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Developments in composite construction and cellular beams

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Abstract. This paper describes recent developments in composite construction and their effect on codified design procedures in the UK. Areas of particular interest include: rules on shear connection, design of beams with web openings, serviceability limits, such as floor vibrations, and fire safe design. The design of cellular beams with regular circular openings now includes generalized rules for web-post buckling, and for the development of in-plane moment in the web-post for asymmetric sections. Closed solutions for the maximum shear force due to limits on web-post bending or buckling are presented. The fire resistance of cellular beams is also dependent on the temperature of the web-post, and for closely spaced openings. It is necessary to increase the thickness of fire protection to the web. For serviceability design of beams, deflection limits and natural frequency and response factor for vibration are presented. It may be necessary to use stricter limits for certain applications.

Key words: composite construction; shear connection; web openings; cellular beams; deflections; floor vibrations.

1. Introduction

Composite construction has been well established in the UK since the early 1980's, and the relevant British Standard, BS 5950-3:Design in composite construction, was first published in 1990. It refers closely to BS 5950-1 which was revised in 2000.

The comparative Eurocodes are EN 1993-1-1: Eurocode 3 and EN 1994-1-1: Eurocode 4, which are going through the process of final editing and approvals. The range of application of these standards is presented in Table 1. At a technical level, Eurocode 4 and BS 5950-3 and -4 are similar although there are subtle differences, for example, in methods of test of composite slabs. Eurocode 4 covers composite columns and partially encased beams, which BS 5950-3 does not. There are also fire design parts of Eurocodes 3 and 4, which are linked to European fire test procedures.

This paper reviews some of the recent developments in composite construction and how they have been included in codified or industry-standards.

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Table 1 Range of application of BS 5950-3 and Eurocode 4

Scope	BS 5950-3 and -4	ENV 1994-1-1 Eurocode 4
Composite beams	Simply supported and continuous beams.	Simply supported and continuous composite beams
Shear connection	Shear connector resistance based on BS 5400-5. Interaction with deck profile shape based on AISC formula subject to limits of application.	Shear connector resistance based on empirical formula. Reduced interaction with deck profile shape and thickness of steel sheet.
Degree of shear connection	Limit on degree of shear connection for all cases.	Stricter shear connection limits asymmetric beams than for hot rolled steel sections.
Partially encased beams	Not covered.	Rules for normal and fire design.
Web openings	Not covered-refer to SCI publication P-056.	Covered by draft Annex N, but this is being revised
Composite columns	Not covered-refer to BS 5400-5.	Based on effective slenderness of composite cross-section
Composite slabs	Semi-empirical method based on ‘m and k’ parameters obtained from tests.	Two methods - based on partial shear connector, or ‘m and k’ method.
Fire resistance	Covered by BS 5950-8 and manufacturer’s test data.	Covered by ENV 1994-1-2 and product test procedures.
Composite connections	Not covered.	Covered in principle.
Vibrations	Refers to SCI publication P-076.	Covered in principle only.
Slimfloor or *Slimdek*	Testing of composite slabs covered by BS 5950-4. Refer to SCI publications P-175 and P-248.	Testing of composite slabs covered by Eurocode 4.

2. Recent developments in composite design

2.1. Web openings and cellular beams

A major development in the UK and France has been in the technology of cellular beams manufactured by cutting and re-welding hot-rolled steel sections, or fabricated by welding from steel plate. The first approach leads to beams with regular circular openings (see Fig. 1). For fabricated beams, openings can be positioned at the chosen locations for optimized design (see Fig. 2). In both cases, elongated circular openings can be used, but in fabricated beams, rectangular openings should be located a certain distance from other openings, or from point loads, to avoid interaction effects.

The primary design checks include the influence of the openings on the pure shear and bending resistance of the composite beams, which were first presented by Lawson (1989) in the UK. However, based on recent research at a European level, further checks are required that may be new to the designer:

- Vierendeel bending resistance due to composite action.
- Web-post buckling between closely spaced openings.
- Influence of asymmetry of the cross-section on web-post moment.

Fig. 1 Cellular beam with regular openings for services

Fig. 2 Fabricated beam with elongated and rectangular openings

2.2. Vierendeel bending

Vierendeel bending resistance depends on the development of plastic hinges at the four corners of the opening together with an additional factor, M_{vc} due to local composite action of the top web-flange section (Tee) with the slab. Chung and Lawson (2001) presented a simplified equation for equilibrium as follows:

$$Vl_o \leq \sum M_p + M_{vc} \tag{1}$$

where

$$M_{vc} = k_l N_{SC} P_d (D_s - 0.5y_c + y_{et}) \tag{2}$$

V = shear force at an opening
N_{sc} = number of shear connectors placed over the opening
P_d = design resistance of a shear connector
$\sum M_p$ = sum of plastic bending resistances of the Tees, reduced for axial forces
D_s = slab depth
y_c = neutral axis depth in the slab

y_{et} = elastic neutral axis depth of the top web-flange section (≈ flange thickness)
k_l = reduction factor due to the length of the opening arising from second-order effects

$$= \left(1 - \frac{l_o}{25D_t}\right) \text{ for unstiffened openings} \qquad (3)$$

= 1.0 for $l_o \leq 5D_t$

where l_o = effective length of opening (= 0.5 d_o for circular openings)
D_t = depth of top Tee
d_o = diameter (or depth) of opening

2.3. Web-post buckling

Web-post buckling is expressed in terms of horizontal shear, V_h, acting on the web-post, as illustrated in Fig. 3. The horizontal shear stress acting on the web-post is given by:

$$\sigma = \frac{V_h}{s_o t_w} \quad \text{where } V_h \approx \frac{Vs}{h + D_s - 0.5y_c} \qquad (4)$$

where V_h = horizontal shear force in the web-post
s = centre-to-centre spacing of openings
s_o = edge-to-edge spacing of openings (= s-l_o)
t_w = web thickness
h = effective depth of the steel section.

The buckling strength may be established from an effective buckling length of the web-post given by:

$$l_{eff} = 0.5\sqrt{s_o^2 + d_o^2} \leq 0.7d_o \qquad \text{for circular openings} \qquad (5)$$

$$l_{eff} = 0.7\sqrt{s_o^2 + d_o^2} \leq d_o \qquad \text{for rectangular openings} \qquad (6)$$

The effective slenderness of the web is:

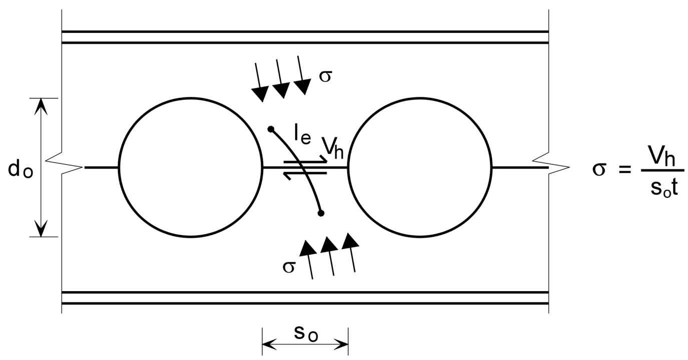

Fig. 3 Web-post buckling model in cellular beams

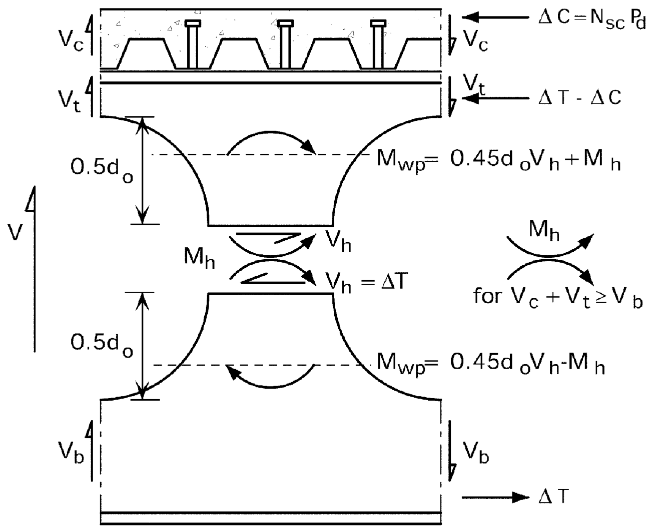

Fig. 4 Web-post moment model in cellular beams

$$\lambda_{eff} = \sqrt{12} l_{eff}/t_w \tag{7}$$

The buckling strength of the web-post, σ_c, is obtained from column curve *c* of BS 5950-1, and it follows that $\sigma \leq \sigma_c$ for an acceptable design. It should be noted that web-post buckling is not considered to occur when $d_o \leq 20\ t_w$.

2.4. Web-post moment

For highly asymmetric cellular beams, a moment is developed in the web-post for equilibrium between the top and bottom Tees, as illustrated in Fig. 4. Generally in a composite beam, most of the applied shear is resisted by the top Tee, as the transfer of shear across the bottom Tee is limited by its Vierendeel bending resistance. The in-plane moment acting on the web-post is given by:

$$M_h = (V - 2V_b)s/2 + V_h e - \Delta C(D_s - 0.5y_c + y_{et})/2 \tag{8}$$

where V_b is the shear force in the bottom Tee

e is the eccentricity of the center-line of the opening above the center-line of the beam

ΔC is the compression force developed in the slab in the distance, s

M_h is the moment in the web-post, which can have a positive or a negative value. Generally a solution can be found where $M_h = 0$, and the shear force in the bottom Tee is at its limiting value of:

$$V_b \leq 2M_{pb}/l_o \tag{9}$$

where M_{pb} is the plastic bending resistance of the bottom Tee, reduced for the effect of axial tension.

A closed solution may be derived for the maximum value of the vertical shear force, *V*, when limited by web-post bending, which is given by:

$$V \le \frac{2\left(\frac{M_{h,e}}{s} + 2\frac{M_{pb}}{l_o}\right)}{\left(1 + \frac{2e}{h}\right)} + \frac{\Delta C}{s}(D_s - 0.5y_c + y_{et}) \tag{10}$$

where $M_{h,e}$ is the elastic bending resistance of the web-post, given by:

$$M_{h,e} = s_o^2 t_w p_y / 6 \tag{11}$$

For rectangular openings, the critical moment occurs at the top of the opening in which case, $2e$ is replaced by $(2e+d_o)$ in the above formula.

A closed solution may also be derived for the maximum value of shear force influenced by web-post buckling, which is given by:

$$V \le \frac{(\sigma_c s_o t_w + 4M_{pb}/l_o)}{\left(1 + \frac{2e + d_o}{h}\right)} + \frac{\Delta C}{s}(D_s - 0.5y_c + y_{et}) \tag{12}$$

It is generally found that the web-post moment check is critical for rectangular openings, and the web-post buckling check is critical for circular openings. Because of this, it is recommended that the edge to edge distance of rectangular openings should exceed l_o. The corresponding web-post buckling limit for circular openings may be reduced to 0.4 d_o.

2.5. Fire engineering

The fire engineering design of cellular beams is relatively complex because the web-post is hotter than the adjacent bottom flange, on which the calculation of the required level of fire protection is usually based. This factor representing the increase in temperature of the web-post can vary between 1.05 and 1.3, depending on the width of the web-post. For designs controlled by web-post shear or buckling, it is necessary to increase the required thickness of fire protection to keep the web temperature below its critical temperature of approximately 550°C.

Recent guidance by the Steel Construction Institute (New Steel Construction, 2003) presents the following increases in fire protection for intumescent coatings, which are generally assessed at a temperature of 620°C. The unity factor is based on the highest of the design checks on the cellular beam. For conventional fire protection materials that are assessed at 550°C, the right hand side of

Table 2 Factor defining increase in fire protection for cellular beams

Spacing : diameter ratio of openings	Materials assessed at 620°C			Materials assessed at 550°C		
	$d/t_w \le 55\varepsilon$	$\le 62\varepsilon$	$\le 70\varepsilon$	$d/t_w \le 55\varepsilon$	$\le 62\varepsilon$	$\le 70\varepsilon$
$1.4 \le s/d_o < 1.5$	1.4	1.5	1.6	1.1	1.3	1.4
$1.5 \le s/d_o < 1.8$	1.1	1.2	1.3	1.0	1.1	1.1
$s/d_o \ge 1.8$	1.0	1.1	1.1	1.0	1.0	1.0

Table 2 may be used. The web slenderness is important because it influences web buckling. Implicit in this table is a load ratio of 0.5 in fire conditions.

This increase in fire protection thickness is expressed as relative to the thickness required for the fully exposed bottom Tee or the unperforated composite section. The web slenderness is expressed as the *d/t* ratio modified by the factor $\varepsilon = \sqrt{275/p_y}$, where p_y is the yield strength of steel.

3. Partial shear connection

Rules for shear connection in composite beams depend on the deformation capacity of the shear connectors, and on the degree of shear connection and span of the beam, which influences the slip in the shear connectors. The conventional deformation capacity adopted for welded shear connectors is 6 mm, but this limit may be difficult to achieve for some types of shear connector and deck profile shapes.

In BS 5950-3, the limit on the degree of shear connection is presented as:

$$K \geq (L-6)/10 \quad \text{but } 1.0 \geq K \geq 0.4 \tag{13}$$

where L is the beam span in metres (m).

This formula applies to a composite beam that is designed to its full bending resistance. For a beam that is under-utilised in bending, it may be possible to argue that the required K value may be further multiplied by the unity factor in bending. Therefore for a unity factor of 0.8, the minimum degree of shear connection may also be multiplied by 0.8.

For less ductile forms of shear connectors, it may be argued that the degree of shear connection should be increased above 100% (i.e., to approach the case of elastic shear flow). In cases of reduced deformation capacity, a generalized formula for the minimum degree of shear connection for plastic design might be:

$$K \geq (L-6)/10 \times UF_b \times (6/\delta_{sc})^{0.5} \tag{14}$$

where UF_b is the unity factor of the composite beam in pure bending.

δ_{sc} is the maximum slip in the shear connectors at their design resistance (≤ 6 mm).

The corresponding limit on the degree of shear connection in Eurocode 4 for single shear connectors per deck rib is $K > 0.04\,L$ for S355 steel. However, in this case, the linear interaction method must be used, which is conservative by 10-20% for partial shear connection design.

For shear connectors that are classified as non-ductile, elastic shear flow should be used to determine the required number of shear connectors along the beam. The influence of deck profile shape is also important. In BS 5950-3, the reduction factor on the shear connector resistance is based on the existing AISC approach with modifications for the number of shear connectors etc.

The reduced design resistance of a shear connector is given by:

$$P_{d,red} = P_d \times \frac{0.7}{\sqrt{N_{sc}}} \times \frac{b_r}{D_p} \times \frac{(h_s - D_p)}{D_p} \tag{15}$$

where b_r is the average width of the rib of deck profile

D_p is the height of deck profile

h_s is the height of shear connector ($\geq D_p$+35 mm)
N_{sc} is the number of shear connectors per rib

In the limit, the maximum design resistance of the shear connectors is taken as:

$$P_{d,red} \leq 0.8\ P_d \text{ for two shear connectors per rib}$$
$$\leq 0.6\ P_d \text{ for three shear connectors per rib}$$

Eurocode 4 further reduces these limits for through-deck welding of shear connectors with steel decking of less than 1 mm, or for pre-cut holes in the decking.

Other interaction formulae have been proposed, which better reflect the failure mechanism.

4. Serviceability design

BS 5950-3 refers to BS 5950-1:2000 for deflection limits, but it is recognized that there are many areas of serviceability design which are left to 'engineering judgment', notably:

- vibration design of floors
- deflection limits for long span beams
- deflection limits for edge beams supporting cladding
- deflection limits for sway frames.

The commonly accepted imposed load deflection limit for beams is span/360, but it is also necessary for practical and visual reasons to introduce limits for total deflection, either as a function of span (i.e., span/200) or as an absolute value (i.e., 50 mm). For beams supporting glazing, stricter limits are required, and the total deflection is generally reduced to as low as 10 to 20 mm. Generally accepted serviceability limits are presented in Table 3.

The check on vibration sensitivity of floors is often presented in terms of a minimum natural frequency (a limit of 4 H_z is often used for composite construction). However, this single natural frequency limit is not in itself sufficient, as an acceptable performance depends on the effective mass of the floor plate, the impulsive actions, the sensitivity of the occupants etc. For example, for lightweight floors, it is necessary to raise the limit to 8 H_z to avoid resonant effects. In specialist buildings, such as hospitals, a higher natural frequency limit should be imposed and a response factor calculation should be made.

Table 3 Proposed deflection limits for beams

Application	Deflection limits		Comments
	Imposed load	Total load	
Beams-exposed	L/360	L/250 but ≤ 40 mm	Limited for visual reasons
Beams-with suspended ceiling	L/360	L/200 but ≤ 50 mm	Depends on type of raised floor and suspended ceiling
Beams supporting cladding	L/500	L/350 but ≤ 25 mm	Depends on type of cladding Stricter limits for glazing
Lightweight floors	L/500	L/350 but ≤ 15 mm	Limits vibration sensitivity

Table 4 Natural frequency and response factor limits for floors

Application (Composite floors)	Natural frequency limit of floor (H_z)	Response factor limit	Comments
Offices	$\geq 4\ H_z$	4-quiet 8-general 12-busy	R=8 is generally specified
Residential (except for lightweight floors)	$\geq 5\ H_z$	4-typically	R=4 for multi-occupancy
Hospitals	$\geq 6\ H_z$ generally	2-wards 1-operating theatre	R=1 achieved by effective mass of floor
Lightweight floors	$\geq 8\ H_z$	Not applicable	Response Factor is not appropriate: Limit deflection-see Table 3.

R is the multiple of the base level of perceptibility of RMS acceleration of 0.005 m/s^2 corresponding to z-axis vibrations

EN1990: Eurocodes - Basis of Structural Design presents partial factors for loads, and gives general design information common to all structural materials. Interestingly, the natural frequency limit of 3 H_z in Eurocode 3 has been dropped in favour of a more general statement of principle.

The natural frequency may be established for a load corresponding to the self weight of the floor and all permanent loads (excluding partitions) plus a proportion of the imposed load (generally taken as 10% or a minimum of 30 kg/m^2). The natural frequency of the floor may be calculated from the following simplified formula:

$$f = \frac{18}{\sqrt{\delta_{sw}}} H_z$$

where δ_{sw} is the deflection of the composite beam subject to the above loads (mm). In a grillage of members, δ_{sw} should take into account the combined deflection of the members in the floor system. Wyatt (1989) states that for frequency modes dominated by the secondary beams, the primary beams may be assumed to be nodal lines (i.e., lines of zero displacement). For frequency modes dominated by the primary beams, the secondary beams may be assumed to be fixed-ended in the calculation of their additional deflection. The composite slab is assumed to be relatively stiff in all cases. For simplified design, the 'system' natural frequency of the floor may be taken as 20% less than the natural frequency of the most flexible beam.

The second level of calculation requires estimation of a Response Factor, R, which depends on the participating mass of the floor. Generally, a number of bays in a particular floor may be assumed to respond to an impulsive effect and a Response Factor, R, of 8 can generally be achieved in composite construction. The R value is expressed as a multiple of the basic perceptibility level for vertical (z-axis) vibrations to BS 6472. A Response Factor of 8 corresponds to a maximum Root Mean Square (RMS) acceleration of 0.005×8=0.04 m/sec^2. Typical Response Factors are also presented in Table 3. Long span floors increase the participating mass of the floor plate and often perform better than medium span floors, despite their lower natural frequency. A damping ratio of 3% can be generally used for open plan offices, increasing to 4.5% for heavily partitioned offices.

5. Conclusions

This paper defines the equilibrium conditions due to transfer of shear forces across large openings in cellular beams and shows that the web-post between the openings can be highly stressed and subject to buckling. Checks on web-post bending and buckling should be made depending on the asymmetry of the cross-section, and closed solutions are presented to avoid an iterative design procedure.

Fire design may also be critical due to the slenderness of the web-post because the thickness of fire protection is usually determined on the basis of the bending stresses rather than shear buckling. For cellular beams with narrow web-posts, it is necessary to increase the thickness of fire protection relative to that of the unperforated section.

For serviceability performance, it is necessary to limit deflections and vibration response. Suggested limits are given for various applications, but it may also be necessary to adopt stricter limits for specific cases, such as hospitals.

References

Chung, K. F. and Lawson, R. M. (2001), "Simplified design of composite beams with large web openings to Eurocode 4", *J. Constructional Steel Research*. **57**, 135-163.

Lawson, R. M. (1987). *Design for Openings in the Webs of Composite Beams*. The Steel Construction Institute Publication 56.

New Steel Construction (December 2003). Use of Intumescent Coatings for Fire Protection of Beams with Circular Openings.

Wyatt, T. A. (1989), *Design Guide on the Vibration of Floors*. The Steel Construction Institute Publication 76.

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