DESIGN OF COMPOSITE BEAMS WITH LARGE WEB OPENINGS

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DESIGN OF COMPOSITE BEAMS WITH LARGE WEB OPENINGS

In accordance with Eurocodes and the UK National Annexes

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FOREWORD

The use of composite beams with large rectangular or circular openings is a practical solution when it is required to pass service ducts through the structural zone of the beams. However, the presence of large openings in the web raises additional design considerations. Design guidance was given in an earlier SCI publication, *Design for openings in webs of composite beams* (P068) and design issues were further explored within a European Coal and Steel Community project, *Large web openings for service integration in composite floors* (ECSC project reference 7210-PR-315) and a Research Fund for Coal and Steel valorisation project (RFCS Project RFS-C2-05037).

The present publication has been written by Prof. R M Lawson of the SCI and Professor of Construction Systems at the University of Surrey, and Dr S J Hicks of HERA, New Zealand (formerly of SCI), with additional contributions from Dr W I Simms and Dr S Bake of SCI. It presents a generalised design method in accordance with the Eurocodes that may be applied to beams with discrete and closely-spaced openings.

This publication supersedes the design guidance given in publication P068.

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CONTENTS

SUMMARY

This publication provides guidance on determining the design resistance of composite beams with large web openings. The composite beams comprise steel I sections with either regular or isolated openings and with a concrete slab on the top flange, connected to the steel section by shear studs. The scope covers rolled steel sections with openings cut in the web, fabricated sections manufactured from rolled sections cut to a profile along the web and re-welded to form a regular pattern of web openings, and plate girders fabricated from three steel plates where the holes are cut in the web plate before welding the beam.

The behaviour of beams with large web openings is described and a design model is presented. Expressions are given for the design resistances of the various elements of the beam – the Tee sections above and below openings, the web posts between openings and the shear connection on the top of the beam, all generally following the principles and terminology of Eurocodes 3 and 4. It is noted that the resistances of non-composite sections can be determined using the same expressions if the terms for the contribution of the concrete are ignored.

A worked example for a secondary beam with pairs of rectangular and circular openings is presented, illustrating the use of the guidance.

NOTATION

The notation follows that of the Eurocodes, where possible.

A_{bT}	cross-sectional area of bottom Tee
A_{tT}	cross-sectional area of top Tee
A_c	cross-sectional area of concrete
A_f	cross-sectional area of flange
A_r	cross-sectional area of horizontal stiffener(s)
A_{sl}	cross-sectional area of tensile reinforcement
A_v	shear area of beam
$A_{v,bT}$	shear area of bottom Tee
$A_{v,tT}$	shear area of top Tee
$A_{w,b}$	cross-sectional area of web of the bottom Tee
$A_{w,T}$	cross-sectional area of web of a Tee
b	beam spacing
b_{eff}	effective slab width at any position along the span
$b_{eff,o}$	effective slab width at an opening
b_f	flange width
b_0	average width of decking rib (minimum width for re-entrant decking); distance between centres of outstand shear connectors in a group
b_r	outstand width of stiffener
b_w	effective width of slab for vertical shear resistance
d	diameter of shank of a stud shear connector; effective depth of concrete resisting vertical shear
e_o	eccentricity of centre of opening above the centreline of the web
e_r	offset distance of centre of stiffener from horizontal edge of opening
E	modulus of elasticity of steel
E_{cm}	secant modulus of elasticity of concrete
f_{cd}	design value of concrete compressive strength
f_{ck}	characteristic compressive cylinder strength of concrete
$f_{ck,cube}$	characteristic compressive cube strength of concrete
f_u	ultimate tensile strength of steel
f_y	yield strength of steel
f_{yr}	yield strength of steel in stiffeners
$F_{r,d}$	design axial force in a stiffener (for verifying end anchorage)
$F_{r,Rd}$	design axial resistance of a stiffener
F_{ten}	tensile force acting on shear connector at the end of an opening when the Vierendeel bending resistance of a composite Tee is fully mobilized
G	permanent action
h	depth of steel beam
h_b	depth of bottom Tee
h_t	depth of top Tee
h_c	depth of concrete above decking profile ($= h_s - h_d$)
h_d	overall depth of decking profile
h_{eff}	effective depth of steel section between centroids of the Tees
h_o	depth of opening
h_p	depth of decking profile (measured to its shoulder)
h_s	total depth of slab
$h_{s,eff}$	effective depth of slab for punching shear
h_{sc}	nominal height of shear connector (typically either 100 mm or 125 mm)

h_w clear web depth of beam between flanges

h_{wb} depth of web of bottom Tee

h_{wt} depth of web of top Tee

$h_{w,T}$ depth of web of Tee (h_{wt} or h_{wb}, as appropriate)

I_y second moment of area about y-y (major) axis

$I_{y,o}$ second moment of area of composite section at an opening about y-y (major) axis

k_o reduction factor on Vierendeel bending resistance of Tee due to the flexibility of a long opening

k_t reduction factor for resistance of a headed stud used with profiled sheeting transverse to the beam

$k_{t,max}$ upper limit to the value of k_t

ℓ_e effective length of rectangular opening

ℓ_o (clear) length of opening

$\ell_{o,eff}$ effective length of opening used in assessing the section class of a Tee in Vierendeel bending

ℓ_v anchorage length of a horizontal stiffener to a rectangular opening (beyond the end of the opening)

ℓ_w buckling length of web-post

L length; span

L_e equivalent span

$M_{pl,a,Rd}$ plastic bending resistance of steel section

$M_{bT,NV,Rd}$ bending resistance of the bottom Tee, reduced for axial tension and shear

$M_{el,Rd}$ elastic bending resistance (of Tee)

M_{Ed} design bending moment

$M_{pl,N,Rd}$ plastic bending resistance (of Tee section) reduced for axial force

$M_{o,Ed}$ design bending moment at the centreline of an opening

$M_{o,Rd}$ bending resistance of the composite section at the centreline of an opening

$M_{pl,Rd}$ plastic bending resistance (of Tee section)

$M_{tT,NV,Rd}$ bending resistance of the top Tee, reduced for axial tension and shear

$M_{wp,Ed}$ design bending moment in web-post at mid-height of openings

$M_{wp,Rd}$ (elastic) bending resistance of a web-post

$N_{bT,Ed}$ tensile force acting in bottom Tee

$N_{a,Rd}$ tensile resistance of the steel beam

$N_{c,Ed}$ design value of the compression force in the concrete flange

$N_{c,f}$ design value of the compression force in the concrete flange corresponding to full shear connection

$N_{c,Rd}$ compression resistance of the concrete slab over its effective slab width (either limited by concrete compressive resistance or by shear connection provided)

$N_{c,s,Rd}$ is the concrete compressive resistance over the effective width of the slab

$N_{bT,Rd}$ axial resistance of bottom Tee

$N_{tT,Rd}$ axial resistance of top Tee

$N_{wp,Ed}$ effective compression force in web-post

$N_{wp,Rd}$ buckling resistance of web-post

$\Delta N_{b,Ed}$ increase in tension force in the bottom Tee between the centrelines of adjacent openings

$\Delta N_{co,Rd}$ increase in compression resistance of the slab, due to the shear connection over an opening

$\Delta N_{cs,Rd}$ increase in compression resistance of the slab, due to the shear connection between the centrelines of the openings

n modular ratio; number of shear connectors

n_f number of shear connectors required for full shear connection

n_r number of shear connectors per rib of transverse decking

n_{sc} number of shear connectors from the support to the centreline of an opening

$n_{sc,o}$ number of shear connectors placed directly over an opening

$n_{sc,s}$ number of shear connectors placed in the slab between centrelines of adjacent openings

P_{Ed} design shear force on a shear connector

P_{Rd}	design shear resistance of shear connector; design shear resistance of shear connector, reduced for transverse decking
q	variable action, load per unit length
Q	variable action (point load or total load)
r	root radius of rolled steel section
r_{o}	corner radius of opening
s	centre-to-centre spacing of adjacent openings
s_{e}	width of end-post
s_{o}	edge-to-edge spacing of adjacent openings
t_{f}	flange thickness
t_{r}	thickness of stiffener
t_{w}	web thickness
$t_{\mathrm{w,eff}}$	effective web thickness (of Tee) reduced due for shear
$v_{\mathrm{c,Rd}}$	shear strength of concrete slab at an opening
V_{b}	shear force in bottom Tee
$V_{\mathrm{b,Rd}}$	shear resistance of bottom Tee
$V_{\mathrm{c,Rd}}$	shear resistance of concrete slab at an opening
V_{Ed}	design shear force
V_{Rd}	vertical shear resistance of beam at an opening
$V_{\mathrm{t,Rd}}$	shear resistance of top Tee
$V_{\mathrm{wp,Rd}}$	longitudinal shear resistance of web-post between openings
$V_{\mathrm{wp,Ed}}$	longitudinal shear force in web-post between openings
x	distance of opening from nearer support
w	deflection of unperforated beam (subscript b for bending, sw for self weight)
w_{add}	additional deflection due to openings (subscript b for bending and v for shear)
z_{c}	depth of concrete in compression
z_{el}	depth of centroid of Tee from outer face of flange
z_{pl}	depth of plastic neutral axis of Tee from outer face of flange

Axes

x	longitudinal axis along the member
y-y	major axis (parallel to flanges)
z-z	minor axis (parallel to web)

Greek Symbols

α	reduction factor on shear connector resistance due to h_{sc}/d ratio
γ_{M0}	partial factor for resistance of steel cross sections
γ_{M1}	partial factor for resistance of steel members to instability
γ_{C}	partial factor for concrete
γ_{V}	partial factor for shear resistance of a headed stud
η	degree of shear connection
μ	utilization factor
σ_{cp}	compressive stress in concrete from axial load
$\bar{\lambda}$	non-dimensional slenderness of web for web-post buckling
λ_1	slenderness value to determine the relative slenderness
ε	$\sqrt{235/f_{\mathrm{y}}}$, where f_{y} is in N/mm^2
$\psi_{0,i}$	factor for a combination value of i-th variable action
ξ	reduction factor for unfavourable permanent actions

INTRODUCTION

1.1 Background

Composite construction has become the commonly preferred method of construction for multi-storey buildings of all types. In composite construction, steel I-sections or H-sections are typically attached to a composite slab by shear connectors. Composite action increases the bending resistance by 50% to 100% relative to that of the steel section alone. Over the last 20 years, many long span composite systems have been developed, and most have been configured in ways that provide for integration of services within the structural depth of the floor.

A common method of incorporating services within the structural depth is by means of circular or rectangular openings cut in the webs of I-sections or H-section beams. There are two main configurations of openings that are used:

- Isolated large rectangular or circular openings at positions where the interaction between the openings is minimised (see Figure 1.1).
- Regular circular openings (sometimes with discrete elongated openings) – these are known as 'cellular beams' (see Figure 1.2).

Figure 1.1 Isolated rectangular openings in a composite beam

Figure 1.2
Cellular beam with regular circular openings formed from two hot-rolled sections

Figure 1.3
Beams fabricated from three plates, with various shapes of openings

There are three methods of manufacture of a beam with web openings:

- Individual openings are cut in the web of a hot-rolled section. The steel section is symmetric in shape. This method is used for beams with isolated openings.
- A fabricated section is formed from three plates that are welded together to form an I-section (see Figure 1.3). The section can be asymmetric (for example, with a larger bottom flange) and the beams can be tapered in depth along their length (see Figure 1.4). Openings are cut in the web either before or after forming the I-section. This method is used both for isolated openings and for regularly spaced openings.
- A hot-rolled section is cut to a profile along the web, the resulting Tee sections are repositioned and re-welded to form a series of regular openings in a deeper section. Historically, the castellated beam was the first of this type, but modern construction uses beams with circular openings (see Figure 1.2). The section can be asymmetric, which is achieved when the Tees are cut from different I-sections. This method is only appropriate for regularly spaced openings.

Large openings cause a significant reduction in the shear resistance of beams, due to the loss of a major proportion of the web, but cause a smaller reduction in bending

Figure 1.4
Large stiffened openings used in tapered beams
(Commerzbank, Frankfurt)

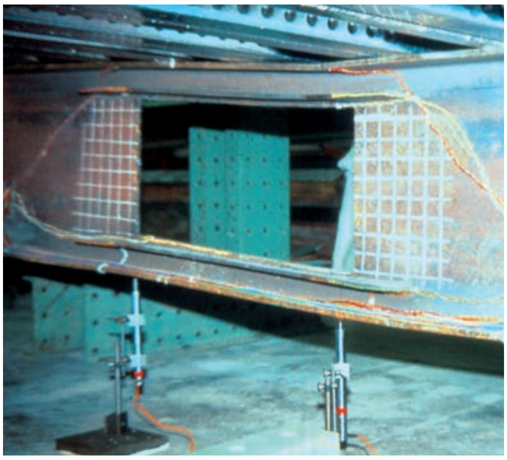


Figure 1.5
Test on a composite beam, showing Vierendeel bending at a large stiffened opening

resistance. The shear transfer across sections with large openings is an important design consideration, and it is good practice to locate large openings remote from the high shear zones of a beam in order to minimise their effect.

Shear transfer across large openings, occurs by Vierendeel or 'four corner' bending in the web flange 'Tee' sections above and below the opening; this leads to a complex interaction of forces at the corners of the opening. The mode of failure by Vierendeel bending is illustrated in Figure 1.5. Horizontal stiffener plates (reinforcement) improve the transfer of shear by increasing the bending resistance of the Tees locally.

Fabricated beams often have slender webs that may require vertical stiffening to prevent buckling of the web close to the opening (see Figure 1.4). However, the need for such stiffening may be minimised by choosing a thicker web and by careful positioning of the openings (i.e. away from point load positions), resulting in more economic design.

1.2 Previous design guidance

The SCI publication *Design of openings in the webs of composite beams* (P068) [1], published in 1988, presented a design method for hot-rolled steel beams with discrete openings in composite construction. The methodology was calibrated against tests on 10 m span composite beams with rectangular openings carried out at the University of Warwick [2].

Modern design and construction of composite beams with web openings differs in important respects from that covered within the SCI publication P068, and from the configurations tested at Warwick:

- The steel sections are often highly asymmetric in terms of the ratio of the bottom to the top flange areas.
- The webs are often relatively slender, so that the effect of local buckling is increased.
- Openings are often relatively long in terms of their aspect ratio (length : depth), as illustrated in Figure 1.6.
- Openings are often asymmetric within the depth of the section and are sometimes close to one of the flanges.
- Elongated openings are often formed by removing the 'web-post' between adjacent circular openings.

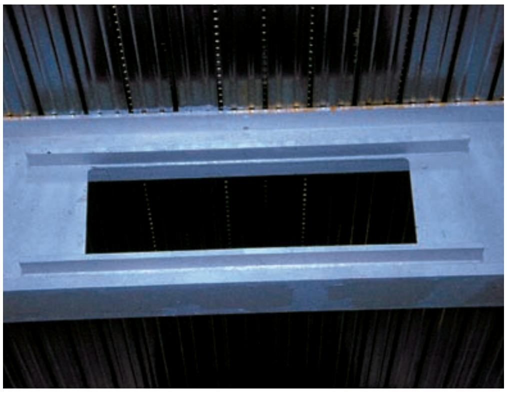


Figure 1.6
Long stiffened opening in a composite beam

These changes in practice have necessitated a re-appraisal of P068, so that guidance can be extended to cover a wider range of applications for both hot-rolled and fabricated steel sections. However, it is not possible to give definitive guidance for all cases and general structural design principles should still be observed. This is particularly true for fabricated sections.

Furthermore, the introduction of Eurocodes requires that design methods are consistent with the principles and application rules of Eurocode 3 and Eurocode 4, for steel and composite construction respectively. A draft amendment to Annex N of the pre-standard ENV 1993-1-1: 1992/A2: 1998 [3] covering the design of beams with holes in the web was started but the committee draft was not published, nor incorporated into the published Eurocode. Other international guidance on beams with large web openings exists in the USA [4], but this does not apply to cellular beams or to beams with closely spaced openings.

General guidance on structure-services integration may be found in *Design of steel framed buildings for service integration* (P166) [5]. It presents layouts of ducts and terminal units for Fan-Coil and VAV air-conditioning systems in various forms of long span steel construction.

1.3 Scope of this publication

This publication extends the guidance in SCI P068 (which is now withdrawn) for both hot-rolled and fabricated sections and follows the principles and relevant application rules of BS EN 1993-1-1 [6] and BS EN 1994-1-1 [7]. The scope of the present publication covers the design of simply-supported composite beams for the following cases:

- Beams fabricated from hot-rolled sections and from plates.
- Symmetric and asymmetric steel sections (in which the ratio of the bottom to top flange areas is less than 3 to 1).
- Steel sections with Class 1, 2 or 3 flanges and Class 1, 2, 3 or 4 webs.
- Openings placed centrally and non-centrally in the depth of the section.
- Rectangular openings, circular openings and elongated circular openings.
- Beams with widely-spaced openings and with closely spaced openings.
- Cellular beams with uniform web thickness.
- Notched beams.

The publication does not directly cover:

- Web-post buckling effects in cellular beams with different web thicknesses in the top and bottom of the web.
- Continuous beams.
- Tapered beams.
- Curved beams.
- Slim floor beams designed with composite action.
- Fire engineering design.

The guidance may be applied to non-composite beams by ignoring the structural effect of the concrete slab. (This application also covers the construction stage for composite beams, before the concrete has hardened.) Where additional considerations apply to non-composite beams, this is noted.

References to Eurocodes in this publication are to the versions published by BSI and include reference to the UK National Annexes, where appropriate.

1.4 Design standards

This publication is prepared in general structural engineering terms and refers to rules in BS EN 1993-1-1 and BS EN 1994-1-1, which supersede BS 5950-1 [8] and BS 5950-3 [9] respectively. As this publication deals with a common design case that is not covered by these Eurocodes, the publication may be considered to provide non-contradictory complimentary information (NCCI).

1.4.1 Eurocodes

The basis of structural design in the Eurocodes is presented in the 'core' document, BS EN 1990 [11], which also defines the combinations of actions (loads). For the design of beams in braced frames at the ultimate limit state, the most commonly applicable combination of actions for buildings in the UK* is given by expression 6.10b:

$$\sum_{j\geq 1} \xi_j \gamma_{G,j} G_{k,j} + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i}$$

where:

$G_{k,j}$	is the characteristic value of the j th permanent action
$Q_{k,1}$	is the characteristic value of the leading variable action
$Q_{k,i}$	is the characteristic value of the i th variable action
$\gamma_{G,j}$	is the partial factor for the j th permanent action
$\gamma_{Q,1}$	is the partial factor for the leading variable action
$\gamma_{Q,i}$	is the partial factor for the i th variable action
$\psi_{0,i}$	is the combination factor for other actions
ξ	is the reduction factor for unfavourable permanent actions.

At the serviceability limit state, the characteristic combination of actions should be used when considering irreversible deformations. The frequent combination applies to cases which occur regularly and the quasi-permanent condition applies to issues such as creep or shrinkage deflections. The combinations of actions for these conditions are defined by the following expressions:

Note: * The expression which gives the most onerous effects depends on the values of the nationally determined parameters. See P361 [10] for a discussion of combinations of actions according to BS EN 1990.

Characteristic: $\sum_{j\geq1} G_{k,j} + Q_{k,1} + \sum_{i>1} \psi_{0,i} Q_{k,i}$

Frequent: $\sum_{j\geq1} G_{k,j} + \psi_{1,1}\, Q_{k,1} + \sum_{i>1} \psi_{2,i}\, Q_{k,i}$

Quasi permanent: $\sum_{j\geq1} G_{k,j} + \sum_{i>1} \psi_{2,i}\, Q_{k,i}$

The characteristic combination is generally used in checking deflections, etc. The value of ψ_0 is taken as for the ultimate limit state (see below) and ψ_1 and ψ_2 depend on the type of loading.

Actions are defined in Eurocode 1, in which Part 1-1 [12] gives values for self weight and imposed loads for buildings. It should be noted that during construction, the concrete self weight and other construction loads are treated as variable actions; these are covered in Part 1-6 [13].

BS EN 1993-1-1 gives general rules for steel structures in buildings and links to other parts of BS EN 1993. BS EN 1994-1-1 gives general rules for composite structures in buildings and links to BS EN 1993-1-1 and BS EN 1992-1-1 [14].

The fire design requirements for steel and composite structures are presented in BS EN 1993-1-2 [15] and BS EN 1994-1-2 [16].

1.4.2 National Annexes

National Annexes are used to implement the Eurocode Parts in each country and give decisions on nationally determined parameters, which include partial factors, as well as other issues and some material and application limits. Where this publication adopts the provisions in the UK National Annexes, this is noted.

Partial factors on actions

The partial factors on actions are given in the National Annex to BS EN 1990 [11]. The partial factor for permanent actions $\gamma_{G,j} = 1.35$. The reduction factor for unfavourable permanent actions $\xi = 0.925$ and the combination factor for accompanying actions is typically $\psi_{0,1} = 0.7$ for most building categories. The overall factor for permanent actions in expression 6.10b is therefore, $\xi \times \gamma_G = 0.925 \times 1.35 = 1.25$.

The partial factor for variable actions $\gamma_{Q,1}$ is given as 1.50, where unfavourable.

The overall factors applied to characteristic values of actions to give design values are summarised in Table 1.1.

Table 1.1 Overall factors applied to actions on buildings in the UK, in expression 6.10b

ACTION	LIMIT STATE		
	ULTIMATE	SERVICEABILITY	FIRE
Self-weight (dead load)	1.25	1.0	1.0
Imposed load (leading variable action)	1.5	1.0	0.5

DESIGN PARAMETER		LIMIT STATE		
		ULTIMATE	SERVICEABILITY	FIRE
Resistance of steel cross sections	γ_{M0}	1.0	1.0	1.0
Resistance of steel members to instability	γ_{M1}	1.0	1.0	1.0
Resistance of concrete	γ_{C}	1.5	1.0	1.0
Shear resistance of a headed stud	γ_{V}	1.25	1.0	1.0
Shear resistance of a composite slab	γ_{VS}	1.25	1.0	1.0
Reinforcement	γ_{S}	1.15	1.0	1.0

Note: The values shown in this table are taken from the UK National Annexes to BS EN 1992-1-1, BS EN 1993-1-1 and BS EN 1994-1-1.

Table 1.2 Partial factors for materials

Partial factors on material strengths

The partial factors on material strength that are used for design of composite beams and slabs are summarised in Table 1.2.

The characteristic material strengths are divided by the relevant partial factors to obtain the design strengths.

In the UK National Annex to BS EN 1993-1-1, the partial factor for resistance of steel cross sections, γ_{M0}, is given as 1.0. The design strength is therefore numerically equal to the characteristic strength.

For serviceability performance and fire engineering design, other partial factors for materials are used, which reflect the required level of reliability at these limit states.

1.4.3 Cross-referencing and symbols

To aid cross-referencing to the Eurocodes, the relevant clauses from the source document are indicated in the text of this publication. Eurocode terminology conventions for symbols are adopted as closely as possible.

Of particular note is the use of the subscript Ed, which refers to design values of effects of actions (internal forces and moments) and the subscript Rd, which refers to design values of resistance. The distinction between symbols for effects and resistances is thus clearly seen.

A more detailed clause by clause review of BS EN 1994-1-1 Eurocode 4 is given in the publication by Johnson and Anderson [17], which should be referred to for background information.

1.5 Material strengths

1.5.1 Steel strength

The strength of steel is usually expressed in terms of its characteristic yield strength f_y in N/mm^2. Various grades of steel can be used in composite beam design. Grade S275 and S355 are commonly specified in the UK, the higher grade often being preferred for composite construction, except where serviceability limits control design.

The UK National Annex to BS EN 1993-1-1 states that the nominal strengths of steel should be taken as the specified minimum values given by the product standard, typically BS EN 10025 [18] for hot-rolled sections. The yield and ultimate tensile strengths of steel grades S235 to S355, according to BS EN 10025, are presented in Table 1.3. Higher steel strengths are not commonly available in the UK, and the rules in BS EN 1994-1-1 do not cover steel with nominal yield strength greater than 460 N/mm^2.

BS EN 1993-1-1, §3.1 states that the nominal values for the strength of steel should be adopted as characteristic values in calculations.

Table 1.3 Yield strength f_y and ultimate tensile strength f_u of steel to BS EN 10025-2

STEEL GRADE	NOMINAL THICKNESS OF ELEMENT, t (mm)			
	$t \leq 16$ mm		16 mm $< t \leq 40$ mm	
	f_y (N/mm^2)	f_u (N/mm^2)	f_y (N/mm^2)	f_u (N/mm^2)
S235	235	360	225	360
S275	275	430	265	410
S355	355	510	345	470

1.5.2 Steel decking

Profiled steel decking is cold formed from steel strip supplied in accordance with BS EN 10346 [19], typically of grade S280 GD + Z275. (Note that EN 10346 replaces the reference standard EN 10147 in BS EN 1994-1-1.) The characteristic yield strength and ultimate tensile strength of this grade should be taken as 280 N/mm^2 and 360 N/mm^2 respectively, according to BS EN 1993-1-3 [20]. The weight of zinc coating for this grade is 275 g/m^2.

The minimum bare metal thickness of sheeting for use in composite slabs is 0.7 mm, according to the UK National Annex to BS EN 1994-1-1, although in practice in the UK, nominal steel thicknesses (bare metal thickness plus coating) of 0.9 to 1.2 mm are generally used.

1.5.3 Concrete strength

In BS EN 1992-1-1 and BS EN 1994-1-1, the compressive strength of concrete is given by its strength class, which relates to the characteristic cylinder strength and its cube strength, e.g. C30/37, which denotes concrete with a cylinder strength equal to 30 N/mm^2 and a cube strength equal to 37 N/mm^2.

In BS EN 1992-1-1, resistances are expressed in relation to the characteristic cylinder strength f_{ck}. The approximate conversion between cylinder and cube strength for normal weight concrete is: $f_{ck} \approx 0.8 f_{ck,cube}$.

BS EN 1992-1-1, Table 3.1 sets out a range of properties for normal weight concrete and the values for characteristic cylinder strength, cube strength and elastic modulus for selected classes are reproduced in Table 1.4. The concrete grades that are covered by BS EN 1994-1-1 are C20/25 to C60/75 for normal weight concrete.

Table 1.4 Concrete properties (taken from Table 3.1 of BS EN 1992-1-1)

PROPERTIES OF CONCRETE	STRENGTH CLASS OF CONCRETE				
	C20/25	C25/30	C30/37	C35/45	C40/50
f_{lck} (MPa)	20	25	30	35	40
$f_{lck,cube}$ (MPa)	25	30	37	45	50
E_{lcm} (GPa)	30	31	33	34	35

Note: 1 MPa = 1 N/mm^2, 1 GPa = 1 kN/mm^2

Values for lightweight aggregate concrete are given in BS EN 1992-1-1, Table 11.3.1 and selected values are reproduced in Table 1.5. (Note that the characteristic strength f_{lck} is used in place of f_{ck} in expressions for resistance in Eurocode 2 and 4.) The modulus of elasticity of lightweight concrete is taken as $(\rho/2200)^2$ multiplied by the elastic modulus for normal weight concrete, where ρ is the dry density expressed in kg/m^3 – values in Table 1.5 are given for a density of 1850 kg/m^3. The lightweight aggregate concrete grades that are covered by BS EN 1994-1-1 are LC20/22 to LC60/66.

Table 1.5 Lightweight aggregate concrete properties (taken from Table 11.3.1 of BS EN 1992-1-1)

PROPERTIES OF CONCRETE	STRENGTH CLASS OF CONCRETE				
	LC20/22	LC25/28	LC30/33	LC35/38	LC40/44
f_{lck} (MPa)	20	25	30	35	40
$f_{lck,cube}$ (MPa)	22	28	33	38	44
E_{lcm} (GPa)	21	22	23	24	25

Note: Values of E_{lcm} are for a dry density of 1850 kg/m^3

In BS EN 1994-1-1, §4.2.1.2, the design compressive strength of concrete is defined as $f_{cd} = f_{ck}/\gamma_c$. However, BS EN 1994-1-1 recommends that for composite beams with a plastic distribution of stress, the compressive stress is limited to 0.85 f_{cd}.

Although not stated, the 0.85 factor takes into account the difference between the parabolic stress-strain curve for concrete and the idealised plastic stress blocks used in the design of a composite beam. For elastic design, the concrete stress is limited to f_{cd}.

To permit the design equations in BS EN 1994-1-1 to be used for headed stud shear connectors, lightweight aggregate concrete should have a dry density of not less than 1750 kg/m^3.

1.6 Shear connection

1.6.1 Stud shear connectors

According to BS EN 1994-1-1, §6.6.3, the design resistance of headed shear connectors (studs) embedded in solid concrete should be determined from the smaller of:

$$P_{Rd} = \frac{0.8 f_u \pi d^2 / 4}{\gamma_V}$$

and

$$P_{Rd} = \frac{0.29 \alpha d^2 \sqrt{f_{ck} E_{cm}}}{\gamma_V}$$

with

$$\alpha = 0.2\left(\frac{h_{sc}}{d} + 1\right) \quad \text{for } 3 \le h_{sc} / d \le 4$$

$$\alpha = 1 \quad \text{for } h_{sc} / d > 4$$

where:

d is the diameter of the shank of the stud ($16 \text{ mm} \le d \le 25 \text{ mm}$)

h_{sc} is the nominal height of the stud

f_u is the specified ultimate tensile strength of the stud material

f_{ck} is the characteristic cylinder strength of the concrete (of density not less than 1750 kg/m^3)

E_{cm} is the secant modulus of elasticity of the concrete (see Table 1.4).

Studs are normally specified as type SD1 in accordance with EN ISO 13918, for which $f_u = 450$ N/mm^2.

The design resistances of standard 19 mm diameter × 100 mm long (length-as-welded is typically 95 mm) shear connectors embedded in solid concrete slabs are presented in Table 1.6 and Table 1.7 (the dry density of lightweight concrete used in the UK is typically 1850 kg/m^3).

Table 1.6
Design resistances of shear connectors embedded in normal weight concrete solid slabs

DESIGN RESISTANCES OF SHEAR CONNECTORS (kN) FOR CONCRETE GRADE				
C20/25	C25/30	C30/37	C35/45	> C40/50
65	74	81	81	81

Note: Values based on the UK National Annex value for γ_V and with $f_u = 450$ N/mm^2

Table 1.7
Design resistances of shear connectors embedded in lightweight concrete (with dry density $\rho = 1850$ kg/m^3)

DESIGN RESISTANCES OF SHEAR CONNECTORS (kN) FOR CONCRETE GRADE				
LC20/22	LC25/28	LC30/33	LC35/38	LC40/45
55	62	70	77	81

Note: Values based on the UK National Annex value for γ_V and with $f_u = 450$ N/mm^2

When composite decking is used, these values may be reduced according to an empirical formula taking account of the shape of the decking profile (see Section 1.6.2).

1.6.2 Influence of decking shape on shear connection

The efficiency of the shear connection between the composite slab and the composite beam may be reduced as a result of the shape and orientation of the decking profile and the number of shear connectors placed in each rib. The resistance of the shear connectors is highly dependent on the area of concrete around them and the embedment of the head of the shear connector into the slab topping. A simple reduction factor formula is used to take account of the decking profile shape and the number of shear connectors in a group.

Strength reduction factor in decking with ribs transverse to the beam

According to BS EN 1994-1-1, §6.6.4.2, the reduction factor k_t for the resistance of shear connectors (relative to resistance in a solid slab) is given by the empirical formula:

$$k_t = \frac{0.7}{\sqrt{n_r}} \frac{b_0}{h_p} \left(\frac{h_{sc}}{h_p} - 1 \right) \quad \text{but not more than } k_{t,max}$$

where:

b_0 is the average rib width (or minimum width for re-entrant profiles)

h_{sc} is the nominal height of the stud

h_p is the profile height (to the shoulder of the profile)

n_r is the number of studs per rib ($n_r = 1$ or 2)

$k_{t,max}$ is the upper limit on k_t (see Table 1.8).

According to BS EN 1994-1-1, §6.6.5.8(1), the nominal height of a shear connector should project at least 2 × stud diameter above the top of the decking (i.e. 38 mm for 19 mm diameter shear connectors).

The coefficient of 0.7 in the above expression has been established according to the requirements given in BS EN 1990 Annex D [11] on the basis of recent test evidence on the performance of shear connectors.

It is recognised that the formula is conservative for single shear connectors per decking rib but may be unconservative for shear connectors in pairs. Therefore, upper limits $k_{t,max}$ are given, as shown in Table 1.8. The upper limit is also presented as a function of the thickness of the steel sheet and whether the shear connectors are through-deck welded, as it is recognised that the decking plays a beneficial role in transferring shear into the slab. The limits to the shear connector diameters reflect the experimental data available in calibrating the expression.

Recent tests in the UK have explored the shear resistance of studs in decking that is transverse to the beam, with particular reference to the level of the mesh

Table 1.8
Upper limits $k_{t,max}$ on reduction factor k_t (taken from Table 6.2 of BS EN 1994-1-1)

NUMBER OF STUD CONNECTORS PER RIB	THICKNESS, t OF SHEET (mm)	STUDS NOT EXCEEDING 20 mm IN DIAMETER AND WELDED THROUGH PROFILED SHEETING	PROFILED SHEETING WITH HOLES AND STUDS 19 mm OR 22 mm IN DIAMETER
$n_r = 1$	≤ 1.0	0.85	0.75
	> 1.0	1.0	0.75
$n_r = 2$	≤ 1.0	0.70	0.60
	> 1.0	0.80	0.60

reinforcement. It was concluded that in some circumstances a reduction factor should be applied to the resistance calculated in the above manner. For further guidance, see document PN001 on: www.steel-ncci.co.uk.

Strength reduction factor in off-centre placing of shear connectors

Many modern decking profiles have a central stiffening fold in the rib that requires the shear connector to be located off-centre in the rib. The preferred position of attachment is where the shear connectors are placed on the side of the rib closest to the nearest support.

The rules in BS EN 1994-1-1 are only given for centrally welded shear connectors. In cases where the studs cannot be placed centrally, it is advised that shear connectors are welded in a ‘favourable location’ (where the zone of concrete in compression in front of the stud is larger than that behind the stud) or one either side of the central position.

Strength reduction factor in decking parallel to beam

For the case where the decking is orientated parallel to the beam, the reduction factor, relative to resistance in a solid slab, is given by BS EN 1994-1-1, §6.6.4.1 as:

$$k_\ell \quad = \quad 0.6 \frac{b_0}{h_p}\left(\frac{h_{sc}}{h_p} - 1\right) \le 1.0$$

This factor applies to shear connectors singly or in pairs.

1.7 Effective slab width

The effective slab width b_{eff} taken to act compositely with the steel beam is defined in BS EN 1994-1-1, §5.4.1.2. It allows for the effects of shear lag in the concrete flange and is a function of the equivalent span L_e, depending on whether the beam is simply supported or continuous. The same effective width at mid-span is used at both the ultimate and serviceability limit states.

At mid-span, the effective width is given by:

$$b_{eff} \quad = \quad b_0 + \Sigma b_{ei}$$

where:

b_{ei} is the effective slab width on either side of the beam ($= L_e/8$ but not more than the geometric width)

b_0 is the distance between centres of outstand shear connectors in a group

L_e is the effective span which, for a simply supported beam, is its span L.

Therefore, for a simply supported beam, and ignoring b_0, the effective slab width at mid-span $b_{eff} = L/4$ (but not more than the beam spacing).

Clause 5.4.1.2(6) states that the effective width of the slab at an end support may be determined as:

$$b_{eff} = b_0 + \Sigma \beta_i b_{ei}$$

where:

$$\beta = 0.55 + 0.025 L_e / b_{ei}$$

For a simply supported beam, it follows that the effective slab width at the end of the span is:

$$b_{eff} = b_0 + 0.187L$$

It follows that the effective slab width at the end of span is approximately 75% of that at mid-span. For the effective width at web openings, see Section 3.2.3.

1.8 Partial shear connection

The design of composite beams is often controlled by the degree of shear connection that is provided. In cases where fewer shear connectors than the number required for full shear connection are provided, it is not possible to develop the full plastic moment resistance of the composite section. This is known as partial shear connection. The degree of shear connection is defined as at the point of maximum moment, but partial shear connection exists at all points in the span, depending on the build-up of longitudinal shear.

The degree of shear connection is defined in BS EN 1994-1-1, §6.6.1.2 as:

$$\eta = \frac{n}{n_f}$$

where:

n_f is the number of shear connectors required for full shear connection

n is the number of shear connectors provided between the points of zero and maximum moment.

For the case when the tensile resistance of the steel beam exceeds the compressive resistance of the concrete slab ($N_{a,Rd} > N_{c,Rd}$), this can be re-expressed as:

$$\eta = \frac{N_{c,max}}{N_{c,s,Rd}} \quad (1)$$

where:

$N_{c,max}$ is the total shear force transferred by the shear connectors between the points of zero and maximum moment ($= nP_{Rd}$)

$N_{a,Rd}$ is the tensile resistance of the steel section

$N_{c,s,Rd}$ is the concrete compressive resistance over the effective width of the slab $N_{c,s,Rd} = 0.85\ f_{cd} b_{eff} h_c$ where f_{cd} is as defined in BS EN 1994-1-1, see Section 1.5.3, b_{eff} is the effective width at the position of maximum moment and h_c is the depth of concrete above the profile

P_{Rd} is the design resistance of the shear connectors used with profiled sheeting (i.e. P_{Rd} as given in Section 1.6 multiplied by k_t or k_ℓ as given in Section 1.6.2).

For the case when the tensile resistance of the steel beam is less than the compressive resistance of the concrete slab ($N_{a,Rd} < N_{c,s,Rd}$), the maximum force that could be developed in the slab = $N_{a,Rd}$ and the degree of shear connection can be re-expressed as:

$$\eta = \frac{N_{c,max}}{N_{a,Rd}} \quad (2)$$

1.8.1 Linear interaction method

There are two methods of determining the bending resistance of a composite section with partial shear connection. The simplest method is the so-called 'linear-interaction' approach given in BS EN 1994-1-1, §6.2.1.3. The reduced bending resistance given by BS EN 1994-1-1, §6.2.1.3(5) may be expressed as:

$$M_{Rd} = M_{pl,a,Rd} + \eta\,(M_{pl,Rd} - M_{pl,a,Rd}) \quad (3)$$

where:

η is as defined in (1) or (2) above

$M_{pl,Rd}$ is the bending resistance of the composite section with full shear connection

$M_{pl,a,Rd}$ is the bending resistance of the steel section.

For adequate design, $M_{Ed} \leq M_{Rd}$, where M_{Ed} is the design bending moment applied to the beam. The verification may be repeated at opening positions by redefining the shear force transferred as $N_c = n_{sc} P_{Rd}$, where n_{sc} is the number of shear connectors from the support to the opening position in the span.

The linear interaction method is conservative with respect to rigid plastic theory (sometimes referred to as the 'stress block' method).

1.8.2 Minimum degree of shear connection

In using the above methods, a minimum degree of shear connection is specified in BS EN 1994-1-1, which is based on research by Johnson and Molenstra [21] together with Aribert [22]. The minimum limit is introduced in order to ensure adequate deformation capacity of the shear connectors, as defined by a characteristic slip capacity of at least 6 mm. In principle, the use of rigid-plastic theory imposes greater deformations on the shear connectors at failure than the linear interaction method. Therefore, these limits are more conservative when using the linear interaction method. (Limits for shear connectors with greater values of slip capacity have been determined from tests in the UK, and are given in document PN002 at: www.steel-ncci.co.uk.)

For symmetric steel sections, the general limit on the degree of shear connection is defined in BS EN 1994-1-1, §6.6.1.2 as:

$$L_e \leq 25: \quad \eta \geq 1 - \left(\frac{355}{f_y}\right)(0.75 - 0.03 L_e), \qquad \eta \geq 0.4$$

$$L_e > 25: \quad \eta \geq 1.0$$

where:

L_e is the distance between points of zero bending moment (beam span for simply supported beams), in metres.

The influence of the steel strength, f_y is introduced because of the higher strains, and hence deformation demands, in plastic design using higher strength steels.

These limits may also be used when the section is asymmetrical with top flange area which exceeds that of the bottom flange.

For asymmetric steel sections in which the bottom flange area equals three times the top flange area, the minimum degree of shear connection is defined by:

$$L_e \leq 20: \quad \eta \geq 1 - \left(\frac{355}{f_y}\right)(0.30 - 0.015 L_e), \qquad \eta \geq 0.4$$

$$L_e > 20: \quad \eta \geq 1.0$$

For steel sections in which the ratio of flange areas is between the limits of 1 and 3, linear interpolation between the above shear connection limits is permitted.

A relaxation of the degree of shear connection is permitted when all the following conditions are met:

- the studs have an overall length after welding not less than 76 mm and a nominal shank diameter of 19 mm;
- the steel section is rolled or welded I-sections or H-sections with equal flanges;
- the concrete slab is composite with profiled steel decking that spans perpendicular to the beam and the concrete ribs are continuous across the beam;
- there is one stud per rib of the decking;

- the decking rib is of proportions $b_0/h_p \geq 2$ and $h_p \leq 60$ mm;
- the linear interaction method, described in Section 1.8.1, is used.

In this case (for single shear connectors), the following limits on the degree of shear connection apply:

$$L_e \leq 25: \quad \eta \geq 1 - \left(\frac{355}{f_y}\right)(1 - 0.04\,L), \qquad \eta \geq 0.4$$

$$L_e > 25: \quad \eta \geq 1.0$$

The various shear connection limits are presented in Figure 1.7, as a function of design span.

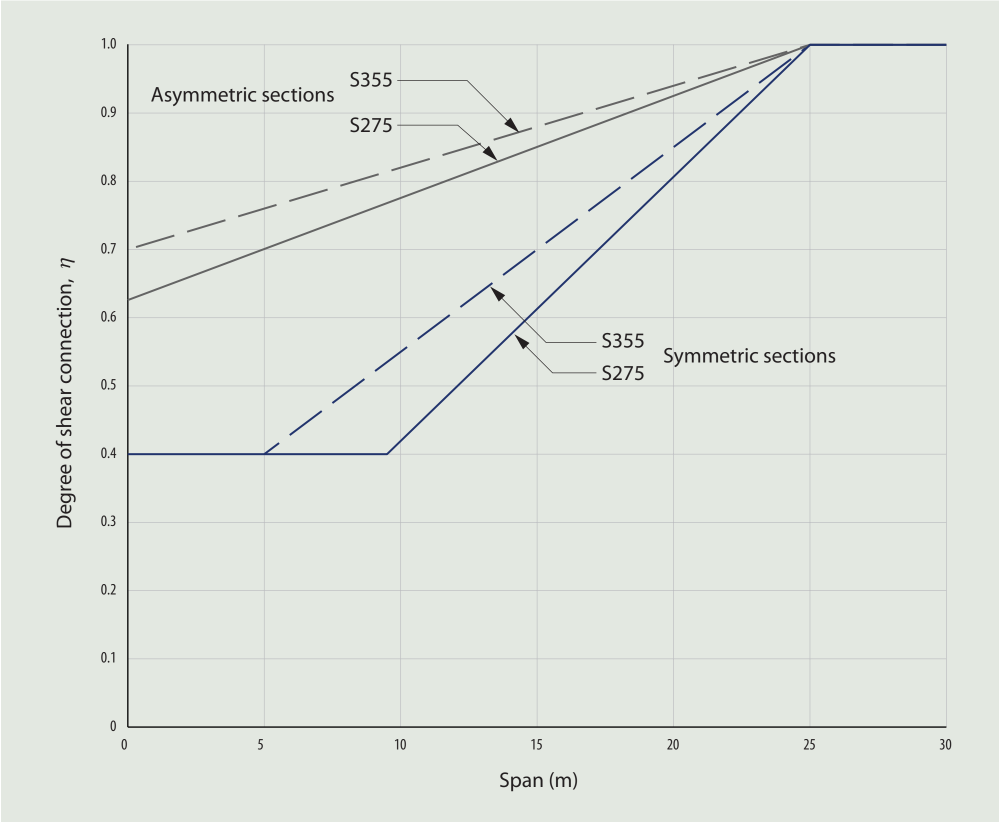


Figure 1.7 Variation of minimum degree of shear connection with the span of a composite beam

1.8.3 Spacing of shear connectors

BS EN 1994-1-1 gives the following limits on the spacing of the shear connectors, which may influence the maximum or minimum degree of shear connection that may be achieved in practice.

Clause 6.6.5.7 states that the spacing of the shear connectors should be not less than $5d$ longitudinally and $4d$ laterally (d is the diameter of the shear connector).

Clause 6.6.5.5(2) states that (for profiled sheeting transverse to the beam), where the longitudinal spacing of the shear connectors does not exceed $15t_f\varepsilon$, the compression flange of the section may be treated as Class 1 or 2, regardless of its actual class.

Clause 6.6.5.5(3) states that the maximum longitudinal spacing of the shear connectors should not be greater than the lesser of 6 times the slab thickness and 800 mm.

Clause 6.6.5.6(2) states that the edge distance (defined as from the edge of the shear connector to the tip of the flange) should not be less than 20 mm.

Clause 6.6.5.2(2) states that if cover is required (for durability), the nominal cover to the top of the shear connectors should not be less than 20 mm, or as recommended by BS EN 1992-1-1 Table 4.4 for reinforcement, less 5 mm. If cover is not required, the top of the connector may be flush with the upper surface of the slab.

Clause 6.6.5.7(5) states that the minimum flange thickness should not be less than 0.4 × stud diameter for adequate welding, and to avoid a reduction in shear connector resistance due to flange bending.

1.9 Transverse reinforcement

The requirement for transverse reinforcement in the slab perpendicular to the axis of the beam ensures an effective transfer of force from the shear connectors into the slab without splitting the concrete longitudinally.

BS EN 1994-1-1, §6.6.6.2(1) states that the design resistance to longitudinal shear should be determined in accordance with EN 1992-1-1, §6.2.4. That method uses a concrete strut analogy in which the reinforcement maintains equilibrium by tying action across the beam. The tensile force in the reinforcement may be based on a 1:2 to 1:1 dispersion angle from the longitudinal shear connector force into the slab. Therefore, the amount of transverse reinforcement is directly linked to the number of shear connectors provided and their design resistance. The upper bound to the force that can be developed is based on the compression resistance of the block of concrete in front of the shear connectors. BS EN 1994-1-1, §6.6.6.3(1) states that the minimum amount of transverse reinforcement should be determined in accordance with BS EN 1992-1-1, §9.2.2(5).

For continuous composite slabs that are designed as simply supported, BS EN 1994-1-1, §9.8.1(2) requires a minimum percentage of reinforcement of 0.2% of the cross sectional area of the slab topping for unpropped construction. The amount of reinforcement should be increased across the beams in the following cases:

- for propped construction (where 0.4% of the slab topping is the recommended minimum area of reinforcement according to §9.8.1(2));
- in the high shear region of primary beams;
- where the required fire resistance period is more than 60 minutes;
- where heavy local loads act on the slab;
- at long openings, where local cracking may occur in the slab.

DESIGN PRINCIPLES FOR WEB OPENINGS

2.1 General

The influence of large or closely spaced web openings on the behaviour of a composite beam is complex. There are many factors that may control failure:

- Whether the opening is in a high shear or a high bending zone.
- Whether the beam is uniformly loaded or point-loaded.
- The shape of the openings i.e. circular, rectangular or elongated circular.
- The position of the opening in the depth of the section.
- The spacing of the openings (interaction effects may occur in the web-post between the openings).
- The asymmetry of the steel section (in terms of the ratio of bottom to top flange areas).
- The longitudinal shear force acting on the slab at the opening.
- The slenderness of the web (which influences its buckling resistance).

The transfer of shear across openings occurs by Vierendeel (or four corner) bending. In a composite beam, the resistance to Vierendeel bending is increased by local composite action between the top Tee (web-flange section) and the slab (which can increase the local bending resistance of the Tee by a factor of 2 to 3). This local composite action permits larger openings to be designed in a composite beam than in a similar non-composite steel beam.

In this publication, the design method is based on an analysis according to first principles that is compatible with BS EN 1993-1-1 [6] and BS EN 1994-1-1 [7]. The method has been validated by finite element analyses and calibration against measurements from a major series of tests on full-scale beams [23].

Beams, with or without openings, should be designed to satisfy the design requirements at the ultimate and serviceability limit states, including the construction condition. The effect of large web openings leads to additional local design verification: the additional considerations are described in this publication.

2.2 Design of unperforated composite beams

A simply supported unperforated composite beam is designed to provide:

- Bending resistance at the point of maximum moment.
- Shear connection between the steel beam and the concrete slab.
- Shear resistance at the supports.
- Resistance to the combinations of moment and shear along the beam.
- Local resistance at the connections and at point loads.
- Adequate transverse reinforcement given the number of shear connectors.

Composite behaviour is achieved by shear connection between the steel beam and the concrete slab. The design bending resistance depends on the section classification of the beam, which determines whether elastic or plastic section properties may be used for global bending resistance. Section classification is a function of the proportions of the flanges and web in compression. Composite beams generally may be treated as Class 1 or 2 in positive (sagging) bending because the plastic neutral axis lies close to or, within the top flange. The bending resistance also depends on the effective width of slab acting with the beam and on the degree of shear connection.

The shear connection may be either 'full' or 'partial'. For full shear connection, the plastic bending resistance of the composite section can be developed at the point of maximum moment. For partial shear connection, the degree of shear connection must satisfy the limits of BS EN 1994-1-1 [7], as discussed in Section 1.8.2, and the bending resistance is reduced. The shear connectors should be distributed in such a way that the bending resistance is satisfied at all points in the span.

At the serviceability limit state, it is also necessary to verify:

- The deflection of the composite beam due to imposed loads and other loads applied to the composite section.
- The total deflection, including the deflection of the steel beam due to its own self weight and that of the concrete (when the beam is unpropped during construction).
- The vibration performance of the floor, which may be assessed according to SCI publication P354 [24].

2.3 Design of beams with large openings

The various modes of failure that may occur at or around large web openings are illustrated in Figure 2.1. Some modes of failure occur due to local effects around single large openings, whereas others arise due to the failure of the web-post between closely spaced openings.

- Global bending (dependant on composite action at the opening position).
- Vertical shear (dependant on the reduced steel section).
- Local Vierendeel bending (dependant on the shear transfer across the opening).

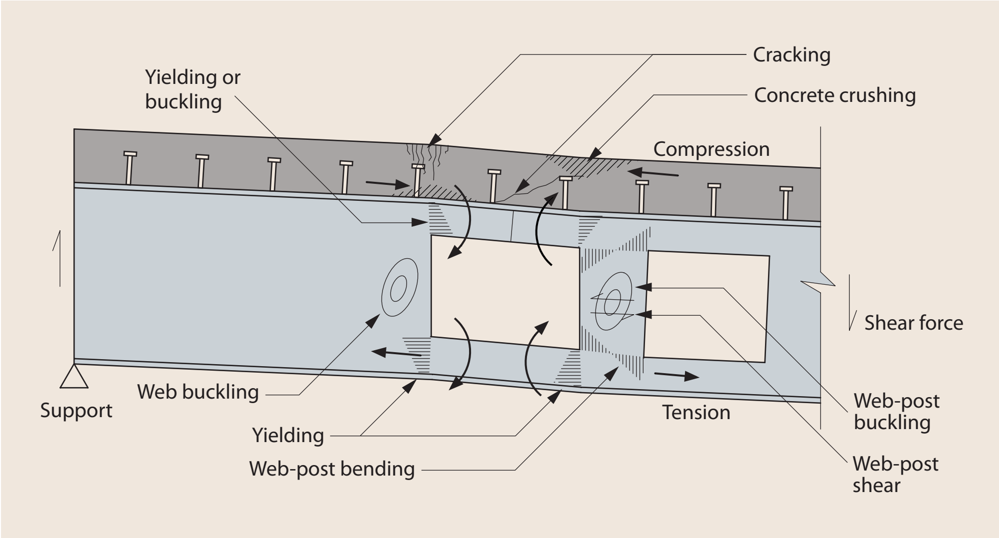


Figure 2.1 Modes of failure at large closely spaced openings

- Web-post horizontal shear (which may govern for closely spaced openings).
- Web-post bending (which may govern between closely spaced rectangular openings).
- Web-post buckling (dependent on the slenderness of the web-post and the shear resistance of the web-post between adjacent openings).
- Shear buckling (which may govern for slender webs).

The local flexibility due to shear and bending deformation may also be significant for large openings, and this is taken into account in serviceability calculations and in the Vierendeel bending check – see Section 6.1.

2.4 Design model

The design rules given in this publication relate to beams with rectangular, circular and elongated circular openings and which have a uniform web thickness. Rules are given for situations where openings are placed adjacent to one another and may be extended to beams with openings at regular spacing, such as in cellular beams.

The configuration of unstiffened openings within the composite beam is illustrated for each type of opening in Figure 2.2. The basic geometric parameters are illustrated in this figure. The eccentricity of the centreline of the opening is defined relative to the centreline of the web (positive is defined as upwards).

The design model is presented for the case of rectangular openings in the web. For circular and elongated circular openings, an equivalent rectangular opening is used in design. The equivalent opening has an identical depth and vertical position, but its effective length is used in calculations of Vierendeel action and local buckling considerations. See Section 3.1.4 for the determination of effective length of an opening.

Although the conditions at the opening may be elastic for the particular loads and opening position in the span, the design model is based on the development of the

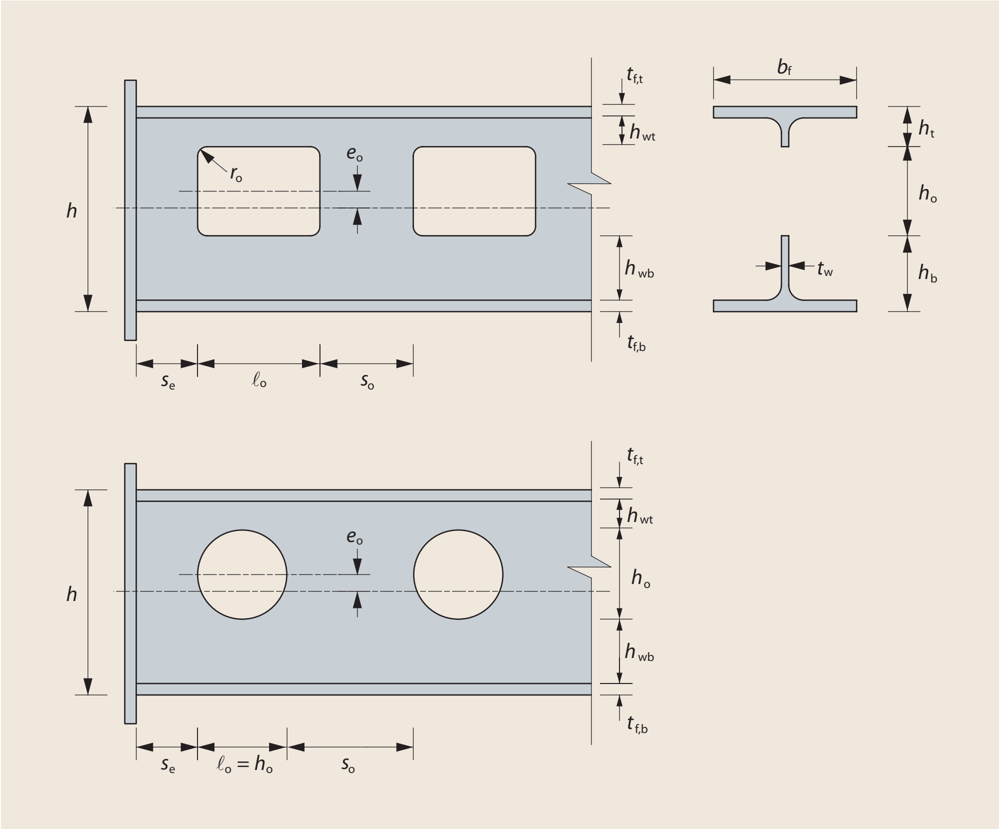


Figure 2.2 Configuration of openings of various shapes

elastic or plastic resistances of the elements, depending on the section classification, which are compared to the internal forces and moments.

The design resistance at the ultimate limit state is discussed in Section 3 for common cases, where the opening is located centrally within the depth of the web. The case where the openings are asymmetric in the depth of the beam is discussed in Section 4. When openings are very long, they may be stiffened horizontally above and below the opening. The design of stiffened openings is discussed in Section 5. Design for the serviceability limit state is discussed in Section 6. Design resistances for special cases are discussed in Section 7.

2.5 Assumptions in the design model

The distribution of internal forces around a large web opening is very complex and therefore certain simplifying assumptions have to be made for practical design, as follows:

- The vertical shear force is established at the higher shear side for a uniformly loaded beam, in order to take account of local loading over the opening. For a point loaded beam, the shear force is constant over the opening length.
- The tensile force in the bottom Tee is established from the design moment acting at the centreline of the opening. This is conservative for all design cases, as the position of zero bending moment in the Tees is towards the lower moment side of the opening as a result of plastic redistribution.

- For compatibility with the calculation of the tensile force in the bottom Tee, the compression force in the slab is established based on the number of shear connectors placed up to the centreline of the opening.
- Having established the bending resistance of the composite cross section at the centreline of the opening, the increase in bending resistance across the opening (Vierendeel bending) is established from the combined bending resistances of the Tees plus a component due to local composite action of the top Tee acting with the slab. This depends on the number of shear connectors placed over the opening.
- The plastic or elastic bending resistances of the web-flange (Tee) sections depend on the section classification and are reduced for coexisting shear and axial forces, based on the forces calculated as above.
- A relatively small vertical shear force acts in the slab at the opening, whose magnitude is limited by punching shear and pull-out of the shear connectors.
- The vertical shear force acting on the opening is resisted mainly by the top Tee, as the bottom Tee resists tension due to the design moment. It is conservative to consider initially that all the design shear is resisted by the top Tee and slab, which simplifies the design process.
- The Vierendeel bending resistance due to local composite action is limited partly by tension in the shear connectors at the end of the opening. This can be taken into account by reducing the design resistance of the shear connection in long openings.
- The forces developed in the web-post between openings are based on the change in forces between the centreline of adjacent openings. The web-post buckling model relates to the longitudinal shear force and moment in the web-post, which is increased due to asymmetry in the web-post position in the beam depth.

2.6 General guidance on positioning of openings

The geometric limits given in Table 2.1 should normally be observed when providing openings in the webs of composite beams. They are practical limits for beams within the scope of the publication. Openings that exceed these limits may be used, provided the design is justified by appropriate calculations, based on the principles of this publication.

At the initial design stage, the following approximations may be used to determine the effect of an opening on the bending resistance of a composite beam:

$M_{o,Rd} = M_{Rd}\,(1 - 0.35h_o/h)$ for unstiffened openings

$M_{o,Rd} = M_{Rd}\,(1 - 0.2h_o/h)$ for horizontally stiffened openings

where:

M_{Rd} is the bending resistance of the unperforated composite beam

$M_{o,Rd}$ is the bending resistance of the composite beam at the opening

h_o is the depth of the opening

h is the depth of the steel beam.

2.7 Design of non-composite beams with large openings

The design of non-composite beams with large web openings may be carried out using essentially the same model and design procedures as are set out in this publication for composite beams but ignoring the contribution of the slab. The principal difference is that, in a non-composite beam, the top Tee resists compression due to global bending (the slab resists most or all of the compression in a composite beam) and this means that the top Tee is less effective in resisting Vierendeel bending and shear.

Composite beams constructed without propping in the construction stage are verified as non-composite beams at that stage.

Table 2.1 Practical geometric limits for beams with web openings

PARAMETER	LIMIT		COMMENT ON LIMIT
	CIRCULAR OPENING	RECTANGULAR OPENING	
Max. depth of opening:	$\leq 0.8h$	$\leq 0.7h$	Experience shows this to be a practical limit for economic design. This is consistent with the other geometrical limits below.
Min. depth of Tees:	$\geq t_f + 30$ mm	$\geq 0.1h$	Practical limit, based on draft Annex N of ENV 1993-1-1:1992.
Min. depth of Top Tee:	As above	As above and $\geq 0.1\ell_o$ if unstiffened	To limit local deformation and stability of the top Tee during construction.
Max ratio of depth of Tees:			Asymmetry of opening position can cause web-post moments. It is preferable to provide an upward (positive) eccentricity of the opening in the web depth.
h_b/h_t	≤ 3	≤ 2	
h_b/h_t	≥ 0.5	≥ 1	
Max. unstiffened opening length, ℓ_o	—	$\leq 1.5h_o$ high shear*	The limit of the aspect ratio, ℓ_o/h_o, limits the deformation across the opening and also tension in the shear connectors. Stricter limits are required for openings in high shear regions*.
	—	$\leq 2.5h_o$ low shear	
Max. stiffened opening length, ℓ_o	—	$\leq 2.5h_o$ high shear*	
	—	$\leq 4h_o$ low shear	
Min. width of web-post:			The design of the web-posts is subject to further checks. It is recommended that stricter limits are adopted for openings in high shear regions*.
- Low shear regions	$\geq 0.3h_o$	$\geq 0.5\ell_o$	
- High shear regions*	$\geq 0.4h_o$	$\geq \ell_o$	
Corner radius of rectangular openings:	—	$r_o \geq 2t_w$ but $r_o \geq 15$ mm	Pre-drilled holes at the corners of the opening ensure no over-cutting and avoid a reduction in local plastic resistance of the Tee.
Min. width of end-post, s_e:	$\geq 0.5h_o$	$\geq \ell_o$ and $\geq h$	The minimum width of web-post depends also on the type of end connection and the build-up of forces in the shear connectors.
Min. horizontal distance to point load:			The distance to the point load is measured from the nearer edge of the opening. A separate check is required on web buckling at point load positions.
– no stiffeners	$\geq 0.5h$	$\geq h$	
– with stiffeners	$\geq 0.25h_o$	$\geq 0.5h_o$	

* A high shear region is where the design shear force is greater than half the maximum value of design shear force acting on the beam.

DESIGN RESISTANCE OF BEAMS WITH UNSTIFFENED OPENINGS

This Section discusses the determination of the local resistance of beams with unstiffened openings at the ultimate limit state (ULS). In this Section, the openings are considered to be located centrally within the depth of the web. The effect of eccentric location of the openings is discussed separately in Section 4, and stiffened openings are discussed in Section 5.

Sections 3.1 to 3.4 discuss effects and resistances at an individual opening; Section 3.5 discusses effects and resistances at the web-post between adjacent openings and Section 3.6 considers the reduction in shear buckling resistance of a slender web adjacent to an unstiffened opening.

3.1 Design effects at openings

3.1.1 Rectangular openings

The design forces and moments acting around an opening in the web of a composite beam with a single rectangular opening are shown in Figure 3.1. Under the action of

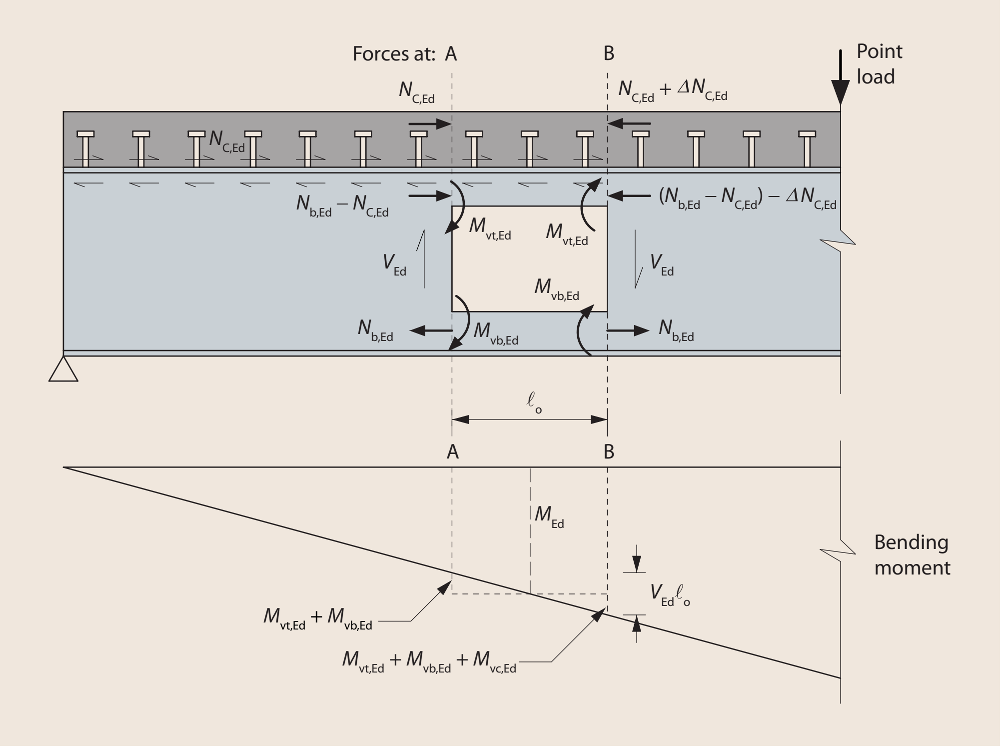


Figure 3.1
Equilibrium of forces at a single opening

the positive (sagging) bending moment, a tensile force is developed in the bottom Tee (i.e. the web-flange section), which remains constant across the width of the opening. This force is balanced by a compression force in the concrete and, depending on the proportions of the slab and the Tees, a compression force in the top Tee. Generally, the top Tee is not assumed to develop any tensile force, as it would require excessively large plastic strains to do so.

The compressive force in the concrete is developed through shear connectors welded to the top flange of the beam. The shear connectors are usually placed uniformly along the beam or, in the case of beams subject to point loads, in groups of uniform spacing dependant on the shear force. In some cases, the force that can be transferred from the beam to the slab is limited by the number of shear connectors provided; this is known as partial shear connection (see Sections 1.8).

The variation in bending moment across the opening due to the design shear force is achieved by ‘Vierendeel action’, resulting in local bending of the Tees in the four corners. In the top Tees, the Vierendeel bending is also resisted by local composite action with the slab.

The global bending resistance is verified at the centre of the opening. Assuming zero Vierendeel moments at that point, the axial force in the bottom Tee thus depends on the bending moment at that position. In reality, this will not always correspond to the point of zero bending in the Tees, which will actually move towards the lower moment side of the opening, when the component of Vierendeel bending resistance due to local composite action is relatively high and plastic redistribution occurs. However, this assumption simplifies the analysis and results in a conservative prediction of resistance.

3.1.2 Distribution of internal forces

At the ultimate limit state, failure occurs due either to the formation of plastic hinges around the opening in zones of high shear and low moment or as a result of tensile yielding in the bottom Tee, in zones of low shear and high moment. Each of the Tees is subject to combined axial force, bending and shear, which affects the development of plasticity and this allows some opportunity for redistribution of Vierendeel moments around the opening. The effect of this behaviour is to make an accurate analysis of the local design forces highly complex, and certain simplifications are necessary.

Therefore, to determine the distribution of internal forces at an opening, certain simplifying assumptions are made, as follows:

- The axial forces in the Tees and in the concrete are based on plastic analysis principles at all load levels, i.e. rectangular stress blocks are considered.
- The compression force in the slab is based on the shear connection provided from the support to the centreline of the opening.
- At the opening, vertical shear is resisted by the two Tee sections and by the concrete slab due to the local composite action at the openings.

- The shear force in the slab at the opening is based on its punching shear resistance.
- The distribution of shear between the Tees is based on their resistance to Vierendeel bending. Local composite action increases the Vierendeel bending resistance of the top Tee. Conservatively, the shear resisted by the bottom Tee can be neglected for large openings.

3.1.3 Minimum values of co-existent shear and bending

It is also necessary to ensure that the design value of the shear force V_{Ed} used to determine Vierendeel bending effects takes into account the possibility of non-uniformity in the loading during construction and in service. Therefore, it is recommended that, at any opening, the minimum combinations of design values of shear and co-existing bending moment given in Table 3.1 are considered for zones of the beam subject to low shear.

Table 3.1 Minimum combinations of design effects to be considered at an opening

CONDITION	SHEAR FORCE	BENDING MOMENT
Construction (steel section)	$0.25V_{Ed,max}$	$0.75M_{Ed,max}$
Composite Condition	$0.15V_{Ed,max}$	$0.85M_{Ed,max}$

$V_{Ed,max}$ is the maximum shear force at the supports for the relevant design situation
$M_{Ed,max}$ is the maximum moment in the span for the relevant design situation

3.1.4 Circular and elongated openings

Generally, circular and elongated openings may be treated in the same way as a rectangular opening, i.e. the forces are carried across the opening by Vierendeel action. For consideration of Vierendeel bending, the effective length of the equivalent rectangular opening may be taken as:

$$\ell_e = 0.45h_o \quad \text{for circular openings}$$

or (4)

$$\ell_e = \ell_o - 0.55h_o \quad \text{for elongated openings}$$

where:

ℓ_o is the length of the elongated opening
h_o is the diameter of the circular opening or of the ends of an elongated opening.

For circular openings, the effective height of the equivalent rectangular opening, for determining the Vierendeel bending resistance of the Tees, may be taken as $0.9h_o$.

The effective length and height for circular openings has been justified by research on non-composite beams with circular openings [25]. The effective length for elongated openings is based on that for a circular opening plus the distance between the two semi-circular ends of the opening.

3.2 Bending resistance at the opening

The bending resistance of the composite beam cross section should be verified at the centre of the opening. This verification determines the tensile force in the bottom Tee (which will affect its bending resistance and also that of the web-post – see further discussion in Section 3.5).

Generally, the plastic bending resistance of the perforated cross section can be utilized, although this depends on the classification of the top Tee and whether bending resistance is limited by partial shear connection. Sections are classified generally in accordance with BS EN 1993-1-1 [6]. Class 1 and 2 sections permit use of plastic bending resistance and Class 3 and 4 sections are restricted to elastic bending resistance.

3.2.1 Classification of the cross section

In an unperforated composite beam, the plastic neutral axis normally lies in or above the top flange of the steel section; only when using a fabricated section with a much smaller top flange than bottom flange will the plastic neutral axis lie in the web. This is equally true for a perforated section, as the plastic neutral axis is higher than in the unperforated section. Consequently, the perforated composite section will generally be Class 2 or better. If the plastic neutral axis of an asymmetric section is in the web of the top Tee, the top flange in compression may be treated as at least Class 2, even if its outstand is Class 3, if it is adequately connected to the slab; this requires shear connectors within the width of each outstand (i.e. a single line of connectors along the beam centreline is ineffective for this purpose) at a spacing that satisfies the limits in BS EN 1994-1-1, §6.6.5.

Local Vierendeel action in the bottom Tee may cause net compressive stress in the Tee at one end of the opening, but this does not require consideration for classification of the whole beam section.

3.2.2 Equilibrium of internal forces providing bending resistance

To determine the plastic bending resistance at the centreline of the opening, the forces in the Tees and the slab must be in equilibrium. Two situations are possible: with the plastic neutral axis in the slab and with the plastic neutral axis in the top Tee. The two situations are illustrated in Figure 3.2 and are discussed separately below.

Case 1: $N_{c,Rd} > N_{bT,Rd}$ (p.n.a. in slab)

In this case, the compression resistance of the full depth of the effective width of slab is greater than the tension resistance of the bottom Tee. For this situation, it may be considered that the p.n.a. is within the slab at a height such that all the concrete above it develops a stress of $0.85 f_{cd}$ (where f_{cd} is as defined in BS EN 1994-1-1).

The tensile resistance of the bottom Tee is given by:

$$N_{bT,Rd} = \frac{A_{bT} f_y}{\gamma_{M0}} \qquad (5)$$

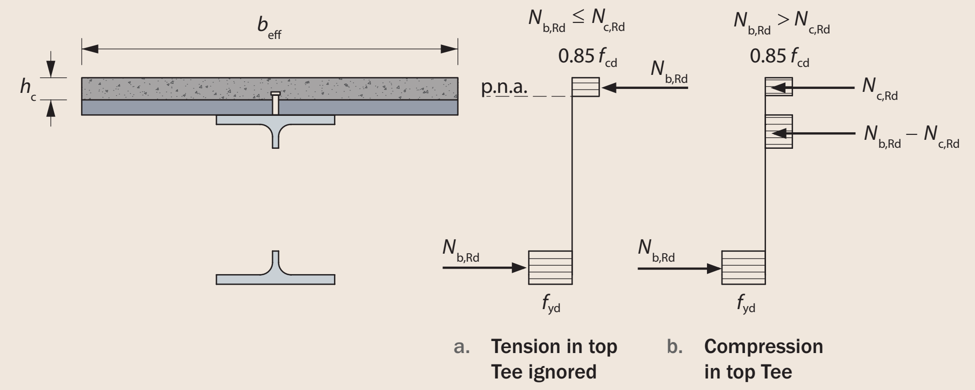


Figure 3.2 Plastic stress blocks in a perforated composite beam

where:

A_{bT} is the cross sectional area of the bottom Tee

f_y is the design value of the yield strength of steel.

The compression resistance of the full thickness of the slab at the position of the opening is the lesser of the compression resistance of the effective width of slab and the resistance provided by the shear connectors between the end of the beam and the centreline of the opening. For a beam with profiled decking with ribs transverse to the beam, the resistance is given by:

$$N_{c,Rd} = \min\left\{0.85 f_{cd} b_{eff,o} h_c\,; \quad n_{sc} P_{Rd}\right\} \tag{6}$$

where:

f_{cd} is the design strength of the concrete ($= f_{ck}/\gamma_C$, with f_{ck} and γ_C as defined in BS EN 1992-1-1 and its National Annex)

$b_{eff,o}$ is the effective slab width at the opening, as defined in Section 3.2.3

h_c is depth of the concrete topping ($h_c = h_s - h_d$)

h_s is the slab depth

h_d is the overall depth of the decking profile

n_{sc} is the number of shear connectors placed over the distance from the nearer support to the centreline of the opening (see discussion in Section 3.2.4)

P_{Rd} is the design resistance of the shear connectors used with profiled sheeting (i.e. the value of P_{Rd} given in Section 1.6.1 multiplied by k_t or k_ℓ as given in Section 1.6.2).

The plastic bending resistance is then given by:

$$M_{o,Rd} = N_{bT,Rd}\left(h_{eff} + z_t + h_s - 0.5 z_c\right) \tag{7}$$

where:

h_{eff} is the effective depth of the beam between the centroids of the Tees

z_t is the depth of the centroid of the top Tee from the outer edge of the flange

z_c is the depth of concrete in compression, which is given by:

$$z_c = \frac{N_{c,Rd}}{0.85 f_{cd} b_{eff,o}} \leq h_c$$

Using this model, it follows that for a given design moment, M_{Ed}, the axial force in the bottom Tee is given by:

$$N_{bT,Ed} = \frac{M_{Ed}}{\left(h_{eff} + z_t + h_s - 0.5z_c\right)} \qquad (8)$$

As a simplification, z_c may be taken as equal to h_c in this expression.

For a beam with decking parallel to the beam, the above expressions should be modified appropriately if the p.n.a. lies below the topping.

Case 2: $N_{c,Rd} < N_{bT,Rd}$ (p.n.a. in top Tee)

In this case, the compression resistance of the full depth of the effective width of slab is less than the tension resistance of the bottom Tee and equilibrium is achieved by developing compression in the top Tee. For this situation, it is conservative to assume that the top Tee is uniformly stressed and subject to a force equal to the difference between the tension resistance of the bottom Tee and the compression resistance of the slab, i.e. it provides a resistance equal to $N_{bT,Rd} - N_{c,Rd}$.

where:

$N_{bT,Rd}$ is defined in (5) and $N_{c,Rd}$ is as defined in (6).

With this assumption, the plastic bending resistance is given by:

$$M_{o,Rd} = N_{bT,Rd} h_{eff} + N_{c,Rd}\left(z_t + h_s - 0.5h_c\right) \qquad (9)$$

where:

h_{eff}, z_t, h_s and h_c are all as defined above.

It is also necessary to verify that for highly asymmetric sections, the compression resistance of the top Tee is adequate, as follows:

$$\frac{A_{tT} f_y}{\gamma_{M0}} \geq N_{bT,Rd} - N_{c,Rd}$$

where:

A_{tT} is the cross sectional area of the top Tee.

To determine the force in the bottom Tee for this case, expression (8) may be used unless this gives $N_{bT,Ed} > N_{c,Rd}$, in which case the force may be taken as:

$$N_{\mathrm{bT,Ed}} = \frac{M_{\mathrm{Ed}} - N_{\mathrm{c,Rd}}\left(z_{\mathrm{t}} + h_{\mathrm{s}} - 0.5h_{\mathrm{c}}\right)}{h_{\mathrm{eff}}} \qquad (10)$$

where:

$N_{\mathrm{c,Rd}}$ is as given by expression (6).

For a beam with decking parallel to the beam, the above expressions should be modified appropriately to take account of the area of concrete in the full depth of slab.

3.2.3 Effective width of slab at an opening

For openings close to the supports, the effective slab width is less than at mid-span. As noted in Section 1.7, BS EN 1994-1-1, §5.4.1.2(6) gives a formula for the effective width at the end of the span. For a simply supported span with a sufficient available width of slab on both sides, and ignoring b_0, the effective slab width acting with the beam at an opening, at a distance x from the support is given by:

$$b_{\mathrm{eff,o}} = 3L_{\mathrm{e}}/16 + x/4 \qquad \text{for } x \leq L_{\mathrm{e}}/4 \qquad (11)$$

and

$$b_{\mathrm{eff,o}} = L_{\mathrm{e}}/4 \qquad \text{for } x > L_{\mathrm{e}}/4$$

where:

L_{e} is the effective span.

3.2.4 Shear connection at an opening

When calculating the bending resistance at an opening, the number of shear connectors should be taken as the number from the support to the centreline of the opening. This is used to calculate the force in the bottom Tee, which influences its Vierendeel bending resistance.

The minimum shear connection required at an opening is determined by the need to develop a compression resistance $N_{\mathrm{c,Rd}}$ that will result in adequate bending resistance at critical cross sections, such as openings. For uniformly loaded composite beams, the shear connection requirements at intermediate positions between the support and the maximum moment position are usually satisfied by the minimum degree of shear connection rules defined in BS EN 1994-1-1 for unperforated sections. However, where point loads are placed close to the supports, the bending resistance of the perforated beam at the first opening position may be critical to the design.

For the composite design of beams with large openings close to the supports, it is likely that a shear connection resistance equal to 40% of the tensile force developed in the bottom Tee (i.e. $N_{\mathrm{c,Rd}} \geq 0.4N_{\mathrm{bT,Ed}}$) will be required to ensure adequate composite bending resistance. This is not a minimum degree of shear connection limit for the beam, but aims to ensure adequate composite action close to supports. If this is not achieved by the application of current minimum degree of shear connection rules, then additional shear connectors should be provided for critical parts of the span or composite action should be neglected when calculating the global bending resistance at opening locations with an inadequate number of shear connectors.

When the precise location of shear connectors is not known, it is good practice to assume that connectors may not be placed in the last 300 mm of the span. In secondary beams, this allows for the most unfavourable arrangement of steel decking and for the possibility of notching of secondary beams at their connections.

3.2.5 Bending resistance envelopes

The bending resistance envelope of an unperforated composite beam increases approximately linearly along the beam, reaching a maximum value at the point where sufficient shear connection has been provided to achieve the full compression resistance of the effective width of the slab, or the tensile resistance of the steel (whichever is the smaller). A similar envelope may be derived for a perforated section. Both of these envelopes are illustrated in Figure 3.3.

The design bending moment diagram depends on whether the loading is uniformly distributed, point loads or a combination of the two. A typical bending moment diagram for

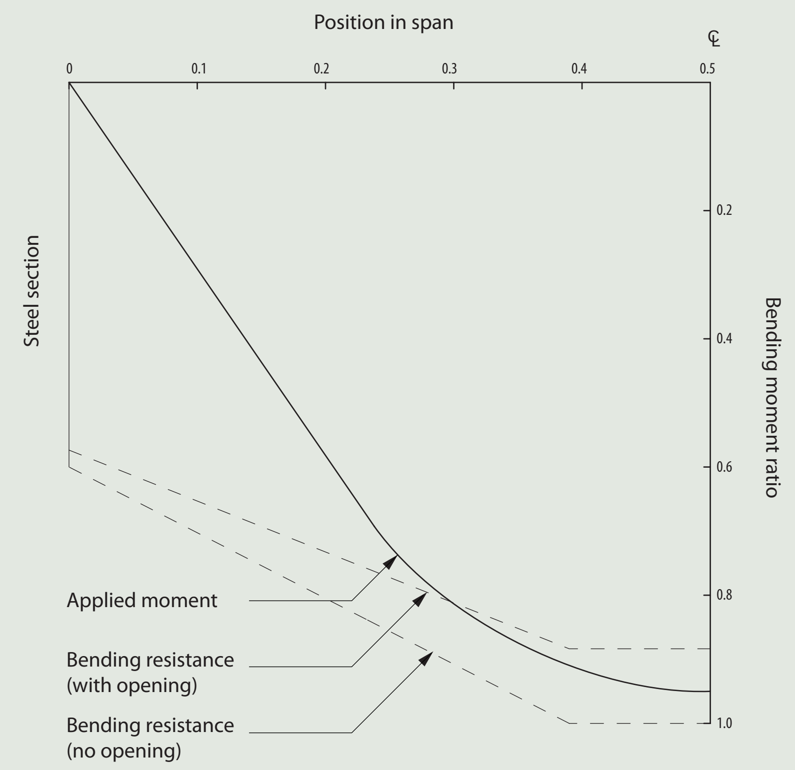


Figure 3.3 Bending moment diagram for a composite beam with and without openings

uniformly distributed loading is illustrated in Figure 3.3; it assumes that the design moment is almost equal to the resistance of the unperforated section at mid-span. In practice, the bending resistance of the beam at mid-span should be sufficiently greater than the design moment that the effect of the openings on the bending resistance is not critical. Then, the local effects at the perforated section become the governing consideration.

3.3 Shear resistance

The vertical shear resistance of the composite section at an opening may be taken as the sum of the shear resistances of the perforated steel section $V_{\mathrm{pl,o,Rd}}$ and of a narrow width of concrete slab $V_{\mathrm{c,Rd}}$.

The shear resistance of the steel section is greatly reduced by the presence of large openings. However, for most long span beams, the utilisation of the unperforated web in shear is relatively low, and so a considerable reduction in shear capacity may be acceptable, especially in the lower shear zones of the beam.

In a composite beam, it is usually the case that the greater part of the design shear is resisted by the top Tee, because it acts compositely with the slab and because it is less heavily stressed in resisting global bending than the bottom Tee.

Where the slenderness of the unperforated web $h_{\mathrm{w}}/t_{\mathrm{w}}$ exceeds the limit of $72\varepsilon/\eta$ given in BS EN 1993-1-1, §6.2.6(6), the shear buckling resistance should be determined in accordance with BS EN 1993-1-5 [30], §5.2. For shear buckling resistance adjacent to a circular opening, see Section 3.6.

3.3.1 Shear resistance of perforated steel section

The shear resistance should be established from the shear area of the perforated steel section. According to BS EN 1993-1-1, §6.2.6 the design plastic shear resistance of a cross section is given as:

$$V_{\mathrm{pl,Rd}} = \frac{A_{\mathrm{v}}\, f_{\mathrm{y}}/\sqrt{3}}{\gamma_{\mathrm{M0}}}$$

where:

A_{v} is the shear area.

For an unperforated I-section beam, the shear area corresponds to the area of the web (see §6.2.6 for exact definition). However, the perforated cross section is effectively two Tee sections, for which §6.2.6 gives an effective shear area of a rolled section Tee as:

$$A_{\mathrm{v}} = (A - b_{\mathrm{f}}t_{\mathrm{f}} + (2r + t_{\mathrm{w}}) \times 0.5t_{\mathrm{f}})$$

and for a welded Tee section as:

$$A_{\mathrm{v}} = t_{\mathrm{w}}(h_{\mathrm{w,T}} - 0.5t_{\mathrm{f}})$$

where:

A	is the cross-sectional area
b_f	is the overall width of the Tee
t_w	is the web thickness of the Tee
t_f	is the flange thickness of the Tee
r	is the root radius of the Tee
$h_{w,T}$	is the overall depth of the Tee.

The shear areas for the Tees for rolled and welded sections are illustrated in Figure 3.4.

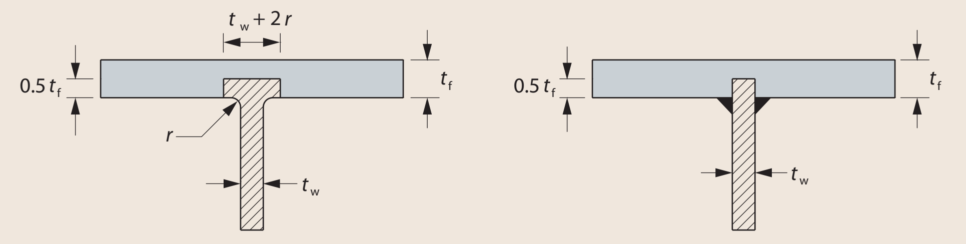
The plastic shear resistance of the perforated section is thus:

$$V_{pl,Rd} = \frac{\left(A_{v,tT} + A_{v,bT}\right) f_y / \sqrt{3}}{\gamma_{M0}}$$

Where $A_{v,tT}$ and $A_{v,bT}$ are the shear areas of the two Tees.

The plastic shear resistance may be used unless the web is susceptible to shear buckling. This is not commonly the case, but see Section 3.6 for consideration of shear buckling.

The nature of shear transfer across large openings in a composite beam generally means that Vierendeel bending effects (see Section 3.4) rather than plastic shear resistance normally govern the design.



a. Shear area of rolled section

b. Shear area of welded section

Figure 3.4 Shear area of Tees of rolled and welded sections

3.3.2 Shear resistance of the concrete slab

BS EN 1994-1-1, §6.2.2.2(1) allows the design plastic shear resistance of a beam to include a contribution from the concrete slab. It has been shown in tests on composite beams that the shear resistance of the concrete slab at an opening can be included and that its value may be calculated conservatively by considering an effective slab width equal to the beam flange width plus twice the effective depth of the concrete slab. For reinforced concrete members without shear reinforcement, BS EN 1992-1-1, §6.2.2 gives the design shear resistance (here referred to as $V_{c,Rd}$) as:

$$V_{c,Rd} = [C_{Rd,c} k(100 \rho_1 f_{ck})^{1/3} + k_1 \sigma_{cp}] b_w d \quad (12)$$

with a minimum of:

$$V_{c,Rd} = [v_{min} + k_1 \sigma_{cp}] b_w d \quad (13)$$

where:

$C_{Rd,c}$ $= 0.18/\gamma_c$ (as given in the National Annex to BS EN 1992-1-1)

k $= 1 + \sqrt{\frac{200}{d}}$ but ≤ 2.0 (with d in mm)

ρ_l $= \frac{A_{sl}}{b_w d}$ but ≤ 0.02

A_{sl} is the area of tensile reinforcement provided by the crack control mesh, which extends $\geq (\ell_{bd} + d)$ beyond the section considered

d is the effective depth of the slab. Based on test performance of beams supporting slabs cast on trapezoidal composite decking, it is considered that the effective depth of the slab may be taken as equal to h_c

ℓ_{bd} is the design anchorage length of the tensile reinforcement

k_1 = 0.15 (as given in the National Annex to BS EN 1992-1-1)

σ_{cp} $= \frac{N_{c,Ed}}{b_{eff} h_c} < 0.2 f_{cd}$

b_w is the effective width of the concrete flange for shear $= b_f + 2h_{s,eff}$

b_f is the flange width of the top Tee

$h_{s,eff}$ is the effective slab depth for punching shear $\approx 0.75 h_s$

v_{min} $= 0.035 k^{3/2} f_{ck}^{\ 1/2}$ (recommended value given in UK National Annex).

For general design of composite beams, σ_{cp} may be neglected for openings placed close to beam supports, but may be a significant factor in high bending moment regions.

Effect of high shear force on global bending resistance

The effect of the high shear forces in the Tees is to reduce the effectiveness of the web for bending. The simple interaction formula given by BS EN 1993-1-1, §6.2.8 may be used to calculate the effect of high shear on global bending resistance. No reduction in bending resistance is required for $\rho \leq 0.5$.

3.4 Bending resistance of Tee sections

Vierendeel bending occurs at the four corners of the opening and the stresses in the resistance model at each corner must be such as to maintain equilibrium across the opening; the stresses and associated forces are illustrated in Figure 3.5 for plastic resistance and Figure 3.6 for elastic resistance.

As noted in Section 3.1.4, circular and elongated openings may be represented by an equivalent rectangular opening.

3.4.1 Vierendeel bending resistances

Vierendeel bending is the means by which shear force is transferred across a large opening. The sum of the Vierendeel bending resistances at the four corners of the opening, plus the contribution due to composite action between the top Tee and the slab, must therefore not be less than the design value of the difference in bending moment from one side of the opening to the other due to that shear force; this may be expressed as:

$$2M_{\mathrm{bT,NV,Rd}} + 2M_{\mathrm{tT,NV,Rd}} + M_{\mathrm{vc,Rd}} \geq V_{\mathrm{Ed}}\ell_{\mathrm{e}} \qquad (14)$$

where:

$M_{\mathrm{bT,NV,Rd}}$ is the bending resistance of the bottom Tee, reduced for coexisting axial tension and shear

$M_{\mathrm{tT,NV,Rd}}$ is the bending resistance of the top Tee, reduced for coexisting axial compression and shear

$M_{\mathrm{vc,Rd}}$ is the local composite Vierendeel bending resistance, as defined in Section 3.4.6

V_{Ed} is the design value of the vertical shear force (taken as the value at the lower moment side of the opening)

ℓ_{e} is the effective length of the opening for Vierendeel bending.

See Section 3.4.4 for guidance on determining $M_{\mathrm{NV,Rd}}$.

In practice, the Vierendeel bending resistances are usually higher than the values used in expression (14), due to strain hardening of the deformed steel at the corners of the opening. However, this effect is not taken into account in design.

The Vierendeel bending resistances depend on the classification of the webs of the Tees (see Section 3.4.2). Where the web is Class 3 or 4, only the elastic bending resistances can be used. However, the axial forces in the Tees (which reduce the bending resistance) are generally determined on the basis of ‘plastic’ stress blocks (as discussed in Section 3.2). It is considered conservative to use the axial forces determined from this plastic stress model, even when evaluating the elastic Vierendeel bending resistances.

3.4.2 Classification of the webs of Tees

The outstand of the web of the Tee may be classified depending on the effective length of Tee $\ell_{\mathrm{o,eff}}$ and the outstand depth h_{t} or h_{b}, as given in Table 3.2.

For this local buckling classification, the effective length of the opening is defined conservatively as:

$\ell_{\mathrm{o,eff}} = \ell_{\mathrm{o}}$ for rectangular openings

or

$\ell_{\mathrm{o,eff}} = 0.7h_{\mathrm{o}}$ for circular openings (15) or

$$\ell_{o,eff} = \ell_o - 0.3h_o \qquad \text{for elongated openings}$$

This effective length takes account of the condition where the outstand of a top or bottom Tee is subject to compression over at least part of its length.

Co-existent axial compression due to global bending does not normally need to be considered in the top Tee because compressive stresses in the top flange of a composite beam due to global bending will be relatively small.

For the purpose of calculating the Vierendeel bending resistance, it is permissible to reduce the depth of a Class 3 web to that appropriate for the Class 2 limit, so that the resistance calculation may be based on plastic resistance of the reduced section, provided the flange is Class 1 or 2 (this reduction does not need to be applied to the consideration of the global bending resistance at the centre of the opening).

For a Class 4 web, the effective elastic section properties are calculated using the limiting value of h_w for a Class 3 web.

For a non-composite beam, there will be significant compression in the top Tee and the above classification should not be used. The usual classification in accordance with BS EN 1993-1-1, §5.5.2, as an outstand in compression, should be used for the web of the Tee.

Effects of axial tension on section classification of bottom Tee

Tension in the bottom Tee modifies the section classification for unstiffened Tees and improves the classification of the section. Conservatively this may be ignored, to simplify the hand calculation process; use the classification in Table 3.2. Alternatively, the effect of tension may be taken into account, as follows.

Class 3 webs may be treated as Class 2 if the plastic stress block for the Tee section subject to the combined bending and tension is such that the depth of web in

CLASS	LIMIT ON DEPTH OF WEB h_w ACCORDING TO LENGTH OF OPENING		
	$\ell_{o,eff} \le 32\varepsilon t_w$	$32\varepsilon t_w < \ell_{o,eff} \le 36\varepsilon t_w$	$\ell_{o,eff} > 36\varepsilon t_w$
2	(no limit)	$h_w \le \dfrac{10\varepsilon t_w}{\sqrt{1-\left(\dfrac{32\varepsilon t_w}{\ell_{o,eff}}\right)^2}}$	
3	(no limit)		$h_w \le \dfrac{14\varepsilon t_w}{\sqrt{1-\left(\dfrac{36\varepsilon t_w}{\ell_{o,eff}}\right)^2}}$
4	(no limit)		

Table 3.2 Classification of Tee webs

h_w is the web depth of the top or bottom Tee, as relevant.

compression is limited to a maximum of $10\varepsilon t_w$. This requirement may be more simply expressed as:

$$\frac{N_{bT,Ed}}{A_{bT}\ f_y / \gamma_{M0}} \geq 1 - \frac{20\varepsilon t_w^{\ 2}}{A_{bT}} \qquad (16)$$

where:

$N_{bT,Ed}$ is the axial tension in the bottom Tee

A_{bT} is the cross sectional area of the bottom Tee.

Class 4 webs may be treated as Class 3 when also subject to axial tension, provided that:

$$h_{wb} \leq \frac{14 t_w \varepsilon'}{\sqrt{1 - \left(\frac{36 t_w \varepsilon'}{\ell_{o,eff}}\right)^2}} \qquad \text{if } \ell_{o,eff} > 36 t_w \varepsilon' \qquad (17)$$

where:

$$\varepsilon' = \varepsilon \Big/ \left(1 - \frac{N_{bT,Rd}}{A_b\ f_y / \gamma_{M0}}\right)^{0.5}$$

h_{wb} is the depth of the web of the bottom Tee above the flange.

3.4.3 Effect of shear force on the bending resistance of Tees

The effect of the high shear forces in the Tees is to reduce the effectiveness of the web. However, the reduction of the web thickness for high shear influences the Vierendeel bending resistance of the Tees and in turn the distribution of shear.

As a first approximation, the following simple interaction formula may be used to calculate of the effective web thickness:

$$t_{w,eff} = t_w (1 - (2\mu - 1)^2) \qquad \text{for } \mu > 0.5 \qquad (18)$$

No reduction in web thickness is required for $\mu \leq 0.5$.

where:

μ is the utilisation of the cross section in shear

$= V_{Ed}/V_{Rd}$

V_{Ed} is the design value of the shear force at the opening

V_{Rd} is the shear resistance of the perforated cross section

$= V_{c,Rd} + V_{t,Rd} + V_{b,Rd}$

$V_{c,Rd}$ is the shear resistance of the concrete slab (see Section 3.3.2)

$V_{t,Rd}$ is the plastic shear resistance of the top Tee

$V_{b,Rd}$ is the plastic shear resistance of the bottom Tee.

The resulting value of $t_{w,eff}$ is applied to the calculation on moment resistance for both tees.

The actual distribution of shear forces between the Tees depends on their Vierendeel bending resistances and these in turn may depend on the reduced effective web thickness. The utilization in shear may differ for the two Tees. See further comment in Section 3.4.4.

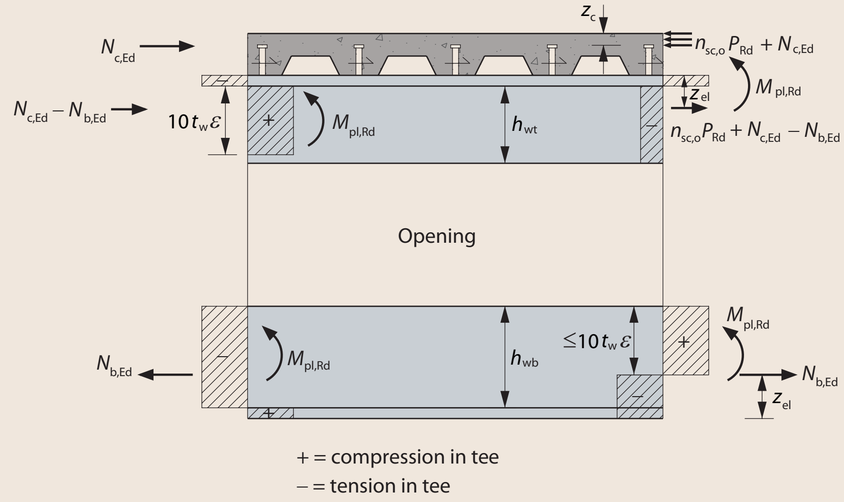


Figure 3.5 Stresses due to plastic Vierendeel bending around an opening

3.4.4 Plastic bending resistance of Tees

Plastic stress blocks can be considered when the cross section of the Tees satisfies the section classification limits for a Class 2 section. The plastic stress block regime around a rectangular opening is illustrated in Figure 3.5.

Plastic bending resistance in the absence of axial force

The plastic bending resistance of a top or bottom Tee section in the absence of axial force (and in the absence of high shear) is given by the following expression, assuming that the plastic neutral axis is in the flange of the Tee:

$$M_{\mathrm{pl,Rd}} = \frac{A_{\mathrm{w,T}}\,f_{\mathrm{y}}}{\gamma_{\mathrm{M0}}}\,(0.5h_{\mathrm{w,T}} + t_{\mathrm{f}} - z_{\mathrm{pl}}) + \frac{A_{\mathrm{f}}\,f_{\mathrm{y}}}{\gamma_{\mathrm{M,0}}}\,(0.5h_{\mathrm{f}} - z_{\mathrm{pl}} + z_{\mathrm{pl}}^{\ 2}/t_{\mathrm{f}}) \qquad (19)$$

where:

z_{pl} is the distance between the plastic neutral axis and the extreme fibre of the steel flange $= (A_{\mathrm{f}} + A_{\mathrm{w,T}})/(2b_{\mathrm{f}})$

$A_{\mathrm{w,T}}$ is the cross sectional area of web of the Tee ($= h_{\mathrm{w,T}}\,t_{\mathrm{w}}$)

A_{f} is the cross sectional area of the flange

b_{f} is the breadth of the flange

$h_{\mathrm{w,T}}$ is the depth of web of the Tee (h_{wb} or h_{wt} respectively for the bottom and top Tees)

t_{f} is the flange thickness.

Plastic bending resistance in the presence of axial force

The reduction of plastic bending resistance for co-existent axial force may be determined according to the following approximate formula:

$$M_{\mathrm{pl,N,Rd}} = M_{\mathrm{pl,Rd}}\,(1 - (N_{\mathrm{Ed}}/N_{\mathrm{pl,Rd}})^2) \qquad \text{for Class 1 and 2 sections} \qquad (20)$$

where:

$M_{pl,N,Rd}$ is the reduced plastic resistance of the Tee taking account of axial forces

$N_{pl,Rd}$ is the axial resistance of the Tee (for a Tee in compression, the depth of a Class 3 web may be reduced to the Class 2 limit, or a Class 4 web reduced to Class 3 respectively)

N_{Ed} is the design value of the axial force in the Tee due to the global bending action, either the compression force in the top Tee, $N_{tT,Ed}$ or the tension force in the bottom Tee, $N_{bT,Ed}$.

For Case 1 in Section 3.2.2 (p.n.a. in the slab), $N_{tT,Ed} = 0$ and $N_{bT,Ed}$ is given by expression (8). For Case 2 (p.n.a. in the top Tee) $N_{bT,Ed}$ is given by either expressions (8) or (10) (see discussion in Section 3.2.2) and the compression in the top Tee is given by $N_{tT,Ed} = N_{bT,Ed} - N_{c,Rd}$, where $N_{c,Rd}$ is given by expression (6).

Reduction of bending resistance due to shear

As noted in Section 3.4.3, the utilization of the web of a Tee in shear may reduce its effective thickness for bending and axial resistances. The effective thickness $t_{w,eff}$ is given by (18) dependent on the utilization factor μ. The reduced cross-sectional area of web of the Tee $(= h_{w,T}\, t_{w,eff})$ is then used in (19) and this reduced plastic bending resistance is used in (20), leading to the plastic bending resistance of the Tee in the presence of axial force and shear, $M_{pl,NV,Rd}$.

However, the shear force which may be resisted by a Tee is limited by Vierendeel bending resistance of the Tee over the length of the opening. Therefore, a process of iteration is required to determine a shear force distribution between the top and bottom Tees that is compatible with the Vierendeel bending resistance. A procedure is needed to evaluate the interactions and to determine the distribution of shear between the Tees, as follows.

Distribution of shear between top and bottom tees

Conservatively $V_{b,Rd}$ can be first set to zero to calculate μ and the associated effective web thickness $t_{w,eff}$ of the top Tee, as shown in Section 3.4.3. The plastic bending resistance of the top Tee $M_{tT,NV,Rd}$ can then be determined. The value of plastic bending resistance for the bottom Tee, $M_{bT,NV,Rd}$ may then be calculated for the same utilization factor and the associated shear force in the bottom Tee may be evaluated as:

$$V_{b,Ed} = 2M_{bT,NV,Rd}/\ell_e \qquad (21)$$

After this, the coexisting shear force in the top composite Tee can then be evaluated as: $V_{t,Ed} = V_{Ed} - V_{b,Ed}$.

The adequacy of Vierendeel bending resistance is verified using expression (14).

If expression (14) is not satisfied at this stage, the utilization of the Tees may be determined from the calculated values of $V_{t,Ed}$ and $V_{b,Ed}$ and the bending resistances re-evaluated. A single iteration is generally adequate.

3.4.5 Elastic bending resistance of Tees

Elastic design should be used when one or both Tee sections are Class 3 (this includes Class 4 sections where an effective section that complies with Class 3 limits is considered). The elastic stress regime around an opening is shown in Figure 3.6. The limit on the effective depth of web in sections that would otherwise be Class 4 is conservatively taken as that for pure compression.

As noted previously, local composite action of the top Tee and the slab occurs due to the shear connectors placed over the opening, and is added to the elastic bending resistance of the Tees. For this reason, the point of zero bending is not at the centreline of the opening.

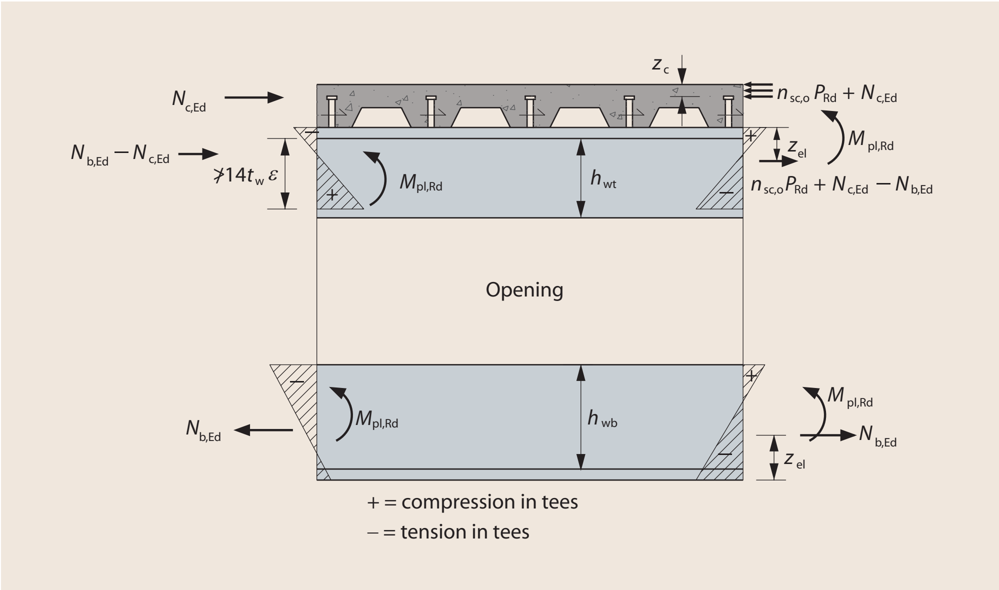


Figure 3.6 Stresses due to elastic Vierendeel bending around an opening

Elastic bending resistance in the absence of axial force

The elastic bending resistance of the top or bottom Tee section in the absence of axial force is given by:

$$M_{el,Rd} = \frac{A_{w,T} f_{yd} (0.5 h_{w,T} + t_f - z_{el})^2 + A_f f_{yd} (z_{el} - 0.5 t_f)^2}{h_{w,T} + t_f - z_{el}} \quad (22)$$

where:

z_{el} is the distance from the centroid of the Tee to the extreme fibre of the flange, given by:

$$z_{el} = \frac{A_{w,T}(0.5 h_{w,T} + t_f) + 0.5 t_f A_f}{(A_f + A_{w,T})}$$

Elastic bending resistance in the presence of axial force

The reduced elastic bending resistance is given by:

$$M_{el,N,Rd} = M_{el,Rd} (1 - (N_{T,Ed}/N_{Rd})) \quad \text{for Class 3 and 4 sections} \quad (23)$$

where:

$M_{el,N,Rd}$ is the elastic resistance of the Tee section (for the outstand of the web in compression, the depth of a Class 4 web may be reduced to the Class 3 limit)

N_{Rd} is the axial resistance of the Tee (for a Tee in compression, the depth of a Class 3 web may be reduced to the Class 2 limit, or a Class 4 web reduced to Class 3 respectively)

$N_{T,Ed}$ is the design value of the axial compression or tension force in the Tee due to the global bending action.

Elastic bending resistance in the presence of high shear

Combined effects due to shear may be neglected in elastic design, provided the global shear resistance is satisfied.

3.4.6 Vierendeel resistance due to local composite action

A contribution to the Vierendeel bending resistance occurs due to local composite action of the top Tee with the slab. The magnitude of this component is dependent on the number of shear connectors placed directly over the opening. However, its magnitude is also influenced by the flexibility of the beam at the opening, and it is necessary to introduce a modification factor to avoid over reliance on local composite action for long openings.

The bending resistance due to composite action of the top Tee with the slab is given conservatively by:

$$M_{vc,Rd} = \Delta N_{c,Rd}(h_s + z_t - 0.5h_c)k_o \qquad (24)$$

where:

$\Delta N_{c,Rd}$ is the compression force developed by the shear connectors placed over the opening $(= n_{sc,o} P_{Rd})$

$n_{sc,o}$ is the number of shear connectors placed over the opening

P_{Rd} is the design resistance of the shear connectors used with profiled sheeting (i.e. P_{Rd} as given in Section 1.6.1 multiplied by k_t or k_ℓ as given in Section 1.6.2)

h_s is the overall depth of the slab

z_t is the depth of the centroid of the top Tee from the outer edge of the flange (as an approximation, take z_t = top flange thickness)

h_c is the depth of concrete above the decking profile

k_o is a reduction factor due to the flexibility of the opening, which takes account of second order effects together with the combination of shear and tension forces at the edge of the opening. Expressions for k_o are given below.

For circular openings, $n_{sc,o}$ is calculated over the effective length of the opening $\ell_e (= 0.45h_o)$. However, because it is impossible to guarantee the exact position of the shear connectors over the opening, together with the fact that the zone of influence of the opening extends outside the opening, it is recommended that the exact value of ℓ_e divided by stud spacing is used to calculate $n_{sc,o}$ (rather than the integer value).

It is conservative to ignore composite action over the opening if the Vierendeel bending resistance of the steel Tees alone is adequate. This will often be the case for circular openings – see the Worked example in Appendix A.

In the top Tee, the effect of the additional axial force due to composite action may be neglected in expression (23).

For isolated large openings, an additional component of Vierendeel bending occurs due to the change in the line of action of the compression force, N_c along the slab. This can be significant for large single openings but should be neglected for closely spaced openings. Furthermore, high tensile forces are developed locally in the shear connectors due to this effect, which increase when openings are placed close together (see the following Section).

Influence of long openings

At long openings, a reduction factor should be applied to the local Vierendeel resistance due to composite action to allow for the flexibility of the opening. This effect is partly due to the tensile forces developed in the shear connectors and partly because of the need to control the relative displacements across the opening to avoid concrete cracking and possible shear failure. For unstiffened openings, the value of the reduction factor k_o to be used in expression (24) is conservatively given by:

$$k_o = \left(1 - \frac{\ell_o}{25h_t}\right) \qquad (25)$$

where:

h_t is the depth of the top Tee.

k_o may be taken as 1.0 when $\ell_o \leq 5h_t$.

Alternative method – influence of tension in shear connectors

As a more accurate (but more complex) alternative to the use of expression (25), the reduction factor k_o in expression (24) may be determined by considering the tension force developed in the shear connectors, which is dependent on the opening length. Tension in the shear connectors is developed at the end of the opening furthest from the support and equates closely to the shear force developed in the slab.

The tension force in the shear connectors arising from local Vierendeel bending may be taken as occurring in the shear connectors at the edge of the openings. When the Vierendeel moment is fully mobilised, this tensile force is given by:

$$F_{ten,max} = M_{vc} / n\ell_o \qquad (26)$$

where:

$F_{ten,max}$ is the design tensile force per shear connector considered to be developed when the Vierendeel bending moment $= M_{vc}$

M_{vc} is the Vierendeel bending moment due to composite action, equal to the value of resistance given by expression (24) when $k_o = 1.0$)

n is the number of shear connectors per rib (= 1 or 2).

The pull out resistance of a shear connector may be taken as $0.85P_{Rd}$. This represents the upper bound to the Vierendeel bending resistance that can be developed due to composite action. For other cases, the presence of tension forces on stud connectors will reduce their design shear resistance. The interaction between shear force P_{Ed} and tension F_{ten} in the shear connectors is limited by the following criterion, which is based on the limiting interaction curve given in Johnson and Buckby [28]:

$$\left(\frac{F_{ten}}{0.85P_{Rd}}\right)^{5/3} + \left(\frac{P_{Ed}}{P_{Rd}}\right)^{5/3} \leq 1.0 \qquad (27)$$

where:

F_{ten} is the design tensile force acting on a shear connector

P_{Ed} is the design shear force acting on a shear connector

P_{Rd} is the design resistance of a shear connector used with profiled sheeting (i.e. P_{Rd} as given in Section 1.6 multiplied by k_t or k_ℓ as given in Section 1.6.2).

At the limit, $F_{ten} = k_o F_{ten,max}$ and $P_{Ed} = k_o P_{Rd}$ and thus the value of k_o is given by:

$$k_o = \left[\left(\frac{F_{ten,max}}{0.85P_{Rd}}\right)^{5/3} + 1\right]^{-3/5} \leq 1.0 \qquad (28)$$

Additionally, according to BS EN 1994-1-1, §6.6.3.2(2), the effect of the tensile force on shear resistance may be neglected if $F_{ten} \leq 0.1P_{Rd}$.

The reduction factor k_o may thus be set to the limiting value given by the above expression, which is evaluated in Table 3.3. It is generally found that use of this factor is less conservative than that given by expression (25).

3.5 Web-posts between openings

The web-post between adjacent openings is subject to high stresses due to:

- Longitudinal (horizontal) shear on its narrowest width.
- Compression due to transfer of vertical shear.
- Bending developed due to Vierendeel action.

The interaction is more complex because of the possibility of buckling due to combinations of all these effects (see Figure 2.1).

Table 3.3 Reduction factor k_o according to tension force on shear connectors

TENSILE FORCE ON SHEAR CONNECTORS	REDUCTION FACTOR
$F_{ten,max}/P_{Rd}$	k_o
0.1	1.0
0.2	0.95
0.3	0.91
0.4	0.86
0.5	0.81
0.6	0.77
0.7	0.72
0.8	0.68
0.9	0.64
1.0	0.60

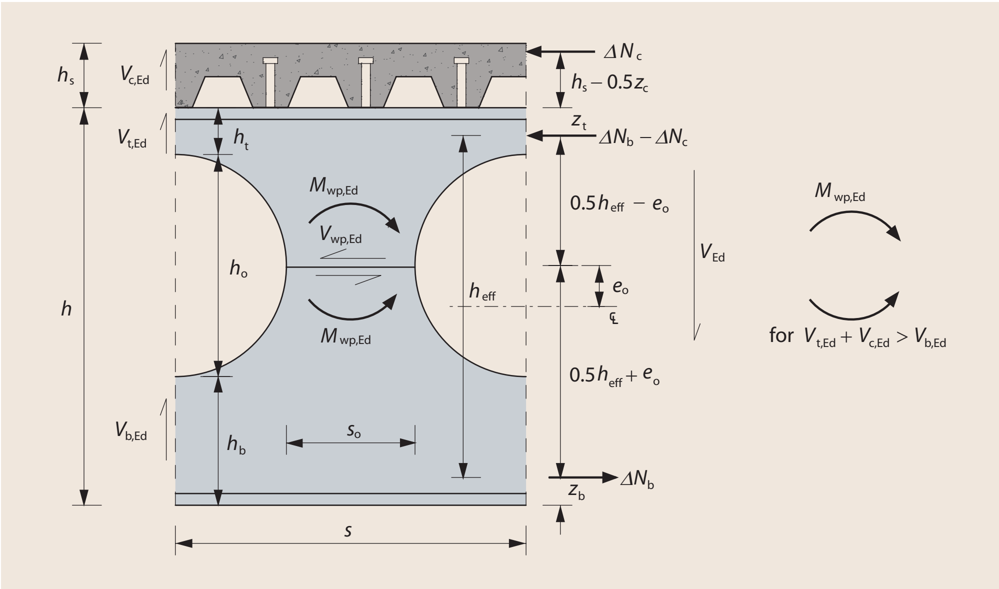


Figure 3.7 Forces in web-post between circular openings for a composite beam

Note: The direction of positive $M_{wp,Ed}$ recognises that, in a composite perforated beam, the shear in the top tee ($V_{t,Ed} + V_{c,Ed}$) is usually greater than in the bottom tee ($V_{b,Ed}$).

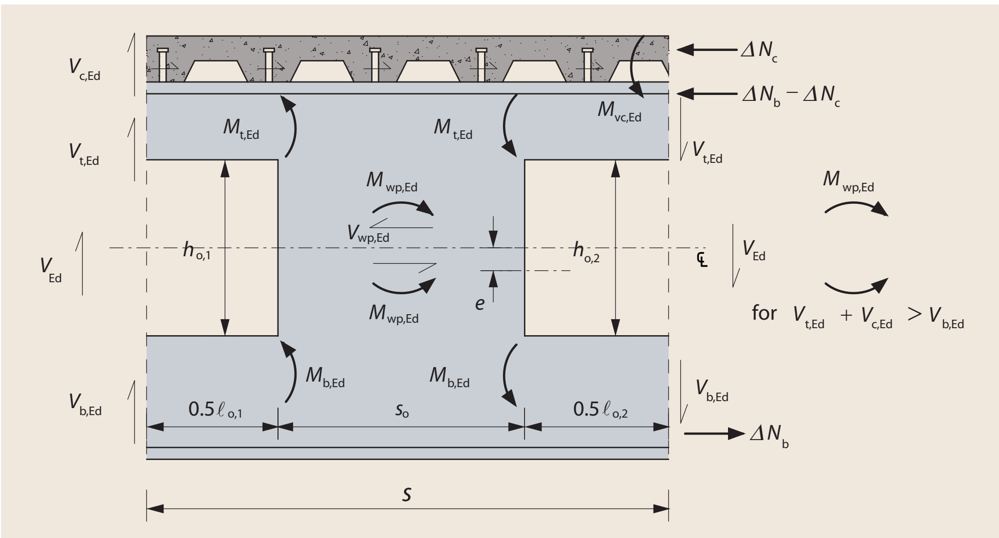


Figure 3.8 Forces in web-post between rectangular openings for a composite beam

The design forces for circular and rectangular openings are shown in Figure 3.7 and Figure 3.8. Equilibrium of the top and bottom Tees may be evaluated separately. The web-post horizontal shear is equal to the increase in tension in the bottom Tee between the centrelines of the adjacent openings.

3.5.1 Design longitudinal shear force in web-post

The design value of the longitudinal shear force, $V_{\mathrm{wp,Ed}}$ acting on the web-post may be determined from the consideration of the build up of tension forces in the bottom Tee.

As a first approximation, assume that there is sufficient shear connection to develop a force in the slab equal to the incremental force in the bottom Tee between the openings. The longitudinal shear force acting on the web-post is then given by:

$$V_{\mathrm{wp,Ed}} = \frac{V_{\mathrm{Ed}}\,s}{\left(h_{\mathrm{eff}} + z_{\mathrm{t}} + h_{\mathrm{s}} - 0.5h_{\mathrm{c}}\right)} \qquad (29)$$

where:

V_{Ed} is the design value of the average of the shear forces at the centrelines of adjacent openings

s is the centre-to-centre spacing of the openings

h_{eff} is the effective depth of the beam between the centroids of the Tees (which may conservatively be taken as the depth between their elastic neutral axes)

z_{t} is the depth of the centroid of the top Tee from the outer edge of the flange (which may conservatively be taken as the elastic neutral axis position).

h_{c} and h_{s} are as previously defined.

The incremental compression force that can be mobilised in the slab due to the resistance of the shear connectors between openings is given by:

$$\Delta N_{\mathrm{cs,Rd}} = n_{\mathrm{sc,s}} P_{\mathrm{Rd}} \qquad (30)$$

where:

$n_{\mathrm{sc,s}}$ is the number of shear connectors between the centrelines of adjacent openings

P_{Rd} is the design resistance of a shear connector used with profiled sheeting (i.e. P_{Rd} as given in Section 1.6 multiplied by k_{t} or k_{ℓ} as given in Section 1.6.2).

If there is insufficient shear connection between the openings, the longitudinal shear force developed in the web-post is increased, and becomes:

$$V_{\mathrm{wp,Ed}} = \frac{V_{\mathrm{Ed}}\,s - \Delta N_{\mathrm{cs,Rd}}\left(z_{\mathrm{t}} + h_{\mathrm{s}} - 0.5h_{\mathrm{c}}\right)}{h_{\mathrm{eff}}} \qquad (31)$$

The larger of the values given by expressions (29) and (31) should be used.

3.5.2 Design compression force in web-post

Compression in the web-post (i.e. forces generally in a vertical direction) potentially leads to web-post buckling. This is a complex phenomenon which is discussed in Section 3.5.6. For verification of adequacy against buckling in the design model, it is necessary to distinguish between closely spaced openings, where the compression force is resisted by the full width of the end post, and widely spaced openings, where forces are resisted by effective widths of web adjacent to each of the openings.

The transition from widely spaced to closely spaced openings may be taken to occur at an edge-to-edge spacing equal to the opening length, corresponding to $s_o = h_o$ for circular openings and $s_o = \ell_o$ for rectangular openings, where h_o and ℓ_o are the depth and length of the openings respectively. This transition is not exact but is used in order to simplify the analysis.

Design compression force for widely spaced openings

For widely spaced or discrete openings, web-post buckling is independent of the spacing of the openings. In this case, it is considered that a compression force acts at the edge of the opening over an effective width of $h_o/2$. The magnitude of the compressive force may be taken as equal to the larger of the vertical shear forces in the top and bottom Tees. The use of the larger of the shear forces in the Tees takes account of any asymmetry in the opening position. The compressive force in the post at the edge of the opening is thus given by:

$$N_{wp,Ed} = V_{T,Ed} \tag{32}$$

where:

$V_{T,Ed}$ is the larger of the shear forces in the two Tees (composite top Tee and steel bottom Tee).

Design compression force for closely spaced openings

For closely spaced openings, the longitudinal shear force on the web-post $V_{wp,Ed}$ is used to determine the design compression force on the web-post, rather than the vertical shear force, as in expression (32). This takes account of the higher forces acting in the web-post between the openings. For openings placed centrally in the beam depth, the compression stress acting on the web-post is taken as equal to the longitudinal shear stress acting on the web-post. The compression force acting on the web-post is thus given by:

$$N_{wp,Ed} = V_{wp,Ed} + |M_{wp,Ed}|/(h_o/2) \tag{33}$$

where:

$V_{wp,Ed}$ is given by expressions (29) or (31)

$M_{wp,Ed}$ is the value of the web-post moment at the mid-height of the opening.

3.5.3 Design moment in web-post

The web-post is subject to bending as a result of Vierendeel bending action. When the shear forces resisted by the Tee sections above and below the opening are equal (which is the case for a symmetric steel beam with central openings), there is zero moment at the mid-height of the web-post. Equal and opposite moments, equal to $V_{\mathrm{wp,Ed}}$ $h_{\mathrm{o}}/2$, act on the top and bottom of the web-post. When the shear forces are not equal, an additional moment $M_{\mathrm{wp,Ed}}$ acts at mid-height of the web-post, given by:

$$M_{\mathrm{wp,Ed}} = (V_{\mathrm{t,Ed}} + V_{\mathrm{c,Ed}} - V_{\mathrm{b,Ed}})\, s/2 + V_{\mathrm{wp,Ed}}\, e_{\mathrm{o}} - \Delta N_{\mathrm{cs}}(z_{\mathrm{t}} + h_{\mathrm{s}} - 0.5h_{\mathrm{c}})/2 \qquad (34)$$

where:

V_{Ed} is the design value of the shear force acting mid-way between the centrelines of adjacent openings

$V_{\mathrm{c,Ed}}$ is the design value of the shear force in the concrete slab, which may be taken as $V_{\mathrm{c,Rd}}$ (see Section 3.3.2)

$V_{\mathrm{b,Ed}}$ is the design value of the shear force in the bottom Tee

$V_{\mathrm{t,Ed}}$ is the design value of the shear force in the top Tee

$V_{\mathrm{wp,Ed}}$ is the design value of the longitudinal shear force acting on the web-post (which may be taken to be $V_{\mathrm{wp,Ed}} = \Delta N_{\mathrm{b,Ed}}$)

$\Delta N_{\mathrm{b,Ed}}$ is the increase in tension force in the bottom Tee between the centrelines of the adjacent openings

e_{o} is the eccentricity of the centreline of the opening measured from mid-height of the beam (taken as positive when the centreline of the opening is above the mid-height of the beam)

ΔN_{cs} is the incremental compression force developed by the shear connectors over a length, s between adjacent openings. The value may be taken as $\Delta N_{\mathrm{cs,Rd}}$ (see expression (30)).

s, z_{t} h_{s}, and h_{c} are as previously defined.

The web-post moment will increase with asymmetry of the cross section (see Section 4). Any distribution of shear force between the Tees that satisfies equilibrium may be assumed, such that the value of $M_{\mathrm{wp,Ed}}$ is minimized.

From equilibrium of the forces, the shear force in the top Tee is given by:
$V_{\mathrm{t,Ed}} = V_{\mathrm{Ed}} - V_{\mathrm{c,Rd}} - V_{\mathrm{b,Ed}}$, and thus:

$$M_{\mathrm{wp,Ed}} = (V_{\mathrm{Ed}} - 2V_{\mathrm{b,Ed}})\, s/2 + V_{\mathrm{wp,Ed}}\, e_{\mathrm{o}} - \Delta N_{\mathrm{cs}}(z_{\mathrm{t}} + h_{\mathrm{s}} - 0.5h_{\mathrm{c}})/2 \qquad (35)$$

Generally, for low values of design shear force, the moment in the web-post may be ignored.

If the height of adjacent openings differs or one of the openings is stiffened and the other unstiffened, then the distribution of shear between the top and bottom Tees may be different for each opening. In such cases, the mean of the values for the two openings should be used in the above expressions.

3.5.4 Shear resistance of web-post

The design longitudinal shear resistance of the web-post may be taken as:

$$V_{wp,Ed} = \frac{(s_o t_w) f_y / \sqrt{3}}{\gamma_{M0}} \tag{36}$$

where:

s_o is the edge-to-edge spacing of the openings ($s - \ell_o$)

t_w is thickness of the web.

Tests have shown that this resistance may be developed in the web-post, despite the complex stress state that exists locally in the web.

3.5.5 Buckling resistance of web-post

Buckling of the web-post is a complex phenomenon and is dependent on:

- shape of the opening, i.e. circular or rectangular;
- slenderness of the web at the opening;
- asymmetry of opening position in the beam depth.

Design model for web-post buckling

A simple compression field model has been developed from an approach presented in SCI Publication P100 [26] and uses an equivalent strut with a buckling length defined by the opening dimensions and spacing. It has been calibrated against FEA results and is conservative relative to the FEA [27]. It has been calibrated over the range $h_o \geq s_o \geq 0.3h_o$ and has been extended to consider openings placed asymmetrically in the depth of the beam. In this case in-plane moments are generated in the web-post.

Basis of the strut model

The design model is based on the following assumptions:

- A compression stress acts in the web-post, taken as equal in magnitude to the longitudinal shear stress acting on the web-post.
- Buckling occurs over an effective length of web, which depends on the spacing of the openings.
- An additional in-plane bending stress may act on the web-post (for highly asymmetric opening positions, see Section 4).

Three design cases may be considered:

- Widely spaced openings.
- Closely spaced openings.
- Asymmetrically placed openings in the beam depth.

The analysis for rectangular and circular openings differs only in the buckling length ℓ_w of the web-post in compression.

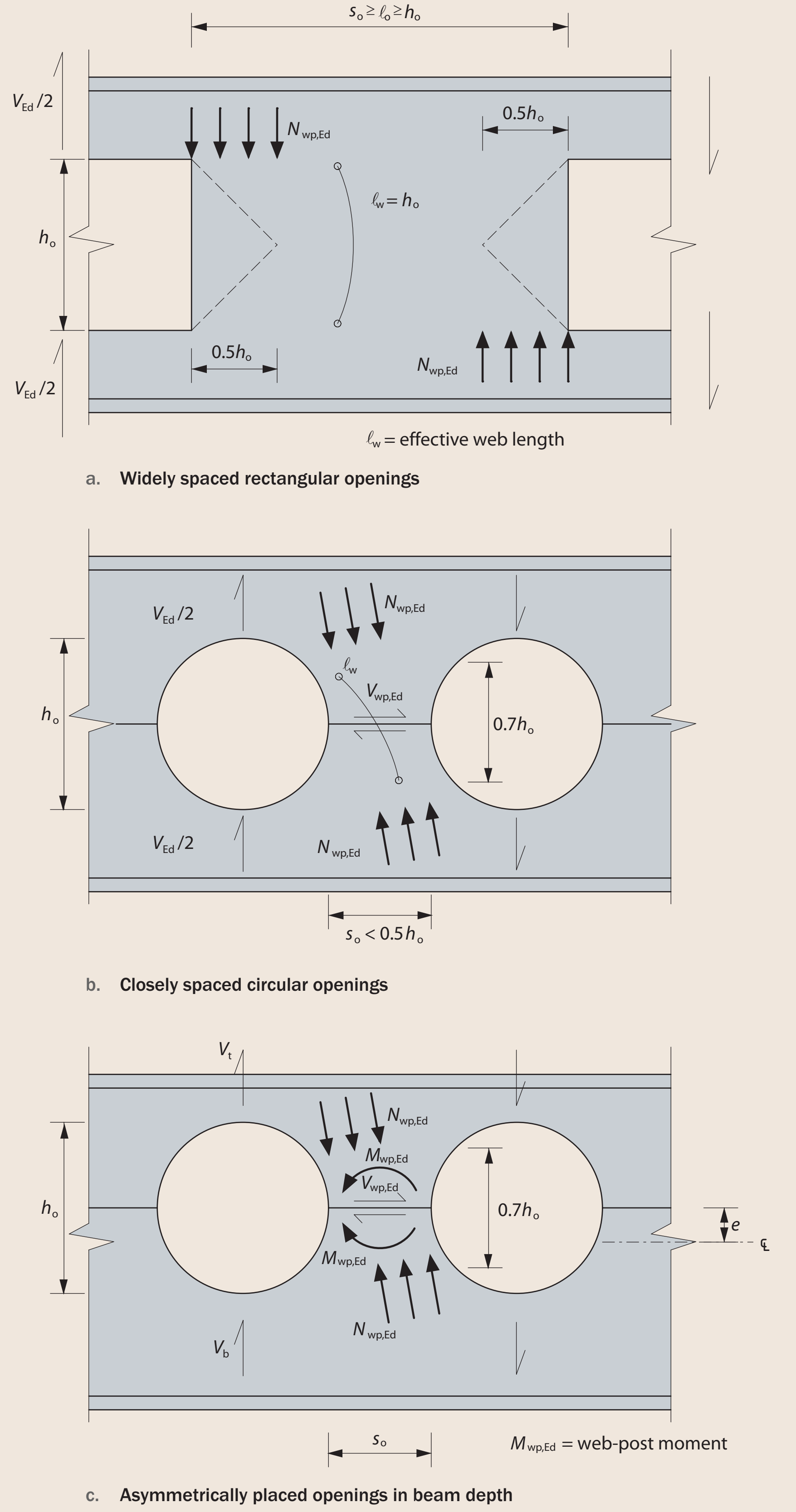


Figure 3.9 Web-post bending model for buckling analysis

These design cases are illustrated in Figure 3.9.

Buckling resistance for widely spaced openings

For widely spaced openings, a buckling length of $\ell_w = h_o$ is appropriate for rectangular openings and $\ell_w = 0.7h_o$ for circular and elongated circular openings.

For the determination of the reduction factor on buckling resistance χ, the non-dimensional slenderness of the web-post $\bar{\lambda}$ may be expressed for these buckling lengths as:

a. For circular openings and elongated circular openings:

$$\bar{\lambda} = \frac{2.5h_o}{t_w}\frac{1}{\lambda_1} \quad (37)$$

b. For rectangular openings:

$$\bar{\lambda} = \frac{3.5h_o}{t_w}\frac{1}{\lambda_1} \quad (38)$$

where λ_1 is defined in BS EN 1993-1-1, §6.3.1.3 as:

$$\lambda_1 = \pi\sqrt{\frac{E}{f_y}} = 94\,\varepsilon \qquad \text{and } \varepsilon = \sqrt{(235/f_y)}$$

The design buckling resistance of the web-post $N_{wp,Rd}$ is determined from BS EN 1993-1-1, §6.3.1. Based on test evidence, buckling curve b may be used for rolled sections; buckling curve c sould be used for beams fabricated from plates. The buckling resistance of the web post is therefore:

$$N_{wp,Rd} = \chi\frac{0.5h_o t_w f_y}{\gamma_{M1}} \quad (39)$$

The verification may be expressed as $N_{wp,Rd} \geq N_{wp,Ed}$

Buckling resistance for closely spaced openings

For closely spaced openings, a buckling length of $\ell_w = 0.7\sqrt{(s_o^2 + h_o^2)}$ is appropriate for rectangular openings and $\ell_w = 0.5\sqrt{(s_o^2 + h_o^2)}$ for circular and elongated circular openings.

The non-dimensional slenderness of the web-post $\bar{\lambda}$ may then be expressed as:

a. For circular openings and elongated circular openings:

$$\bar{\lambda} = \frac{1.75\sqrt{s_o^2 + h_o^2}}{t_w}\frac{1}{\lambda_1} \quad (40)$$

b. For rectangular openings:

$$\bar{\lambda} = \frac{2.5\sqrt{s_o^2 + h_o^2}}{t_w}\frac{1}{\lambda_1} \quad (41)$$

where λ_1 is as defined in BS EN 1993-1-1, §6.3.1.3 (see above).

The design buckling resistance of the web post, $N_{wp,Rd}$, is determined from buckling curve b in BS EN 1993-1-1, §6.3.1.2 for rolled sections and beams fabricated from rolled sections, or from buckling curve c for beams fabricated from plates. The buckling resistance of the web-post is given by:

$$N_{wp,Rd} = \chi \frac{s_o t_w f_y}{\gamma_{M1}} \qquad (42)$$

The verification may be expressed as $N_{wp,Rd} \geq N_{wp,Ed}$.

Minimum opening width for web-post buckling

Web post buckling effects are small and can be ignored for widely spaced openings for the following conditions:

$$h_o/t_w \leq 25 \text{ for circular openings} \qquad (43)$$
$$h_o/t_w \leq 20 \text{ for rectangular openings}$$

3.5.6 Bending resistance of web-post

The design bending resistance of the web-post should be taken as its elastic value. For rectangular openings, and for circular openings at the mid-height of the opening, this is given by:

$$M_{wp,Rd} = \frac{s_o^2 t_w}{6} \frac{f_y}{\gamma_{M0}} \qquad (44)$$

For rectangular openings, the bending resistance of the web-post should be such that:

$$M_{wp,Rd} \geq V_{wp} h_o/2 + |M_{wp,Ed}| \qquad (45)$$

where:

$V_{wp,Ed}$ is given by expressions (29) or (31)

$M_{wp,Ed}$ is the value of the web-post moment at the mid-height of the opening.

Web-post bending resistance may control for closely spaced rectangular openings.

The design moment at the mid-height of the web-post does not modify its ability to resist shear at that section, provided the elastic bending resistance of the web-post is used.

For circular openings, web-post bending does not control the design of the web-post because the increasing width of the web-post from its mid-height increases its bending resistance.

3.6 Shear buckling adjacent to an isolated opening

As an alternative methodology for slender webs with isolated openings, the shear buckling resistance of a web may be modified to take account of an isolated opening. The shear buckling resistance of a slender unperforated web is given by BS EN 1993-1-1, §6.2.6(6) and BS EN 1993-1-5, §5. The shear buckling resistance of the web with isolated openings may be taken conservatively as:

$$V_{w,o,Rd} = 0.9V_{bw,Rd}\left(1 - \frac{\sqrt{h_o \ell_o}}{h_w}\right) \quad (46)$$

where $V_{bw,Rd}$ is the shear buckling resistance given by BS EN 1993-1-5, §5.2.

For a circular opening, $\ell_o = h_o$ in this formula.

It should be noted that this formula is based on inclined stress field action in the web, and is therefore only appropriate for isolated openings because of the complementary compression developed in the web. When the opening is at least $0.8h_w$ from the end of the beam (where h_w is the clear depth between flanges) $V_{w,o,Rd}$ may be taken as that for a rigid end-post, as defined in BS EN 1993-1-5, irrespective of the actual end-post width.

LIMITING SHEAR RESISTANCE FOR BEAMS WITH CLOSELY SPACED OPENINGS

This Section presents expressions for the limiting value of vertical shear resistance of beams with closely spaced openings, dependent on web-post bending resistance and web-post buckling resistance.

4.1 Resistance governed by web-post bending

4.1.1 Resistance for circular openings

In the limit, the shear force in the bottom Tee reaches its maximum value due to Vierendeel bending and the moment in the web-post reaches its elastic bending resistance. By re-arranging the previous equilibrium equations, closed solutions can be established for the maximum vertical shear that can be resisted (i.e. when $M_{wp,Ed} = M_{wp,Rd}$).

For partial shear connection, when $V_{wp,Ed} > \Delta N_{cs,Rd}$, the limiting value of design vertical shear resistance of the perforated beam is given by:

$$V_{Rd} = \frac{2M_{wp,Rd} / s + 4M_{b,NV,Rd} / \ell_o}{1 + 2e_o / h_{eff}} + \frac{\Delta N_{cs,Rd}}{s}\left(z_t + h_s - 0.5h_c\right) \qquad (47)$$

where:

$M_{wp,Rd}$ is the elastic bending resistance of the web-post at mid-height of the opening, given by expression (44)

$M_{bT,NV,Rd}$ is the bending resistance of the bottom Tee, reduced due to the effect of axial tension (given by expression (20) or (23) depending on the section classification). Initially, no reduction for shear should be made, provided that the shear resisted by the bottom Tee does not exceed half its plastic shear resistance

$\Delta N_{cs,Rd}$ is the incremental compression force that can be mobilised in the slab due to the resistance of the shear connectors between openings.

e_o, h_c, h_{eff}, h_s, ℓ_o, s and z_t are all as previously defined.

For full shear connection, when $V_{wp,Ed} \leq \Delta N_{cs,Rd}$, the compression force in the web-post is taken as equal to $V_{wp,Ed}$, which leads to the following equation for limiting vertical shear resistance of the perforated beam:

$$V_{Rd} = \left[2M_{wp,Rd}/s + 4M_{bT,NV,Rd}/\ell_o\right]\frac{\left[h_{eff} + h_s - 0.5h_c\right]}{\left[h_{eff} + 2e_o\right]} \qquad (48)$$

The limiting value of V_{Rd} should be taken as the smaller of the values given by these two equations.

If the shear force V_{Ed} can be resisted by evaluating V_{Rd} with $M_{wp,Rd}$ set to zero, then it follows that no web-post moment is required for equilibrium.

4.1.2 Resistance for rectangular openings

The behaviour of beams with closely spaced rectangular openings is different from that of circular openings because of the greater importance of Vierendeel bending and web-post bending.

In a composite beam with rectangular openings, high in plane moments are generated in the web-post in order to maintain equilibrium in the bottom Tee. The critical point for web-post moment generally becomes the top of the web-post due to the combined moment and longitudinal shear effects, which should satisfy the following limit for a rectangular opening:

$$M_{wp,Rd} \geq V_{wp,Ed} h_o/2 + |M_{wp,Ed}| \quad (49)$$

where:

$M_{wp,Rd}$ is the elastic bending resistance of the web-post, given by expression (44)

$M_{wp,Ed}$ is the web-post moment at the mid-height of the opening.

By rearranging the previous equilibrium equations, it can be shown [27] that the shear resistance that can be developed between adjacent rectangular openings is as follows.

For partial shear connection, when $V_{wp,Ed} > \Delta N_{c,s,Rd}$, the limiting value of design vertical shear resistance of the perforated beam is given by:

$$V_{Rd} = \frac{2M_{wp,Rd} / s + 4M_{bT,NV,Rd} / \ell_o}{1 + (2e_o + h_o)/h_{eff}} + \frac{\Delta N_{cs,Rd}}{s}(z_t + h_s - 0.5h_c) \quad (50)$$

Where one of the adjacent openings is stiffened and the other unstiffened, $M_{bT,NV,Rd}$ may be taken as the mean value for the two bottom Tees.

For full shear connection, when $V_{wp,Ed} \leq \Delta N_{c,s,Rd}$, the limiting value of design shear resistance is given by:

$$V_{Rd} = \left[2M_{wp,Rd}/s + 4M_{bT,N,Rd}/\ell_o\right] \frac{\left[h_{eff} + h_s - 0.5h_c\right]}{\left[h_{eff} + 2e_o + h_o\right]} \quad (51)$$

The value of V_{Rd} should be taken as the smaller of the values given by these two expressions.

4.2 Resistance governed by web-post buckling

The same approach as above may be adopted to derive a closed solution for the vertical shear resistance of the perforated beam, limited by web-post buckling, as follows.

For partial shear connection:

$$V_{\mathrm{Rd}} = \frac{N_{\mathrm{wp,Rd}}\left(h_{\mathrm{o}} / s\right) + 4M_{\mathrm{bT,NV,Rd}} / \ell_{\mathrm{o}}}{1 + h_{\mathrm{o,eff}} / h_{\mathrm{eff}}} + \frac{\Delta N_{\mathrm{cs,Rd}}}{s}\left(z_{\mathrm{t}} + h_{\mathrm{s}} - 0.5h_{\mathrm{c}}\right) \qquad (52)$$

For full shear connection, the compression force $\Delta N_{\mathrm{c,Ed}}$ is taken as equal to $V_{\mathrm{wp,Ed}}$, which leads to the following equation for limiting vertical shear resistance:

$$V_{\mathrm{Rd}} = \left[N_{\mathrm{wp,Rd}}\left(h_{\mathrm{o}} / s\right) + 4M_{\mathrm{bT,NV,Rd}} / \ell_{\mathrm{o}}\right]\frac{\left[h_{\mathrm{eff}} + h_{\mathrm{s}} - 0.5h_{\mathrm{c}}\right]}{\left[h_{\mathrm{eff}} + h_{\mathrm{o}} + 2e_{\mathrm{o}}\right]} \qquad (53)$$

where:

$N_{\mathrm{wp,Rd}}$ is the web-post buckling resistance, as defined in Section 3.5.6

$M_{\mathrm{b,NV,Rd}}$ is the bending resistance of the bottom Tee, reduced due to the effect of axial tension and shear as appropriate (given by expression (20) or (23) depending on the section classification).

The limiting value of V_{Rd} should be taken as the smaller of the values given by these two expressions.

As noted above, if one opening is stiffened and one unstiffened, the average value of $M_{\mathrm{bT,NV,Rd}}$ may be used.

The web-post buckling verification for eccentric openings is the same for both circular and rectangular openings, using an appropriate value of $N_{\mathrm{wp,Rd}}$. No web-post buckling verification is required when the spacing of the openings satisfies the limits for widely spaced openings.

4.3 Verification of shear resistance

The value of the shear resistance V_{Rd} should be taken as the smaller of the limiting values for web-post bending and web-post buckling given by Sections 4.1 and 4.2 but not greater than ($V_{\mathrm{pl,Rd}} + V_{\mathrm{c,Rd}}$) for the perforated section (see Section 3.3). It is generally found that web-post bending will control for rectangular openings, whereas web-post buckling will control for circular openings.

For verification of shear resistance, the value of V_{Rd} should be at least equal to the shear force midway between the centrelines of adjacent openings, V_{Ed}.

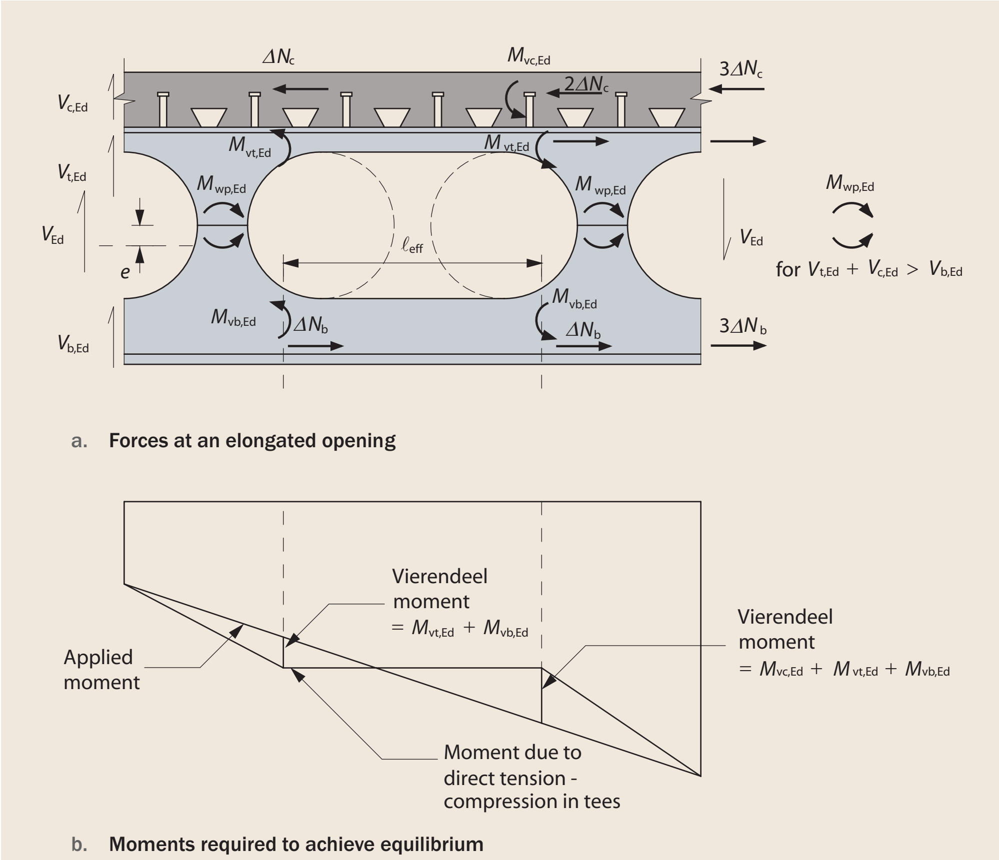


Figure 4.1
Forces in web-post at an elongated opening

4.4 Eccentricity effects for elongated openings

Elongated openings may be created by removing the web-post between adjacent circular openings. The same approach as described for closely spaced openings applies to elongated openings except that the web-post moment developed due to asymmetry effects will increase. This behaviour is illustrated in Figure 4.1.

The previous expressions may be modified as follows: the value of s should be taken as the distance between the centrelines of the circular and elongated openings (which is often twice the regular spacing, when a web-post has been removed); the effective length of the opening is given by $\ell_e = \ell_o - 0.55h_o$. It follows that the maximum value of V_{Rd} in the above expressions is considerably reduced, unless $M_{b,N,Rd}$ is increased by the use of horizontal stiffeners above and below the elongated opening.

DESIGN RESISTANCE OF BEAMS WITH STIFFENED OPENINGS

5.1 Geometric limitations

Where circular or rectangular openings are stiffened in order to ensure adequate design resistance in composite construction, it is appropriate to apply geometric limits on the positions and sizes of the openings and stiffeners. The limits given in Figure 5.1 are based on practical requirements for welding and effective transfer of forces around the openings. They are not precise limits, but are sensible limits for design cases where beams are subject to uniformly distributed load or multiple point loads.

5.2 Beams with horizontal stiffeners

5.2.1 General requirements

Horizontal stiffeners are formed from rectangular plates, welded to the top and/or the bottom edges of the openings, on one or both sides. They have the effect of:

- Increasing the local resistance to Vierendeel bending.
- Preventing local buckling of the web of the Tee.

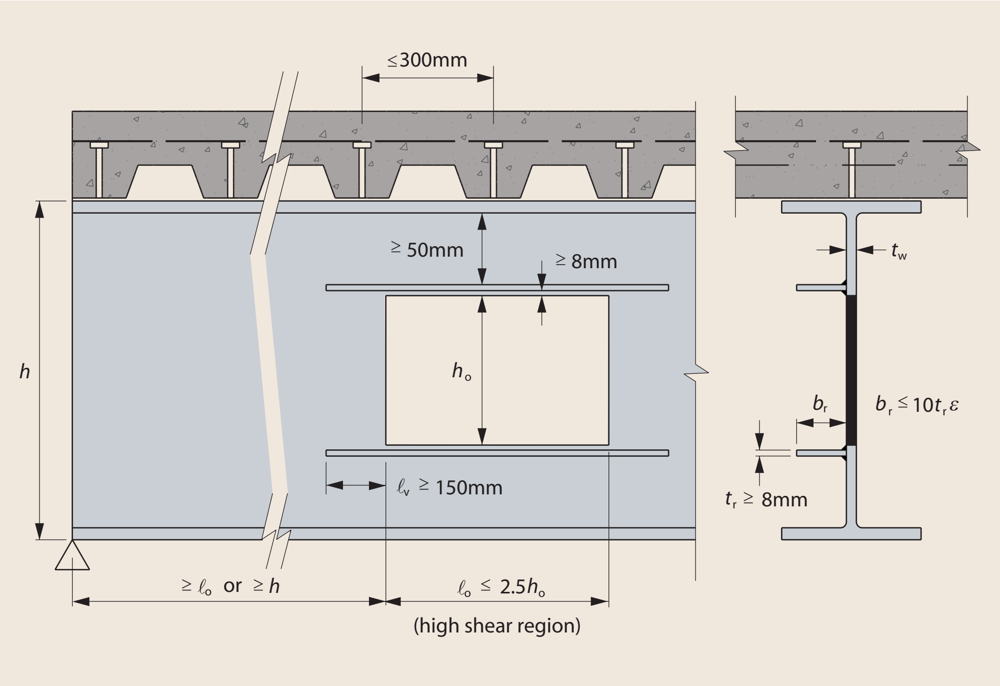


Figure 5.1 Detailing limits for rectangular openings in high shear zones

In choosing the size and position of horizontal stiffeners, consideration must be given to the practicality of connecting the stiffeners to the beam web (i.e. there must be sufficient access to make the welds). Where openings are close to the section flanges, partial penetration butt welds from one side of the stiffener may be the only solution.

The outstand/thickness ratio of the stiffeners should not exceed the limits for a Class 3 (semi-compact) section. If the beam is designed as a Class 1 or 2 section, then any part of the outstand in excess of the Class 2 limit must be considered as ineffective. In the region of the opening, the web is considered to be Class 2 when stiffened.

The maximum size of stiffener is generally controlled by the ability of the web to resist the local anchorage forces, which are transferred by the weld between the stiffener and the web at the ends of the opening. A suitable anchorage length is required in order to develop the axial resistance of the stiffener.

Stiffeners welded on one side of the web can be more efficient from a fabrication point of view but where single-sided stiffeners are used it is necessary to ensure that the web is sufficiently strong in transverse bending that the eccentric forces can be transferred from the stiffener to the web.

5.2.2 Stiffener size and anchorage length

A practical upper bound to the stiffener area is $A_r \leq 0.5 h_o t_w$. This will ensure that the adjacent web is strong enough to resist the force transferred from the stiffener.

In practice, horizontal reinforcement (stiffeners) in excess of 200 mm wide and 20 mm thick are not recommended. A minimum width of 80 mm or $\ell_o/10$ is recommended.

In selecting the size of stiffener, the following rule may be used for stiffeners on both sides of the web:

$$\frac{t_r}{t_w} \leq 1.2\left(\frac{\ell_v}{2b_r}\right) \quad (54)$$

where:

ℓ_v is the anchorage length of the stiffener
t_r is the thickness of the stiffener
t_w is the thickness of the web
b_r is the width of the stiffener.

Stiffeners should preferably be attached by fillet welds to both sides of the stiffener (although butt welds might be unavoidable in some cases, as noted above). The throat thickness of the fillet weld should be equal to at least half the thickness of the stiffener. The minimum offset distance from the edge of the opening should be at least 8 mm, to allow for at least a 5 mm leg length fillet weld.

The anchorage length ℓ_v of the stiffener beyond each end of the opening should generally be at least $0.25\ell_o$ and should satisfy the following criteria:

a. For shear resistance of the fillet welds $\ell_v \geq \dfrac{F_r}{2naf_{vw,d}}$

b. For shear resistance of the stiffeners $\ell_v \geq \dfrac{F_r}{nt_r f_{yr} / \left(\gamma_{M0}\sqrt{3}\right)}$ (55)

c. For shear resistance of the web $\ell_v \geq \dfrac{F_r}{2\,nt_w f_{yr} / \left(\gamma_{M0}\sqrt{3}\right)}$

where:

F_r is the design force in the stiffener, which may be taken as:

$$F_r = F_{r,Rd} = \frac{A_r f_{yr}}{\gamma_{M0}}$$

A_r is the cross-sectional area of the stiffener, or effective area of a Class 3 stiffener

f_{yr} is the yield strength of the stiffener

n is 1 for a single-sided stiffener; and 2 for double-sided stiffeners

a is the throat thickness of the fillet weld

$f_{vw,d}$ is the design shear strength of a fillet weld, as given by BS EN 1993 1 8, §4.5.3.3 [29] (note that the value depends on the value of γ_{M2} given by the National Annex).

Therefore, ℓ_v should generally be taken as not less than $0.25\ell_o$ or $2b_r$ or a minimum of 150 mm, and the welds should be checked for the transfer of forces over this length.

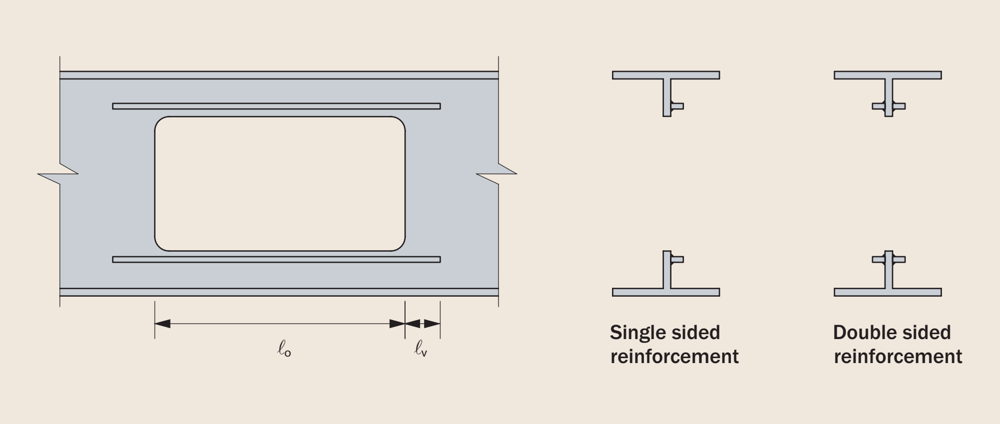


Figure 5.2 Criteria for anchorage of stiffeners

5.2.3 Additional requirements for single-sided stiffeners

Single-sided stiffeners are often preferred when using relatively small stiffeners, but their effectiveness may be reduced by the ability of the web to resist the out of plane bending effects due to the eccentric axial force in the stiffener. In such cases, it is recommended that:

- the web depth : thickness does not exceed 70ε;
- the stiffener width b_r does not exceed $\ell_o/6$ for fabricated beams and $\ell_o/4$ for rolled beams;

- the stiffener thickness t_r does not exceed t_w (see below);
- the anchorage length ℓ_v should be taken as the smaller of $2b_r$ and the distance to the edge of the adjacent opening, and should satisfy the limits in expression (55).

The thickness of the stiffener should satisfy the following limit:

$$\frac{t_r}{t_w} \leq 0.96\left(\frac{\ell_v}{2b_r}\right) \text{ but } \leq 1.0 \quad \text{for stiffeners on one side of the beam} \tag{56}$$

5.2.4 Bending resistance of beams with horizontal stiffeners

Vierendeel bending

Plastic resistance of Tees

The expressions for plastic bending resistance of the Tee given in Section 3.4.4 are modified when longitudinal stiffeners are used, as follows:

$$M_{pl,Rd} = \frac{A_{w,T} f_y}{\gamma_{M0}}\left(\frac{h_{wt}}{2} + t_f - z_{pl}\right) + \frac{A_f f_y}{\gamma_{M0}}\left(0.5t_f - z_{pl} + z_{pl}^2/t_f\right) + \frac{A_r f_y}{\gamma_{M0}}\left(t_f + h_{w,T} - e_r - z_{pl}\right) \tag{57}$$

where:

z_{pl} is the distance between the plastic neutral axis and the extreme fibre of the steel flange given by:

$z_{pl} = (A_f + A_{w,T} + A_r)/(2b_f)$ for $A_r \leq A_f - A_{w,T}$

$A_{w,T}$ is the cross-sectional area of web of the Tee $= h_{w,T}\, t_{w,eff}$

A_f is the cross-sectional area of the flange

b_f is the breadth of the flange

$h_{w,T}$ is the depth of web of the Tee, excluding the flange ($= h_{wb}$ or h_{wt} as appropriate)

t_f is the thickness of the flange

A_r is the cross-sectional area of horizontal stiffeners

e_r is the offset distance of centre of stiffener from tip of web.

Elastic resistance of Tees

The expressions for elastic bending resistance of the Tee given in Section 3.4.5 are modified when there are longitudinal stiffeners, as follows:

$$M_{el,Rd} = \frac{A_{w,T} f_y (0.5h_{w,T} + t_f - z_{el})^2 + A_f f_y (z_{el} - 0.5t_f)^2 + A_r f_{y,r} (h_{w,T} - e_r + t_f - z_{el})^2}{(h_{w,T} + t_f - z_{el})\gamma_{M0}} \tag{58}$$

where:

z_{el} is the distance between the elastic neutral axis and the extreme fibre of the steel flange given by:

$$z_{el} = \frac{A_{w,T}(0.5h_{w,T} + t_f) + 0.5t_f A_f + A_r\left(h_{w,T} - e_r + t_f\right)}{(A_f + A_{w,T})} \tag{59}$$

5.2.5 Effect of axial force on bending resistance

For a stiffened bottom Tee, a linear reduction in bending resistance with axial tension is conservatively used because of the influence of the stiffener area [23], as follows:

$$M_{pl,N,Rd} = M_{pl,Rd}(1 - N_{Ed}/N_{pl,Rd}) \quad (60)$$

This moment axial force interaction formula is conservative for small stiffener areas, where the interaction tends to be quadratic in form.

5.2.6 Shear resistance of Tees

The horizontal stiffeners may conservatively be ignored in calculation of the shear resistance of the Tees. As a consequence of this, the shear area of the Tees may be calculated in accordance with Section 3.3.1 and, where the Tee is subject to high shear, the bending resistance should be reduced as shown in Section 3.4.3.

5.2.7 Effect of long openings

At long stiffened openings, a reduction factor is applied to the local Vierendeel bending resistance due to composite action to allow for the flexibility of the opening. For horizontally stiffened openings, this reduction factor, k_o is given by:

$$k_o = \left(1 - \frac{\ell_o}{35h_t}\right) \quad (61)$$

where:

h_t is the depth of the top Tee;

k_o may be taken as 1.0 for $\ell_o \leq 5h_t$ for stiffened openings.

An alternative, more accurate (but complex), approach is to consider the effect of tensile forces in the shear connectors on the composite component of Vierendeel bending resistance, $M_{vc,Rd}$, as presented in Section 3.4.6.

5.3 Vertical stiffeners

Vertical stiffeners are not generally required in beams with Class 1 or Class 2 sections, but may be required for Class 4 sections. However, vertical stiffeners may be needed at heavy concentrated point loads and at secondary beam connections. This generalisation applies also to perforated beams. When provided, vertical stiffeners should generally be full depth, unless the unstiffened part of the web is sufficient to prevent local buckling.

5.4 Ring stiffeners

5.4.1 Effect of ring stiffeners on web-post behaviour

Ring stiffeners welded around the inside of circular openings have the effect of suppressing web-post buckling (for Class 3 webs). Two cases may be considered:

Adjacent ring stiffened openings

Based on finite element analysis (FEA) of typical cases, it is considered that the ring stiffeners reduce the slenderness of the web-post by a factor of 0.7 by preventing any buckling along the edge of the opening and thus the value of $\overline{\lambda}$ given by expression (37) is multiplied by a factor of 0.7. Openings with a diameter less than $40t_w\varepsilon$, may be considered as fully restrained against web-post buckling if ring stiffeners are provided.

Ring stiffened opening on one side of a web-post

Based on FEA, it is considered that the ring stiffeners on one side of the web-post have a partial effect on web-post buckling. It is considered that the buckling length of the strut is reduced by a factor of 0.85 for $s/h_o \leq 1.5$ and thus the value of $\overline{\lambda}$ given by expression (40) is multiplied by a 0.85. For wider web-posts with $s/h_o > 1.5$, no effect of the adjacent ring stiffener should be taken into account.

BEHAVIOUR AT SERVICEABILITY LIMIT STATE

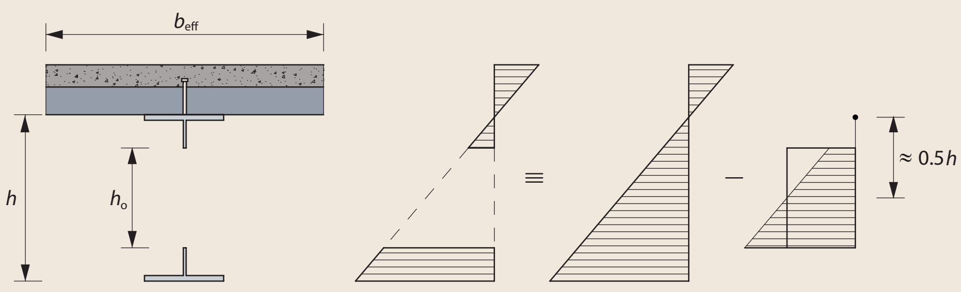
The serviceability limits likely to be of relevance to the design of composite beams are those concerning control of deflections, control of cracking in the slab, and minimising of perceptible floor vibrations. In the case of beams with large web openings, the principal influences for design at the Serviceability Limit State (SLS) are:

- Additional deflection due to the loss of flexural stiffness at the openings.
- Additional deflection due to Vierendeel bending effects at large openings (shear deflection).
- Reduction in overall stiffness (leading to greater dynamic response).

6.1 Calculation of additional deflections

6.1.1 Use of elastic section properties

Serviceability performance is established using elastic properties, taking account of the loss of web on deflections. For pure bending, the elastic stiffness is defined as in Figure 6.1. In tests on composite beams with large openings, little deviation from elastic behaviour in the load-deflection response of the beams occurs until close to failure, and therefore local yielding is not important as regards overall serviceability behaviour. Furthermore, for beams with stiffened webs, the loss of stiffness of the perforated section is reduced.



a. Cross-section at opening
b. Elastic stresses
c. Equivalent elastic stress blocks

Figure 6.1 Elastic properties of a perforated beam

The second moment of area of a composite beam at an opening may be established as shown schematically in Figure 6.1. The reduced second moment of area at the opening is given approximately by:

$$I_{y,o} = I_y - 0.3 h\, h_o^{\,2} t_w$$

Simplifying assumptions can be made to calculate the additional deflections due to circular and rectangular openings. Empirical formulae can be established that are accurate for most practical applications.

6.1.2 Increased deflection due to openings

The increased deflection of a composite beam comprises components resulting from bending and shear, as shown in Figure 6.2.

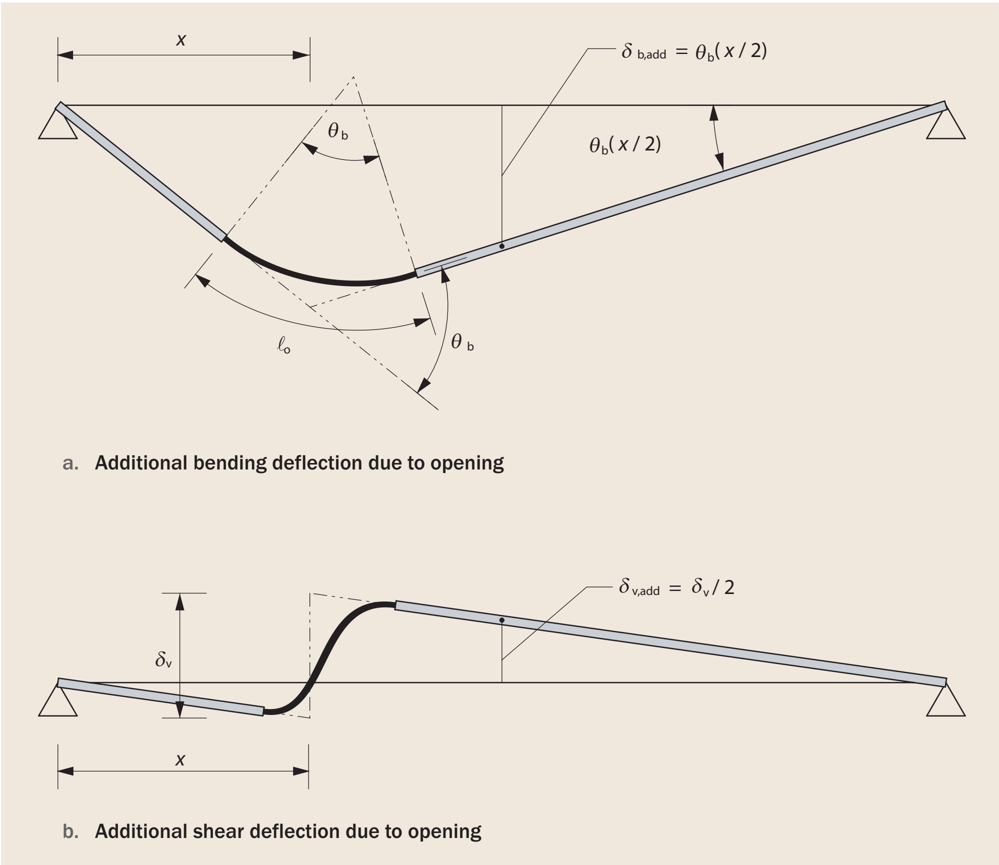


Figure 6.2 Component deflections due to the flexibility of the beam at an opening

Additional bending deflection

For a uniformly loaded beam with a single opening placed at a distance, x, from each support, it may be shown that the additional pure bending deflection $w_{b,add}$ relative to the central deflection, w_b of a composite beam without openings is approximately given by:

$$\frac{w_{b,\,add}}{w_b} = 19.2\left(1-\frac{x}{L}\right)\left[\frac{x}{L}\right]^2\left(\frac{\ell_o}{L}\right)\left\{\frac{EI_y}{EI_{y,o}}-1\right\} \quad (62)$$

where:

I_y is the second moment of area of the full composite section

$I_{y,o}$ is the second moment of area of the composite section at an opening.

$$w_b = \frac{5F_{d,ser}\,L^3}{384EI}$$

where:

$F_{d,ser}$ is the design value of the total load on the beam at SLS

L is the span of the beam.

For a beam subject to a central point load, the additional deflection is given by:

$$\frac{w_{b,\,add}}{w_b} = 12.0\left(1-\frac{x}{L}\right)\left[\frac{x}{L}\right]^2\left(\frac{\ell_o}{L}\right)\left\{\frac{EI_y}{EI_{y,o}}-1\right\} \qquad (63)$$

and the central deflection is given by:

$$w_b = \frac{F_{sd}\,L^3}{48EI_y}$$

Additional shear deflection

The additional shear deflection $w_{v,add}$ for a single opening, expressed as relative to the pure bending deflection, is given by:

$$\frac{w_{v,\,add}}{w_b} = 1.6\left(1-\frac{2x}{L}\right)\left[\frac{\ell_o}{L}\right]^3\frac{EI_y}{EI_{y,\,T}} \quad \text{for a uniformly distributed load}$$

$$\frac{w_{v,\,add}}{w_b} = 1.0\left(1-\frac{2x}{L}\right)\left[\frac{\ell_o}{L}\right]^3\frac{EI_y}{EI_{y,\,T}} \quad \text{for a central point load} \qquad (64)$$

where $I_{y,T}$ is the second moment of area of a Tee, averaged above or below the opening. This ignores the significant contribution of the concrete in local composite action, which may be included in a more refined analysis. The composite stiffness may be expected to double the effective stiffness of the Tee, EI_T. This shear component may be significant for long openings.

The total additional deflection is therefore $w_{b,add} + w_{v,add}$.

6.2 Approximate method

6.2.1 Single openings in a composite beam

As a more simple alternative, the following approximate empirical formula is established from an analysis of the additional deflection of a composite beam, which is based on the pure bending deflection due to the loss of stiffness at the opening. The approximation also allows for the influence of the deflection in the high shear regions by the term $(1 - x/L)$, where x is the distance from the nearer support to the centreline of the opening. As a good approximation, the additional deflection due to an opening, w_{add}, may be expressed as relative to that of the unperforated composite beam in pure bending, w_b as follows:

$$\frac{w_{add}}{w_b} = k_o\left(\frac{\ell_o}{L}\right)\left(\frac{h_o}{h}\right)\left(1-\frac{x}{L}\right) \quad \text{for } x \leq 0.5L$$

$$\frac{w_{add}}{w_b} = k_o\left(\frac{\ell_o}{L}\right)\left(\frac{h_o}{h}\right)\left(\frac{x}{L}\right) \quad \text{for } x > 0.5L \qquad (65)$$

where the coefficient, k_o, is given by:

k_o = 1.0 for longitudinally stiffened openings

k_o = 1.5 for unstiffened openings

and:

ℓ_o is the effective length of opening ($= 0.45h_o$ for circular openings when calculating deflections)

w_b is the bending deflection of unperforated composite beam.

The formula for additional deflection becomes more conservative for shorter openings, where Vierendeel deflections are less.

6.2.2 Additional deflection due to multiple openings

For multiple regular sized openings, the additional deflections may be taken as:

$$\frac{w_{add}}{w_b} = 0.7n_o k_o\left(\frac{\ell_o}{L}\right)\left(\frac{h_o}{h}\right) \qquad (66)$$

where n_o is the number of regular openings along the beam. The factor of 0.7 represents the combined effect of the distribution of moment and shear along the beam.

Typically, for a cellular beam (where $\ell_o = 0.45h_o$) the above expression reduces to:

$$\frac{w_{add}}{w_b} = 0.47n_o\left(\frac{h_o}{h}\right)^2\left(\frac{h}{L}\right) \qquad (67)$$

The additional deflection due to the openings in a cellular beam is typically 12 to 15% of that of an unperforated beam of the same depth.

SPECIAL CASES

7.1 Highly asymmetric openings

Highly asymmetric positions of openings may be necessary if the opening has to be close to one flange, as illustrated in Figure 7.1. In this case, it is recommended that a horizontal stiffener is introduced in order that the local bending resistance of the remaining section is increased. The Vierendeel bending resistance is then entirely dependent on the remaining Tee.

In such cases, the openings should be placed sufficiently far apart that web-post bending is not critical (see Section 4). This is satisfied when the edge-to-edge spacing of the openings exceeds twice the depth of the larger opening.

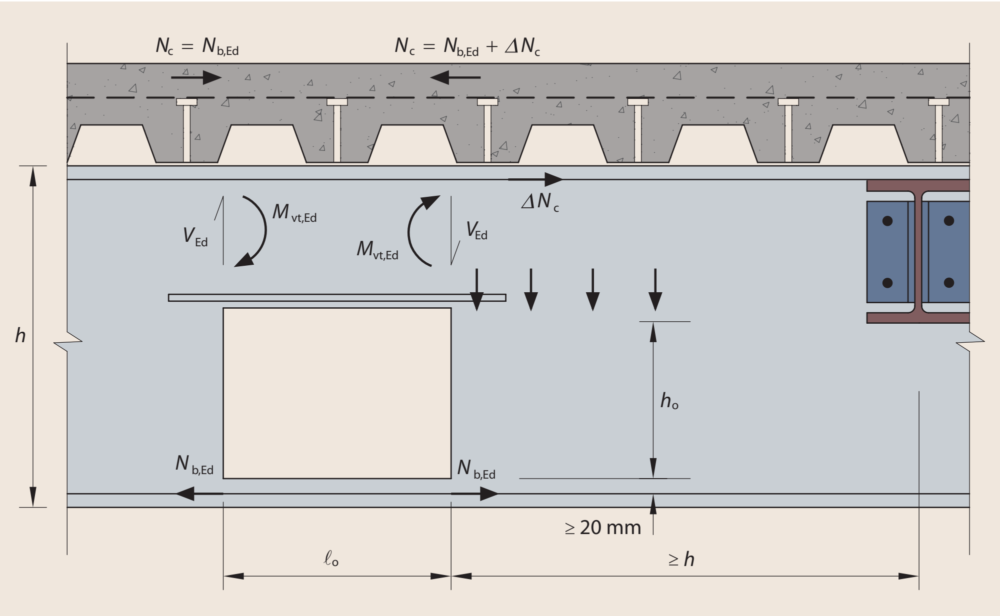


Figure 7.1 Highly asymmetric rectangular opening

7.2 End-posts

The end-post width defines the distance of the edge of the first opening from the end of the beam. The longitudinal shear and buckling resistance of the end-post depends on the type of connection that is used and on notching of the ends of the beam.

Full depth end-plates are preferred for primary beams supporting high shear forces and for connections to columns. In this case, the end-plate transfers shear forces smoothly into the web and partially restrains the web against buckling. Therefore, the stability of the end-post is probably greater than the first web-post between openings.

Conversely, fin plates, web cleats and partial depth end plates offer much less restraint to the end-post. Furthermore, secondary beams are often notched at their top flange, which will increase the effective length of the end-post, certainly for non-composite beams.

It is not possible to give definitive guidance on the design of end-posts in composite beams with web openings, depending on the connection type. It is recommended that the minimum width of the end-post is not less than $0.5h_o$, for a circular opening where h_o is the diameter of the first opening and, for a rectangular opening, not less than its length, ℓ_o.

For cellular beams with multiple openings, a conservative approach is to ignore composite action at the first opening, which will increase the longitudinal shear forces used in the design checks for first web-post. Also, the final opening can be half-filled to increase the width of the end-post. As noted above, a full depth end plate connection will stiffen the end-post locally.

7.3 Notched beams

Large notches are sometimes cut in the beam adjacent to a column to allow the passage of services, etc. The notch is formed by cutting away the bottom flange and web adjacent to the support, as illustrated in Figure 7.2. Generally a horizontal stiffener is provided to increase the local bending resistance of the reduced section.

For analysis purposes, there is a similarity between the local bending at the notch and the Vierendeel bending around openings; here, the Vierendeel bending resistance reduces due to tension arising from local composite action between the slab and the top Tee. The requirement may be expressed as:

$$V_{Ed}\ell_o \quad \leq M_{tT,NV,Rd} + M_{vc,Rd} \qquad (68)$$

where:

$M_{tT,NV,Rd}$ is the bending resistance of the top Tee, reduced for axial compression and for shear

$M_{vc,Rd}$ is the local composite Vierendeel moment resistance, as defined in Section 3.4.6.

Recommended dimensional limits for notched beams are shown in Figure 7.2 in order to avoid local deformation at the notch.

Also, it is recommended that the first adjacent opening is located at a distance not less than the notch length from the end of the notch.

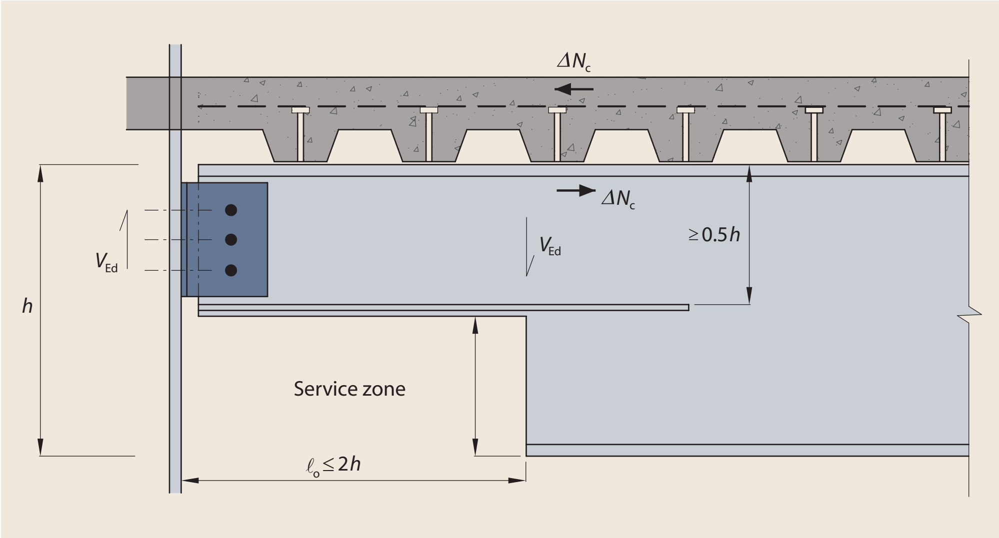


Figure 7.2 Notched beam showing proposed dimensional limits

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CREDITS

Cover Detail of openings in beam
Broadgate Tower, London

xii Broadgate Tower, London

1 London School of Economics
renovation, London

2 Longacre, London

2 Belgrave House, London

2 Commerzbank, Frankfurt

3 Web opening test,
Warwick University

3 Large web opening

56 Kings Place, London

80 Bishops Square, London

APPENDIX A

Design example for composite beam with web openings

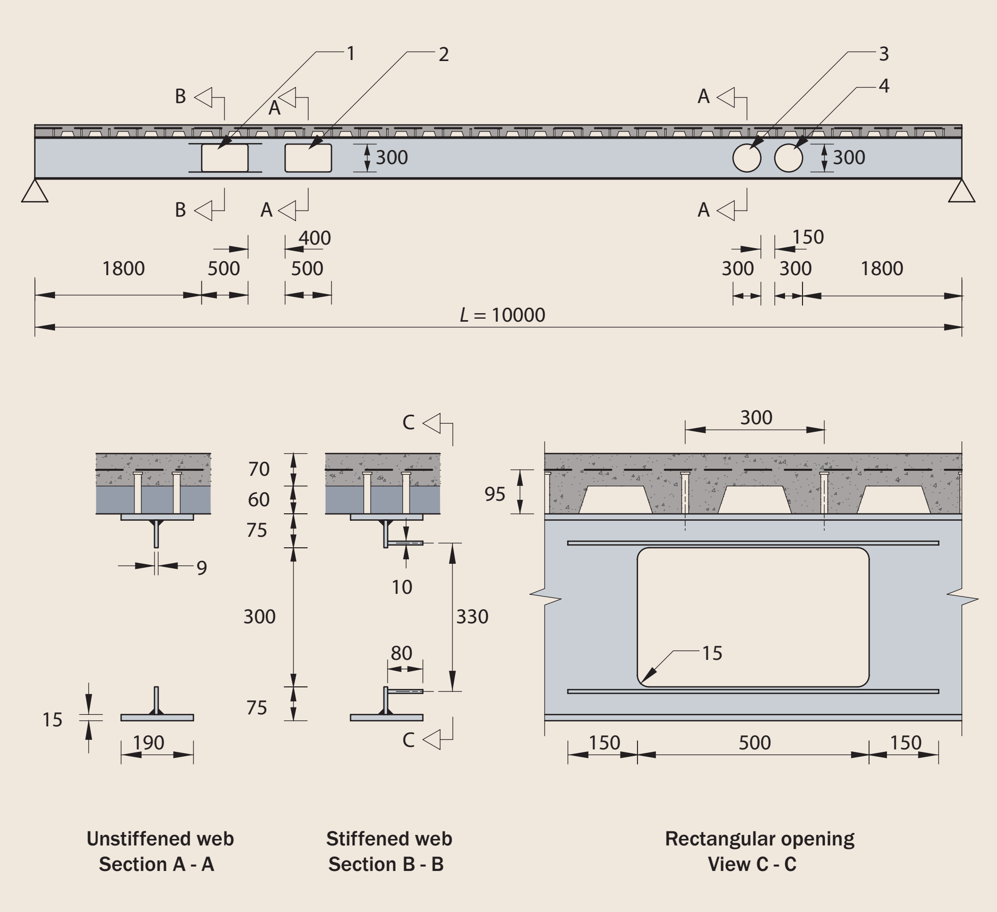
The worked example is carried out for a composite beam with circular and rectangular openings to illustrate the use of the design method presented in this publication.

Contents list for worked example

1 Scope

Verify the adequacy of a 10 m span composite secondary beam with web openings in accordance with BS EN 1993-1-1 and BS EN 1994-1-1. The beam is subject to a uniform imposed load of 5.0 kN/m^2. The beam has two circular openings on one side of mid-span and two rectangular openings, which may need to be stiffened. The adequacy in both normal and construction stages is to be verified.

The chosen beam size is 457 × 191 × 74 kg/m UKB in S355 steel and the configuration is shown below.



The design aspects covered in this example are:

- Bending resistance of the unperforated section at midspan.
- Bending resistance of the perforated section at holes.
- Vierendeel bending resistance of Tees above and below the openings.
- Resistance of web-post in shear, bending and compression.
- Deflection (as influenced by the openings).
- Verification at construction stage.

2 Dimensions and properties

2.1 General dimensions

Beam span	L	$= 10$ m
Beam spacing	b	$= 3$ m
Slab depth	h_s	$= 130$ mm
Decking depth	h_d	$= h_p = 60$ mm (assume simple trapezoidal profile)
Average trough width	b_0	$= 160$ mm
Trough pitch		$= 300$ mm
Decking thickness	t	$= 0.9$ mm
Slab depth above profile	h_c	$= 70$ mm
Position of mesh reinforcement	35 mm below surface of slab	
Shear studs (2 per trough)	19 mm diameter, 100 mm overall nominal height	
Transverse spacing of studs:	100 mm	

2.2 Openings

Two rectangular openings, 300 × 500 mm, edge-to-edge spacing 400 mm

Two circular openings, 300 mm diameter, edge-to-edge spacing 150 mm

2.3 Section properties of beam

Section properties P363

Beam size 457 × 191 × 74 kg/m UKB:

h	$= 457$ mm	A	$= 94.6$ cm^2
b_f	$= 190$ mm	I_y	$= 33300$ cm^4
t_f	$= 14.5$ mm	$W_{pl,y}$	$= 1650$ cm^3
t_w	$= 9.0$ mm		
r	$= 10.2$ mm		

2.4 Material properties

3-1-1/NA

Steel grade S355	f_y	$= 355$ N/mm^2
Concrete class C30/37	f_{ck}	$= 30$ N/mm^2 (normal weight concrete)
Reinforcement	A252 mesh to BS 4483	
Shear studs	19 mm diameter, Type SD1 to BS EN ISO 13918	
	f_u	$= 450$ N/mm^2

3 Actions

3.1 Construction stage

Permanent actions

Self weight of beam	0.75 kN/m	≡ 0.25 kN/m^2
Self weight of decking	0.1 kN/m^2	
Self weight of mesh	0.05 kN/m^2	
Total permanent actions	g_k	= 0.4 kN/m^2

Variable actions

Weight of wet concrete	2.55 kN/m^2	(25 kN/m^3 × 0.102 m^2/m width)
Construction load	0.75 kN/m^2	
Total variable actions	q_k	= 3.3 kN/m^2

3.2 Composite stage

Permanent actions

Self weight of beam	0.25 kN/m^2	
Self weight of decking	0.1 kN/m^2	
Self weight of mesh	0.05 kN/m^2	
Self weight of slab	2.45 kN/m^2	(24 kN/m^3 × 0.102 m^2/m width)
Finishes, services etc.	1.3 kN/m^2	
Total permanent actions	g_k	= 4.15 kN/m^2

Variable actions

Imposed load	q_k	= 5.0 kN/m^2

3.3 Partial factors for actions

BS EN 1990 Table NA.A1.2(B)

Partial factor for permanent actions	γ_G	= 1.35
Partial factor for variable actions	γ_Q	= 1.50
Reduction factor	ξ	= 0.925

3.4 Design values of combined actions

(6.10) Construction stage $\gamma_G\, g_k + \gamma_Q\, q_k$

$= 0.4 \times 1.35 + 3.3 \times 1.5 = 5.49$ kN/m^2

(6.10b)	Composite stage	$\gamma_G \xi g_k + \gamma_Q q_k$	
		$= 1.35 \times 0.925 \times 4.15 + 1.5 \times 5.0$	$= 12.68$ kN/m^2

3.5 Design bending moments and shear forces

Construction stage

Maximum shear force	V_{Ed}	$= 5.49 \times 3 \times 10/2$	$= 82.4$ kN
Maximum moment (mid-span)	M_{Ed}	$= 5.49 \times 3 \times 10^2/8$	$= 206$ kNm

Composite stage

Maximum shear force	V_{Ed}	$= 12.68 \times 3 \times 10/2$	$= 190$ kN
Maximum moment (mid-span)	M_{Ed}	$= 12.68 \times 3 \times 10^2/8$	$= 476$ kNm

4 Shear connection

4.1 Shear connector resistance

Shear resistance of a headed stud connector is the smaller of:

4-1-1/§6.6.3.1

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

$P_{Rd} = 0.29 \alpha d^2 \sqrt{f_{ck} E_{cm}} / \gamma_v$

2-1-1/ Table 3.1

where:

$E_{cm} = 33$ kN/m^2

Appendix 2.4 — $f_u = 450$ N/mm^2

4-1-1/NA.2.9 — $\gamma_v = 1.25$

$h_{sc}/d = 100/19 = 5.3$ (>4) Therefore, $\alpha = 1$

$$P_{Rd} = \frac{0.8 \times 450 \times \pi \times 19^2 / 4}{1.25} \times 10^{-3} = 81.7 \text{ kN, or}$$

$$P_{Rd} = \frac{0.29 \times 1 \times 19^2 \sqrt{30 \times 33 \times 1000}}{1.25} \times 10^{-3} = 83.3 \text{ kN}$$

Therefore, the shear resistance of a shear connector in a solid slab is $P_{Rd} = 81.7$ kN.

4-1-1/§6.6.4.2

Reduction factor due to decking profile – decking ribs transverse to the supporting beams:

Eq. (9)

$$k_t = \left(\frac{0.7}{\sqrt{n_r}}\right)\left(\frac{b_0}{h_p}\right)\left(\frac{h_{sc}}{h_p} - 1\right)$$

$$= \left(\frac{0.7}{\sqrt{2}}\right)\left(\frac{160}{60}\right)\left(\frac{100}{60} - 1\right) = 0.88$$

BS EN 1994-1-1 Table 6.2

However, the upper limit for k_t is 0.7 when $n_r = 2$, and sheet thickness ≤ 1.0 mm. Therefore, $k_t = 0.7$ and the shear resistance of a single shear connector, when placed in pairs per decking rib, is:

$P_{Rd} \quad = 81.7 \times 0.7 = 57.2$ kN

Document PN001 would apply a further reduction on account of the position of the mesh relative to the heads of the studs but that reduction is not applied here.

4.2 Number of shear connectors

To the point of maximum moment (mid-span)

Number of connector spacings over full span (allowing 300 mm for end distances):

$\frac{10000 - 2 \times 300}{300} = 31.33$ or 31 (rounded)

Section 1.8

Therefore, number of connectors in half span (between zero and maximum moment)

$n \quad = (15 + 1) \times 2 = 32$

At rectangular openings 1 and 2

Section 3.2.4

Number of spacings to centreline of rectangular opening 1:

$= (2050 - 300)/300 = 5$

Therefore, number of shear connectors $n_{sc,1} \quad = 6 \times 2 = 12$

Similarly $n_{sc,2} \quad = 9 \times 2 = 18$

Over openings 1 and 2

Number of spacings over a rectangular opening $= \frac{500}{300} = 1.67$

but, $n_{sc,o1}$ and $n_{sc,o2}$ may be taken as the decimal number rather than the nearest integer. For pairs of shear connectors, $n_{sc,o} = 3.3$.

Between rectangular openings

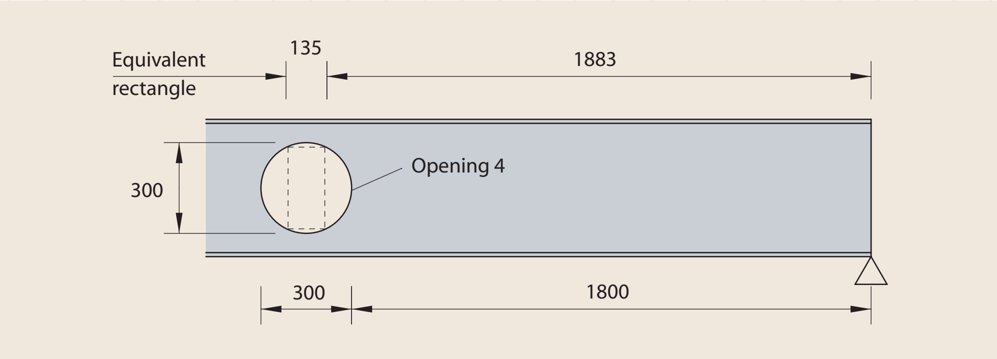
Section 3.4.6

Number of spacings between rectangular openings:

$n_{sc,1\text{-}2} \quad = \frac{900}{300} = 3$ (centre-centre)

Therefore, take number of shear connectors between centreline of openings,

$n_{sc,1\text{-}2} \quad = 3 \times 2 = 6$



At circular openings 3 and 4

Eq. (4) Equivalent rectangular opening length $\ell_e = 0.45h_o = 0.45 \times 300 = 135$ mm

Section 3.2.4 Number of spacings from support to centreline of opening 4, ignoring first 300 mm from support

$= 1800 + 300/2 - 300)/300 = 5.5$

Therefore $n_{sc,4} = 2 \times 6 = 12$

Similarly $n_{sc,3} = 2 \times 8 = 16$

Between circular openings 3 and 4

Number of spacings between circular openings 3 and 4 (centre to centre)

$$= \frac{450}{300} = 1.5$$

Therefore, number of shear connectors between openings $n_{sc,3\text{-}4} = 1.5 \times 2 = 3$

Over openings 3 and 4

Number of spacings over circular opening $= \dfrac{135}{300} < 1$

Conservatively for the purposes of this analysis, assume that, no shear connectors are placed over the circular openings, i.e. $n_{sc,o4} = n_{sc,o3} = 0$.

4.3 Detailing of shear connection

4-1-1/§6.6.5.7(4) Transverse spacing of studs $b_0 = 100$ mm (Appendix 2.1) $> 4d = 76$ mm

Nominal height of stud above shoulder of profile $= 100 - 60 = 40$ mm

4-1-1/§6.6.5.8 Required minimum height $= 2d = 2 \times 19 = 38$ mm OK

5 Bending resistance at mid-span (composite stage)

Section 1.7 Effective width of compression flange at mid-span, ignoring spacing of studs b_0:

b_{eff} $= \text{span}/4 = 2.5 \text{ m} < 3 \text{ m}$ (beam spacing)

Compressive resistance of slab

$$N_{c,s,Rd} = \frac{0.85 f_{ck} b_{eff} h_c}{\gamma_c} = \frac{0.85 \times 30 \times 2500 \times 70}{1.5} \times 10^{-3} = 2975 \text{ kN}$$

Tensile resistance of steel section

$$N_{a,Rd} = \frac{A f_y}{\gamma_{M0}} = \frac{9460 \times 355}{1.0} \times 10^{-3} = 3358 \text{ kN}$$

Since $N_{a,Rd} > N_{c,s,Rd}$, the plastic neutral axis lies in the steel section.

Degree of shear connection

Section 1.8 $N_{c,max} = nP_{Rd} = 32 \times 57.2 = 1830 \text{ kN}$

The degree of shear connection at mid-span is:

Eq. (1) $$\eta = \frac{N_{c,max}}{N_{c,s,Rd}} = \frac{1830}{2975} = 0.62$$

4-1-1/§6.6.1.2 Limit on degree of shear connection:

For an equivalent span L_e less than 25 m, the limit of η for symmetric sections with pairs of shear connectors is:

4-1-1/(6.12) $$\eta \geq 1 - \left(\frac{355}{f_y}\right)(0.75 - 0.03 L_e) \geq 0.4$$

$$\eta \geq 1 - \left(\frac{355}{355}\right)(0.75 - 0.03 \times 10) = 0.55$$

Since the actual degree of shear connection is 0.62, which exceeds 0.55, the minimum degree of shear connection is satisfied.

Bending resistance for full shear connection

Tensile resistance of steel flange:

$$N_{f,Rd} = \frac{b_f \, t_f \, f_y}{\gamma_{M0}} = \frac{190 \times 14.5 \times 355}{1.0} \times 10^{-3} = 978 \text{ kN}$$

Tensile resistance of web:

$$N_{w,Rd} = N_{a,Rd} - 2N_{f,Rd} = 3358 - 2 \times 978 = 1402 \text{ kN}$$

Since $N_{a,Rd} > N_{cm,Rd}$ and $N_{w,Rd} < N_{cm,Rd}$ the plastic neutral axis lies in the steel flange.

Taking moments about the top flange:

$$M_{pl,Rd} \approx N_{cm,Rd}(h_s + h_p)/2 + N_a h/2$$

$$= [2975 \times (130 + 60)/2 + 3358 \times 457/2\,] \times 10^{-3} = 1050 \text{ kNm}$$

Bending resistance for partial shear connection

4-1-1/§6.2.1.3 For partial shear connection, the bending resistance of the composite section is obtained by interpolation between that of the steel section and that of the composite section with full shear connection.

Bending resistance of steel section:

Appendix 2 (for properties of steel section)

$$M_{pl,a,Rd} = W_{pl,y}\frac{f_y}{\gamma_{M0}} = 1650 \times 355 \times 10^{-3} = 586 \text{ kNm}$$

4-1-1/§6.2.1.3 Use linear interpolation:

Eq. (3)

$$M_{Rd} = M_{pl,a,Rd} + \eta(M_{pl,Rd} - M_{pl,a,Rd})$$

$$= 586 + 0.62 \times (1050 - 586) = 873 \text{ kNm}$$

Appendix 3.5 Since $M_{Rd} = 873$ kNm $> M_{Ed} = 476$ kNm, the design bending resistance at mid-span is satisfactory.

6 Design of beam at circular openings 3 and 4 (composite stage)

6.1 Geometric data

Centres of the openings are at $x_{0,4} = 1.95$ m and $x_{0,3} = 2.4$ m from the nearest end of the beam.

Diameter of opening $= 300$ mm

Height of equivalent rectangular opening, $h_{eo} = 0.9h_o = 0.9 \times 300 = 270$ mm

Eq. (4) Length of equivalent rectangular opening for Vierendeel bending:

$$\ell_e = 0.45 \times 300 = 135 \text{ mm}$$

Eq. (15) Effective length of opening for section classification of outstand of Tees:

$\ell_{o,eff} = 0.7h_o = 0.7 \times 300 = 210$ mm

Area of each of the two Tees:

$A_T = (A - h_o t_w)/2 = (9460 - 270 \times 9.0)/2 = 3515\ \text{mm}^2$

Depth of Tee $h_T = (h - h_{eo})/2 = (457 - 270)/2 = 93.5$ mm

Depth of web of Tee $h_{w,T} = h_T - t_f = 93.5 - 14.5 = 79$ mm

Depth of elastic neutral axis of Tee from outer face of flange is given by:

$$z_{el} = \frac{b_f t_f^2/2 + \left(t_f + h_{w,T}/2\right) h_{w,T}\, t_w}{A_T}$$

Neglecting the fillets of the Tee:

$$z_{el} = \frac{190 \times 14.5^2/2 + (14.5 + 79/2) \times 79 \times 9.0}{190 \times 14.5 + 79 \times 9.0} = 17\ \text{mm}$$

Effective depth between centroids of Tees, $h_{eff} = 457 - 2 \times 17 = 423$ mm

6.2 Design effects

Bending moment at the centreline of opening 4 ($x_{0,4} = 1.95$ m) is:

$M_{Ed} = 190 \times 1.95 - 12.69 \times 3 \times 1.95^2/2 = 299$ kNm

Similarly, the moment at opening 3 is $M_{Ed} = 347$ kNm

Shear force at edge of equivalent opening 4 (at $x_{0,4} - 0.135/2$) is:

$V_{Ed} = 190 - 12.69 \times 3 \times (1.95 - 0.135/2) = 119$ kN

Shear force midway between the openings: (at $x_{3\text{-}4} = 2.175$) is:

$V_{Ed} = 190 - 12.69 \times 3 \times 2.175 = 107$ kN

6.3 Section classification

The top flange is Class 2, due to its attachment to the slab (shear connectors in mid-outstand at 300 mm spacing along the beam).

Classification of web outstand of the top Tee in Vierendeel bending (ignoring axial compression):

For the web to be Class 2, independent of its depth:

Table 3.2 $\ell_{o,eff} < 32\varepsilon t_w$

where:

$$\varepsilon = \sqrt{235/f_y} = \sqrt{235/355} = 0.81$$

Therefore $32\varepsilon t_w = 32 \times 0.81 \times 9.0 = 233 \text{ mm} > \ell_{o,eff} = 210 \text{ mm}$

The Tee is Class 2 in Vierendeel bending.

The bottom Tee is thus also Class 2, irrespective of the tension force in the Tee (which would improve the classification).

6.4 Bending resistance at centreline of circular opening

The tensile resistance of the bottom Tee is given by:

Eq. (5)

$$N_{bT,Rd} = \frac{A_{bT} f_y}{\gamma_{M0}} = \frac{3515 \times 355}{1.0} \times 10^{-3} = 1250 \text{ kN}$$

Eq. (6) The compression resistance of the effective width of slab at the opening is given by:

$$N_{c,Rd} = \min\left\{0.85 f_{cd} b_{eff,o} h_c \,;\; n_{sc} P_{Rd}\right\}$$

Eq. (11) Neglecting b_0, the effective width is given by:

$$b_{eff,o} = 3L_e/16 + x/4 \quad \text{for } x \leq L_e/4$$

Distance from end support to centre of opening 4, $x_{0,4} = 1950 \text{ mm}$

$$b_{eff,o} = 3 \times 10000/16 + 1950/4 \qquad = 2363 \text{ mm}$$

Appendix 4.2

$$n_{sc,4} = 12$$

Thus:

$$N_{c,Rd} = \min\left\{0.85 \times (30/1.5) \times 2363 \times 70; \quad 12 \times 57.2\right\}$$
$$= \min\left\{2811; \quad 686\right\} = 686 \text{ kN}$$

The p.n.a. is therefore in the flange of the top Tee (Case 2 in Section 3.2.2). The plastic bending resistance is therefore given by:

Eq. (9)

$$M_{c,Rd} = N_{bT,Rd} h_{eff} + N_{c,Rd}\left(z_t + h_s - 0.5h_c\right)$$

Appendix 6.1

$$z_t = z_{el} = 17 \text{ mm}$$

$$M_{c,Rd} = \left[1250 \times 423 + 686 \times (17 + 130 - 0.5 \times 70)\right] \times 10^{-3} = 605 \text{ kNm}$$

This bending resistance is adequate at both opening 4 and opening 3.

For the consideration of coexistent global bending, Vierendeel bending and shear at opening 4, the design tension force in the bottom Tee is required.

For Case 2, the value of $N_{bT,Ed}$ is given by the following (unless this exceeds $N_{c,Rd}$, see Section 3.2.2)

Eq. (8) $$N_{bT,Ed} = \frac{M_{Ed}}{(h_{eff} + z_t + h_s - 0.5z_c)}$$

$$N_{bT,Ed} = \frac{299}{(423 + 17 + 130 - 0.5 \times 70)} \times 10^3 = 559 \text{ kN}$$

The compression resistance of the slab is 686 kN and thus there is no requirement for the top Tee to resist axial compression at this opening. (If the tension force in the bottom Tee were greater than the slab resistance, then equation (10) would be used to determine the force in the bottom Tee and thus the compression in the top Tee.)

6.5 Shear resistance of perforated composite beam section

The design shear resistance is the sum of the resistances of the top and bottom Tees and the concrete slab:

$$V_{Rd} = V_{t,Rd} + V_{b,Rd} + V_{c,Rd}$$

Plastic shear resistance of a Tee section is given by:

3-1-1/§6.2.6 $$V_{pl,Rd} = \frac{A_v \, f_y / \sqrt{3}}{\gamma_{M0}}$$

In which $A_v = A - b_f t_f + (2r + t_w) \times 0.5 \times t_f$

For plastic shear resistance, consider the full height of the circular opening, for which the area is given by:

$$A_T = (A - h_o t_w)/2 = (9460 - 300 \times 9.0)/2 = 3380 \text{ mm}^2$$

$$A_v = 3380 - 190 \times 14.5 + (2 \times 10.2 + 9.0) \times 7.25 = 838 \text{ mm}^2$$

$$V_{t,Rd} = V_{b,Rd} = \frac{838 \times 355 / \sqrt{3}}{1.0} = 172 \text{ kN}$$

Shear resistance of the concrete slab at an opening:

Eq. (12) $$V_{c,Rd} = [C_{Rd,c}\, k\, (100\rho_l f_{ck})^{1/3} + k_1 \sigma_{cp}] b_w d$$

With a minimum value of:

Eq. (13) $V_{c,Rd} = [v_{min} + k_1\sigma_{cp}]\, b_w d$

Effective width of the slab $b_w = b_f + 2h_{s,eff}$

$h_{s,eff} = 0.75h_s = 0.75 \times 130 = 97.5$ mm

$b_w = 190 + 2 \times 97.5 = 385$ mm

2-1-1/Table NA.1 (§6.2.2(1)) $C_{Rd,c} = \dfrac{0.18}{\gamma_c} = \dfrac{0.18}{1.5} = 0.12$ and $k_1 = 0.15$

2-1-1/§6.2.2(1) $k = 1 + \sqrt{\dfrac{200}{d}}$ but ≤ 2.0

Section 3.3.2 Effective depth of slab for shear, $d = h_c = 70$ mm

$k = 1 + \sqrt{\dfrac{200}{70}} = 2.69 > 2$ and therefore, $k = 2$

Proportion of reinforcement:

$\rho_1 = \dfrac{A_{sl}}{b_w d} = \dfrac{252}{1000 \times 70} = 0.0035 > 0.02$

The value of the first term in the expression for $V_{c,Rd}$ is:

$C_{Rd,c}k\,(100\rho_1 f_{ck}) = 0.12 \times 2 \times (100 \times 0.0035 \times 30)^{1/3} = 0.52$ N/mm^2

The value of the first term in the expression for the minimum value, v_{min} is:

2-1-1/Table NA.1 (§6.2.2(1)) $v_{min} = 0.035k^{3/2} f_{ck}^{1/2} = 0.035 \times 2^{3/2} \times \sqrt{30} = 0.54$ N/mm^2

So, use 0.54 N/mm^2.

From above (Appendix 6.5), assuming no compression in the top Tee, the force in the slab at the opening equals the force in the bottom Tee = 556 kN.

Appendix 6.4 Compression force acts over an effective width of $b_{eff} = 2363$ mm, and so:

$A_c = h_c b_w = 70 \times 1876 = 131 \times 103$ mm^2

$\sigma_{cp} = \dfrac{556 \times 10^3}{2363 \times 70} = 3.36$ N/mm^2

$k_1\sigma_{cp} = 0.15 \times 3.36 = 0.50$ N/mm^2

Total shear strength of concrete is:

Hence:

$V_{c,Rd} = (0.54 + 0.50) \times 385 \times 70 \times 10^{-3} = 28$ kN

Section 3.4.3

Hence:

$V_{Rd} = V_{c,Rd} + V_{t,Rd} + V_{b,Rd} = 28 + 172 + 172 = 372$ kN

This exceeds the design shear force at opening 4, $V_{Ed} = 119$ kN.

6.6 Bending resistance of Tees

Assumed distribution of shear force

Initially, assume that 50% of the shear force V_{Ed} is resisted in each Tee.

Since the shear force in each Tee (119/2 = 59.5 kN) is less than $0.5V_{pl,Rd}$ the web thickness does not need to be reduced when determining plastic bending resistance and axial resistance.

If the shear force in the bottom Tee is limited by Vierendeel bending resistance across the Tee, the shear forces may need to be redistributed.

Plastic bending resistance

For a Class 2 cross section, the plastic bending resistance of an unstiffened Tee, in the absence of axial force and high shear is given by:

Eq. (19)

$$M_{pl,Rd} = \frac{A_{w,T} f_y}{\gamma_{M0}}\left(0.5h_{w,T} + t_f - z_{pl}\right) + \frac{A_f f_y}{\gamma_{M0}}\left(0.5t_f - z_{pl} + z_{pl}^2/t_f\right)$$

The depth of the plastic neutral axis of the Tee from the outer surface of the flange is given by:

$$z_{pl} = \frac{\left(A_f + A_{w,T}\right)}{2b_f} = \frac{190 \times 14.5 + 79 \times 9.0}{2 \times 190} = 9.1 \text{ mm}$$

Hence:

$$M_{pl,Rd} = \frac{79 \times 9 \times 355}{1.0}\left(0.5 \times 79 + 14.5 - 9.1\right) \times 10^{-6} + \frac{190 \times 14.5 \times 355}{1.0}\left(0.5 \times 14.5 - 9.1 + 9.1^2/14.5\right) \times 10^{-6} = 15.1 \text{ kNm}$$

The plastic bending resistance of the bottom Tee is reduced for axial tension, as follows:

Eq. (26)

$$M_{bT,N,Rd} = M_{pl,N,Rd} = M_{pl,Rd}\left[1 - \left(\frac{N_{Ed}}{N_{pl,Rd}}\right)^2\right]$$

Appendix 6.4

in which $N_{Ed} = 559$ kN and $N_{pl,Rd} = N_{bT,Rd} = 1250$ kN

Therefore, $M_{bT,N,Rd} = 15.1 \times (1 - (559/1250)^2) = 12.1$ kNm

The plastic resistance of the top Tee is not reduced for axial force and thus:

$$M_{tT,N,Rd} = 15.1 \text{ kNm}$$

Composite bending resistance

As in the worst case, there are no shear connectors over the circular openings, thus no local composite action is developed. Therefore the component of Vierendeel bending resistance due to composite action is neglected (i.e. $M_{vc,Rd} = 0$).

6.7 Verification of resistance to Vierendeel bending

The criterion for adequacy of Vierendeel bending resistance is:

Eq. (14)

$$2M_{bT,NV,Rd} + 2M_{tT,NV,Rd} + M_{vc,Rd} \geq V_{Ed}\,\ell_e$$

Thus, using the above values for the Tees, the criterion is:

$$2 \times 12.1 + 2 \times 15.1 + 0 = 54.2 \text{ kNm} > 119 \times 0.135 = 16.1 \text{ kNm}$$ OK

6.8 Web-post shear, bending and buckling between openings 3 and 4

For closely spaced circular openings, the resistance of the web-post in shear, bending or buckling might govern. Each of these resistances needs to be verified. (But web-post bending will only be critical when the distribution of shear between the Tees is radically different.)

Web-post bending resistance

Elastic bending resistance of web-post (at mid-height between circular openings):

Eq. (44)

$$M_{wp,Rd} = s_o^{\,2} t_w f_y / 6 = (450–300)^2 \times 9.0 \times 355 \times 10^{-6}/6 = 11.9 \text{ kNm}$$

Since the Vierendeel bending resistance was verified above for equal shear force in each of the Tees, with no shear force in the slab, the web-post moment at mid-height $M_{wp,Ed} = 0$. Hence verification of bending resistance at the top and bottom of the openings is not needed for circular openings.

Web-post shear resistance

Check whether there is sufficient shear connection to develop a force in the slab equal to the incremental force in the bottom Tee.

There is sufficient connection if $V_{wp.Ed} \leq \Delta N_{cs,Rd}$

Eq. (29)

$$V_{wp,Ed} = \frac{V_{Ed}\, s}{\left(h_{eff} + z_t + h_s - 0.5h_c\right)}$$

Taking the value of V_{Ed} mid-way between the openings:

$$V_{wp,Ed} = \frac{107 \times 450}{\left(423 + 17 + 130 - 0.5 \times 70\right)} = 90 \text{ kN}$$

$$\Delta N_{cs,Rd} = n_{sc,s} P_{Rd}$$

Appendix 4.2

For $n_{sc,3\text{-}4} = 3$:

$$\Delta N_{cs,Rd} = 3 \times 57.2 = 171 \text{ kN}$$

So there is sufficient shear connection between the centrelines of the openings.

Shear resistance of web-post

The longitudinal shear resistance of the web-post is given by:

Eq. (36)

$$V_{wp,Rd} = \frac{\left(s_o t_w\right) f_y / \sqrt{3}}{\gamma_{M0}} = 150 \times 9.0 \times 0.577 \times 355 \times 10^{-3}$$

$$= 276 \text{ kN} > V_{wp,Ed} = 90 \text{ kN}$$

Web-post buckling resistance

Web-post buckling may be treated by considering the compressive stress due to an effective longitudinal force in the web-post.

Eq. (33)

Since the openings are placed centrally in the web depth and the web-post moment required at mid-height $M_{wp,Ed} = 0$, $N_{wp,Ed} = V_{wp,Ed}$.

Hence $N_{wp,Ed} = 90$ kN.

Section 3.5.3

Edge-to-edge distance between openings 3 and 4 = 150 mm, which is less than h_o and thus the openings are classed as 'closely spaced'.

Non-dimensional slenderness ratio of web-post:

Eq. (40)

$$\bar{\lambda} = \frac{1.75\sqrt{s_o^2 + h_o^2}}{t_w} \frac{1}{\lambda_1}$$

$$\lambda_1 = 76 \text{ for S355}$$

$$\bar{\lambda} = \frac{1.75\sqrt{150^2 + 300^2}}{9.0} \frac{1}{76} = 0.86$$

The reduction factor for flexural buckling is calculated using the factor ϕ, given by:

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

3-1-1/§6.3.1.2 For a welded section, use buckling curve c, for which $\alpha = 0.49$

Thus $\phi = 0.5(1 + 0.49(0.86 - 0.2) + 0.86^2) = 1.03$

The reduction factor χ is given by:

$$\chi = \frac{1}{\phi + \left(\phi^2 - \bar{\lambda}^2\right)^{0.5}}$$

$$= \frac{1}{1.03 + \left(1.03^2 - 0.86^2\right)^{0.5}} = 0.63$$

Eq. (42) $$N_{wp,Rd} = \chi \frac{s_o t_w f_y}{\gamma_{M1}}$$

$$N_{wp,Rd} = \frac{0.63 \times 150 \times 9 \times 355}{1.0} \times 10^{-3} = 302 \text{ kN} > N_{wp,Ed} = 90 \text{ kN}$$ OK

7 Design of beam at rectangular openings 1 and 2

7.1 Geometric data

Centres of the openings are at $x_{0,1} = 2.05$ m and $x_{0,2} = 2.95$ m from the nearest end of the beam.

Height of openings $h_o = 300$ mm

Area of each of the two Tees:

$$A_T = (A - h_o t_w)/2 = (9460 - 300 \times 9.0)/2 = 3380 \text{ mm}^2$$

Depth of Tee $h_T = (h - h_o)/2 = (457 - 300)/2 = 78.5$ mm

Depth of web of Tee $h_{w,T} = h_T - t_f = 78.5 - 14.5 = 64$ mm

Depth of elastic neutral axis of Tee from outer face of flange is given by:

$$z_{el} = \frac{b_f t_f^2/2 + \left(t_f + h_{w,T}/2\right) h_{w,T}\, t_w}{A_T}$$

Neglecting the fillets of the Tee:

$$z_{el} = \frac{190 \times 14.5^2/2 + (14.5 + 64/2) \times 64 \times 9.0}{190 \times 14.5 + 64 \times 9.0} = 14.0 \text{ mm}$$

Effective depth between centroids of Tees, $h_{eff} = 457 - 2 \times 14 = 429$ mm

The effective length of the openings for section classification of the Tees is $\ell_{o,eff} = 500$ mm.

7.2 Design effects

Bending moment at centreline of opening 1 ($x_{0,1} = 2050$ mm) is:

M_{Ed} $= 190 \times 2.05 - 12.69 \times 3 \times 2.05^2/2 = 310$ kNm

Similarly, the moment at opening 2 is 395 kNm.

Shear force at the edge of opening 1 (at $x_{0,1} - 0.500/2$) is:

V_{Ed} $= 190 - 12.69 \times 3 \times (2.05 - 0.250) = 121$ kN

Similarly, the shear force at opening 2 is 87 kN.

The shear force midway between the openings is 95 kN.

7.3 Section classification

The top flange is treated as Class 2 due to its attachment to the slab.

Classification of web outstand of the top Tee in Vierendeel bending (ignoring axial compression).

For the unstiffened web to be Class 2, independent of its depth:

Table 3.2 $\ell_{o,eff}$ $< 32\varepsilon t_w$

From Appendix 6.7 this limit is $\ell_{o,eff} = 210$ mm and thus the limiting depth for Class 2 has to be determined.

The limiting depth of the web of the Tee is given by:

$$\frac{10 t_w \varepsilon}{\sqrt{1-\left(\frac{32 t_w \varepsilon}{\ell_{o,eff}}\right)^2}} = \frac{10 \times 9.0 \times 0.81}{\sqrt{1-\left(\frac{32 \times 9.0 \times 0.81}{500}\right)^2}} = 82 \text{ mm} > h_w = 64 \text{ mm}$$

Therefore the Tee sections at the rectangular openings 1 and 2 are Class 2.

7.4 Bending resistance at centreline of rectangular opening

Unstiffened openings

The tensile resistance of the bottom Tee is given by:

Eq. (5) $N_{bT,Rd}$ $= \dfrac{A_{bT} f_y}{\gamma_{M0}} = \dfrac{3380 \times 355}{1.0} \times 10^{-3} = 1200$ kN

The compression resistance of the effective width of slab at the opening is given by:

Eq. (6) $N_{c,Rd}$ $= \min\left\{0.85 f_{cd} b_{eff,o} h_c \,;\ n_{sc} P_{Rd}\right\}$

Neglecting b_0 the effective width is given by:

Eq. (11) $b_{eff,o} = 3L_e/16 + x/4$ for $x \leq L_e/4$

Distance from end support to centre of opening 1, $x_{0,1} = 2050$ mm

$b_{eff,o} = 3 \times 10000/16 + 2050/4 = 2388$ mm

Appendix 4.2 $n_{sc,1} = 12$

Thus:

$$N_{c,Rd} = \min\{0.85 \times (30/1.5) \times 2388 \times 70; \quad 12 \times 57.2\}$$
$$= \min\{2842; \quad 686\} = 686 \text{ kN}$$

Appendix 4.2 For opening 2, $n_{sc,2} = 18$

The p.n.a. is therefore in the flange of the top Tee (Case 2 in Section 3.2.2). The plastic bending resistance is therefore given by:

Eq. (9)
$$M_{o,Rd} = N_{bT,Rd}\, h_{eff} + N_{c,Rd}\,(z_t + h_s - 0.5h_c)$$

$$M_{o,Rd} = [1200 \times 429 + 686 \times (14 + 130 - 0.5 \times 70)] \times 10^{-3} = 590 \text{ kNm}$$

This bending resistance is adequate at both opening 1 and opening 2.

For the consideration of coexistent global bending, Vierendeel bending and shear at opening 1 (which has higher shear than opening 2), the design tension force in the bottom Tee is required.

For Case 2, the value of $N_{bT,Ed}$ is given by the following (unless this exceeds $N_{c,Rd}$, see Section 3.2.2):

Eq. (8)
$$N_{bT,Ed} = \frac{M_{Ed}}{(h_{eff} + z_t + h_s - 0.5z_c)}$$

$$N_{bT,Ed} = \frac{310}{(429 + 14 + 130 - 0.5 \times 70)} \times 10^3 = 576 \text{ kN at opening 1}$$

The compression resistance of the slab is 686 kN and thus there is no requirement for the top Tee to resist axial compression at this opening. (If the tension force in the bottom Tee were greater than the slab resistance, then equation (10) would be used to determine the force in the bottom Tee and thus the compression in the top Tee.)

By a similar calculation, $N_{bT,Ed} = 734$ kN at opening 2, which is greater than $N_{c,Rd}$ (= 1030 kN, for $n_{sc,2} = 18$), so equation (8) may still be used.

7.5 Shear resistance of perforated composite beam section

Appendix 6.5

As for the circular openings:

$V_{pl,Rd}$ $= 172$ kN (for each Tee)

The effective width of slab for shear is the same as for the circular openings:

b_w $= 385$ mm

Depth of concrete for shear resistance $= 70$ mm

Assuming no stress in the top Tee, the stress in the concrete at opening 1 is given for an effective width of 2388 mm by:

$$\sigma_{cp} = \frac{N_{bT,Ed}}{b_{eff,o}\,d} = \frac{576 \times 10^3}{2388 \times 70} = 3.44 \text{ N/mm}^2$$

Thus $k_1 \sigma_{cp} = 0.15 \times 3.44 = 0.52$ N/mm^2

Hence the concrete shear resistance is:

$V_{c,Rd}$ $= (0.54 + 0.52) \times 385 \times 70 \times 10^{-3} = 28$ kN for opening 1

Similarly, for opening 2, $V_{c,Rd} = 32$ kN

7.6 Bending resistance of unstiffened Tees

Initial assumption of distribution of shear force

Initially, assume that 50% of the shear force V_{Ed} is resisted in each Tee.

Later, the shear force may be re-distributed between the Tees, depending on the shear force that can be transferred by Vierendeel bending of the bottom Tee. The amount of redistribution may also be limited by the need to minimize the web-post moment at mid-height of the opening; the adequacy of shear resistance for an assumption of $M_{Wp,Ed} = 0$ will be verified in Section 7.8.

Since this shear force in each Tee ($121/2 = 60.5$ kN) is less than $0.5V_{pl,Rd}$ (= 172 kN, see Appendix 6.5), the web thickness does not need to be reduced for determining plastic bending resistance and axial resistance.

Plastic bending resistance

For a Class 2 cross section, the plastic bending resistance of an unstiffened Tee, in the absence of axial force and high shear is given by:

Eq. (19)

$$M_{pl,Rd} = \frac{A_{w,T} f_y}{\gamma_{M0}} \left(0.5h_{w,T} + t_f - z_{pl}\right) + \frac{A_f f_y}{\gamma_{M0}} \left(0.5t_f - z_{pl} + z_{pl}^2 / t_f\right)$$

The depth of the plastic neutral axis of the Tee from the outer surface of the flange is given by:

$$z_{pl} = \frac{(A_f + A_{w,T})}{2b_f} = \frac{190 \times 14.5 + 64 \times 9.0}{2 \times 190} = 8.8 \text{ mm}$$

Hence:

$$M_{pl,Rd} = \frac{64 \times 9 \times 355}{1.0}(0.5 \times 64 + 14.5 - 8.8) \times 10^{-6}$$

$$+ \frac{190 \times 14.5 \times 355}{1.0}\left(0.5 \times 14.5 - 8.8 + 8.8^2/14.5\right) \times 10^{-6} = 11.4 \text{ kNm}$$

The plastic bending resistance of the bottom Tee is reduced for axial tension, as follows:

Eq. (20) $$M_{bT,N,Rd} = M_{pl,N,Rd} = M_{pl,Rd}\left[1 - \left(\frac{N_{Ed}}{N_{pl,Rd}}\right)^2\right]$$

Appendix 7.4 N_{Ed} = 576 kN at opening 1

N_{Ed} = 734 kN at opening 2

$N_{b,Rd} = N_b = 1200$ kN

Therefore $M_{bT,N,Rd} = 11.4 \times (1 - (576/1200)^2) = 8.8$ kNm at opening 1

and $M_{bT,N,Rd} = 11.4 \times (1 - (734/1200)^2) = 7.1$ kNm at opening 2

Composite Vierendeel bending resistance

The compression force developed in the slab due to composite action over the opening:

$$\Delta N_{c,Rd} = n_{sc,o} P_{Rd} = 3.3 \times 57.2 = 188 \text{ kN}$$

For unstiffened openings, reduction factor due to the length of the openings, k_o is given by:

Eq. (25) $$k_o = 1 - \ell_o/(25h_t) = 1 - 500/(25 \times 78.5) = 0.75$$

where h_t = depth of top Tee = 78.5 mm.

Vierendeel bending resistance due to local composite action is given by:

Eq. (24) $$M_{vc,Rd} = \Delta N_{c,Rd}(h_s + z_t - 0.5\, h_c) k_o$$

$$= 188 \times (130 + 14 - 35) \times 10^{-3} \times 0.75 = 15.4 \text{ kNm}$$

Verification of Vierendeel bending resistance

Eq. (14) The criterion for adequacy of Vierendeel bending resistance is:

$$2M_{bT,NV,Rd} + 2M_{tT,NV,Rd} + M_{vc,Rd} \geq V_{Ed}\,\ell_e$$

Thus, using the above values for the Tees, the total Vierendeel resistances are:

At opening 1:

$$2 \times 8.8 + 2 \times 11.4 + 15.4 = 55.6 \text{ kNm}$$

At opening 2:

$$2 \times 7.1 + 2 \times 11.4 + 15.4 = 52.4 \text{ kNm}$$

The design values of the Vierendeel moments are:

$$M_v = V_{Ed}\ell_o = 121 \times 0.5 = 60.5 \text{ kNm at opening 1 } (> 55.6 \text{ kNm})$$

$$M_v = V_{Ed}\ell_o = 87 \times 0.5 = 43.5 \text{ kNm at opening 2 } (< 52.4 \text{ kNm})$$

Opening 1 will therefore need to be stiffened.

Opening 2 does not need to be stiffened.

In mobilizing the full Vierendeel bending resistance of the top Tee, it will resist a shear of $2 \times 11.4/0.500 = 45.6$ kN. This is only 27% of the plastic shear resistance, so there is no need to reduce the bending resistance for coexistent shear stress.

7.7 Bending resistance of stiffened opening 1

Section properties for stiffened Tees

A 80 × 10 flat plate is welded to both Tees such that there is a clear distance of 10 mm from the edge of the opening to the face of the web. The centroid of the flat is thus 63.5 mm from the outer face of the flange.

The outstand ratio of the plate $= 80/10 = 8$, which is within the class 2 limit for outstands (i.e. $10\varepsilon = 8.1$ for S355).

Area of stiffener $A_r = 80 \times 10 = 800 \text{ mm}^2$

The depth of the elastic neutral axis from the outer face of the flange will have increased, due to the addition of the stiffener, and is now:

$$z_{el} = \frac{14.0 \times 3380 + 800 \times 63.5}{3380 + 800} = 24 \text{ mm}$$

The distance between the centroids of the two stiffened Tees is:

$$h_{eff} = 457 - 2 \times 24 = 409 \text{ mm}$$

Since the area of the web ($64 \times 9 = 576$ mm^2) plus the stiffener, is less than that of the flange ($190 \times 14.5 = 2755$ mm^2) the plastic axis of the Tee is in the flange and the depth of the p.n.a. is given by:

$$z_{pl} = \frac{190 \times 14.5 + 64 \times 9.0 + 80 \times 10}{2 \times 190} = 10.9 \text{ mm}$$

Tensile resistance of stiffened bottom Tee:

$$N_{bT,Rd} = 1200 + 80 \times 10 \times 355 \times 10^{-3} = 1484 \text{ kN}$$

The plastic resistance of the Tee is:

Eq. (57)

$$M_{pl,Rd} = \frac{A_{w,T} f_y}{\gamma_{M0}}\left(\frac{h_{wt}}{2} + t_f - z_{pl}\right) + \frac{A_f f_y}{\gamma_{M0}}\left(0.5t_f - z_{pl} + z_{pl}^2/t_f\right) + \frac{A_r f_y}{\gamma_{M0}}\left(t_f + h_{w,T} - e_r - z_{pl}\right)$$

$$M_{pl,Rd} = \left[\frac{64 \times 9 \times 355}{1.0}\left(\frac{64}{2} + 14.5 - 10.9\right) + \frac{190 \times 14.5 \times 355}{1.0}\left(\frac{14.5}{2} - 10.9 + \frac{10.9^2}{14.5}\right) + \frac{80 \times 10 \times 355}{1.0}(14.5 + 64 - 15 - 10.9)\right] \times 10^{-6}$$

$$= 7.3 + 4.4 + 14.9 = 26.6 \text{ kNm}$$

Force in bottom Tee

With the stiffened top Tee, the p.n.a. of the composite section is still in the flange (Case 2 in Section 3.2.2) and the bending resistance of the composite section is given by:

Eq. (9)

$$M_{o,Rd} = N_{bT,Rd} h_{eff} + N_{c,Rd}(z_t + h_s - 0.5h_c)$$

$$M_{o,Rd} = [1480 \times 409 + 686 \times (14 + 130 - 0.5 \times 70)] \times 10^{-3} = 682 \text{ kNm}$$

Since the resistance of the unstiffened section was adequate, this check is not strictly needed but is included for completeness.

For Case 2, the tension force in the bottom Tee is given by:

Eq. (8)

$$N_{bT,Ed} = \frac{M_{Ed}}{(h_{eff} + z_t + h_s - 0.5z_c)}$$

$$N_{bT,Ed} = \frac{310}{(409 + 24 + 130 - 0.5 \times 70)} \times 10^3 = 587 \text{ kN at opening 1}$$

Vierendeel bending resistance

Section 5.2.5

The bending resistance of the stiffened bottom Tee subject to axial tension is reduced by linear interaction, according to:

Eq. (60)

$$M_{\mathrm{bT,N,Rd}} = M_{\mathrm{pl,Rd}}\left[1-\left(\frac{N_{\mathrm{bT,Ed}}}{N_{\mathrm{bT,Rd}}}\right)\right] = 26.6 \times (1 - (587/1484)) = 16.1 \text{ kNm}$$

For stiffened openings, the reduction factor on the Vierendeel bending resistance due to composite action due to the length of opening, k_{o} is given by:

Eq. (61)

$$k_{\mathrm{o}} = 1 - \ell_{\mathrm{o}}/(35\,h_{\mathrm{t}}) = 1 - 500/(35 \times 78.5) = 0.82$$

Vierendeel bending resistance due to local composite action is given by:

Eq. (24)

$$M_{\mathrm{vc,Rd}} = \Delta N_{\mathrm{c,Rd}}(h_{\mathrm{s}} - 0.5\,h_{\mathrm{c}} + z_{\mathrm{t}})\,k_{\mathrm{o}}$$

$$= 188 \times (14 + 95) \times 10^{-3} \times 0.82 = 16.8 \text{ kNm}$$

A less conservative reduction due to the length of the opening could be obtained using the alternative method in Section 3.4.6 and the values given by Table 3.3 but this is not evaluated here.

Eq. (14)

The criterion for adequacy of Vierendeel bending resistance is:

$$2M_{\mathrm{bT,NV,Rd}} + 2M_{\mathrm{tT,NV,Rd}} + M_{\mathrm{vc,Rd}} \geq V_{\mathrm{Ed}}\ell_{\mathrm{e}}$$

Thus, using the above values for the Tees, the total resistance is:

$$2 \times 16.1 + 2 \times 26.6 + 16.8 = 102.2 \text{ kNm}$$

The design value of Vierendeel moment at opening 1 is:

$$M_{\mathrm{v}} = V_{\mathrm{Ed}}\ell_{\mathrm{o}} = 121 \times 0.5 = 60.5 \text{ kNm}$$

Therefore, the stiffened opening is adequate for Vierendeel bending resistance.

If the resistance were fully utilized, the shear in the top Tee would be:

$$2 \times 26.6/0.5 = 106 \text{ kN}$$

This is greater than 50% of $V_{\mathrm{pl,Rd}}$ ($= 172$ kN, see Appendix 6.5) and $M_{\mathrm{tT,NV,Rd}}$ would need to be reduced for shear. Here the utilization of Vierendeel bending resistance is only $60.5/102 = 59\%$ and thus no reduction is necessary.

7.8 Web-post shear, bending and buckling

For closely spaced rectangular openings, the Vierendeel bending resistance is potentially limited by the bending resistance of the adjacent web-post.

Check whether there is sufficient shear connection to develop a force in the slab over the distance between the centrelines of the openings that is equal to the incremental force in the bottom Tee.

If there is sufficient shear connection, $V_{\mathrm{wp,Ed}} \leq \Delta N_{\mathrm{cs,Rd}}$

Eq. (29) $$V_{\mathrm{wp,Ed}} = \frac{V_{\mathrm{Ed}}\, s}{\left(h_{\mathrm{eff}} + z_{\mathrm{t}} + h_{\mathrm{s}} - 0.5h_{\mathrm{c}}\right)}$$

Taking the value of V_{Ed} mid way between the centrelines of the openings and h_{eff} for the stiffened opening:

$$V_{\mathrm{wp,Ed}} = \frac{95 \times 900}{\left(409 + 24 + 130 - 0.5 \times 70\right)} = 162 \text{ kN}$$

$$\Delta N_{\mathrm{cs,Rd}} = n_{\mathrm{sc,s}} P_{\mathrm{Rd}}$$

Appendix 4.2 For $n_{\mathrm{sc,1\text{-}2}} = 6$

$$\Delta N_{\mathrm{cs,Rd}} = 6 \times 57.2 = 343 \text{ kN}$$

So there is sufficient shear connection between the centrelines of the openings.

Shear resistance of web-post

The longitudinal shear resistance of the web-post is given by:

Eq. (36) $$V_{\mathrm{wp,Rd}} = \frac{\left(s_{\mathrm{o}} t_{\mathrm{w}}\right) f_{\mathrm{y}} / \sqrt{3}}{\gamma_{\mathrm{M0}}} = \frac{400 \times 9.0 \times 355}{\sqrt{3}} \times 10^{-3} = 738 \text{ kN}$$

Shear resistance of web-post is adequate.

Bending resistance of web-post

Elastic bending resistance of web-post:

Eq. (44) $$M_{\mathrm{wp,Rd}} = \frac{s_{\mathrm{o}}^2 t_{\mathrm{w}}}{6} \frac{f_{\mathrm{y}}}{\gamma_{\mathrm{M0}}}$$

$$= 400^2 \times 9.0 \times 355 / 6 \times 10^{-6} = 85.2 \text{ kNm}$$

The shear resistance of the beam, as limited by web-post bending, is the smaller of:

Eq. (50) $$V_{\mathrm{Rd}} = \frac{2M_{\mathrm{wp,Rd}} / s + 4M_{\mathrm{bT,NV,Rd}} / \ell_{\mathrm{o}}}{1 + \left(2e_{\mathrm{o}} + h_{\mathrm{o}}\right) / h_{\mathrm{eff}}} + \frac{\Delta N_{\mathrm{cs}}}{s}\left(z_{\mathrm{t}} + h_{\mathrm{s}} - 0.5h_{\mathrm{c}}\right)$$

and:

Eq. (51) $V_{Rd} = \left[2M_{wp,Rd}/s + 4M_{bT,N,Rd}/\ell_o\right] \dfrac{[h_{eff} + h_s - 0.5h_c]}{[h_{eff} + 2e_o + h_o]}$

Appendix 7.6 and 7.7

Since one opening is stiffened and one unstiffened, use the mean value of Vierendeel resistance of both Tees:

$M_{bT,N,Rd} = (16.1 + 7.1)/2 = 11.6 \text{ kNm}$

Thus the shear resistance, as limited by web-post bending, is the lesser of:

$$V_{Rd} = \frac{2 \times 85.2/900 + 4 \times 11.6/500}{1 + (0 + 300)/409} \times 10^3 + \frac{342}{900}(24 + 130 - 0.5 \times 70) = 208 \text{ kN}$$

and

$$V_{Rd} = \left[2 \times 85.2/900 + 4 \times 11.6 \times 10^3/500\right] \times 10^3 \times \frac{[409 + 130 - 0.5 \times 70]}{[409 + 0 + 300]} = 201 \text{ kN}$$

Appendix 7.2

Hence, $V_{Rd} = 201 \text{ kN} > V_{Ed} = 95 \text{ kN}$

The elastic bending resistance of the web-post $M_{wp,Rd}$ does not limit the shear resistance.

The compression force in the web-post is minimized by minimizing the value of the web-post moment at the mid-height of the opening. If a value of $M_{wp,Ed} = 0$ is assumed, the moment at the top of the web-post is given by $V_{wp,Ed} \times h_o/2$ ($= 162 \times 0.15 = 24.2$ kNm). Inserting this value into the above expressions for the shear resistance limited by web-post bending gives $V_{Rd} = 104 \text{ kN} > V_{Ed} = 95 \text{ kN}$. Thus the assumption of $M_{wp,Ed} = 0$ is consistent with the verification of shear resistance and the following verification of resistance to web-post buckling.

Web-post buckling

For closely spaced openings, the effective compression force in the web-post is given by:

Eq. (33) $N_{wp,Ed} = V_{wp,Ed} + |M_{wp,Ed}|/(h_o/2)$

As noted above, it may be assumed that $M_{wp,Ed} = 0$.

Appendix 7.8

Hence, $N_{wp,Ed} = V_{wp,Ed} + 0 = 162 \text{ kN}$

Eq. (41)

Since $s_o < \ell_o$, the openings are closely spaced and the web-post slenderness is given by:

$$\bar{\lambda} = \frac{2.5\sqrt{s_o^2 + h_o^2}}{t_w} \frac{1}{\lambda_1}$$

$$\bar{\lambda} = \frac{2.5\sqrt{400^2 + 300^2}}{9.0} \frac{1}{76} = 1.83$$

3-1-1/§6.3.1.2 Thus:

$\phi = 0.5(1+\alpha(\bar{\lambda}-0.2)+\bar{\lambda}^2) = 2.57$, where $\alpha = 0.49$ (buckling curve c), and;

$$\chi = \frac{1}{\phi + \left(\phi^2 - \bar{\lambda}^2\right)^{0.5}} = [2.57 + (2.57^2 - 1.83^2)^{0.5}]^{-1} = 0.23$$

Eq. (42)

$$N_{\mathrm{wp,Rd}} = \chi \frac{s_o t_w f_y}{\gamma_{M1}} = 0.23 \frac{400 \times 9 \times 355}{1.0} \times 10^{-3} = 292\ \mathrm{kN} > N_{\mathrm{wp,Ed}} = 162\ \mathrm{kN} \quad \text{OK}$$

The closed solution for vertical shear resistance, as limited by web-post buckling, for the case where there is full shear connection between the openings is given by:

Eq. (53)

$$V_{\mathrm{Rd}} = \left[N_{\mathrm{wp,Rd}}\left(h_o / s\right) + 4M_{\mathrm{bT,NV,Rd}} / \ell_o\right] \frac{\left[h_{\mathrm{eff}} + h_s - 0.5h_c\right]}{\left[h_{\mathrm{eff}} + h_o + 2e_o\right]}$$

$$V_{\mathrm{Rd}} = \left[292\left(300/900\right) + 4 \times 11.6 / 0.500\right] \frac{\left[409 + 130 - 0.5 \times 70\right]}{\left[409 + 300 + 0\right]} = 134\ \mathrm{kN}$$

Hence, $V_{\mathrm{Rd}} = 134\ \mathrm{kN} > V_{\mathrm{Ed}} = 95\ \mathrm{kN}$ OK

This value of shear resistance is greater than the value limited by web-post bending when it is assumed that $M_{\mathrm{wp,Ed}} = 0$ ($V_{\mathrm{Rd}} = 104$ kN). The lower value governs.

8 Construction stage

The construction stage is rarely a critical design situation and the verification is performed here for completeness, having earlier determined the section properties of the beam at the opening.

Consider the design of the stiffened rectangular opening:

Appendix 3.4 Design loads at construction stage $q_d = 5.49\ \mathrm{kN/m^2}$

Maximum moment in mid span $M_{\mathrm{Ed}} = 206\ \mathrm{kNm}$

Maximum moment at a rectangular opening:

$$M_{\mathrm{o,Ed}} = (5.49 \times 3 \times 5) \times 2.05 - 5.49 \times 3 \times 2.05^2/2 = 134\ \mathrm{kNm}$$

Shear force at opening, $V_{\mathrm{Ed}} = 5.49 \times 3 \times (5 - 1.8) = 52.7\ \mathrm{kN}$

Bending resistance of unperforated steel beam:

$$M_{\mathrm{Rd}} = W_{\mathrm{pl,y}} f_y / \gamma_{M0} = 1650 \times 10^3 \times 355 \times 10^{-6} = 586\ \mathrm{kNm} > 195\ \mathrm{kNm}$$

Bending resistance of steel beam at an opening:

$$M_{\mathrm{o,Rd}} = 586 - 9.0 \times 300^2 \times 355 \times 10^{-6}/6 = 538\ \mathrm{kNm} > 137\ \mathrm{kNm}$$

Vierendeel moment at a rectangular opening:

$M_{v,Ed} = V_{Ed}\ell_o = 52.7 \times 0.5 = 26.4$ kNm

Tensile force in bottom Tee:

$N_{b,Ed} = M_{o,Ed}/h_{eff} = 134 \times 10^3/409 = 328$ kN

Appendix 7.7

Bending resistance of stiffened Tee, $M_{pl,Rd} = 27.9$ kNm

Tensile resistance of stiffened Tee, $N_{pl,Rd} = 1484$ kN

Bending resistance of stiffened Tee with coexistent axial force:

Eq. (60)

$M_{pl,N,Rd} = M_{pl,Rd}\,(1 - N_{Ed}/N_{pl,Rd})$

$M_{pl,N,Rd} = 27.9 \times (1 - 328/1484) = 21.7$ kNm

Combined Vierendeel bending resistance of Tees:

$M_{V,Rd} = 4M_{pl,N,Rd} = 4 \times 21.7 = 86.8 \text{ kNm} > 26.4$ kNm OK

9 Serviceability limit state

9.1 Deflections

Modulus of elasticity:

3-1-1/§3.2.6
2-1-1/Table 3.1
4-1-1/§5.4.2.2

Steel $E_a = 210\,000$ N/mm²

Concrete $E_{cm} = 33$ GPa (short term loading)

$E_{c,eff} = E_{cm}/2 = 16.5$ GPa (long term loading)

Modular ratio (for short and long term loads – see 4-1-1/§5.4.2.2(11)):

$$n = \frac{E_a}{E_{c,eff}} = \frac{210}{16.5} = 12.7$$

Non-composite stage

Consider only the permanent actions (ignore construction loads).

Self weight of composite slab and beam = 0.4 + 2.45 = 2.85 kN/m²

$$w_a = \frac{5g_k bL^4}{384E_a I_y} = \frac{5 \times 2.85 \times 3 \times 10^4 \times 10^9}{384 \times 210 \times 33300 \times 10^4} = 15.9 \text{ mm}$$

Composite stage

Finishes g_k $= 1.3$ kN/m²

Imposed load q_k $= 5.0$ kN/m²

Composite second moment of area (remote from opening), can be expressed as:

$$I_y = \frac{A\left(h + h_s + h_p\right)^2}{4\left(1 + nr\right)} + \frac{b_{eff}\, h_c^3}{12n} + I_y$$

where:

$$r = \frac{A}{b_{eff}\ h_c} = \frac{9460}{2500 \times 70} = 0.054$$

$$I_y = \frac{9460 \times \left(457 + 130 + 60\right)^2}{4\left(1 + 12.7 \times 0.054\right)} + \frac{2500 \times 70^3}{12 \times 12.7} + 33300 \times 10^4$$

$$I_y = 926 \times 10^6 \text{ mm}^4$$

Deflection due to finishes and imposed load:

$$w_i = \frac{5q_k bL^4}{384 E_a I_y} = \frac{5 \times 6.3 \times 3 \times 10^4 \times 10^9}{384 \times 210 \times 926 \times 10^6} = 12.7 \text{ mm}$$

(Assuming full shear connection.)

4-1-1/§7.3.1(4) For this design situation, the effects of partial shear connection on slip may be neglected, as the degree of shear connection exceeds 0.5. Hence:

Total deflection $w = 15.9 + 12.7 = 28.6$ mm

Additional deflection due to openings, w_{add}

For each opening, the additional deflection due to the presence of the opening is given by:

Eq. (65)

$$\frac{w_{add}}{w} = k_o\left(\frac{\ell_o}{L}\right)\left(\frac{h_o}{h}\right)\left(1 - \frac{x}{L}\right)$$

where $k_o = 1.0$ for stiffened openings and $k_o = 1.5$ for unstiffened openings.

For opening 1 (stiffened rectangular):

$$\frac{w_{add}}{w} = 1.0\left(\frac{0.5}{10}\right)\left(\frac{0.3}{0.457}\right)\left(1 - \frac{2.05}{10}\right) = 0.026$$

For opening 2 (unstiffened rectangular):

$$\frac{w_{add}}{w} = 1.5\left(\frac{0.5}{10}\right)\left(\frac{0.3}{0.457}\right)\left(1 - \frac{2.95}{10}\right) = 0.035$$

For opening 3 (circular):

$$\frac{w_{\text{add}}}{w} = 1.5\left(\frac{0.135}{10}\right)\left(\frac{0.3}{0.0457}\right)\left(1-\frac{2.40}{10}\right) = 0.010$$

For opening 4 (circular):

$$\frac{w_{\text{add}}}{w} = 1.5\left(\frac{0.135}{10}\right)\left(\frac{0.3}{0.457}\right)\left(1-\frac{1.95}{10}\right) = 0.011$$

Total deflection due to openings:

$$w_{\text{add}} = (0.026 + 0.035 + 0.010 + 0.011) \times 31.2 = 0.082 \times 28.6 = 2.3 \text{ mm}$$

The deflection due to imposed load alone, after allowing for the openings, is:

$$12.7 \times (1 + 0.082) \times 5.0/6.3 = 10.9 \text{ mm}$$

This is less than span/360 (= 27.7 mm), which is a commonly applied limit for imposed loads.

Total deflection $= 28.6 + 2.3 = 30.9$ mm

This is equivalent to L/323, which is less than L/250, a commonly applied total deflection limit for beams with raised floor and suspended ceiling.

9.2 Vibrations

BS EN 1990/ A1.4.4

For the serviceability limit state, the natural frequency for vibrations should be kept above an appropriate value for comfort to users.

NA.2.2.6

The frequent combination of actions should be used.

The frequent combination is expressed as:

$$\sum_{j\geq 1} G_{k,j} + P + \psi_{1,1} Q_{k,1} + \sum_{i>1} \psi_{2,i} Q_{k,i}$$

BS EN 1990/ Table NA.A1.1

For Category A and B buildings, $\psi_1 = 0.5$ and thus the total design value of actions are:

Self weight	2.85×1.0	$= 2.85$ kN/m²
Finishes	1.3×1.0	$= 1.3$ kN/m²
Imposed load	5.0×0.5	$= 2.5$ kN/m²
		6.45 kN/m²

For natural frequency, use the dynamic modulus for concrete to determine the second moment of area of the composite section.

P354

From P354, $E_{c,dyn} = 38$ kN/mm² and thus $n = 210/38 = 5.53$

Use $r = 0.054$, as above, thus:

$$I_y = \frac{9460 \times (457 + 130 + 60)^2}{4(1 + 5.33 \times 0.054)} + \frac{2500 \times 70^3}{12 \times 5.33} + 33300 \times 10^4$$

$$I_y = 1120 \times 10^6 \text{ mm}^4$$

The effect of the openings is to add approximately 8% to the deflection of the beam. Therefore, use $I_{y1} = 0.92 \times 1120 \times 10^6 \text{ mm}^4 = 1030 \times 10^6 \text{ mm}^4$

$$w_{sw} = \frac{5q_{sw}bL^3}{384E_aI_{y1}} = \frac{5 \times 6.45 \times 3.0 \times 10^4 \times 10^9}{384 \times 210 \times 1030 \times 10^6} = 11.6 \text{ mm}$$

$$f = \frac{18}{\sqrt{w_{sw}}} = \frac{18}{\sqrt{11.6}} = 5.3 \text{ Hz}$$

A detailed assessment of the dynamic response of the whole floor may be needed, taking account of floor configuration and building use (see SCI publication P354 [24]).

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