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Impact of Structural Eurocodes on Steel and Composite Structures

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The introduction of Structural Eurocodes to replace the existing British Standard design codes will benefit the construction industry in the long run. However, in the short term, to help our engineers migrate over to the new Eurocodes, it is useful to conduct a comparative study on both Eurocodes and British Standards to identify some of the key changes and impact on the performance of the steel and composite structures. In this connection, this paper investigates five key design aspects comprising material properties, flexural buckling resistance of compression members with Grade S460 steel, shear buckling resistance of plated structures, web bearing and buckling resistance of the plated structures under transverse force and lateral-torsional buckling of composite beam. It is found that the flexural buckling resistance, web bearing and buckling resistance and lateral-torsional buckling resistance determined from Eurocodes 3 and 4 are much larger than those from BS5950. However, for shear buckling resistance of plated structures, Eurocode 3 produced a lower value compared to BS5950, and for shear resistance of headed stud, Eurocode 4 produced a lower value compared to BS5950-3.1.

Keywords: Eurocodes, BS5950, material property, flexural buckling resistance, shear buckling resistance, web bearing and buckling resistance, lateral-torsional buckling.

1. Introduction

Eurocodes are a set of harmonized technical design rules established by the European Committee for the standardization of design of structures and structural components in the European Union. The main objective of drawing up these set of harmonized technical specifications for building and civil engineering works is to eliminate technical barriers and to promote trades across member countries. Each Eurocode contains provisions which are open for national choice. Such provisions contain country specific data on those parameters, known as Nationally Determined Parameters, and such data could include weather aspects, seismic

zones, geo-informatics, etc. in a relating National Annex. The National Standard implementing Eurocodes must comprise the full, unaltered text of that Eurocode, including all annexes. All European countries are required to develop their own National Annexes. Although a non-European country, Singapore has chosen to adopt the Eurocodes as her national design codes and after an initial 2 year co-existence period, the Eurocodes will be fully implemented in Singapore by 1 April 2015.

Although many of the Eurocodes design rules are based on the same design concepts as the British Standards, the Eurocodes have incorporated more up-to-date research on many respects of structures behavior. The

Eurocodes clauses are structured slightly differently in which they specified principles that must be satisfied first and followed by application rules that offer a way of satisfying the principles. The Eurocodes are also less prescriptive than the British Standards, with more design aspects left open to the engineers to decide. Consequently, for this reason, it is useful to discuss the design approaches and compare the results obtained by both codes to assess the impact of making some of the key changes. Slender or thin-walled elements under compressive stresses are susceptible to buckling; hence it is imperative to evaluate accurately the buckling strength in order to optimize the design. For structural element, there are basically two different buckling modes, the overall buckling and local buckling. Overall buckling includes lateral buckling, torsional buckling and other buckling modes. Local plate buckling may occur on the slender elements first before the overall beam or column buckling, or yielding.

In this paper, the key changes for four design aspects using both Eurocodes and BS5950 are investigated in details, and they are all related to structural member buckling resistances. Flexural buckling resistance of compression members with grade S460 steel, shear buckling resistance of plated structures web bearing and buckling resistance of the plated structures under transverse force are compared using BS5950-1 and EC3, and lateral-torsional buckling resistance of composite beam. In addition, the material properties of the concrete and steel materials as well as the headed stud shear connectors are also investigated using both Eurocodes and BS5950.

2. Materials

Compared to BS5950, Eurocodes provide a wider range of both concrete strength and steel strength. For structural steel, the yield strength should not be taken as being more than 460 N/mm^2 in BS5950-1, whereas steel grades with nominal yield strength up to 690 N/mm^2 can be used according to EC3. Similarly, concrete with cylinder strength up to 60 N/mm^2 can be adopted in EC4, while maximum 50 N/mm^2 of concrete cube strength is given in BS5950-3.1. These ranges in BS5950 are narrower than those given in Eurocodes because there is limited knowledge and experience of the behaviour of structures with high strength steel and strong concrete. The key changes in material properties between BS5950 and Eurocodes are summarized below.

2.1 Ductility of structural steel

For the property of structural steel, EC3 has additional ductility requirements compared to BS5950 in terms of stress ratio, elongation and strain ratio. The limiting values of the ratio f_u/f_y , the elongation at failure and the ultimate strain ϵ_u may be defined as following:

(1) For normal strength steel ($f_y < 460 \text{ N/mm}^2$):

- a) $f_u/f_y \geq 1.10$;
- b) elongation at failure not less than 15%;
- c) $\epsilon_u \geq 15\epsilon_y$, where ϵ_y is the yield strain.

(2) For high strength steel ($460 \text{ N/mm}^2 < f_y < 690 \text{ N/mm}^2$):

- a) $f_u/f_y \geq 1.05$ (EC3-1-12), $f_u/f_y \geq 1.10$ (UK NA to EC3-1-12),;
- b) elongation at failure not less than 10%;
- c) $\epsilon_u \geq 15\epsilon_y$.

However, some product standards only have requirements on the nominal yield and tensile strength, or their minimum values. The stress ratio calculated according to these

nominal values cannot comply with the EC3 ductility requirement, as shown in Table 1. In composite slab, the profiled steel sheeting manufactured from these product standards may face the problem with EC3 ductility requirement, for example, the stress ratio of a type of profiled steel sheeting which is

manufactured from G550 steel in accordance with AS 1397 can not comply with the EC3 ductility requirement, as shown in Figure 1. Therefore, the yield strength of profiled steel sheeting should be amended to satisfy the ductility requirement.

Table 1 Stress-ratio of some product standards

Standard	Grade	Nominal yield strength (MPa)	Nominal tensile strength (MPa)	Stress ratio
AS 1397	G450	450	480	1.07
	G500	500	520	1.04
	G550	550	550	1.00
AS 1595	CA 500	500	510	1.02
EN 10326	S550GD	550	560	1.02
ISO 4997	CH550	550	550	1.00

Figure 1. Stress-strain curve

2.2. Resistance of headed studs

Shear connectors are used for providing the composite action between steel and concrete. This connection which is referred to as shear connection is provided mainly to resist longitudinal shear. The most common shear connector used in construction is the headed studs. It can be welded to the upper flange of steel beams either directly or through profiled steel sheeting.

In BS5950-3.1, the characteristic resistance of headed studs in solid slabs is given for various combinations of height, diameter and concrete strength, but the physics behind the numbers is not presented. The characteristic resistance of a headed shear stud embedded in a solid slab of normal weight concrete should be taken from Table 5 of BS5950-3.1. In EC4, the resistance of headed studs in solid slabs are calculated by design equation. It is influenced by some factors, such as a shank diameter d and an ultimate strength f_u of headed stud, a characteristic strength f_{ck} and a mean secant modulus E_{cm} of concrete, and failure either in the steel alone or in the concrete alone.

For a solid concrete slab, the design shear resistance of a headed stud should be determined from:

$$P_{Rd} = \frac{0.8f_u\pi d^2/4}{\gamma_v} \quad (1)$$

$$P_{Rd} = \frac{0.29\alpha d^2 \sqrt{f_{ck} E_{cm}}}{\gamma_v} \quad (2)$$

Whichever is smaller, with:

$$\alpha = 0.2(h_{sc}/d + 1) \leq 1 \quad (3)$$

The two equations represent different failure modes. Equation (1) is based on failure of the shank of headed stud, while equation (2) is based on failure in concrete.

The characteristic resistance of headed stud determined by BS5950-3.1 is different from that calculated by EC4. The comparison of characteristic resistance of different types of headed stud and various concrete strength used in the BS5950-3.1 and EC4 is shown in Figure 2. It is found that the characteristic resistance of headed studs calculated by EC4 is lower than those determined by BS5950-3.1. The resistance of headed studs is mainly influenced by the types of headed studs and the concrete strength. The larger the values of diameter and height of headed stud, the smaller the difference between BS5950-3.1 and EC 4 becomes. Additionally, with the decreasing of concrete strength, the resistance of headed stud also decreases significantly compared to BS5950-3.1.

Taking a headed stud with 19 mm diameter and 100mm height for example, the exact value of characteristic resistance determined by BS5950-3.1 and EC4 is given in Table 2. EC4 leads to a 15% maximum reduction of characteristic resistance of headed stud compared to BS5950-3.1.

Figure 2 Comparison of characteristic resistance of headed studs in solid slabs

Table 2 Comparison of characteristic resistance of headed stud

Characteristic resistance of headed stud P_{Rk} (kN)				
Headed stud in solid slab	Characteristic strength of concrete strength (N/mm ²)			
	25	30	35	40
BS5950-3.1	95	100	104	109
EC4	81	92.1	100.6	102.1
(BS-EC4) / BS %	15%	8%	3%	6%

If profiled steel sheeting is used, headed stud connectors are located in the trough of sheeting. Based on the information from tests, it shows that the shear resistance of headed studs in composite slab is lower than the resistance in a solid slab for materials of the same strength. This is because that local failure of the concrete rib occurs. For this reason, reduction factors are applied to the resistance P_{Rd} according to EN 1994-1-1.

The load-slip relationship of headed stud connector in profiled steel sheeting is more complex than in a solid slab. It is influenced by the following factors:

- The direction of the ribs relative to direction of span of the composite beam;
- The mean breadth b_0 and depth h_p of profiled steel sheeting;
- The diameter d and height h_{sc} of the headed shear stud;
- The number n_r of the headed studs in one trough;
- Whether or not a headed stud is central within a trough.

Base on testing and experience, reduction factors k are given to calculate the shear resistance of a headed stud connector in composite slab with profiled steel sheeting. There are two situations should be considered in the calculation of reduction factor k .

(1) For profiled steel sheeting with ribs parallel to the supporting beams, the reduction factor, k_t is taken as:

$$k_t = 0.6 \frac{b_0}{h_p} \left(\frac{h_{sc}}{h_p} - 1 \right) \leq 1.0 \quad (4)$$

In BS5950-3.1, the reduction factor is calculated using the same equation given in EC4.

(2) For profiled steel sheeting with ribs transverse to the supporting beams, the reduction factor, k_t is taken as:

$$k_t = \frac{0.7}{\sqrt{n_r}} \frac{b_0}{h_p} \left(\frac{h_{sc}}{h_p} - 1 \right) \leq k_{t,max} \quad (5)$$

Where n_r is the number of stud connectors in one rib at a beam intersection, not exceed 2. The factor k_t should not be taken greater than $k_{t,max}$ given in Table 3.

In BS5950-3.1, the reduction factors that are applied to the resistance of the shear connector are calculated using identical equations to those EC4, but with a different constant coefficient applied. For re-entrant trough profiles, the constant coefficient is 0.85 and 0.6 for singles and pairs

respectively, and for open trough profiles, 0.63 and 0.34 for singles and pairs respectively. However, the constant coefficient is 0.7 and 0.5 for singles and pairs respectively in EC4, as shown in Equation (5).

Table 3 Upper limits $k_{t,max}$ for the reduction factor k_t

Profiled steel sheeting	Number of stud per rib	Thickness (mm)	EC4	BS
Re-entrant trough	$n_r=1$	≤ 1.0 > 1.0	0.85 1.0	1.0
	$n_r=2$	≤ 1.0 > 1.0	0.70 0.8	0.8
Open trough	$n_r=1$	≤ 1.0 > 1.0	0.85 1.0	0.82
	$n_r=2$	≤ 1.0 > 1.0	0.70 0.8	0.45

Compared to BS5950-3.1, EC4 leads to a 17% reduction for re-entrant trough profiles if the deck geometry is such that the limiting values do not apply, whereas an increase for open-trough profiles. However, most decks commonly used in the UK are designed such that the limiting value dominates, so the reduction factor is independent of the geometry and is based only on the number of studs and the orientation of the deck. For re-entrant trough profiles, these values of EC4 are the same as the BS for decks thicker than 1.0 mm, but about 15% lower than BS5950 for decks with a sheet thickness of 1.0 mm or thinner. However, for open trough profiles, these values of EC4 are higher than that of BS.

In composite slab with profiled steel sheeting, the resistance of headed stud ($d=19\text{mm}$ and $h=100$) determined by BS5950-3.1 and EC4 are compared in Table 5. The values of headed stud resistance is the product of the resistance of headed stud in

solid slab and the reduction factor $k_{t,max}$ in Table 3. From Table 4, it is found that the resistance of headed stud in EC4 is lower than that in BS5950-3.1, except for open trough profiled steel sheeting with two headed studs per rib.

Table 4 Comparison of resistance of headed studs

Headed stud in composite slab with profiled steel sheeting			Characteristic strength of concrete strength (N/mm ²)			
			25	30	35	40
BS 5950-3.1	Re-entrant	n _r =1	95	100	104	109
		n _r =2	76	80	83.2	87.2
	Open trough	n _r =1	77.9	82	85.3	89.4
		n _r =2	42.8	45	46.8	49.1
EC4	Any type	n _r =1	68.9	78.3	85.5	86.8
		n _r =2	56.7	64.5	70.4	71.5

In general, for both solid slab and composite slab, the resistance of headed studs determined by EC4 is lower than that given in BS5950-3.1. Therefore, for a composite member with identical dimension and material, more headed studs are needed in the construction according to EC4.

3. Flexural buckling resistance of universal columns with S460 steel

The behaviour of high strength steel with respect to effect of buckling and residual stresses is studied by Grotmann D, Sedlacek, G (1994) and Beg D, Hladnik L (1995). It is found that HSS has a better performance than ordinary steel, which can be interpreted as the smaller influence of imperfections, such as residual stresses. The residual stresses of HSS were lower if made dimensionless with respect to the strength of the steel, as shown in the work by Clarin M.

(2004). This is reflected in the calculation of flexural buckling resistance by using higher buckling curves for S460 according to EN 1993-1-1. Therefore, the buckling resistance of columns with high strength steels are expected to be higher than those of columns with normal steels.

According to BS5950-1, members in compression are analysed by considering the nonlinear diagram of steel behaviour, the accidental eccentricities as well as the shape of cross-sections. Design value of compression buckling resistance is determined by:

$$N_{b,Rd} = A p_c \quad (6)$$

Where, A is the gross cross-sectional area; p_c is the compressive strength determined from Table 23 and Table 24 in BS5950-1, which is related to the buckling curves, the design strength f_y and the slenderness λ .

The principle in EC3 for determining the buckling resistance of compression members is similar with that of BS5950-1. According to EN 1993-1-1, for compression members the design buckling resistance should be taken as:

$$N_{b,Rd} = A \frac{\chi f_y}{\gamma_{M1}} \quad (\gamma_{M1} = 1.0) \quad (7)$$

Where, A is the gross cross-sectional area; χ is the reduction factor related to the non-dimensional slenderness of the member:

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} \quad (8)$$

where

$$\phi = 0.5 \left[1 + \alpha (\bar{\lambda} - 0.2) + \bar{\lambda}^2 \right]$$

$$\bar{\lambda} = \sqrt{\frac{A f_y}{N_{cr}}} = \frac{L_{cr}}{i} \frac{1}{\lambda_1}$$

$$\lambda_1 = \pi \sqrt{E / f_y} = 93.9 \varepsilon \quad \varepsilon = \sqrt{235 / f_y}$$

α is the imperfection factor.

The buckling curves defined by EC3 are equivalent to those given in Table 24 in BS5950-1. However, except curves a, b, c and d, a more favourable buckling curve a_0 for S460 steel is introduced in EC3.

To compare the compression buckling resistance calculated from BS5950-1 and EC3, a column with pinned boundary conditions is analysed. A series of Rolled H-sections (356×406UC) and Rolled I-sections (457×191UB) with grade S460 and S355 steel are adopted as the internal column. Assuming the nominal buckling length is 7.5m. The buckling resistance are illustrated in Figure 3 and Figure 4.

Figure 3: Compression buckling resistance of Rolled H-sections

Figure 4: Compression buckling resistance of Rolled I-sections

It can be seen from Figure 3 and 4 that the buckling resistance obtained from EC3 has a significantly improvement compared with that determined from BS5950-1. The buckling resistance for S460 steel calculated from EC3 is about 15% higher than that obtained from BS5950-1. However, for S355 steel, almost identical values are obtained from both codes. This is because that a favourable buckling curve a_0 is adopted for S460 steel in EC3, as mentioned in above.

4. Shear buckling resistance of plates

The usual slender design of the web panels in plate structures, such as plate girder, makes the web susceptible to instability phenomena: shear buckling. Buckling in slender plates is a local and sudden phenomenon followed by large out-of-plane displacements and loss of stiffness. Slender plates are capable of carrying considerable post-buckling loads due to stresses in the inclined tension fields as shown in the work by Alinia MM, Habashi HR (2009).

In BS5950-1, if the web depth-to-thickness ratio $h_w/t_w > 70\varepsilon$ for a rolled section, or 62ε for a welded section, it should be assumed to be susceptible to shear buckling. The factor ε is taken as $\varepsilon = \sqrt{275/f_y}$, which is different from the expression in EC3. The shear buckling resistance $V_{b,Rd}$ of a web with or without intermediate transverse stiffeners may be taken as the simple buckling resistance V_w given by:

$$V_{b,Rd} = V_w = h_w t_w q_w \quad (9)$$

Where, q_w is the shear buckling strength which should be obtained from Table 21 or Annex H.1 of BS5950-1 depending on the values of h_w/t_w and a/h_w where a is the stiffener spacing.

In EC3, two methods are provided to determine shear resistance of a cross-section. One is the plastic shear resistance of cross-

sections where the web is not prone to shear buckling. The other one is the shear buckling resistance of cross-sections where the web is prone to shear buckling. Plate with $h_w/t_w > 72\varepsilon/\eta$ for unstiffened web, or $h_w/t_w > 31\varepsilon\sqrt{k_\tau}/\eta$ for a stiffened web, it becomes susceptible to shear buckling. Where, k_τ is the shear buckling coefficient defined in Annex A3 of EN 1993-1-5; η is taken as 1.0 given in UK National Annex.

There are many tension field theories which aim to describe the ultimate resistance of plates under shear, as shown in the work by Höglund (1981). The resistance of steel beams to shear buckling is covered by the stiffened plate rules in EN 1993-1-5 and is based on the rotated stress field theory proposed by Höglund. In this method, the shear resistance $V_{b,Rd}$ comprises contributions from the web $V_{bw,Rd}$ and the flanges $V_{bf,Rd}$. The expression given in EN 1993-1-5 takes the following form, clearly identifying the two separate contributions:

$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \eta \frac{f_{yw} h_w t_w}{\sqrt{3} \gamma_{M1}} \quad (10)$$

With:

$$V_{bw,Rd} = \chi_w \frac{f_{yw} h_w t_w}{\sqrt{3} \gamma_{M1}} \quad (11)$$

$$V_{bf,Rd} = \frac{b_f t_f^2}{c} \frac{f_{yf}}{\gamma_{M1}} \left[1 - \left(\frac{M_{Ed}}{M_{f,Rd}} \right)^2 \right] \quad (12)$$

The value $V_{bw,Rd}$ depends on web slenderness and end post condition. Rigid end posts are typically used at the ends of girders to improve their shear resistance. The reduction factor χ_w considers components of pure shear and anchorage of membrane forces by transverse stiffeners due to tension field action. The partial safety factor γ_{M1} is taken as 1.0. The value $V_{bf,Rd}$ depends on the plastic resistance of the flanges bending out

of its plane. However, the contribution to shear resistance from the flange is much less than the web and can always be conservatively ignored to avoid the additional calculation effort. Hence, the shear resistance of cross-sections can be transferred as:

$$V_{b,Rd} \approx V_{bw,Rd} = \chi_w \frac{f_{yw} h_w t_w}{\sqrt{3} \gamma_{M1}} \leq \eta \frac{f_{yw} h_w t_w}{\sqrt{3} \gamma_{M1}} \quad (13)$$

From equation (13), it's noted that the shear buckling resistance can never be larger than the plastic shear resistance of the cross-section $\eta \frac{f_{yw} h_w t_w}{\sqrt{3} \gamma_{M1}}$.

EC3 classify the beam end support conditions as rigidity and non-rigidity according to the end post could or not provide adequate anchorage for the longitudinal component of the tensile stresses developed in the web during the post-buckling range. The reduction factor χ_w of rigid and non-rigid end post is determined from different expressions, as shown in Table 5.1 and Figure 5.1 of EN 1993-1-5 depending on web slenderness $\bar{\lambda}_w$. The web slenderness $\bar{\lambda}_w$, for both unstiffened and stiffened webs is given by:

$$\bar{\lambda}_w = \frac{h_w}{37.4 t_w \varepsilon \sqrt{k_\tau}} \quad (14)$$

For beams without intermediate stiffeners and longitudinal stiffeners at supports only, this expression could simplify to:

$$\bar{\lambda}_w = \frac{h_w}{86.4 t_w \varepsilon} \quad (15)$$

To compare the shear buckling resistance calculated by the methods given in EC 3 and BS5950-1, a simply supported plate girder with grade S275 steel is analysed. To find the connection with BS5950, the following expression is used:

$$V_{b,Rd} = \chi_w \frac{f_{yw} h_w t_w}{\sqrt{3} \gamma_{M1}} = h_w t_w f_{sb} \quad (16)$$

With: $f_{sb} = \frac{f_{yw}\chi_w}{\sqrt{3}\gamma_{M1}} = 158.8\chi_w$

The stiffener spacing is assumed to be equal to web depth, hence $a/h_w = 1$. Then, the shear buckling coefficient k_τ is $5.34 + 4.00(h_w/a) = 9.34$. Hence, the equation (14) can be rewritten as $\bar{\lambda}_w = h_w/105.7t_w$. Then, the Table 5.1 of EN1993-1-5 can be rewritten as shown in Table 5 using the factor h_w/t_w and the shear buckling strength $f_{sb}(q_w)$ of the plate girder obtained from EC3 and BS5950-1 are given in Figure 5.

Table 5: Shear resistance function for S275 steel by h_w/t_w

	χ_w	
	Rigid end post	Non-rigid end post
$\frac{h_w}{t_w} < 87.7$	1	1
$87.7 \leq \frac{h_w}{t_w} < 114.1$	$87.7 \frac{t_w}{h_w}$	$87.7 \frac{t_w}{h_w}$
$\frac{h_w}{t_w} \geq 114.1$	$\frac{1.37}{0.7 + 105.66 h_w/t_w}$	$87.7 \frac{t_w}{h_w}$

Figure 5: Shear buckling strength curves for S275 steel

It can be seen from Figure 5 that the values of shear buckling strength obtained from EC 3 in plateau are generally smaller than that determined from BS5950-1. For non-rigid end post girder, the values of shear buckling resistance are consistently lower than that got from BS5950-1. However, for rigid end post web, with the increasing of the ratio h_w/t_w , shear buckling resistance may exceed the value obtained from BS5950-1. It means that the design shear resistance obtained from EC3 is conservative compared with BS5950-1 only for welded stocky beams.

5. Web buckling resistance under transverse loading

Buckling of the web happens when the web is too slender to carry the transverse loading being transferred from the flange. In BS5950-1, the web buckling occurs if the flanges are restrained, and the buckling resistance of the unstiffened web should be taken as:

$$F_{Rd,BS} = \frac{25\epsilon t_w}{\sqrt{(S_s + nk) h_w}} F_{w,Rd} \quad (17)$$

With

$$F_{w,Rd} = (S_s + nk) t_w f_{yw} \quad (18)$$

Where, S_s is the stiff bearing length determined from Figure 13 of BS5950-1; k is $t_f + r$ for rolled section, or t_f for weld section; nk is the additional length assuming a dispersion of the bearing at 1: n through the flange thickness.

In EC3, three types of load application are provided: (a) load is applied through one flange; (b) through both flanges and transferred through the web directly; (c) through one flange adjacent to an unstiffened end. With a reduction factor χ_F , the design

resistance to local buckling under transverse loading should be determined as:

$$F_{Rd,EC} = \chi_F \frac{l_y t_w f_{yw}}{\gamma_{M1}} \quad (19)$$

Where

$$\chi_F = \frac{0.5}{\bar{\lambda}_F} \leq 1.0 \quad (20)$$

$$\bar{\lambda}_F = \sqrt{\frac{l_y t_w f_{yw}}{F_{cr}}} \quad (21)$$

$$F_{cr} = 0.9 k_F E \frac{t_w^3}{h_w} \quad (22)$$

k_F is a factor related to load application;

l_y is the effective loaded length calculated as follows:

For load application type (a) and (b), it should be taken as:

$$l_{y1} = s_s + 2t_f \left(1 + \sqrt{m_1 + m_2}\right) \quad (23)$$

For load application type (c), it should be taken as the smallest value obtained from equation (23), (24), and (25), which is given in the Corrigendum to EN 1993-1-5 (CEN, 2009).

$$l_{y2} = l_e + t_f \sqrt{\frac{m_1}{2} + \left(\frac{l_e}{t_f}\right)^2 + m_2} \quad (24)$$

$$l_{y3} = l_e + t_f \sqrt{m_1 + m_2} \quad (25)$$

Where, l_e is a factor related to k_F ; the factor

m_1 is taken as $\frac{f_{yf} b_f}{f_{yw} t_w}$, and m_2 is taken as 0

for $\bar{\lambda}_F \leq 0.5$, or $0.02 \left(\frac{h_w}{t_f}\right)^2$ for $\bar{\lambda}_F > 0.5$. It

should be noted that $\bar{\lambda}_F$ and m_2 are interdependent. The plate

slenderness $\bar{\lambda}_F$ depends on l_y , which in turn is affected by m_2 .

To compare the web buckling resistance predicted by EC3 and BS5950-1, an expression can be proposed as following:

$$R = \frac{F_{Rd,EC}}{F_{Rd,BS}} \quad (26)$$

Based on equation (17) and (18), the web buckling resistance given in BS5950-1 can be written as:

$$F_{Rd,BS} = \frac{25 \varepsilon t_w}{\sqrt{(S_s + nk) h_w}} (S_s + nk) t_w f_{yw} \quad (27)$$

Similarly, with $\chi_F \leq 1.0$, from equation (19)-(22), the web buckling resistance obtained from EC3 can be rewritten as:

$$F_{Rd,EC} = \frac{0.5 \sqrt{0.9 k_F E} t_w}{\sqrt{l_y h_w f_{yw}}} l_y t_w f_{yw} \quad (28)$$

If $\chi_F > 1.0$, equation (28) is not applicable. Then,

$$R = \frac{F_{Rd,EC}}{F_{Rd,BS}} = \sqrt{0.275 k_F} \sqrt{\frac{l_{y,EC}}{l_{y,BS}}} \quad (29)$$

Where, E is taken as 210GPa; $l_{y,BS}$ is taken as $S_s + nk$. For unstiffened web with load application through one flange web, the buckling coefficient k_F is taken as 6. Then the ratio R is only related to the effective loaded length l_y as follows:

$$R = \frac{F_{Rd,EC}}{F_{Rd,BS}} = 1.284 \sqrt{l_{y,EC} / l_{y,BS}} \quad (30)$$

Based on analysis of various steel sections, it is found that the effective loaded length obtained from EC3 is larger than that calculated from BS5950-1. Therefore, as shown in equation (29), the value of R is greater than 1.0, which means the web buckling resistance of EC3 is higher than that of BS5950-1. Taking the universal beams 762×267UB, 457×191UB and 457

×152UB for instance, the calculation results of web buckling resistance under transverse loading are shown in Figure 6 and Figure 7. In Figure 6 and Figure 7, the load application type (a) is adopted according to EC3, and n is taken as 5 according to BS5950-1 for calculating $l_{y,BS}$.

It can be seen that the web buckling resistance of EC3 is always larger than that of BS5950-1 for both S275 and S460 steel sections. About 40% improvement of web buckling resistance can be achieved by using the equation of EC3.

Figure 6 Web buckling resistance of rolled section with S275 steel

Figure 7 Web buckling resistance of rolled section with S460 steel

6. Lateral-torsional buckling of composite beams

In composite beam, the upper flange of steel section attached to slab by shear connection may be assumed to be laterally stable. Generally, the profiled steel sheeting is assumed to prevent any lateral-torsional buckling in the design situation of construction. Therefore, the composite beam can be taken as fully restrained against lateral buckling, though lateral-torsional buckling can occur before fixing the profiled steel sheeting. The bottom flange of steel section is in compression in hogging moment regions of continuous composite beams. The region of hogging moment at the internal supports may be considerable when only the dead loads act on one of the spans. In this situation, the lateral-torsional buckling of the bottom flange may easily occur at internal supports. Therefore, the slab is usually assumed to prevent the upper flange of the steel section from lateral-torsional buckling. But, the stability of bottom flange should be checked in the hogging moment region.

In BS5950-3.1, no equation is provided to calculate the lateral-torsional buckling resistance of continuous composite beams. When checking the lateral-torsional buckling, the method given in BS5950-1 is used. However, EC4 provides equation to check lateral-torsional buckling.

According to BS5950-3.1, when checking the lateral stability of the bottom flange in negative moment regions, the methods given in appendix G of BS5950-1 may be used. For a uniform member with one flange laterally restrained and a non-restrained compression flange, the buckling resistance moment is determined by the following equation:

$$M_{b,Rd} = p_b S_x \quad (31)$$

Where p_b is the bending strength; S_x is the plastic modulus about the major axis. The bending strength p_b for the relevant values of λ_{TB} and the yield strength of steel p_y should be obtained from Table 16 or Table 17 of BS5950-1. For a uniform member, the equivalent slenderness λ_{TB} is determined by the following expression:

$$\lambda_{TB} = n_t u v_t \lambda \quad (32)$$

with:

$$v_t = \left[\frac{4a/h_s}{1 + (2a/h_s)^2 + 0.05(\lambda/x)^2} \right]^{0.5}$$

$$n_t = \left[\frac{R_1 + 3R_2 + 4R_3 + 3R_4 + R_5 + 2(R_s - R_E)}{12R_{\max}} \right]^{0.5}$$

Where u is the buckling parameter, $u = 0.9$ for rolled sections; λ is taken as L_y/r_y ; a is the distance between the reference axis and the axis of restraint; h_s is the distance between the shear centres of flanges; x is the ratio of the section depth to flange thickness; R_1 to R_5 , R_E and R_s can be determined from the G.4.3 of BS5950-1.

According to EC4, the design buckling resistance moment of a laterally unrestrained continuous composite beam should be taken as:

$$M_{b,Rd} = \chi_{LT} M_{Rk} \quad (33)$$

With:

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \bar{\lambda}_{LT}^2}}$$

$$\bar{\lambda}_{LT} = \sqrt{\frac{M_{Rk}}{M_{cr}}}$$

$$\phi_{LT} = 0.5 \left[1 + \alpha_{LT} (\bar{\lambda}_{LT} - 0.2) + \bar{\lambda}_{LT}^2 \right]$$

Where: M_{Rd} is the design resistance moment under hogging bending at the relevant internal support; α_{LT} is the imperfection

factor related to the buckling curve; M_{Rk} is the resistance moment of the composite section using characteristic material properties; M_{cr} is the elastic critical buckling moment at an internal support of a continuous beam.

According to EN1994-1-1, the calculation of the elastic critical moment M_{cr} is based on a rather complex approach called “continuous inverted U-frame” model. This model takes into account the lateral displacement of the bottom flange causing bending of the steel web, and the rotation of the top flange that is resisted by bending of the slab. The elastic critical buckling moment at an internal support of a continuous beam is given by:

$$M_{cr} = (k_c C_4 / L) \left[(G_a I_{at} + k_s L^2 / \pi^2) E_a I_{afz} \right]^{1/2} \quad (34)$$

where: k_c is a property of the composite section; C_4 is a property of distribution of bending moment within length L ; G_a is the shear modulus for steel, $G_a = E_a / [2(1+\nu)] = 80.8 \text{ kN/mm}^2$; I_{at} is the torsional moment of area of the steel section; k_s is the rotational stiffness; L is typically, the span length; I_{afz} is the minor-axis second moment of area of the steel bottom flange, $b_f^3 t_f / 12$.

A worked example of a two-span continuous composite beam is given to compare the lateral-torsional buckling resistance calculated according to EC4 and BS5950-3.1. In the worked example, the span of composite beam is 9 m, and the thickness of composite slab with profiled steel sheeting is 130 mm. A steel section $406 \times 178 \text{ UB } 67$ with steel strength $f_y = 355 \text{ N/mm}^2$ is adopted. The cylinder compression strength of concrete used in the beam f_{ck} and the strength of reinforcement f_{sk} is 25 N/mm^2 and 500 N/mm^2 respectively. The result is illustrated in Table 6.

It is found that value of the lateral-torsional buckling resistance determined by EC4 is larger than that calculated from BS5950-3.1. EC4 leads to 14 % increase of the resistance compared to BS5950-3.1. As mentioned above, BS5950-3.1 adopted the method given in BS5950-1 for steel beam with one flange laterally restrained and a non-restrained compression flange. The influence of concrete is not considered in the calculation. However, in EC4, although the upper flange of steel section attached to slab by shear connection is assumed to be laterally stable and the bottom flange of steel section is non-restrained, which is similar to the assumption of BS5950-1, the contribution of concrete slab is considered in the calculation of the bending moment resistance M_{Rd} and the elastic critical buckling moment M_{cr} . This is the reason why the value of the lateral-torsional buckling resistance obtained from EC4 larger than that determined from BS5950-3.1.

Table 6: Comparison of lateral-torsional buckling resistance

	EC 4	BS5950-3.1	(EC 4- BS)/ BS
$M_{b,Rd}$ (kN)	546	479	14%

7. Conclusions

The impact of Eurocodes on steel and composite structures is investigated in this paper. The analysis of flexural buckling resistance of uniform compression members with grade S460 steel, the shear buckling resistance of plated structures, and the web bearing and web buckling resistance of the plated structures under transverse loading are conducted based on EC3 and BS5950-1. The study of shear resistance of headed studs and lateral-torsional buckling resistance of continuous composite beams is also

conducted according to EC4 and BS5950-3.1. Based on the results of analysis, the following conclusions can be drawn.

- (1) EC3 has additional ductility requirements on structural steel compared to BS5950 in terms of stress ratio, elongation and strain ratio. However, the stress ratio calculated according to some product standards cannot comply with the EC3 ductility requirement. Therefore, the yield strength should be modified to satisfy the ductility requirements.
- (2) For both solid slab and composite slab, the resistance of headed studs determined by EC4 is lower than that given in BS5950-3.1.
- (3) Considering the effect of residual stress, a favourable buckling curve a_0 is adopted to calculate the flexural buckling resistance for compression members with S460 steel in EC3. Therefore, the buckling resistance obtained from EC3 has a significant improvement compared with that determined from BS5950-1.
- (4) Support condition (rigidity or non-rigidity) has a significant influence on determining shear buckling resistance. Under the same web depth-to-width ratio h_w/t_w , the shear buckling resistance for non-rigid end post web is consistently lower than the value obtained from BS5950-1. However, the shear buckling resistance of the rigid end post web may exceed the value obtained from BS5950-1 if the ratio h_w/t_w exceeds a certain value.

- (5) With the increase of steel strength, the web is more susceptible to shear buckling. Results from checking the UBs using grade S460 steel show that the depth-to-width h_w/t_w of some cross-section exceed the shear buckling limit 72ε in EC3, which means shear buckling resistance must be checked under the design process. However these rolled sections are not susceptible to shear buckling according to BS5950-1.
- (6) The resistance to transverse loading is determined by two main factors, the effective loaded length l_y and the reduction factor χ . The deduction and worked example show that the effective loaded length and reduction factor for rolled section predicted by EC3 are always larger than the length predicted by BS5950-1. Therefore, the resistance to transverse loading obtained by BS5950 is more conservative compared with EC3.
- (7) The value of lateral-torsional buckling resistance of continuous composite beams determined by EC4 is larger than that calculated from BS5950-3.1. This is because that the contribution of concrete is considered in EC4, while not taken into account in the design of BS5950-3.1.

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