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Continuous Composite Beams for Buildings

Poutres mixtes continues dans le bâtiment

Durchlaufende Verbundträger für Hochbauten

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SUMMARY

The treatments in the first draft of Eurocode 4 of the classification of steel cross sections, lateral torsional buckling, and partial shear connection are shown to have much influence on the conception and design of continuous composite beams. Further research on lateral torsional buckling and on the stiffening of steel webs is needed, to enable design methods to be improved and costs to be reduced.

RÉSUMÉ

Le déversement et la connection partielle ont une très grande influence sur la conception et le dimensionnement des poutres mixtes continues dans la version préliminaire de l'Eurocode 4. Dans le but d'améliorer les méthodes de calcul et de réduire les coûts de construction, il est nécessaire d'entreprendre des recherches sur le déversement et sur les raidisseurs d'âme.

ZUSAMMENFASSUNG

Die im ersten Entwurf des Eurocodes 4 bearbeitete Klassifizierung von Stahl-Querschnitten, das Torsionsdrillknicken und die Teilverdübelungen haben erwiesenermassen grossen Einfluss auf den Entwurf und die Ausführung von durchlaufenden Verbundträgern. Weitere Untersuchungen in Bezug auf das Torsionsdrillknicken und die Stegversteifungen sind zur Verbesserung der Berechnungsmethoden und zur Reduzierung der Kosten erforderlich.

1. INTRODUCTION

One of the first decisions that has to be made in the design of a composite floor structure for a building, with or without steel sheeting, is whether to use simply-supported or continuous beams.

The advantages of continuity are:
- the structure can be made shallower or its deflections reduced, or both;
- the susceptibility of the structure to vibration should be reduced;
- close control of the width of cracks in the slab near internal supports becomes possible without the use of joints in the slab; and
- the weight of steel in the beams can be reduced.

The apparent disadvantages (with comments on them) are:
- the beam-to-column joints become more expensive, unless the beams are designed to pass either side of the columns;
- if end-plate joints are used, the steel frame is more sensitive to inaccuracies of fabrication;
- erection of the frame may be more difficult; but when erected, it needs less temporary bracing; and
- there may be an increase in the design bending moments in some columns; but the influence of this on the weight of steel in columns is rarely significant.

It appears that in many structures, the use of continuity should save money. Except in respect of lateral buckling, nearly all the necessary research has been done; much of it before 1967, when continuous composite beams were first treated in a British Code of Practice (CP 117: Part 2, for bridges); but in 1985 there is still no British code for continuous beams in buildings, which partly explains why they are so rarely used in the U.K. They were fully treated in the European Model Code of 1981 [1], and its scope was extended in the draft Eurocode 4 [2] to include composite frames.

Another problem of continuity is that at present, design takes longer, partly because designers get so little relevant experience. The remedy lies with computers. The development of software for this purpose is likely to accelerate as soon as Eurocode 4 is finalised.

The object of this paper is to discuss the implications of some of the clauses of draft Eurocode 4 relevant to continuous beams, with reference to:
- the influence of the classification of steel cross sections;
- design for lateral torsional buckling; and
- partial shear connection.

The scope of the paper is limited to fully continuous composite beams in frames braced against sidesway, using rolled steel sections. Semi-rigid joints are not considered, nor is the use of elastic-plastic analysis of the structure, as these are subjects for current research.

2. CLASSIFICATION OF CROSS SECTIONS OF COMPOSITE BEAMS

2.1 The four Classes of Eurocodes 3 and 4

In these and other recent codes, account is taken of local buckling of steelwork by placing each steel web and compression flange into one of four classes: Class 1, Plastic; Class 2, Compact; Class 3, Semi-compact; and Class 4, Slender. The methods of analysis given in Eurocode 4 for a continuous beam are determined by the classes of its critical cross sections, which for this purpose can be assumed to be sections at each internal support and near the centre of each span.

Plastic hinge analysis of the structure is allowed when all relevant sections are in Class 1, and plastic analysis can be used for the resistance in bending (or bending and shear) of sections in Class 1 or 2. Elastic analysis can be used without restriction. The limiting slendernesses proposed in draft Eurocode 4 are given in Table 1. Those for Classes 1 and 2 are 10% to 25% lower than their counterparts in draft Eurocode 3 [3], for reasons explained elsewhere [4]. In Table 1:

b_0 is the overall breadth of a flange of mean thickness t,
d is the depth between fillets of a web of thickness w,
αd is the depth of the web in compression, and
ϵ takes account of the specified yield strength of the steel, with values:
1.0 for steel Fe 360, with yield strength 235 N/mm^2,
0.814 for steel Fe 510, with yield strength 355 N/mm^2.

		Class 1	Class 2	Class 3
Flanges,	b_0/t	16ϵ	20ϵ	30ϵ
Webs,	d/w	$30\epsilon/\alpha$	$33\epsilon/\alpha$	As in EC3

Table 1 Maximum b_0/t and d/w ratios for steel sections in composite beams

2.2 Steel compression flanges

Any steel flange that is attached to a concrete slab by shear connection in accordance with EC3 is assumed to be in Class 1. The class of other flanges depends only on the specified yield strength of the steel. Guidance is given in Table 2 on the classification of the flanges of the European standard sections IPE, IPE-A, and HEA [5] and of the British UB sections, in terms of the overall depth of the section, h.

Steel	Class 1		Class 2	
Fe 360	IPE,	all	IPE,	all
	IPE-A,	$h \geq 330$ mm	IPE-A,	all
	HEA,	$h \geq 390$ mm	HEA,	$h \geq 310$ mm
	UB	nearly all	UB,	all
Fe 510	IPE,	$h \geq 190$ mm	IPE,	all
	IPE-A,	$h \geq 600$ mm	IPE-A,	$h \geq 330$ mm
	HEA,	$h \geq 490$ mm	HEA,	$h \geq 390$ mm
	UB,	heavier end of each size range	UB,	nearly all

Table 2 Classification of flanges of rolled steel sections

2.3 Steel webs

To demonstrate that a web is in Class 1 or Class 2, the depth of the web in compression is determined from the position of the plastic neutral axis of the composite section; so that no account need be taken of the modular ratio or of the effects of sequential construction of the concrete slab. If the depth exceeds the limit for Class 2, the web will normally be in Class 3 if the steel member is a rolled section.

For midspan cross sections of composite T-beams for buildings, the plastic neutral axis is usually in the slab or steel top flange, so the section is in Class 1. Even in L-beams, the depth of web in compression is rarely enough to put the section into Class 2.

At an internal support, the class of the section depends on the amount of longitudinal reinforcement in the slab. This is shown by the following example. An internal span of length 12 m of a T-beam consists of a slab 120 mm thick and more than 3.0 m wide, composite with an IPE 550 steel section, for which the web has a depth between fillets of 468 mm and a thickness of 11.1 mm (d/w = 42.2). The corresponding UB section is a 533 x 210 UB 109, with d = 476.5 mm, w = 11.6 mm, and d/w = 41.1. If the section is in Class 1 or Class 2, the effective breadth of the concrete flange is given by Eurocode 4 as L/4, or 3.0 m. Let there be r% of longitudinal reinforcement at an internal support (i.e., an area of 36r cm^2), with a design yield strength of 425/1.15 = 370 N/mm^2.

When the section reaches its plastic moment of resistance in hogging bending, the net compressive force in the web equals the tensile force to cause yield of the reinforcement, so that the proportion of the depth of the web in compression, α, increases with r. Maximum values of r for the web to be just in Classes 1 and 2 are given in Table 3, for steels Fe 360 and Fe 510.

	Class 1		Class 2	
Fe 360	r = 0.39%,	α = 0.71	r = 0.52%,	α = 0.78
Fe 510	r = 0.22%,	α = 0.58	r = 0.38%	α = 0.64

Table 3. Influence of reinforcement ratios on class of cross section

The signficance of these results is that the four values of r all lie within the practical range. The lightest slab reinforcement possible is that required for crack-width control, which could be less than 0.2% if half or more of the overall depth of the slab is taken up with profiled steel sheeting. The section would then be in Class 1, as are the sections at midspan, so plastic hinge analysis could be used.

If the designer sought to take maximum advantage of continuity because the span/depth ratio was high, a reinforcement ratio exceeding 0.8% might be considered, which would put the composite section well into Class 3, with consequences for design that are now considered.

2.4 Design of a beam with one or more sections in Class 3

In beams with critical sections in Classes 1 and 2 only, plastic behaviour can occur without buckling. Such beams are more tolerant of the effects of sequence of construction, unforeseen load distribution, shrinkage of concrete, and temperature differences than are beams with sections in Classes 3 or 4. The boundary between Class 2 and Class 3 also corresponds roughly to the transition from beams for buildings to beams for bridges. For these reasons there is in both Eurocodes 3 and 4 a marked increase in the complexity of design methods, when even one section of a continuous beam is moved from Class 2 into Class 3. The main requirements for Class 3 composite beams that differ from those for Class 2 are now outlined.

(1) The classification of a web depends on the depth of the web in compression as given by elastic analysis *for the load case considered*, and so is not independent of loading or of the sequence of construction.

(2) If continuous beams are analysed using uncracked flexural stiffnesses, redistribution of moments from internal supports is allowed up to 30% when these sections are in Class 2; but none is allowed for Class 3 unless the midspan sections are designed elastically, as if in Class 3, even though they are likely to be in Class 1. The reason is that when regions near internal supports remain elastic (as Class 3 sections must do), a midspan region is likely to shed bending moment to them before it can reach its plastic moment of resistance.

(3) It can no longer be assumed in design that all load is carried by the composite

member when unpropped construction is used.

(4) The secondary (hyperstatic) effects of shrinkage of concrete have to be considered.

(5) Elastic analysis of the section is used, with a more accurate but less simple evaluation of effective width. It may be necessary to use different modular ratios for live and for dead loading, and so to keep the effects of these loads distinct in calculations.

(6) No provision is made in Eurocode 4 for the use of partial shear connection in members with sections in Classes 3 or 4, because of the lack of relevant research.

The consequences for design are obvious. The entrance fee to Class 3 is high!

For a continuous beam of equal spans, the most critical sections are at the penultimate supports. Often, one would like to use a steel beam of constant section, probably just in Class 2, and to stiffen these two regions while keeping them in Class 2. It is clear from Table 3 that there is little scope for doing this by increasing the longitudinal reinforcement in the slab, unless:

- a bottom-flange plate of equivalent area is also added (which may be visually unacceptable), or
- the web is made less slender, so that the increased depth in compression is still in Class 2.

If the vertical shear is high, it is to be expected that vertical stiffeners would improve the stability of the web; but their spacing would have to be less than its depth, because local buckles in regions of moment gradient are of short wavelength. A cheaper alternative would be to provide a longitudinal stiffener just below mid-depth of the web (Fig. 1). These have been shown in tests [6] to be most effective in delaying web buckling until after much plastic rotation has occurred. No design methods for such stiffeners, related to the classification of the web, are given in the draft Eurocodes because further research on them is needed.

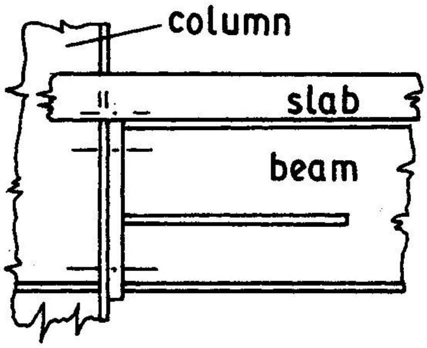

Fig. 1 Longitudinal stiffener

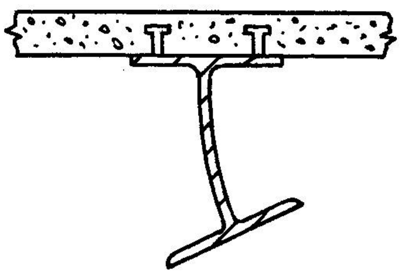

Fig. 2 Lateral buckling

3. LATERAL BUCKLING OF BOTTOM FLANGES NEAR INTERNAL SUPPORTS

3.1 Eurocode 4

This form of buckling involves lateral and torsional displacement of a bottom flange in a region of steep moment gradient. Most rolled sections used for composite beams are not susceptible, so a quick method is needed for identifying those which are. Design methods given for all-steel beams can be used, but are likely to be over-conservative, because most of them, including that of Eurocode 3, are based on theory in which the top (tension) flange of the steel member is assumed to be free to twist about a longitudinal axis and to deflect sideways, so that buckling can occur without distortion of the cross section. None of these assumptions is true for composite beams. The slab cannot deflect sideways and provides stiff resistance

to twisting, so that the buckling is *distortional* and involves bending of the web, which provides vertical, lateral, and torsional restraint to the bottom flange (Fig. 2).
It is stated in Eurocode 4 which are the IPE and HE steel sections that can be assumed not to need permanent bracing against lateral buckling. These exemptions are based on the rules for continuous torsional restraint given in Eurocode 3, modified to allow for distortion of the section, and checked by a parametric study done during drafting of the Netherlands code of practice of 1983. They are conservative, in that the steel top flange is still assumed to be free to deflect laterally. They apply to members in Fe 510 or weaker steel, supported at both ends, and connected to a concrete slab not less than 100 mm thick.

When elastic analysis of the structure is used, all IPE and HE sections qualify (Table 4). The list in Eurocode 4 of sections that qualify when plastic hinge analysis is used is misleading, because lateral buckling rarely governs. The webs of many of the sections that qualify are found to be in Class 2, when account is taken of the reinforcement in the slab. For example, typical calculations show that even with lightly reinforced slabs, the level of the plastic neutral axis of the composite section in hogging bending is usually such that $0.6 < \alpha < 0.7$, where α is as in Table 1. If α is taken as 0.65, the slenderness limits for Class 1 webs become:

$$\left.\begin{array}{ll} d/w \ngtr 46.1 & \text{when } f_y = 235 \text{ N/mm}^2 \\ d/w \ngtr 37.6 & \text{when } f_y = 355 \text{ N/mm}^2. \end{array}\right\} \quad (1)$$

These limits are shown in Fig. 3, together with the ranges of values of *d* and *w* for UB sections (horizontal lines), and for IPE-A and IPE sections (dashed lines).

Steel	Plastic hinge analysis (governed by local buckling)	Elastic analysis (governed by lateral buckling)
Fe 360	IPE, all IPE-A, $d \leq 300$ mm HEA, $d \leq 700$ mm UB, most with $d \leq 550$ mm	IPE, all IPE-A, all HEA, all UB, all
Fe 510	IPE, $d \leq 300$ mm IPE-A, none HEA, $d \leq 490$ mm UB, few, see Figure 3	IPE, all IPE-A in Class 2, $d \leq 300$ mm IPE-A in Class 3, all HEA, all UB, most with $d \leq 550$ mm

Table 4 Sections unlikely to require bracing against lateral buckling

3.2 Other steel sections that should not need lateral bracing

Exemptions for other types of steel section are now discussed. The writer studied this subject by means of parametric numerical analyses for elastic critical stresses for distortional lateral buckling of bottom flanges of fixed-ended composite T-beams [7]. It was found that for floor slabs of typical stiffness, the only significant parameter was the depth/thickness ratio (d/w) of the steel web.

It was assumed that the relationship between the critical stress and the true buckling stress was given by the Perry-Robertson equation that is used in the British Bridge Code (BS 5400) for other forms of lateral buckling. The equivalent lateral-torsional slenderness was found to be

$$\lambda = 3.08\ (d/w)^{0.7}\ . \quad (2)$$

In the Bridge Code, this form of buckling is neglected when

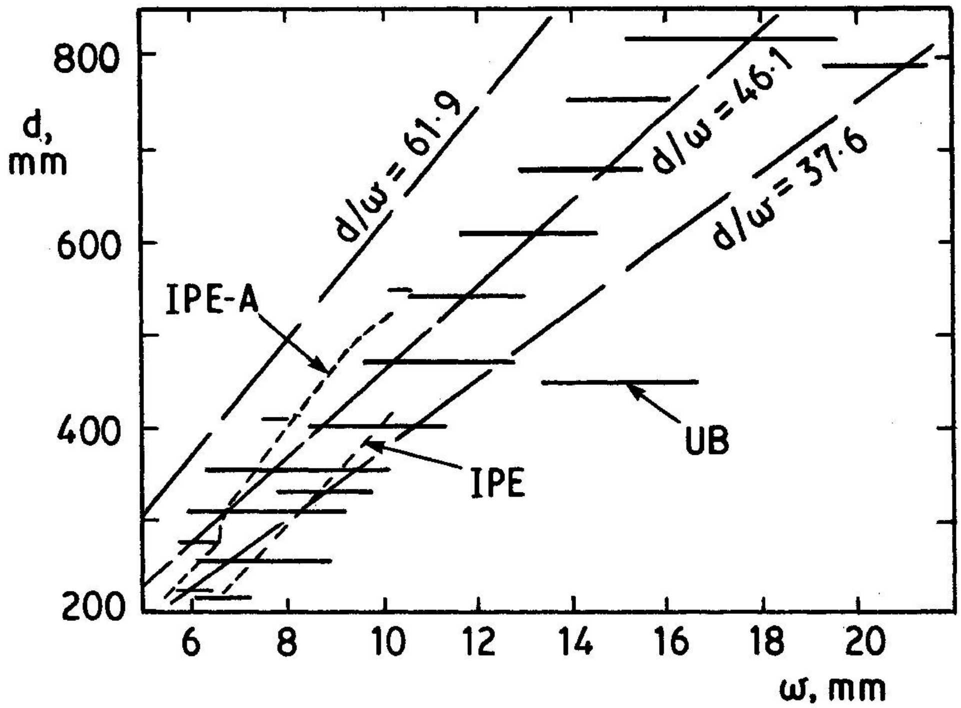

Fig. 3. Limiting slendernesses for web

$\lambda_{LT}(f_y/355)^{1/2} \leq 45$, with the yield strength f_y in N/mm² units. From equation (2) this gives

$$d/w \leq 46.1\ (355/f_y)^{0.714} \tag{3}$$

which is:

$$\left.\begin{aligned} d/w &\leq 61.9 \text{ when } f_y = 235 \text{ N/mm}^2 \\ d/w &\leq 46.1 \text{ when } f_y = 355 \text{ N/mm}^2 . \end{aligned}\right\} \tag{4}$$

These results take account of the use of plastic analysis of sections at each internal support, but not of the use of plastic hinge theory, so they are relevant to sections in Class 2, and conservative for Class 3. The limits (4) are shown in Fig. 3. They confirm the recommendation in Eurocode 4 that all IPE and HE sections qualify, and give the results for IPE-A and UB sections shown in column 2 of Table 4.

4. PARTIAL SHEAR CONNECTION

The design methods of Eurocode 4 for partial shear connection in continuous composite beams [8] are developed from those given in the Model Code [1]. They are applicable to beams with all critical cross sections in Classes 1 or 2. They often enable only 50% of full shear connection to be used, which simplifies detailing when profiled steel sheeting is used.

The methods are explained with reference to stud shear connectors and a propped cantilever subjected to uniformly distributed load (Fig. 4). The relevant levels of load per unit length are:

w, the design ultimate load, for which the number N of uniformly-spaced studs is to be calculated;

w_f, the ultimate load calculated from plastic hinge analysis of the composite member, for which N_f uniformly-spaced studs would be needed; and

w_a, the ultimate load from plastic hinge analysis of the steel beam alone.

Connectors are classified as "ductile" or "stiff". Welded studs of the usual sizes are ductile, provided that the specified cylinder strength of the concrete does not exceed

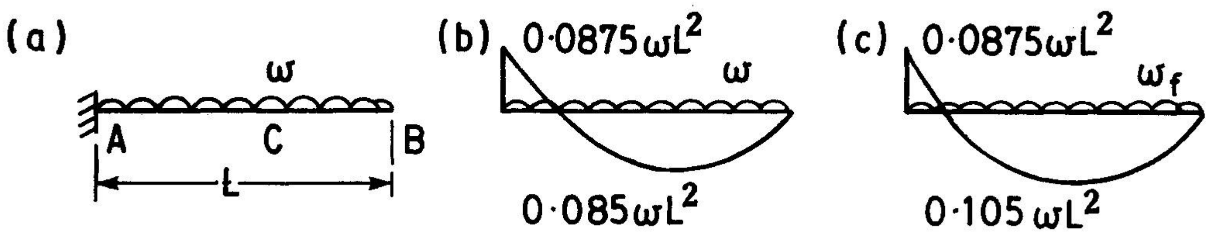

Fig. 4. Propped cantilever in Class 2

30 N/mm². This limit takes account of the reduced strain capacity of stronger concrete. Most bar-type connectors are stiff.

The principal design equations are:

- for ductile connectors, $N/N_f = (w - w_a)/(w_f - w_a)$, but ≥ 0.5, (5)
- for stiff connectors, $N/N_f = w/w_f$, but ≥ 0.5. (6)

As an example, it is assumed that the cross section at A (Fig. 4(a)) is in Class 2. Elastic analysis of a uniform member with 30% redistribution to midspan gives the design moments of Fig. 4(b). A member is designed with full-interaction bending resistance at A, M_a, equal to the required value $0.0875\ wL^2$; a sagging resistance at C, $M_C = 1.2\ M_a$; and resistance of the steel beam alone, $M_s = 0.9\ M_a$.

Plastic hinge analyses give $w_f = 1.163\ w$, Fig. 4(c), and $w_a = 0.918\ w$. Equation (5) then gives $N/N_f = 0.33$, so 50% shear connection is provided.

The savings in shear connection are due mainly to the fact that when the structure is analysed elastically, there is usually surplus flexural resistance at midspan, even when redistribution is used.

5. CLOSURE

Accounts have been given of the treatment in the draft Eurocode 4 of three aspects of the design of continuous composite beams for buildings, and of their expected implications in practice. These show why "the limiting slendernesses given for Class 2 are the most significant numbers in the code" [4]. The use and calibration of these methods in trial designs should enable them and their presentation to be further improved.

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