3}} \end{vmatrix} \right._{min} \qquad \text{(Eq. J.24)}$$

The smallest value of effective area of the steel sheet in tension, $A_p$, and the mean cross-sectional area per unit length of the concrete shear surface being considered, $A_{cv}$, that can occur within 1 m are considered, and are calculated based on straight deck corners, and neglecting imperfections such as deck stiffeners.

$$A_{cv,\,top\,eq.} = 99426 \text{ mm}^2 \text{ (154 in.}^2\text{)}$$

$$A_{cv,\,bottom\,eq.} = 76200 \text{ mm}^2 \text{ (118 in.}^2\text{)}$$

$$\eta = 1.0$$

$$\tau_{Rd} \approx 0.30 \text{ N/mm}^2 \text{ (43.5 psi)}$$

$$A_e = 6(9.03) = 54.2 \text{ mm}^2 \text{ (0.084 in.}^2\text{)}$$

$$v_{pd} = \frac{A_p f_{yp}}{\gamma_{ap}} = \frac{1188(0.91)(276)}{(1.10)1000} = 271 \text{ kN (61.0 kips)} \qquad \text{(Eq. J.25)}$$

$$v_{Rd} = \left. \begin{vmatrix} 2.5(99426)(1.0)\left(\dfrac{0.30}{1000}\right) + (54.2)\dfrac{414}{1.15(1000)} + 271 \\ 0.2(76200)(1.0)\dfrac{27.58}{1.50(1000)} + \dfrac{271}{\sqrt{3}} \end{vmatrix} \right._{min}$$

$\{v_{Rd} = 365 \text{ kN/m (25.0 kips)}\} > \{v_{Sd} = 237 \text{ kN (16.2 kips/ft)}\} \rightarrow$ o.k.

**6. Check for combined bending and shear:**

1999 AISCS: No check is required.

EC4-1994-1-1: Since $V_{z,Sd} > 0.5V_{pl,z,Rd}$, the beam must be checked for interaction of vertical shear and bending moment. Using the same plastic flexural theory as in step 3, $M_{f,Rd}$ is calculated:

$$M_{f,Rd} = 241 \text{ kNm (178 kip} \cdot \text{ft)}$$

Reduced moment strength based on presence of shear is computed:

$$M_{v,Rd} = M_{f,Rd} + (M_{pl,y,Rd} - M_{f,Rd})(1 - \rho_z) \quad \text{(Eq. J.26)}$$

As shown in Fig. J.4, the presence of shear is reducing the moment strength between 2.3 m and the mid-span. This further reduces the sustainable uniform distributed load to 33.6 kN/m (2.30 k/ft).

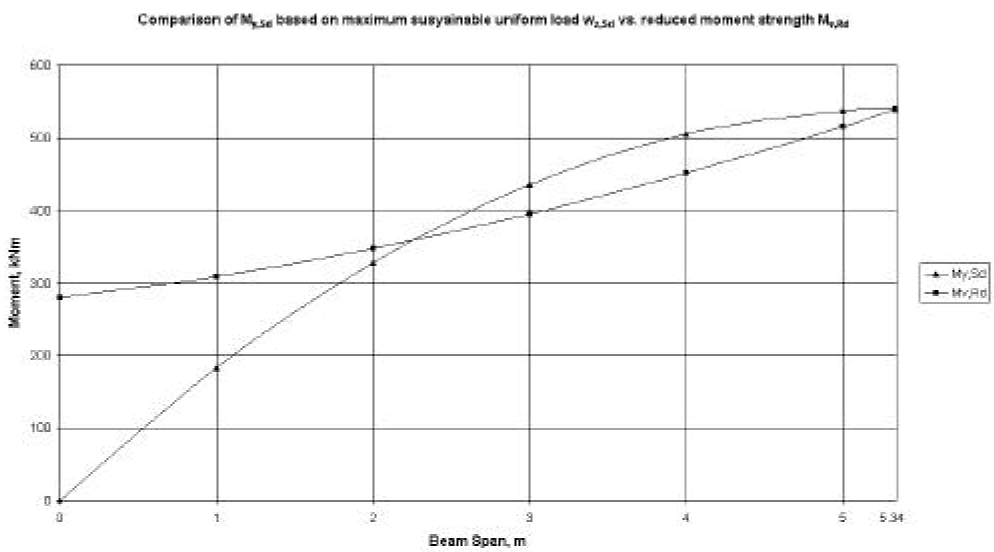


Fig. J.4 Effect of shear on moment strength per EC 4-1994-1.1

**7. Check of applicable spacing and detailing requirements:**

1999 AISCS: $\{\text{Stud spacing} = 305 \text{ mm (12 in.)}\} \leq \{915 \text{ mm (36 in.)}\} \rightarrow$ o.k.

$\{H_s - h_r = 38.1 \text{ mm (1.5 in.)}\} \geq \{38.1 \text{ mm (1.5 in.)}\} \rightarrow$ o.k.

$$\{d_s - h_r = 76.2\text{ mm }(3\text{ in.})\} \geq \{50.8\text{ mm }(2\text{ in.})\} \rightarrow \text{o.k.}$$

EC4-1994-1.1: $$\{d_s - H_s = 38.1\text{ mm }(1.5\text{ in.})\} \geq \{20\text{ mm }(0.79\text{ in.})\} \rightarrow \text{o.k.}$$

$$\{h_c = 76.2\text{ mm }(3\text{ in.})\} \geq \{50\text{ mm }(1.97\text{ in.})\} \rightarrow \text{o.k.}$$

$$\{h_p = 50.8\text{ mm }(2\text{ in.})\} \leq \{85\text{ mm }(3.35\text{ in.})\} \rightarrow \text{o.k.}$$

$$\{H_s - (d_s - x) = 30.0\text{ mm }(1.18\text{ in.})\} \geq \{30\text{ mm }(1.18\text{ in.})\} \rightarrow \text{o.k.}$$

$$\{H_s - h_r = 38.1\text{ mm }(1.5\text{ in.})\} \geq \{38.1\text{ mm }(1.5\text{ in.})\} \rightarrow \text{o.k.}$$

$$\{H_s - h_p = 38.1\text{ mm }(1.5\text{ in.})\} \geq \{2D = 38\text{ mm }(1.5\text{ in.})\} \rightarrow \text{o.k.}$$

$$\{\text{min. stud spacing} = 305\text{ mm }(12\text{ in.})\} \geq \{5D = 95\text{ mm }(3.75\text{ in.})\} \rightarrow \text{o.k.}$$

$$\{e_T = b_f/2 = 89\text{ mm }(3.48\text{ in.})\} \geq \{20\text{ mm }(0.79\text{ in.})\} \rightarrow \text{o.k.}$$

Studs should also be located at least at a distance of 2.2D away from butt joints in decking. It is assumed in this example that a joint does not occur at the beam considered.

Thus, the maximum factored line load based on the 1999 AISCS is governed by the composite beam flexural resistance and equals 41 kN/m (2.8 kips/ft). The equivalent strength per EC4-1994-1.1 is limited by combined effect of shear and bending and equals 34 kN/m (2.3 kips/ft), or 17% less than that obtained using 1999 AISCS.

## J.10 SUMMARY AND CONCLUSIONS

This paper provided an insight into significant differences between EC4-1994-1.1 and 1999 AISCS rules for design of composite steel-concrete beams. The goal of this comparison was to identify the areas in which 1999 AISC may be improved, taking advantage of available body of knowledge and experimental results about behavior of composite steel-concrete beams. Many if these aspects are already recognized and addressed in EC4-1994-1.1.

Several modifications to the 1999 AISCS are suggested. First, it is necessary to revise, or replace the stud strength computational model. The current modified version of model by Grant et al. (1977) still has significant shortcomings. A possible solution would be to implement the new model proposed by Rambo-Roddenberry et al. (2002). Second, the minimum required degree of shear connection should be raised to 40-50%. A member length dependant function could be implemented, upon which the minimum degree of shear connection would vary. In absence of such requirements, the design of shear connection could be significantly unconservative, even if the strength of shear connection is computed fairly accurately. The reason for this is the inability of horizontal shear to be progressively transferred towards interior studs due to lack of slip capacity in excessively long members. Third, 1999 AISCS should establish a required check for longitudinal slab splitting. Such a check is stipulated by EC4-1994-1.1 and can be of significance, especially in members with high $d/t_s$ ratios. Finally, a check for combined shear and bending can be significant and should be investigated further. The example presented in this paper shows that this check can result in reduction of load carrying capacity of 11%, which is significant.

## J.11 NOTATION

Notations used in this paper are as follows:

$A_b$ = area of beam bottom flange, $mm^2$

$A_c$ = area of concrete compressive block, $mm^2$

$A_{cv}$ = mean cross-sectional area per unit length of beam concrete shear surface considered, $mm^2/m$

$A_e$ = cross-sectional area of slab reinforcement per beam unit length, $mm^2/m$

$A_p$ = effective area of the steel sheet in tension, $mm^2$

$A_s$ = area of beam cross-section, $mm^2$

$A_t$ = area of beam top flange, $mm^2$

$A_{sc}$ = area of stud cross-section, $mm^2$

$A_{v,z}$ = shear stress area, $mm^2$

$A_{wire}$ = cross-sectional area of reinforcement wire, $mm^2$

C = total horizontal force between support and mid-span, kN

D = stud diameter, mm

E = steel modulus of elasticity, $N/mm^2$

$E_c$, $E_{cm}$ = concrete modulus of elasticity, $N/mm^2$

$F_a$ = steel beam tensile force, $N/mm^2$

| | |
|---|---|
| $F_c$ | = concrete compressive force, N/mm$^2$ |
| $F_s$, $f_{sk}$ | = wire mesh yield strength, N/mm$^2$ |
| $F_u$, $f_u$ | = stud fracture stress strength, N/mm$^2$ |
| $F_{yf}$, $f_{yf}$ | = flange yield stress strength, N/mm$^2$ |
| $F_{yp}$, $f_{yp}$ | = deck yield strength stress, N/mm$^2$ |
| $F_{yw}$, $f_{yw}$ | = web yield stress strength, N/mm$^2$ |
| $F_w$ | = steel beam web force, N/mm$^2$ |
| $H_s$ | = stud height, mm |
| L | = beam span, m |
| $M_{f,Rd}$ | = moment strength calculated using flange and reinforcement forces, kNm |
| $M_n$ | = nominal moment strength, kNm |
| $M_{pl,y,Rd}$ | = design moment strength, kNm |
| $M_u$, $M_{y,Sd}$ | = applied factored moment, kNm |
| $M_{v,Rd}$ | = moment strength reduced by presence of shear, kNm |
| N | = number of studs per half-span |
| $N_f$ | = number of studs required for fully composite action |
| $N_r$ | = number of studs per rib |
| $Q_n$, $P_{Rd}$ | = design stud strength, kN |
| $S_b$ | = typical interior beam spacing, m |
| SRF, $k_l$ | = stud strength reduction factor |
| $V_n$ | = nominal shear strength, kN |
| $V_{pl,z,Rd}$ | = plastic shear strength, kN |
| $V_{z,Sd}$ | = applied factored shear, kN |
| a | = depth of concrete compressive block, mm |
| $b_{e1}$ | = effective slab width left of beam, mm |
| $b_{e2}$ | = effective slab width right of beam, mm |
| $b_{eff}$ | = effective slab width, mm |
| $b_f$ | = beam flange width, mm |
| d, $h_a$ | = beam depth, mm |
| $d_s$ | = slab depth, mm |

| | |
|---|---|
| $e_L$ | = maximum distance between consecutive studs, mm |
| $e_{mid\text{-}ht}$ | = distance from the edge of stud shank to the steel deck web, measured at mid height of the deck rib, mm |
| $e_T$ | = distance from edge of flange to the center of stud, mm |
| $f'_c$, $f_{ck}$ | = concrete compressive strength, N/mm$^2$ |
| h | = beam web depth between fillets, mm |
| $h_c$ | = slab depth above rib, mm |
| $h_r$, $h_p$ | = deck rib height, mm |
| t | = deck thickness, mm |
| $t_f$ | = beam flange thickness, mm |
| $t_w$ | = beam web thickness, mm |
| x | = distance from top of slab to reinforcement centroid, mm |
| $v_{pd}$ | = longitudinal shear resistance of steel sheeting, kN/m |
| $v_{Rd}$ | = longitudinal shear resistance, kN/m |
| $v_{Sd}$ | = applied longitudinal shear per unit length, kN/m |
| $w_b$ | = beam unit weight, kN/m |
| $w_c$ | = unit weight of dry concrete, kg/m$^3$ |
| $w_r$, $b_0$ | = average rib width, mm |
| $w_u$, $w_{z,Sd}$ | = maximum applied factored load, kN/m |
| $z_c$ | = distance from top of slab to P.N.A., mm |
| a | = factor to determine the position of neutral axis |
| ? | = light-weight concrete coefficient |
| ?$_z$ | $= \left( \frac{2V_{z,Sd}}{V_{pl,z,Rd}} - 1 \right)^2$ |
| $t_{Rd}$ | = basic shear strength of concrete, N/mm$^2$ |
| ?$_a$ | = partial factor of safety for steel |
| ?$_a$ | = partial factor of safety for profiled steel decking |
| ?$_c$ | = partial factor of safety for concrete |
| ?$_s$ | = partial factor of safety for reinforcement |
| ?$_v$ | = partial factor of safety for shear connectors |
| $\phi_b$ | = resistance factor for flexure |

$\phi_v$ = resistance factor for shear

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