

# Foreword

**This manual provides technical information for designers and contractors to design and plan the construction operation.**

**The body of the manual is split into eight distinct sections:**

1. Components
2. Structural design
3. Deck span and beam selector tables
4. Connection design
5. Services design
6. Acoustic design
7. Construction
8. Supplier information

The manual is intended to provide all the information needed for designers and constructors using Slimdek in grids up to 9m x 9m in a wide range of applications. Where Slimdek is used for applications outside the scope of this manual, reference should be made to the Corus Construction Services & Development staff and to other supporting documents and computer software listed below.

The Construction Services & Development department at Corus provides free technical and commercial advice to help specifiers acquire best practice in design and construction using steel. Experts with in-depth knowledge and experience in civil, structural and building design are available to help speed up the design process, facilitate innovation and encourage an integrated approach. The Construction Services & Development department is directly linked to Corus offices around the world to give rapid sales and technical support at a local level.

## Corus Publications

Slimdek - Engineered Flooring Solutions<sup>[52]</sup>

Slimdek - Structure and Services Integration<sup>[53]</sup>

Slimdek - The Residential Pattern Book<sup>[54]</sup>

The Prevention of Corrosion on Structural Steelwork<sup>[55]</sup>

## Corus Software

Software and further information to assist in the design of the Slimdek system is available from the Corus in Construction website at [www.corusconstruction.com](http://www.corusconstruction.com)

## Design Assistance

Design

- Slimflor Integrated Design Software (SIDS)

Detailing

- Slimdek details

## Steel Construction Institute (SCI) Publications

- SCI-P-169: Design of RHS Slimflor Edge Beams<sup>[18]</sup>
- SCI-P-175: Design of Asymmetric Slimflor Beams using Deep Composite Decking<sup>[19]</sup>
- SCI-P-248: Design of Slimflor Fabricated Beams using Deep Composite Decking<sup>[23]</sup>
- SCI-P-273: Service Integration in Slimdek<sup>[24]</sup>
- SCI-P-279: Value and benefit of Slimdek construction<sup>[25]</sup>
- SCI-P-300: Composite Slabs and Beams using Steel Decking: Best Practice for Design and Construction (publication with MCRMA)<sup>[26]</sup>
- SCI-P-309: Case studies on Slimdek<sup>[27]</sup>
- SCI-P-321: Acoustic Performance of Slimdek<sup>[28]</sup>
- SCI-P-372: Acoustic detailing for steel construction<sup>[31]</sup>
- SCI-P-380: Avoidance of thermal bridging in steel construction<sup>[32]</sup>

SCI-P-169, 175 and 248 can be downloaded from the Corus in Construction website at [www.corusconstruction.com](http://www.corusconstruction.com)

## BCSA publication

Guide to the Installation of Deep Decking<sup>[48]</sup>

## Acknowledgement

This manual has been prepared with the assistance and guidance of The Steel Construction Institute.

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# Introduction

Slimdek is an engineered flooring solution developed to offer a cost-effective, service-integrated, minimal depth floor for use in multi-storey steel-framed buildings with grids up to 9m x 9m. Slimdek extends the range of cost-effective steel options for modern buildings. Ease of planning and servicing, combined with a reduction in building height, gives significant cost and speed of construction benefits.

The Slimdek components form a stable structure once installed; the decking sheets are fixed to the frame to provide lateral stability and end diaphragms not only ensure that the concrete is contained during placement but resist vertical loading and allow the full shear capacity of the deck to be realised during the construction stage.

Slimdek is especially economical for highly serviced buildings. Flexibility of routing services without constraint and the ability to accommodate services between the ribs can lead to substantial savings in the cost of services.

The Slimdek system has safety during construction at its heart. Ongoing review and development of the system has resulted in a method of working which allows the decking units to be placed onto the beams with minimal handling and once installed the decking provides a working platform. If space permits it is possible to preassemble Slimdek bays, minimising work at height.

Slimdek solutions can be designed to incorporate the latest technology in energy-efficient services principles. Research and development into enhanced forms of passive service systems and optimising the contribution of the structure to the operation of the building environment have led to new methods of air distribution on and through the floor construction.

Reductions in height of up to 400mm per storey (over conventional construction) can be achieved by using Slimdek, giving the potential for extra floors within the same building height. Alternatively, a tight floor-to-floor height can reduce total building height and thus save cladding costs.

Speed of construction gives Slimdek a significant advantage over reinforced concrete flat slab structures and the lightweight structure saves on frame and foundation costs.

A cost comparison of various forms of construction is available from [www.corusconstruction.com/coststudy](http://www.corusconstruction.com/coststudy)

## Key features of the system are:

- Shallow composite slab achieves excellent load capacity, diaphragm action and robustness.
- ASB achieves efficient composite action without shear studs.
- ASB(FE) provides 60 minutes inherent fire resistance.
- Lighter, thinner web ASB may be used unprotected in buildings requiring up to 30 minutes fire resistance, in fire-protected applications or in applications where the web accommodates service penetrations.
- ComFlor 225 decking spans up to 6.5m without propping (dependent on slab weight).
- Construction is light in weight.
- Services can be integrated between the decking ribs passing through openings in the ASB.

# The Construction (Design and Management) Regulations 2007 (CDM 2007).

## General

The Construction (Design and Management) Regulations 2007 (CDM 2007)<sup>[33]</sup>, which replace the Construction (Design and Management) Regulations 1994 and the Construction (Health, Safety and Welfare) Regulations 1996, came into force in Great Britain on 6 April 2007.

The Regulations were drafted to simplify and clarify the delivery of occupational health, safety and welfare in construction. They place duties in terms of management arrangements and practical measures on a range of construction project participants, including clients, designers and contractors.

The purpose of this manual is to provide clients, designers and contractors with information about the Slimdek system, its components and safe methods of construction to help them to discharge the duties prescribed by these regulations.

The new CDM 2007 Regulations are divided into 5 parts:

- Part 1 deals with the application of the Regulations and definitions.
- Part 2 covers general duties that apply to all construction projects.
- Part 3 contains additional duties that only apply to notifiable construction projects, i.e., those lasting more than 30 days or involving more than 500 person days of construction work.
- Part 4 contains practical requirements that apply to all construction sites.
- Part 5 contains the transitional arrangements and revocations.

The Regulations require any person on whom a duty is placed to be competent to discharge that duty, to co-operate with other duty holders and to co-ordinate activities to ensure that performance is maximised and risks are minimised.

## Client

The client is required to take reasonable steps to ensure that there are, and continue to be, suitable management arrangements to ensure health, safety and welfare on site, and that the design of any structure intended for use as a workplace complies with the Workplace (Health, Safety and Welfare) Regulations. On notifiable projects the client is also required to appoint a CDM co-ordinator to advise and assist on compliance with the CDM Regulations during the project.

## Designer

The term 'designer' includes everyone who prepares or specifies designs for construction work, including variations. This refers not only to the drawings, design details and specifications, but also to specifiers of quality, including lists of specific requirements and materials which they wish to prohibit.

A client may become a designer by insisting upon a specific material or design detail. A contractor may become a designer by designing specific details of their section of work or by temporary works required for the project.

Manufacturers supplying standardised products that can be used in any project are not designers under CDM 2007, although they may have duties under supply legislation. The person who selects the product is a designer and must take account of health and safety issues arising from its use. If a product is purpose made for a project, the person who prepares the specification is a designer under CDM 2007 and so is the manufacturer who develops the detailed design.

## Contractor

Of the three main parties highlighted, CDM 2007 has least effect on the duties of the contractor. As would be expected, every contractor is required to plan, manage and monitor construction work carried out by them or under their control in a way which ensures that, so far as is reasonably practicable, it is carried out without risks to health and safety. Contractors must also provide every worker carrying out the construction work under their control with any information and training which they need for the particular work to be carried out safely and without risk to health, including suitable site induction and information on the risks to their health and safety.



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# Components

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## Media City, Salford.

The site of the Media City development in Salford Quays is huge and will include three BBC office blocks, other speculative offices, a hotel, residential blocks, a 2,000 space multi-storey car park and a large new public square. By 2010 the site will have been transformed into the UK's first purpose-built media city.

The building with the largest footprint - 12,500m<sup>2</sup> - and at the centre of the project, is a large steel-framed studio building with two conjoined towers, one a 16-storey hotel and the other a 19-storey office development. A major part in the decision to construct the towers in steel was the need to maximise floor to ceiling heights, which led to the Corus Slimdek system being used.

The Slimdek system offered:

- flexibility, simplifying the connections between the studio and the two towers
- aesthetic appeal, the full height glazing allowed the underside of the floor to be viewed from outside
- minimum floor depth

Client:	Peel Holdings
Architect:	Fairhurst Design Group
Structural engineer:	Jacobs
Management contractor:	Bovis Lend Lease
Steelwork contractor:	William Hare
Market sector:	Mixed development



# 1. Components

## 1.1 General

Slimdek is the proprietary name for an engineered flooring system formed with ComFlor 225 deep decking spanning between Asymmetric Slimflor Beams (ASBs) and/or Rectangular Hollow Slimflor Beams (RHSFBs). The various elements of the Slimdek system are described in this section.

## 1.2 ComFlor® 225 decking

ComFlor 225 decking is cold formed from 1.25mm thick, zinc-coated steel to BS EN 10326<sup>[1]</sup>, -S350GD+Z275 with a design yield strength of 350N/mm<sup>2</sup>. The minimum zinc coating mass is 275g/m<sup>2</sup> in total, including both sides.

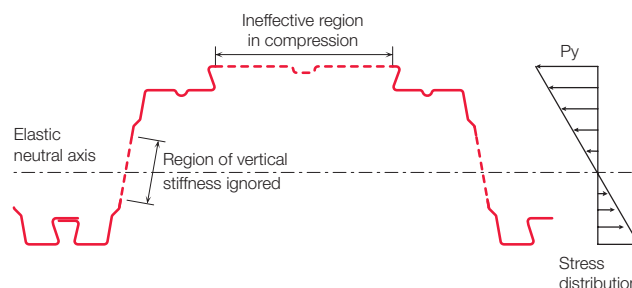
ComFlor 225 can achieve spans of up to 6.5m unpropped and up to 9.6m when propped during construction. The profile has re-entrant sections at the top of the crest and at the bottom of the trough. Its geometry is defined in *Figure 1.1*. Special hangers have been developed to facilitate servicing and fixing of ceilings, see *Section 5.4*. Alternatively, services can be attached using conventional fixings.

The minimum bearing requirement for the decking is 50mm on steelwork and the deep decking must be centred to provide an equal bearing at both ends.

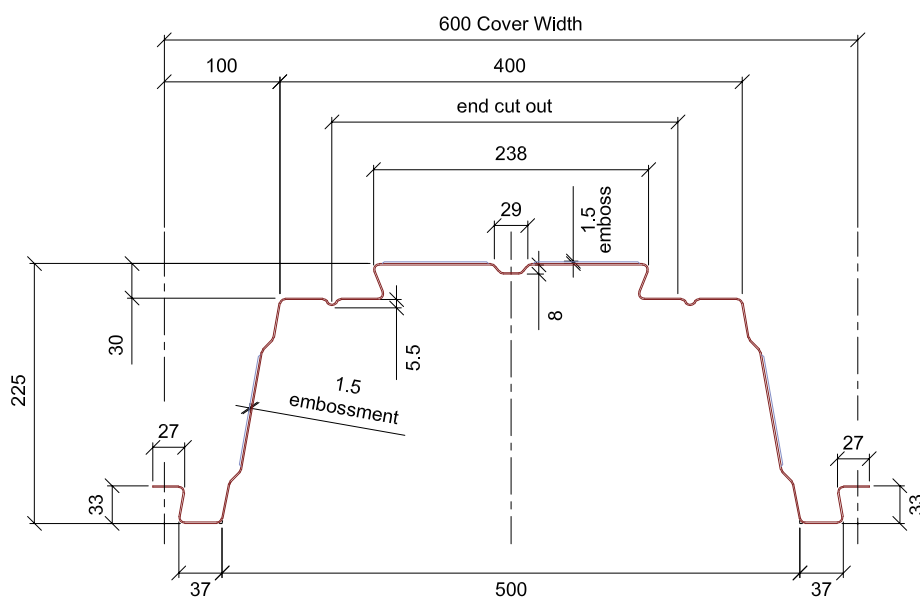
Additional structural support is usually required to the web of columns to take the end bearing of the decking sheet and close off gaps. This is illustrated in *Figure 2.27*. A range of end and side closures, edge trim and accessories is available to complete the deck installation.

## 1.2.1 Section properties

The section properties for stiffness and strength of deep deck profiles rely on the effects of the longitudinal stiffeners and transverse ribs or embossments. Even so, only a proportion of the cross-section is fully effective in bending. The elastic bending resistance of deep decks has to be established by tests, as design to BS 5950-6<sup>[2]</sup> is not directly appropriate to such highly stiffened deep profiles. The ComFlor 225 deep deck profile is considerably more effective in bending than previous deep profiles, largely because of the re-entrant portion in the top flange. The middle portions of the web and upper flange are largely ineffective because of the influence of local buckling. The effective section of the ComFlor 225 profile is illustrated in *Figure 1.2*. Load/span data are given in *Section 3, Tables 3.1 – 3.4*.



**Figure 1.2** Effective section of ComFlor 225 decking



**Figure 1.1** Geometry of ComFlor 225 decking

**Table 1.1 ComFlor 225 properties**

Nominal thickness	Design thickness	Depth profile	Weight profile	Area of steel	Height of neutral axis	Moment of inertia	Elastic modulus	Ultimate moment capacity		Shear resistance span $P_v$	Shear resistance end $P_v$	Shear resistance $P_w$
								Sagging	Hogging			
mm	mm	mm	kN/m <sup>2</sup>	mm <sup>2</sup> /m	mm	cm <sup>4</sup> /m	cm <sup>3</sup> /m	kNm/m	kNm/m	kN/m	kN/m	kN/m
1.25	1.21	225	0.17	2118	107	968	88.0	30.8	30.8	28.7	36.4	29.3

Yield stress = 350 N/mm<sup>2</sup>

#### Composite slab parameters:

$m_r = 367$   $k_r = 0.003$

$m_r$  and  $k_r$  are empirical parameters to BS 5950-4<sup>[3]</sup>, established from six composite slab tests reduced through regression analysis for design purposes. They relate to 1.25mm thick ComFlor 225 decking in S350 steel only:

**ComFlor 225 concrete volumes (nett)**

Slab Depth (mm)	290	300	310	320	330	340	350	360
Nett Volume (m <sup>3</sup> /m <sup>2</sup> )	0.121	0.131	0.141	0.151	0.161	0.171	0.181	0.191

Excluding casing to ASB

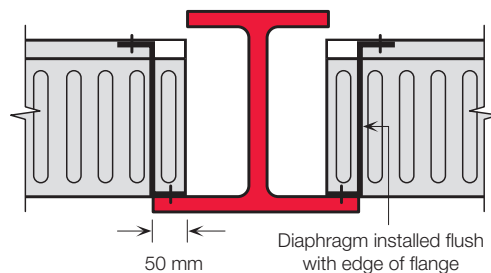
## 1.2.2 Decking accessories

### 1.2.2.1 End diaphragms

Deep decking is installed in single spans and located onto end diaphragms which have been pre-fixed to the lower flange of the support beams. These diaphragms are 1.6mm thick and manufactured in 1800mm lengths. In addition to locating the decking sheets and preventing loss of concrete, they provide a structural role as they resist vertical loading at the deck/support interface, allowing the full shear capacity of the decking to be utilised during the construction stage. Each diaphragm consists of three rib closures and is attached to the beam by shot-fired pins at 600mm centres. The decking is then fixed using self-drilling, self-tapping screws through the crests and the ribs. The closures must be fixed to the lower flanges of the beams at both ends of the steel work bay in advance of the decking. Each length must be fitted accurately so that the 600mm pitch of the decking section is located as shown on the decking layout drawings.



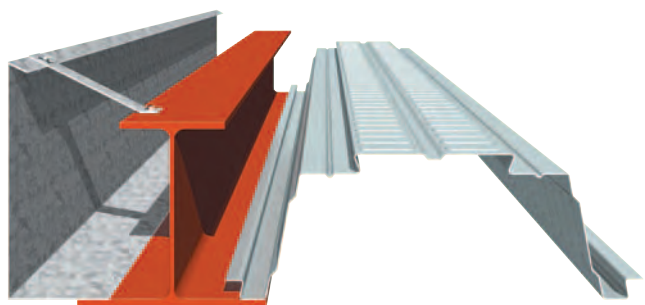
**Figure 1.3** Diaphragm fixing



**Figure 1.4** End diaphragm

### 1.2.2.2 Edge trims

Edge trims are usually supplied in 3m lengths of the appropriate gauge pressed steel, which may be cut down on site to suit the steelwork configuration. Each length should be fixed to the perimeter beams with the edge restraint straps fixed at 500mm centres.



**Figure 1.5** Edge trim

### 1.2.2.3 Closure flashing to cut decking

As buildings are not sized with a whole number of decking units in mind, it is often necessary to cut decking sheets longitudinally, particularly at tie beams or at the slab edge, to achieve the desired dimensions. In this case, a Z section is used to provide local support to the decking and to act as a closure flashing. Fixings should be at 500mm centres. Available options are illustrated in *Figure 1.6, opposite*.



Figure 1.6 Closure options

### 1.3 ASB and ASB(FE) - Asymmetric Slimflor® Beams

Asymmetric Slimflor Beams (ASBs) are an integral part of the *Advance* sections range and are manufactured to the exacting standards demanded for CE marked products. Produced by hot rolling, the degree of asymmetry between the widths of the top and bottom flanges is approximately 60%, and the top flange is embossed to enhance composite action with the concrete encasement. The ASB has been specifically designed for use with deep decking, see Figure 1.7 to create a ribbed slab of 290 to 370mm depth.

Design rules for normal and fire conditions are based on composite action, established from full-scale tests. A summary of tests is contained in Appendix A.

Two types of ASB section are available in S355JR, J0 and J2 steel to BS EN 10025-2<sup>[4]</sup>:

1. ASB - asymmetric sections with thin webs that require fire protection to the exposed bottom flange to achieve more than 30 minutes fire resistance, or where the web accommodates web penetrations, see Figure 1.8(a).
2. ASB(FE) - asymmetric sections that are engineered for optimum characteristics in normal and fire conditions. The beams have a web thickness greater than that found in rolled sections to achieve a fire resistance up to 60 minutes when unprotected, assuming that the integrity of the web is not breached, see Figure 1.8(b).



a) Asymmetric Slimflor® Beam ASB

b) Fire engineered Asymmetric Slimflor® Beam section ASB(FE)

Figure 1.8 Types of Asymmetric Slimflor® Beam

The structural engineer must decide which type of section is most economic for a given design situation. Guidance is available from Corus.

Selector tables are given in Section 3.2, Tables 3.5 – 3.12.

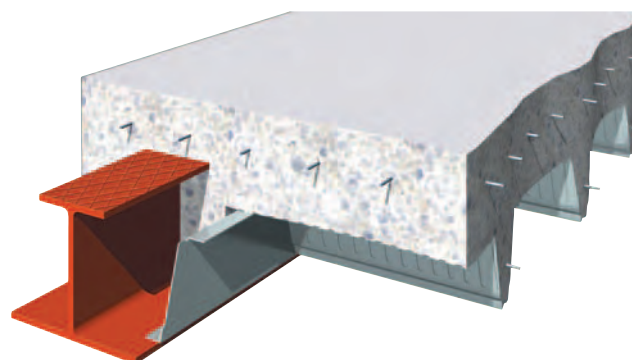
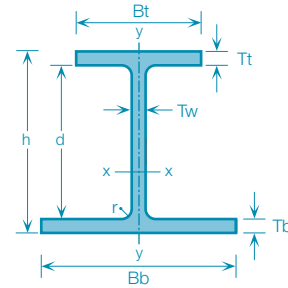


Figure 1.7 Typical Asymmetric Slimflor® Beam

#### 1.3.1 Brittle fracture

The steel sub-grade should be selected from Table 1.4 over, which is in accordance with BS 5950-1<sup>[5]</sup>. This standard relates the service temperature, the material stress, the type of detail and the strain rate to a limiting thickness for each sub-grade. Sub-grades JR and J0 will be suitable for most situations but designers need to be careful when detailing welded joints for the thicker sections when the K factor is less than 1.

## Asymmetric Slimflor® Beams

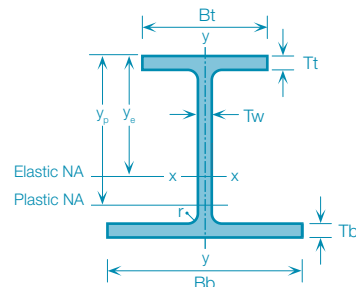


**Table 1.2** ASB dimensions for detailing

Designation		Depth h mm	Width of flange		Thickness		Root radius r mm	Depth between flanges d mm
Serial size	Mass kg/m		Top B <sub>t</sub> mm	Bottom B <sub>b</sub> mm	Web T <sub>w</sub> mm	Flange T <sub>t</sub> & T <sub>b</sub> mm		
300ASB(FE) 249	249.2	342	203	313	40	40	27	262
300ASB 196	195.5	342	183	293	20	40	27	262
300ASB(FE) 185	184.6	320	195	305	32	29	27	262
300ASB 155	155.4	326	179	289	16	32	27	262
300ASB(FE) 153	152.8	310	190	300	27	24	27	262
280ASB(FE) 136	136.4	288	190	300	25	22	24	244
280ASB 124	123.9	296	178	288	13	26	24	244
280ASB 105	104.7	288	176	286	11	22	24	244
280ASB(FE) 100	100.3	276	184	294	19	16	24	244
280ASB 74	73.6	272	175	285	10	14	24	244

Note: ASB(FE) are fire engineered sections

## Asymmetric Slimflor® Beams



**Table 1.3** ASB properties

Designation		Second moment of area		Radius of gyration		Elastic neutral axis y <sub>e</sub> cm	Elastic modulus		
Serial size	Mass kg/m	I <sub>x</sub> cm <sup>4</sup>	I <sub>y</sub> cm <sup>4</sup>	r <sub>x</sub> cm	r <sub>y</sub> cm		Z <sub>x</sub> Top cm <sup>3</sup>	Z <sub>x</sub> Bottom cm <sup>3</sup>	Z <sub>y</sub> cm <sup>3</sup>
300ASB(FE) 249	249.2	52920	13194	12.9	6.45	19.2	2757	3528	843
300ASB 196	195.5	45871	10463	13.6	6.48	19.8	2321	3185	714
300ASB(FE) 185	184.6	35657	8752	12.3	6.10	18.0	1984	2547	574
300ASB 155	155.4	34514	7989	13.2	6.35	18.9	1825	2519	553
300ASB(FE) 153	152.8	28398	6840	12.1	5.93	17.4	1628	2088	456
280ASB(FE) 136	136.4	22216	6256	11.3	6.00	16.3	1367	1777	417
280ASB 124	123.9	23453	6410	12.2	6.37	17.2	1360	1891	445
280ASB 105	104.7	19249	5298	12.0	6.30	16.8	1145	1604	370
280ASB(FE) 100	100.3	15506	4245	11.0	5.76	15.6	995	1292	289
280ASB 74	73.6	12191	3334	11.4	5.96	15.7	776	1060	234

Note: ASB(FE) are fire engineered sections.

**Table 1.4** Choice of sub grade for ASBs dependant upon situation and exposure

				K value for Internal conditions (-5°)							K value for External conditions (-15°)						
				0.5	0.75	1	1.5	2	3	4	0.5	0.75	1	1.5	2	3	4
300 ASB	249	Web,	40	NA	J2	J0	J0	JR	JR	JR	NA	J2	J2	J0	J0	JR	JR
		Flange,	40	NA	J2	J0	J0	JR	JR	JR	NA	J2	J2	J0	J0	JR	JR
300 ASB	196	Web,	20	J0	J0	JR	JR	JR	JR	JR	J2	J0	J0	JR	JR	JR	JR
		Flange,	40	NA	J2	J0	J0	JR	JR	JR	NA	J2	J2	J0	J0	JR	JR
300 ASB	185	Web,	32	J2	J0	J0	JR	JR	JR	JR	NA	J2	J0	J0	J0	JR	JR
		Flange,	29	J2	J0	J0	JR	JR	JR	JR	NA	J2	J0	J0	J0	JR	JR
300 ASB	155	Web,	16	J0	JR	JR	JR	JR	JR	JR	J0	J0	J0	JR	JR	JR	JR
		Flange,	32	J2	J0	J0	JR	JR	JR	JR	NA	J2	J0	J0	J0	JR	JR
300 ASB	153	Web,	27	J2	J0	J0	JR	JR	JR	JR	J2	J0	J0	J0	JR	JR	JR
		Flange,	24	J2	J0	JR	JR	JR	JR	JR	J2	J0	J0	J0	JR	JR	JR
280 ASB	136	Web,	25	J2	J0	JR	JR	JR	JR	JR	J2	J0	J0	J0	JR	JR	JR
		Flange,	22	J0	J0	JR	JR	JR	JR	JR	J2	J0	J0	J0	JR	JR	JR
280 ASB	124	Web,	13	J0	JR	JR	JR	JR	JR	JR	J0	J0	JR	JR	JR	JR	JR
		Flange,	26	J2	J0	J0	JR	JR	JR	JR	J2	J0	J0	J0	JR	JR	JR
280 ASB	105	Web,	11	JR	JR	JR	JR	JR	JR	JR	J0	J0	JR	JR	JR	JR	JR
		Flange,	22	J0	J0	JR	JR	JR	JR	JR	J2	J0	J0	J0	JR	JR	JR
280 ASB	100	Web,	19	J0	J0	JR	JR	JR	JR	JR	J0	J0	J0	JR	JR	JR	JR
		Flange,	16	J0	JR	JR	JR	JR	JR	JR	J0	J0	J0	JR	JR	JR	JR
280 ASB	74	Web,	10	JR	JR	JR	JR	JR	JR	JR	J0	JR	JR	JR	JR	JR	JR
		Flange,	14	J0	JR	JR	JR	JR	JR	JR	J0	J0	JR	JR	JR	JR	JR

Steel grades indicated in Table 1.4 are based on the requirements of Cl. 2.4.4 of BS EN 5950-1<sup>[5]</sup>

JR means that steel grade S355 JR is suitable for the combination of K factor and steel thickness

J0 means that steel grade S355 J0 is suitable for the combination of K factor and steel thickness

J2 means that steel grade S355 J2 is suitable for the combination of K factor and steel thickness

NA means that no suitable grade exists for the combination of K factor and steel thickness

	Plastic neutral axis $y_p$ cm	Plastic modulus		Buckling parameter $u$ -	Torsional index $x$ -	Warping constant $H$ cm <sup>6</sup>	Torsional constant $J$ cm <sup>4</sup>	Area of section $A$ cm <sup>2</sup>	Designation  Serial size
		$s_x$ cm <sup>3</sup>	$s_y$ cm <sup>3</sup>						
	22.6	3761	1512	0.825	6.80	1998105	2004	317	300ASB(FE) 249
	28.1	3055	1229	0.845	7.86	1498135	1177	249	300ASB 196
	21.0	2658	1030	0.822	8.56	1203025	871	235	300ASB(FE) 185
	27.3	2361	949	0.843	9.40	1068165	620	198	300ASB 155
	20.4	2160	816	0.822	9.97	894772	513	195	300ASB(FE) 153
	19.2	1806	740	0.814	10.19	709504	379	174	280ASB(FE) 136
	25.8	1730	761	0.832	10.54	720657	332	158	280ASB 124
	25.4	1441	632	0.831	12.08	573540	207	133	280ASB 105
	18.4	1295	510	0.815	13.13	450943	160	128	280ASB(FE) 100
	21.3	979	402	0.830	16.63	337956	72.2	93.7	280ASB 74

## 1.4 RHSFBs - Rectangular Hollow Slimflor® Beams

At the perimeter of the building, it is often structurally more efficient and often architecturally desirable, to use an RHS which can resist torsional forces caused by eccentric loading on the beam, as shown in *Figure 1.9*.

The RHSFB comprises a Corus Celsius hot-rolled Rectangular Hollow Section in S355J2H steel to BS EN 10210<sup>[6]</sup>, with a 15mm thick S355 steel plate welded to the underside of the RHS. The RHSFB can achieve 60 minutes fire resistance without protection to the bottom plate where the bare outer face of the RHS is fire protected by fire stopping required to ensure the integrity of the compartmentation.

Selector tables are given in *Section 3, Tables 3.13 – 3.16*.

### 1.4.1 Brittle fracture

Table 5 of BS 5950-1<sup>[6]</sup> allows a maximum thickness of 55mm at  $K=1$  for external steelwork using S355J2H structural hollow sections to BS EN 10210. The normal hot-finished range rolled by Corus has a maximum wall thickness of 20mm, so even in the case of welded tubular joints where a  $K$  factor of 0.5 applies, the standard J2 quality rolled by Corus will be suitable.

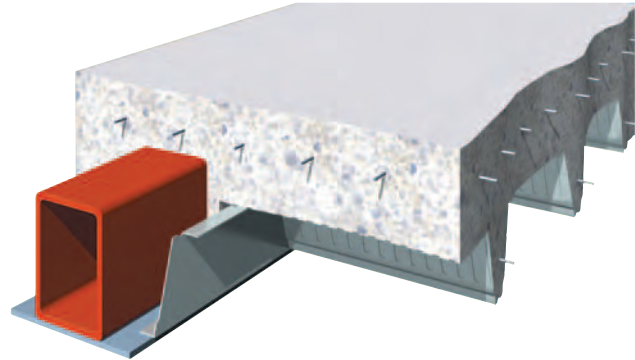
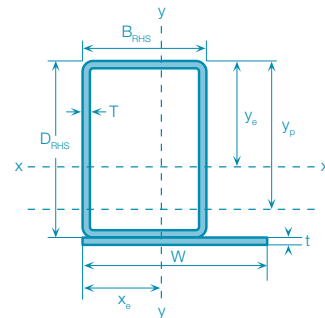


Figure 1.9 Typical RHS edge beam



## Rectangular Hollow Slimflor® Beams

Table 1.5 RHSFB dimensions and properties

Designation	Thickness	Mass	Second moment of area		Radius of gyration		Elastic neutral axis	
$D_{RHS} \times B_{RHS}$ (W x t)	T mm	kg/m	$I_x$ cm <sup>4</sup>	$I_y$ cm <sup>4</sup>	$r_x$ cm	$r_y$ cm	$x_e$ cm	$y_e$ cm
200 x 150 (240 x 15)	8.0	69.7	5451	4055	7.8	6.8	9.3	14.4
	10.0	79.3	6251	4461	7.9	6.6	9.1	13.8
	12.5	90.8	7107	4903	7.8	6.5	8.9	13.3
250 x 150 (240 x 15)	8.0	76.0	9087	4484	9.7	6.8	9.2	17.4
	10.0	87.1	10449	4975	9.7	6.7	9.0	16.8
	12.5	100.6	11938	5517	9.7	6.6	8.8	16.2
300 x 200 (290 x 15)	8.0	94.4	16614	8795	11.8	8.6	11.6	20.7
	10.0	108.6	19226	9931	11.8	8.5	11.4	20.0
	12.5	126.0	22147	11228	11.7	8.4	11.2	19.3
400 x 200 (290 x 15)	8.0	106.9	32322	10308	15.4	8.7	11.4	26.6
	10.0	124.3	37511	11772	15.4	8.6	11.2	25.7
	12.5	146.1	43409	13461	15.3	8.5	11.1	24.9
450 x 250 (340 x 15)	8.0	125.4	48873	17759	17.5	10.5	13.9	29.9
	10.0	146.0	56914	20482	17.5	10.5	13.7	28.8
	12.5	171.0	66155	23677	17.4	10.4	13.6	27.9

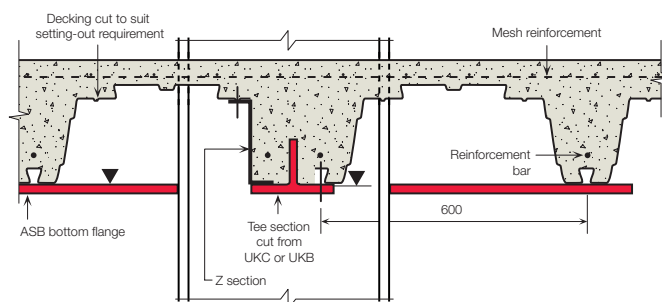
Notes: A full list of available sections is included in the SIDS software package.  
Section properties are based on a plate extending 90mm beyond the RHS.

## 1.5 Tie members

Tie members are required to provide robustness by tying together columns at each floor level. Generally tie members are formed from inverted Tees although smaller UKB or RHS with a welded plate are often used where the tie supports other local loads. The general arrangement for tie members is illustrated in *Figure 1.10* where the Tee section allows for placement of a Z section to suit a deck layout which is not in multiples of 600mm. To avoid visible sag, the depth of the Tee is normally taken to be at least span/40.

The Tee section does not resist loads applied to the slab so reinforcement is placed in the ribs adjacent to the Tee. Fire protection is not normally required where the tie is partially encased in the slab. Tees may be attached by an end plate to the column web or to a stiffener located between the column flanges. The same stiffener may act as a compression stiffener in a moment-resisting connection to the major axis of the column.

The design of tie members is considered further in *Section 2.7*.



**Figure 1.10** Inverted Tee section as a tie member

Elastic modulus				Plastic neutral axis $y_p$ cm <sup>3</sup>	Plastic modulus $S_x$ cm <sup>3</sup>	Torsional constant $J$ cm <sup>4</sup>	Area of section $A$ cm <sup>2</sup>	Designation $D_{RHS} \times B_{RHS}$ ( $W \times t$ )
$Z_x$ Top cm <sup>3</sup>	$Z_x$ Bottom cm <sup>3</sup>	$Z_y$ Left cm <sup>3</sup>	$Z_y$ Right cm <sup>3</sup>					
380	763	435	276	19.4	549	3670	88.8	<b>200 x 150</b> (240 x 15)
452	816	490	300	18.9	661	4436	100.9	
532	872	551	325	17.2	783	5314	115.6	
521	1002	489	302	23.7	775	5048	96.8	<b>250 x 150</b> (240 x 15)
622	1077	555	331	21.5	926	6117	110.9	
736	1162	629	362	19.7	1087	7353	128.1	
803	1538	756	506	28.6	1169	10595	120.3	<b>300 x 200</b> (290 x 15)
964	1665	870	565	25.9	1404	12941	138.4	
1149	1811	1001	631	23.7	1661	15710	160.5	
1214	2173	901	587	33.6	1810	15768	136.3	<b>400 x 200</b> (290 x 15)
1460	2373	1048	663	30.9	2146	19292	158.5	
1746	2610	1218	750	28.7	2507	23471	185.5	
1634	2946	1274	885	38.4	2401	27121	160.0	<b>450 x 250</b> (340 x 15)
1971	3229	1491	1011	35.2	2861	33322	186.0	
2368	3564	1747	1158	32.7	3383	40757	218.0	



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# Structural design

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## Zero4, Plymouth.

Zero4 is a new city centre development with retail outlets on the ground floor and luxury apartments above. The ten-storey residential block will contain 120 flats, from one-bedroom studio flats up to three-bedroom duplex apartments on the upper levels.

The Slimdek flooring system was developed to offer a cost-effective, service integrated, minimal depth floor for use in multi-storey steel-framed buildings with grids up to 9m x 9m. Slimdek allowed the construction team to reduce the overall building height, and the construction programme was completed with speed and efficiency.

The Slimdek system offered:

- speed of construction
- open spans with column-free space, allowing for reconfiguration
- reduced floor depth, minimising building height
- a cost-effective flooring system
- easy service integration
- the ability to achieve complex grid patterns
- good acoustic performance

Client:	Penrose
Structural engineer:	Airey & Coles
Main contractor and developer:	Prestige Homes
Steelwork contractor:	SIAC Tetbury Steel
Market sector:	Mixed residential/retail

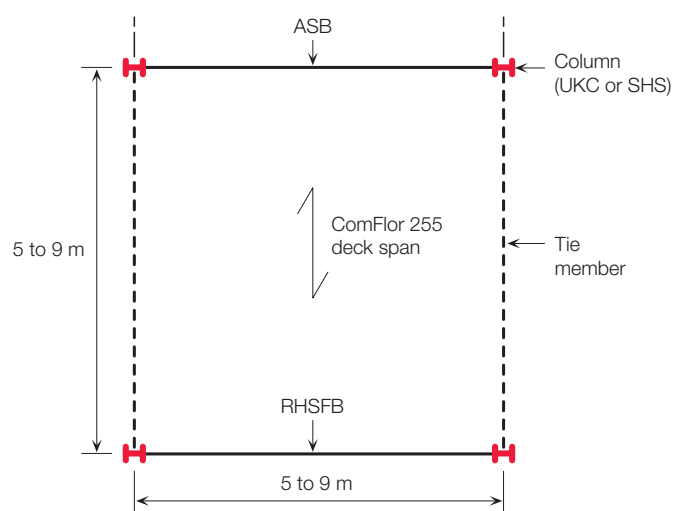


## 2. Structural design

### 2.1 Introduction

In designs using Slimdek, the slab spans between beams that are directly attached to columns. Secondary beams are eliminated where possible. The members provide temporary stability and robustness in line with the requirements of the Building Regulations<sup>[34]</sup> and BS 5950-1<sup>[5]</sup>. A typical layout of a floor using Slimdek components is illustrated in *Figure 2.1*.

Design must be carried out by an appropriately qualified person. The process is simplified by use of the deck span and beam selector tables given in *Section 3* or using the SIDS design software from the Corus in Construction website at [www.corusconstruction.com](http://www.corusconstruction.com). Detailed design methods for all the Slimdek components are presented in SCL publications listed in the References.



**Figure 2.1** Slimdek – beam layout

#### 2.1.1 Design situations

Two fundamental forms of construction are possible with the Slimdek system. In either case, the slab is considered to be composite but the beams may be either composite or non-composite in the completed structure. Composite beams should be designed in accordance with BS 5950-3.1<sup>[7]</sup> and capable of being designed plastically whereas non-composite beams are designed using BS 5950-1<sup>[5]</sup>.

The beam is designed for two load situations, firstly during construction, when the concrete has been placed but the restraining effect caused by solidification of the concrete is not present and secondly, the normal situation, when the slab is cast and the floor is subject to not only the self-weight of the concrete but the imposed loads resulting from use.

In the construction stage, the top flange of the beam is laterally unrestrained and lateral torsional buckling needs to be taken into account. The designer needs to be aware of the construction sequence to be adopted as casting the floor on one side of a beam but not on the other will induce a torsional moment in the beam which could prove to be the governing condition.

For non-composite design, any available longitudinal shear resistance is ignored allowing a simplified design procedure to be adopted.

The concrete surrounding the beam will provide lateral restraint and a stiffening effect in the normal stage as long as adequate shear connection is provided and composite action can be shown to exist. Tests have shown that a significant increase in the moment resistance of the steel section can be obtained by utilising composite action. A ribbed pattern is rolled into the top flange of the beam to ensure that sufficient bond is generated between the steel section and the concrete slab; the design value for bond stress is  $0.6\text{N/mm}^2$  but it may not be necessary to utilise all the increased resistance at the ultimate limit state because of the importance of serviceability and fire-limit states. Design rules given in BS 5950-3.1<sup>[7]</sup> allow both full and partial shear connection dependent upon the moment capacity of the section.

### 2.1.2 Concrete cover

Table 2.1 summarises the requirements for the minimum concrete cover to reinforcement and beams based on BS 8110-1<sup>[6]</sup> and 2<sup>[9]</sup>. The minimum slab depth over the steel decking is normally dependent on the fire-resistance requirements, see 2.2.2.1 or acoustic requirements, see Section 6.

The minimum cover to the beams includes two bar diameters for the mesh reinforcement. It is recommended that the mesh is not lapped across the beams and laps along the beam should be nested.

For scheme design, the following slab depths should be considered as typical:

280 ASB sections ~ 290-320mm deep slab.

300 ASB sections ~ 315-370mm deep slab.

For non-composite applications, 300 ASB sections or 300mm RHSFB may be detailed as flush with the top of the slab. However, the slab depth should also be checked for its fire resistance, see 2.2.2.1.

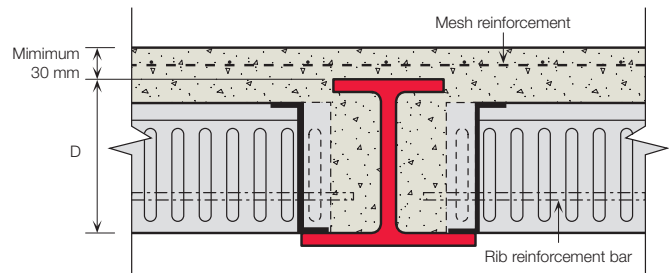


Figure 2.2 Composite arrangement

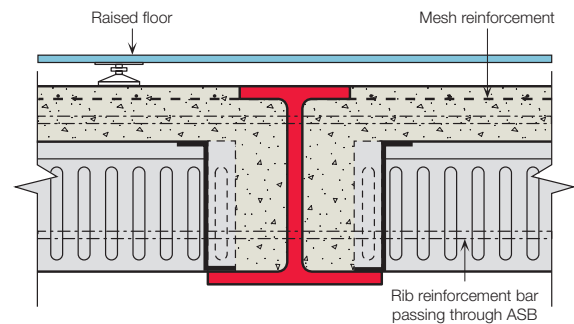


Figure 2.3 Non-composite arrangement

Table 2.1 Minimum cover (mm) to the reinforcement and top flange of ASB and RHSFB for different exposure conditions

Location			Heated Buildings			Moderate Exposure	
			Mesh reinforcement			Mesh reinforcement	
			A142	A193	A252	A252	A353
Reinforcement	LWC	C30 grade	15	15	15	40	40
		C35 grade	15	15	15	35	35
	NWC	C30 grade	25	25	25	-	-
		C35 grade	20	20	20	35	35
ASB or RHSFB flush with slab			0	0	0	N.R.	N.R.
ASB (Composite)	LWC	C30 grade	30	30	35	55	60
		C35 grade	30	30	35	50	55
	NWC	C30 grade	40	40	40	-	-
		C35 grade	35	35	35	50	55
RHSFB (Non-composite)			40	40	40	60	60
RHSFB (Composite)*			85	85	85	85	85

Moderate exposure = subject to periodic wetting and drying.

N.R. = not recommended for moderate exposure.

A142 to A353 is the mesh size in mm<sup>2</sup>/m - see Table 2.3 and 2.2.5.

LWC - lightweight concrete using 12mm max aggregate.

NWC - normal weight concrete using 20mm max aggregate.

\*using 75mm long headed stud (as-welded height 70mm + 15mm cover).

## 2.2 Structural design - floor slab

### 2.2.1 ComFlor 225 deck design

The steel decking is generally designed in accordance with BS 5950-4<sup>[3]</sup>, BS 5950-6<sup>[2]</sup> and BS 5950-8<sup>[10]</sup>. Construction loading for long span applications is taken in accordance with BS EN 1994-1-1<sup>[11]</sup> which states that a construction load of  $1.5\text{ kN/m}^2$  should be applied over a  $3\text{ m}$  square area of the decking with a reduced construction load of  $0.75\text{ kN/m}^2$  elsewhere.

BS 5950:Part 4<sup>[3]</sup> states that an additional load due to the ponding effect of the wet concrete should be taken when the deck deflection exceeds the smallest of  $20\text{ mm}$  or  $\text{Span}/180$  or  $D_s/10$  where  $D_s$  is the overall depth of the composite slab.

This deflection is calculated for the unfactored self-weight of the decking and wet concrete alone and if the limit is not reached, the ponding effect may be ignored in the design of the steel decking.

The partial factors on loads in BS 5950 are 1.6 for construction loads and 1.4 for self-weight loads. The factored construction loading that should be considered for deep decking (for unpropped spans over  $3\text{ m}$ ) is shown graphically in Figure 2.4.

The nominal end bearing distance of the decking on its supports is  $50\text{ mm}$ , which facilitates placing of the decking between the flanges of the ASB. Cut-outs in the top of the decking ensure sufficient space around the beam for placement and observation of the concrete filling as shown in Figure 2.5. The end bearing and other geometry is illustrated in Figure 2.6. To allow for construction tolerances, the absolute minimum end bearing distance, as measured on site, is  $35\text{ mm}$ .

Deck span tables are given in Section 3.

Alternatively, design may be carried out using the SIDS software package.

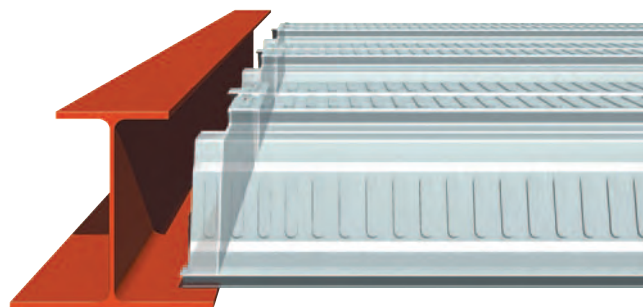


Figure 2.5 End bearing and cut-outs of decking

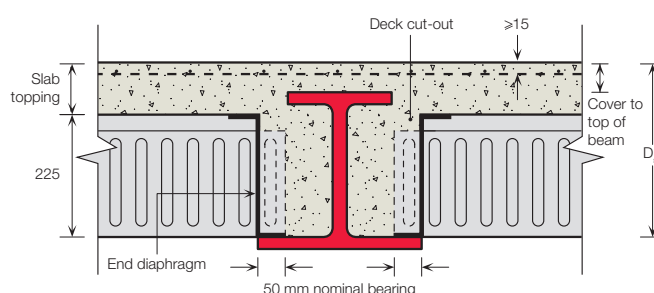


Figure 2.6 Geometry for ASB with ComFlor 225

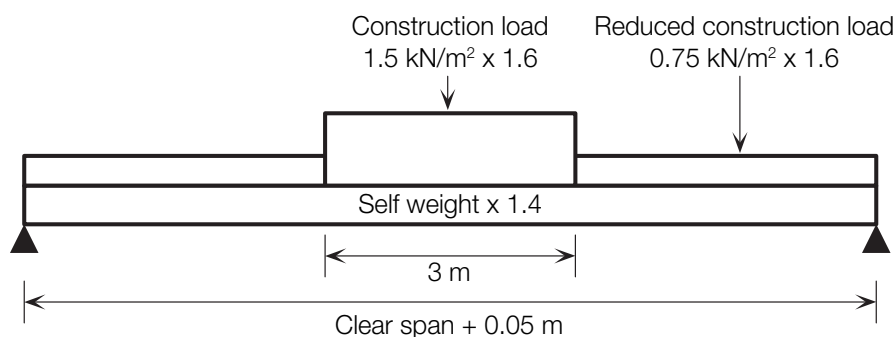


Figure 2.4 Construction stage loading

## 2.2.2 Composite slab design

The design of composite slabs using deep decking follows conventional principles. Three important design cases are:

- The ability of the decking to support the loads during construction, with or without the need for temporary propping.
- The ultimate load resistance of the composite slab, which is influenced by the amount of bar reinforcement placed in the ribs of the decking.
- The fire resistance of the composite slab, which is dependent on the:
  - slab depth
  - reinforcing bar in the ribs, assuming the steel decking is largely ineffective in fire.

The ComFlor 225 deck profile has been designed to achieve optimum strength and stiffness properties with the facility for integrated servicing.

In all cases, the effective span of the decking is taken from the centres of bearing, which is the clear span plus the nominal end bearing distance of 50mm. The slab depth determines the self-weight of the slab and hence the maximum span capabilities of the steel decking. Lightweight concrete is beneficial at the construction stage as its wet density is typically 1900kg/m<sup>3</sup> against 2400kg/m<sup>3</sup> for normal weight concrete.

The parameters used in the slab design are based on rigorous testing of both the decking and composite slab, see *Appendix A* for summary of tests.

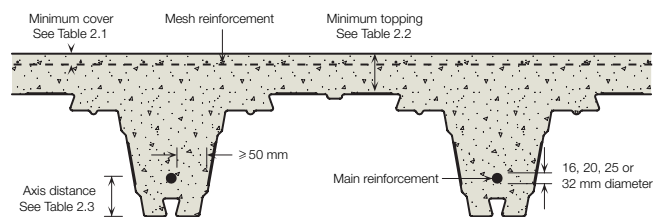
### 2.2.2.1 Fire resistance

Bar reinforcement is required in each rib for the fire-limit state design, as the steel decking is largely ineffective in fire, see *Figure 2.7*. The axis distance to these bars is dependent on the fire resistance requirement. Axis distance is defined as the measurement from the soffit of the slab to the centre-line of the bar.

N.B. The bar reinforcement is utilised in normal design.

Partial continuity is assumed at the fire limit state based on a model that assumes full continuity at one end and a simple support at the other (i.e., a propped cantilever condition).

The minimum slab depth is dependent on the fire resistance requirements and should be in accordance with *Table 2.2*. For insulation in fire, the minimum depth of topping is evaluated from the depth of an equivalent solid slab (in BS 5950:Part 8<sup>(10)</sup>) less 10mm to account for the insulating effect of the adjacent ribs, see *Figure 2.7*.



**Figure 2.7** Details of reinforcement and minimum slab depth

**Table 2.2** Minimum slab depth above the ComFlor 225 decking depending on fire resistance

Type of Cover	Fire Resistance (minutes)	Normal Weight Concrete (mm)	Lightweight Concrete (mm)
Slab depth above	60	70	60
ComFlor 225 decking	90	80	70
for fire insulation purposes	120	95	80

Note: This depth is the minimum for fire insulation purposes. A greater slab depth may be required for spanning capability or to satisfy the acoustic requirements – see the deck span tables. Both the fire insulation and beam cover requirements must be satisfied, e.g., if a 280 ASB(FE)136 is required with composite action and 60 minutes fire resistance, the minimum slab depth is  $288 - 22 + 30 = 296\text{mm}$  for the beam which is greater than slab requirement of  $225 + 60 = 285\text{mm}$  (LWC). Therefore a 296mm minimum depth slab would be required.

### 2.2.2.2 Reinforcement requirements

There are generally two requirements for reinforcement in ComFlor 225 slabs; bars in the deck ribs for structural design in the normal and fire conditions and top mesh for crack control in the slab.

The reinforcement is usually chosen after considering the span and fire resistance requirements of the slab. Typically, a 16, 20, 25 or 32mm bar ( $f_y = 500\text{N/mm}^2$ ) is placed in each rib of the slab, depending on the conditions. The axis distance from the underside of the ComFlor 225 decking to the centre of the bars should be 70, 90 and 120mm for fire resistance of 60, 90 and 120 minutes respectively as given in *Table 2.3*.

Generally, a larger bar diameter is required for propped slabs (normally 25 or 32mm diameter). In most cases, it is the construction stage that will control the design in unpropped slabs. Requirements for propping are covered in the *Section 7.5*.

The design of the rib bar reinforcement is an integral part of the composite slab design, and is included in the SIDS software package. The detailing rules in *Table 2.3* and *Figures 2.8(a), (b) and (c)* are based on BS 8110-1<sup>[8]</sup> and have been shown to achieve excellent composite action and fire resistance.

The minimum anchorage details depend on the diameter of the main reinforcing bars in the rib, which depends on the fire-resistance period, whether or not the slab is propped, and the level of applied shear. For 60 minutes fire resistance, when the level of applied shear is less than 0.5 times the shear resistance,  $V_c$ , straight bars may be used without extra anchorage bars – in accordance with BS 8110-1<sup>[8]</sup>, clause 3.12.9.4. However, anchorage bars are recommended even for low values of applied shear at 90 and 120 minutes fire-resistance periods.

A 100mm projection past the tip of the flange is used for detailing purposes, which allows the main bars to be placed in single lengths in the ribs. BS 8110-1<sup>[8]</sup> states that the cross-sectional area of the end anchorage may be reduced to 40% of that of the main bars. L-bars or U-bars may be used, but should have an effective anchorage length of  $12\phi$  in accordance with BS 8110-1<sup>[8]</sup>. The anchorage bars need not be vertical, and may require slanting to clear the top flange.

*Table 2.3* also presents the vertical shear resistance achieved at the interface with the ASB when these anchorage details are adopted. The resistance is given for both normal weight and lightweight concrete, and includes the bare steel end-crushing resistance of the decking. (Note that the decking is not sufficiently anchored to include anything other than the end-crushing resistance, which also incorporates the stiffening effect from the end diaphragm.) When the applied shear exceeds the resistance, more reinforcement may be provided, and the resistance should be calculated in accordance with BS 8110-1<sup>[8]</sup>, clause 3.4.5.

Additional reinforcement is required in the slab in the following cases:

- Partial continuity
- Transverse reinforcement adjacent to shear connectors
- U-bars at composite edge beams
- Additional crack control reinforcement, see *Section 2.2.5*
- Adjacent to openings
- At positions of concentrated loads

Table 2.3 Detailing requirements and vertical shear resistance for deep composite slabs using Slimdek

Detailing requirement		Fire resistance (minutes)					
		60		90		120	
Minimum main bar diameter $\phi$ (mm) – unpropped slab		16 ( $\phi_L=12\text{mm}$ )		20 ( $\phi_L=16\text{mm}$ )		25 ( $\phi_L=16\text{mm}$ )	
– propped slab		20 ( $\phi_L=16\text{mm}$ )		25 ( $\phi_L=16\text{mm}$ )		32 ( $\phi_L=20\text{mm}$ )	
Axis distance to bar (mm)		70		90		120	
Shear stress in rib, $V \leq 0.5 V_c$		Straight bar		$\phi_L=16\text{mm}$ L-bar		$\phi_L=16\text{mm}$ L-bar	
		$\phi_L=12\text{mm}$ L-bar		$\phi_L=16\text{mm}$ U-bar*		$\phi_L=20\text{mm}$ L-bar*	
		$\phi_L=16\text{mm}$ L-bar				$\phi_L=16\text{mm}$ U-bar	
Shear stress in rib, $V > 0.5 V_c$		$\phi_L=16\text{mm}$ L-bar		$\phi_L=16\text{mm}$ U-bar*		$\phi_L=20\text{mm}$ U-bar*	
End shear resistance	Concrete type	NWC		LWC		NWC	
	Anchorage bar diameter $\phi_L$	0	12	16	0	12	16
	Resistance (kN/m)	55**	67	73	51**	61	66
Minimum mesh size in topping							
		A142		A193		A252 up to 8m span	
						A393 over 8m span	

Axis distance is distance from centre line of reinforcing bar to underside of decking.

$\phi$  = diameter of main bar reinforcement, in mm.

$\phi_L$  = diameter of L-bar or U-bar end anchorage, in mm. (Area  $\phi_L = 0.4 \times \text{Area } \phi$ ).

\*Bars may be slanted from vertical plane to clear top flange of ASB.

\*\*Value assumes 16mm dia straight main bar with 100mm projection past flange tip.

The end shear resistance includes the bare steel end crushing resistance of the decking (including the diaphragm) and assumes grade 30 concrete with the minimum cover from Table 2.1.

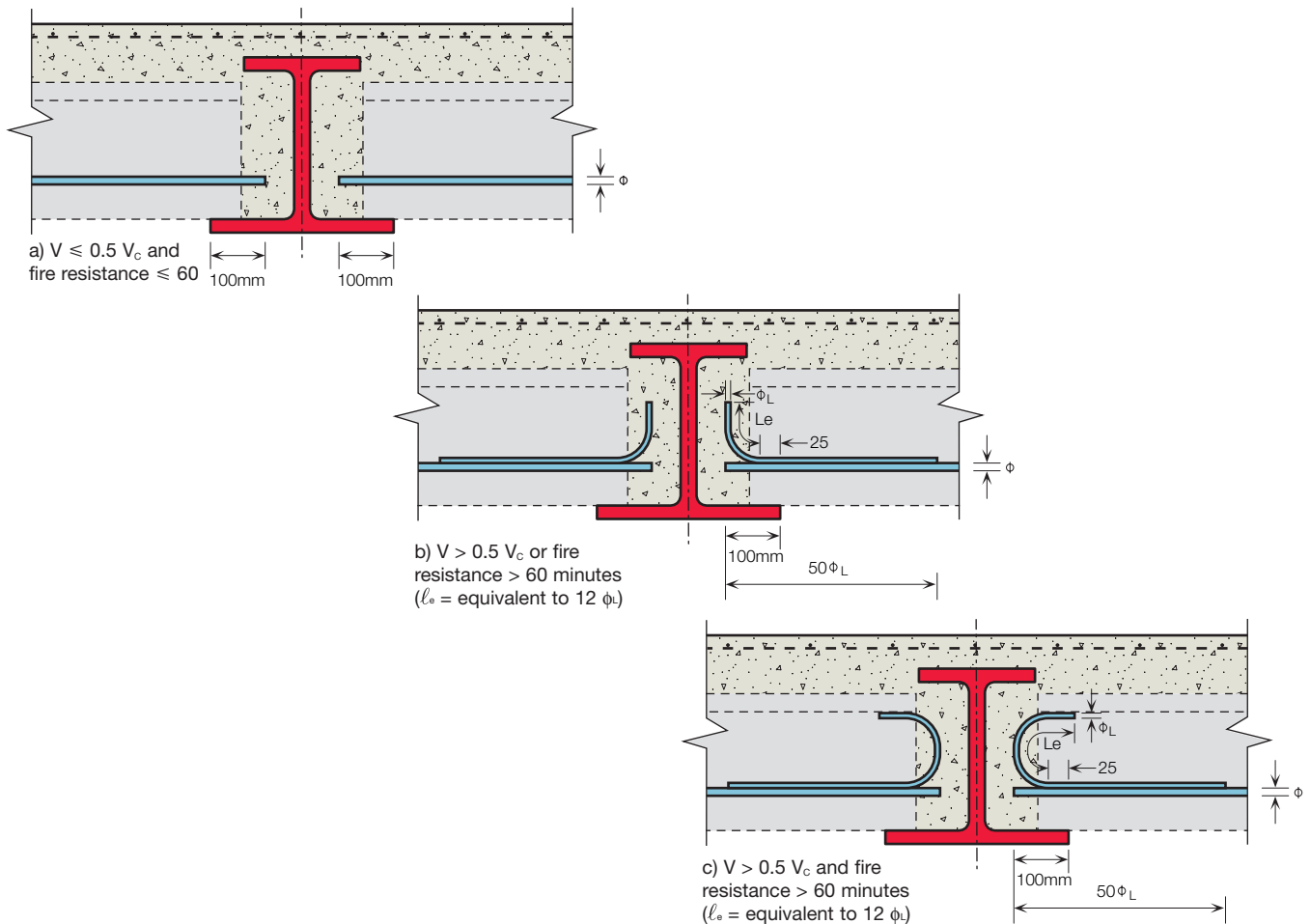


Figure 2.8 Detailing of bar reinforcement in slabs for end anchorage

### 2.2.3 Partial continuity

Tests have shown that the ComFlor 225 composite slab supported on steel beams and provided with adequately detailed continuity reinforcement over the steel beams exhibits a degree of continuity at the supports. The following beneficial effect of partial continuity at the supports may be taken:

- 20% reduction in the deflections of the composite slab at the normal design stage.

The reduction in deflection is justified from a model that assumes partial continuity at the supports.

When reliance on partial continuity is assumed, the top mesh reinforcement should be as given in *Table 2.4: Crack control* but not less than specified in *Table 2.3: Detailing requirements*. Top reinforcement should be adequately anchored and extend at least  $0.15 \times \text{span}$  or  $45 \times \text{bar diameter}$  from the support into the span. Fabric reinforcement should continue over the span as distribution steel.

In all cases, partial continuity should be ignored when assessing the bending capacity of the composite slab at the normal design stage.

### 2.2.4 Punching shear

Where heavy point or line loads are applied to the slab, particularly during construction, punching shear through the slab topping over the crest of the decking should be checked to the rules given in BS 8110–1<sup>[8]</sup>.

### 2.2.5 Crack control

In most Slimdek applications, it is not required to satisfy strict crack control criteria where a raised access floor or some other covering is provided in heated buildings. However, designers should be aware that there are certain cases where control of surface cracking is essential.

BS 8110–2<sup>[9]</sup> recommends limiting crack widths to a maximum of 0.3mm for durability and visual reasons. For crack control, sufficient reinforcement is provided to distribute tensile strains in such a way that a number of fine cracks form, rather than single large cracks.

The common cases where crack control should be considered are those where:

- Durability is a concern, such as in a floor or roof exposed to surface moisture.
- The floor is exposed to view and is power-floated or very smooth, so those cracks are visible.
- The floor covering is extremely brittle.
- The floor is highly trafficked, so that edges of small cracks tend to break off and dust collects in the cracks.

The ability to achieve good crack control depends on:

- The maximum crack width that is acceptable.
- The span/depth ratio of the beam, which influences its flexibility and hence the tensile strains in the concrete at its supports.
- Recognition that locally 'stiff' points in the support structure may accentuate cracking (e.g., stiff tie beams or walls).
- The amount of cover and spacing of the reinforcement in the slab.
- The type of loading applied to the slab (including dynamic or impact effects).
- Whether the slab or beams are propped during construction (as de-propping results in high tensile strains in the concrete at an early age).

Steel decking acts to distribute shrinkage strains in the body of the slab so that only nominal reinforcement is required in these regions. However, composite slabs are not normally designed to resist negative (hogging) moments and, therefore, cracking at the internal supports or other 'stiff' points can occur. Beams designed as simply supported in long span or more flexible applications may experience rotations at the beam-ends sufficient to cause cracking near the connections.

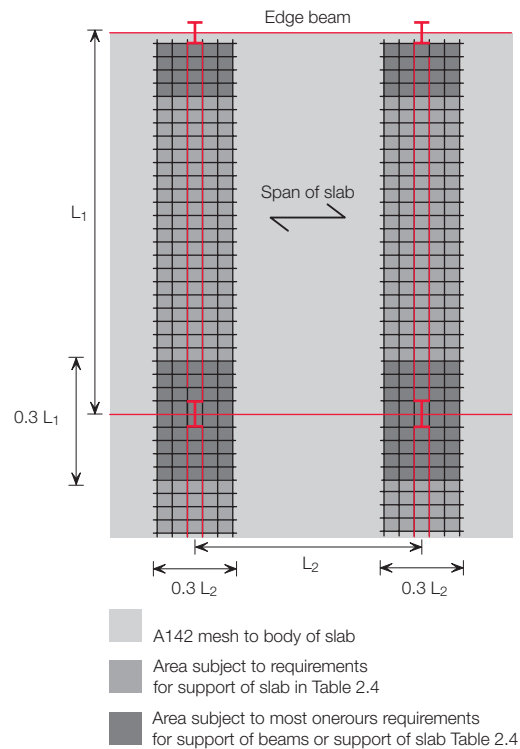
For composite slabs, the nominal reinforcement specified in BS 5950–4<sup>[3]</sup> corresponds to only 0.1% of the cross-sectional area of the slab, which is normally satisfied by using A142 mesh in the concrete topping. Often, additional reinforcement is required in the slab for 'fire engineering' reasons, or local to composite beams to act as transverse reinforcement. This can be effective in reducing cracking.

Where strict crack control is required, the amount of reinforcement in the negative moment regions of the slab or the beams should be increased significantly. This reinforcement should also be positioned as close to the surface of the slab as possible (whilst maintaining durability requirements, *Table 2.1*) in order to be effective in controlling cracking. All secondary 'stiff' points should be identified, as cracking can occur local to these points.

The minimum reinforcement requirements for crack control in deep composite slabs is summarised in *Table 2.4*. The characteristic crack width would be expected to be less than 0.3mm for durability. Clearly, more reinforcement is required if crack widths are to be reduced further.

Crack control reinforcement should extend over the negative moment region, which may be taken as 15% of the span measured from either side of the edge of the support and the mesh should continue over the entire span as distribution reinforcement. Crack control reinforcement at columns should extend along the beam  $0.15 \times$  beam span to each side of the column. The width of crack control reinforcement at columns should extend  $0.15 \times$  slab span to each side of the column, as shown in *Figure 2.9*.

The amount of reinforcement should be distributed uniformly by restricting the maximum bar spacing to 200mm. The cover of the mesh to the top of the slab should be the minimum required for durability, see *Table 2.1*.



**Figure 2.9** Crack control mesh in the slab

Table 2.4 Minimum reinforcement in composite slabs						
Location of reinforcement			Minimum reinforcement			
			Recommended for crack control and/or partial continuity of slab (see note 2)		Other cases	
Body of composite slab			A142		A142	
Supports of unpropped composite slabs (see note 4)			A142		A142	
Supports of propped composite slabs (see note 4)	Slab span up to 7m	Slab span up to 8m	A193		A142	
			A252		A193	
			A393		A193 (see note 3)	
			Recommended for crack control (see notes 1&2) Other cases (see note 1)			
Supports of beams (see note 5)	Unpropped slabs and beams	Slab depth 300mm	0.3%	A252 or 2 x A142	0.1%	A142
		Slab depth 325mm		A393 or 2 x A193		A142
		Slab depth 350mm		A393 or 2 x A193		A142
	Propped slabs or propped beams	Slab depth 300mm	0.5%	A393 or 2 x A193	0.2%	A142
		Slab depth 325mm		2 x A252		A193
		Slab depth 350mm		A252 + A393		A252 or 2 x A142

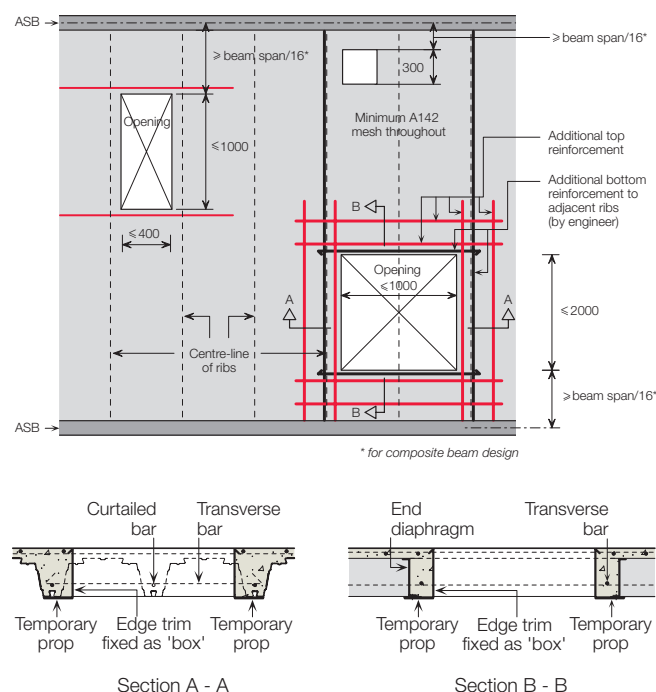
**Notes:**

1. The minimum percentage is based on the cross-sectional area of the slab topping (above the ribs of the decking).
2. The minimum mesh size given is designed to limit the characteristic crack width to 0.3mm providing the normal range of slab thicknesses are used and imposed loads do not exceed  $4\text{kN/m}^2$  (+  $1\text{kN/m}^2$  partition allowance).
3. A393 mesh is required for 120 minutes fire resistance.
4. Reinforcement should extend  $0.15 \times$  slab span on each side of supports.
5. Reinforcement at columns should extend along the beam  $0.15 \times$  beam span to each side of the column. The width of reinforcement at columns should extend  $0.15 \times$  slab span to each side of the column.

Where crack control is required adjacent to the beam-column connections, the area of reinforcement should be increased by providing either: two layers of mesh or a single layer mesh together with loose bars detailed in accordance with good practice. In such cases, the concrete cover over the beam should be increased, see *Table 2.1*.

### 2.2.6 Openings in the slab

Provision for vertical service openings within the floor slab will necessitate careful design and planning. The main cases where openings can be introduced without significant limitations are shown in *Figure 2.10* and are described as follows:



**Figure 2.10** Details of openings in the slab

- Openings up to 300mm x 300mm can be accommodated anywhere in the slab over a crest section of the decking. Additional reinforcement (other than the crack control reinforcement) is not generally required.
- Openings up to 400mm wide x 1000mm long may be taken through the crest of the ComFlor 225 decking. This allows vertical shafts or ducts to be accommodated without the need for trimming around openings. The openings are formed in three stages. Firstly, either polystyrene blocks are secured in place by wire, or shuttering is fixed in the position of the openings. Then the concrete is poured and, finally, when the concrete has gained adequate strength (normally 75% of the minimum required characteristic strength), the deck is cut away, allowing access for the vertical service runs. Additional reinforcement may be required around the

opening, which should be designed in accordance with BS 8110-1<sup>[8]</sup>. If the opening lies within the effective width of the slab, i.e., beam span/8, the supporting beam should be designed as non-composite.

- Openings of up to 1000mm wide x 2000mm long may be accommodated by removing one rib (maximum) of the decking. Standard edge trim is pre-fixed as a 'box' around the opening. Closures are required to support the discontinuous decking and to contain the wet concrete. Additional reinforcing bars are required in the transverse RC beam to transfer forces from the discontinuous rib. Additional reinforcement in accordance with BS 8110-1<sup>[8]</sup> is required in the topping to control crack formation at the corners of the opening. Headers and propping will be required until the concrete has gained sufficient strength and should be in position before the deck is laid so that the 'box' is supported adequately from the propping. The slab should be designed as a ribbed slab in accordance with BS 8110-1<sup>[8]</sup>, with the decking being used as permanent formwork. If the opening lies within the effective width of the slab, i.e., beam span/8, the supporting beam should be designed as non-composite.
- Larger openings will generally require trimming by secondary beams. Care must be taken to identify all the flashing and closure pieces required.

A close grouping of penetrations transverse to the span direction of the deck should be treated as a single large opening. Additional reinforcement or trimming steelwork may be required.

### 2.2.7 Electrical trunking

It is also possible to create routes for electrical trunking which are embedded in the slab. These can be located either within the structural slab or within a structural or non-structural screed. Further information is given in *Section 5.3*.

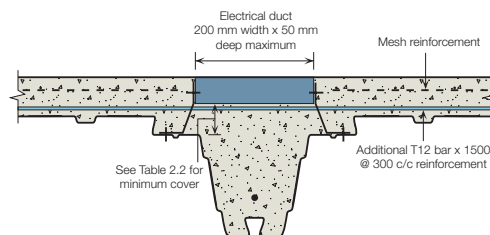
#### 2.2.7.1 Electrical duct within structural slab

Trays should generally pass above the ribs so that the fire resistance of the slab is not affected, *see Figure 2.11*. The structural design of the floor should take account of the reduced cross-sectional area of the slab. In unpropped construction, to achieve a level floor surface care must be taken to pre-camber the electrical tray to allow for deflection of the deck during concreting. Propped construction dispenses with the need to pre-camber but increases the requirement for crack control over supports, *see Section 2.2.5*.

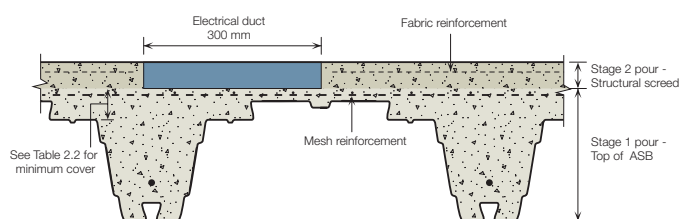
#### 2.2.7.2 Electrical duct within screeds

Trays within non-structural screeds provide full flexibility on size and location. However, reduced concrete depths can be achieved by locating trays within a structural screed as part of a second stage concreting operation, *see Figure 2.12*. Stage 1 concreting is taken to level with the top of the ASB section. Trays are then positioned and the stage 2 concreting completed. The surface of the stage 1 concrete should be suitable for receiving the stage 2 concrete to achieve adequate bond. Lateral trays crossing the ASB should be placed adjacent to columns to avoid affecting the composite action of the ASB.

The decking should be designed for the construction stage 1. The stage 1 slab may be designed compositely to support the stage 2 pour providing the concrete has gained adequate strength (normally 75% of the minimum required characteristic strength).



**Figure 2.11** Electrical duct within structural slab



**Figure 2.12** Electrical duct within structural screed (two stage pour)

**Table 2.5 Recommended deflection limits for slabs**

Design situation	Deflection limit	Comment
Construction stage: Decking only	L/180 or 20mm maximum	Weight of decking plus wet concrete (ponding excluded)
	L/130 or 30mm maximum	Weight of decking plus wet concrete plus allowance for ponding
Composite stage: Imposed load deflection	L/350 or 20mm maximum	Unfactored imposed load only
Composite stage: Superimposed dead load plus imposed load	L/250 or 30mm maximum	Unpropped Unfactored total load deflection less the deflection due to self-weight of the slab
		Propped Unfactored total load deflection less the deflection due to self-weight of the slab plus the deflection due to prop removal

Notes:

1. Contract specifications may give other limits.
2. These limits may be varied where greater deflection will not impair the strength or efficiency of the floor.
3. Where soffit deflection is considered important (e.g., for service requirements or aesthetics) it may be necessary to reduce these limits.
4. The deflection of the decking (construction stage deflection) should be based on unfactored dead loads only – i.e., construction loads are not considered.

## 2.2.8 Serviceability

### 2.2.8.1 Deflections

Composite slabs using ComFlor 225 decking are relatively stiff compared to slabs using shallow decking. The recommended deflection limits for composite slabs are given in *Table 2.5*. If the simply supported deflection exceeds the limit chosen, the slab depth and/or reinforcement can be increased or consideration can be given to concrete type, propping and/or partial continuity of the slab.

It is recommended that the increased weight of concrete resulting from ponding, see *Section 2.2.1* is included in the design of the support structure if the predicted deflection excluding the effect of ponding exceeds 10% of the overall slab depth.

### 2.2.8.2 Stiffness of composite section

Elastic design is used at the serviceability limit state for deflection calculations. The stiffness of composite slabs is found by taking account of the area of the concrete rib and the effective breadth of the concrete flange. The calculation of stiffness is based on the average of the cracked and uncracked section in which the concrete area is divided by an appropriate modular ratio (ratio of the elastic modulus of steel to concrete) given in *Table 2.6*. For propped slabs, the modular ratio should be modified to take account of the long term nature of the self weight loading, see *Table 2.6*.

### 2.2.8.3 Serviceability stresses

When the decking is loaded during construction, it is stressed to a certain level. This stress, which is calculated using serviceability loads, is locked in when the concrete has gained its strength.

The construction stage stresses are added to the normal stage stresses. Combined stresses should not exceed the value of  $p_y$  for the decking in tension and compression. Similarly, the concrete compressive stress should not exceed  $0.5 f_{cu}$ . These stresses are rarely significant and the SIDS software package carries out these checks.

**Table 2.6 Modular ratios of steel to concrete**

Loading Conditions	NWC	LWC
Short-term loading	6	10
Long-term loading	20	25
Normal imposed loading	10	15

## 2.3 Structural design – beams

### 2.3.1 General

There are two distinct phases for which the elements of the Slimdek system have to be designed:

- The construction phase, during which the beams and decking support the loads associated with construction of the floor slab i.e., wet concrete plus loads from personnel plus equipment, and
- The normal phase during which the elements of the system act together to support loading associated with use of the building in addition to the self-weight of the floor.

In the construction phase, the beam is assumed to be unrestrained by the surrounding concrete and subject to torsional loads resulting from placement of concrete on one side of the beam only. In the normal phase, the beam is assumed to be restrained by the surrounding concrete and, dependent upon the selected slab dimensions, the system can be designed to be composite or non-composite.

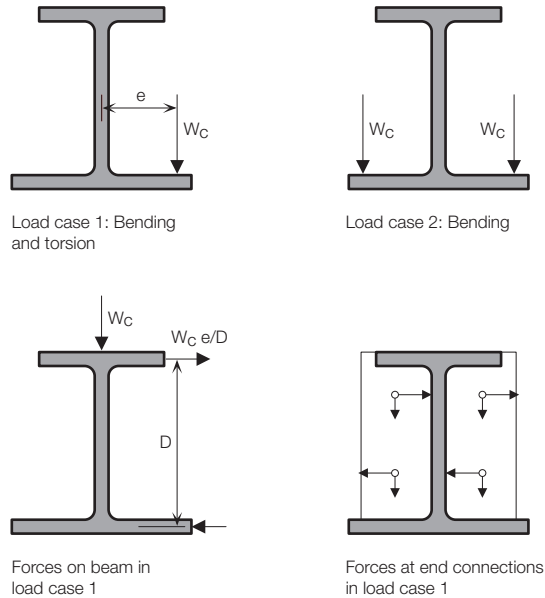
### 2.3.2 Construction stage

#### 2.3.2.1 Lateral torsional buckling (LTB)

Beams are subject to combined bending and torsion, and lateral torsional buckling in the construction stage. For ASBs, this is dealt with in a manner similar to that for conventional construction in BS 5950-1<sup>[6]</sup>. The torsional effect occurs when one side of the beam is loaded and forces are resolved as in *Figure 2.13*. The horizontal force causes transverse bending in the flanges which is combined with the longitudinal bending stresses.

A reduced effective length may be used because of the partial restraint offered by the end connections. The beneficial stabilising effects of the bottom flange loading are generally ignored.

The parameters for design for this condition are contained in the section property tables given in *Table 1.3* and *Table 1.5*. The SIDS software checks all beams for lateral torsional buckling and bending and torsion in the construction stage.

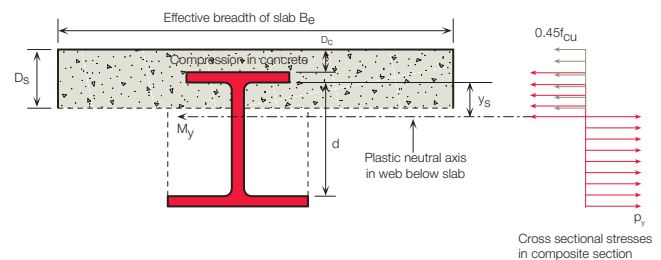


**Figure 2.13** Combined bending and torsion on ASB during construction

### 2.3.3 Normal stage – ASB

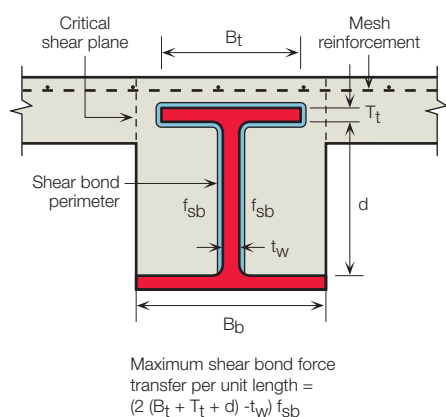
#### 2.3.3.1 ASB – Moment capacity

ASBs achieve a degree of composite action due to the shear bond developed between the steel and concrete around the beam which is enhanced by the embossed pattern rolled into the top surface of the beam. The design value of the bond stress  $f_{sb}$  that is considered to act around the perimeter of the beam excluding the bottom flange is taken as  $0.6N/mm^2$ . The composite action demonstrated in full-scale tests was over  $1.0N/mm^2$ , see *Appendix A*. This bond strength is sufficient to achieve the minimum degree of shear connection in BS 5950-3.1<sup>[7]</sup>. The plastic stress blocks of the composite ASB section are shown in *Figure 2.14*.

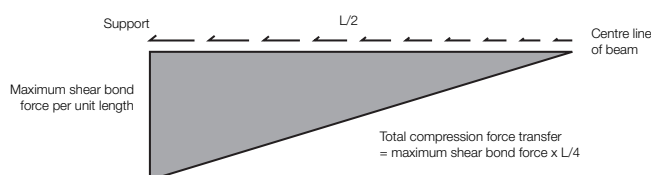


**Figure 2.14** Composite design using ASB

Shear bond failure is potentially non-ductile and hence shear flow along the beam is assumed to be linear rather than uniform, which is the case for conventional shear connectors, see *Figure 2.15*.



a) Shear bond transfer around the internal surface of the section



b) Elastic shear transfer along the beam

**Figure 2.15** Shear bond in ASB used in determining composite properties (uniformly loaded beam).

Where composite action is required, the cover to the top surface of the upper flange should be a minimum of 30mm (or as required by *Table 2.1* for durability). Where the depth of concrete is between 30 and 60mm, composite action is achieved and the full value of the compression resistance provided by the concrete flange may be used. If the concrete depth above the top flange is greater than 60mm, composite action is still developed but only the first 60mm of the concrete compressive flange should be used in the design. The effective breadth of the slab is taken as  $\text{span}/8$ , which is half the amount used in conventional composite construction. This limits the degree of composite action and compensates for the modest amount of reinforcement placed over the beams.

### 2.3.3.2 ASB – Bottom flange loading, torsional effects and web bending

In Slimdek construction, the bottom flange is loaded, giving rise to different failure modes that must be considered in the design procedure. Loading from one side during construction results in an out of balance moment being applied to the beam that causes transverse bending moments in the flanges. These moments are resisted by the torsional rigidity of the beam and by the connections at the ends of the beams.

Web bending can occur where sections are predominantly loaded on one side. Its effects are most significant in thin web ASB sections. This aspect is covered by the SIDS software and can be ignored for scheme design.

### 2.3.3.3 ASB – Shear resistance

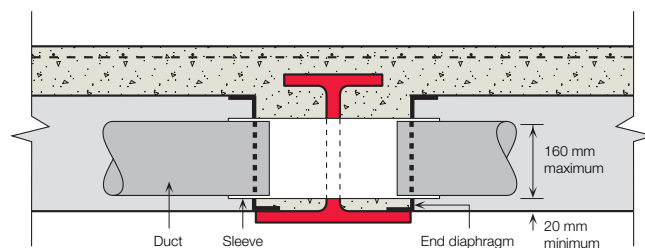
The vertical shear resistance of the steel section should be checked for its influence on the moment capacity of the section. This is only likely to be critical where secondary beams cause high local loads, or where the section is highly perforated for services.

### 2.3.3.4 ASB – Transverse reinforcement

Transverse reinforcement is required in composite construction in order to transfer the longitudinal force generated by the shear-bond into the concrete compression flange without splitting of the concrete. The concrete, the mesh reinforcement and any additional reinforcing bars that cross each shear plane contribute to the total longitudinal shear resistance. For ASBs, use of A142 or A193 mesh is usually sufficient, except where required for crack control, see *Table 2.4*.

### 2.3.3.5 ASB – Web openings

Full integration of services can be achieved by providing openings through the ASB midway between the ribs of the deep decking. During fabrication, an opening (usually circular or oval) is cut in the web of the ASB. The position and detailing rules of openings in the webs of ASB and ASB(FE) sections are given below.



**Figure 2.16** Forming openings through ASB

The same-sized openings are also cut in the diaphragms that fit between the ribs and a 'sleeve' is placed through the beam and diaphragms before the concrete is placed. The elements that form the opening in an ASB are shown in *Figure 2.16*. Flat, oval or circular ducts may be placed inside the sleeve and sealed externally.

### 2.3.3.6 ASB – Maximum sizes of openings

The maximum acceptable size of the openings in the web of the ASB has been established by full-scale tests. Although the loss of area of the web mainly affects the shear resistance of the beams, tests have shown that the concrete encasement also provides considerable shear resistance in this region. The following empirical rules have been established for the maximum size of openings in ASB. The thicker web ASB(FE) sections possess sufficient shear resistance even when perforated by large openings.

#### ASB sections:

Either:

- 160mm deep x 320mm long elongated openings (suitable for 152mm x 318mm flat oval ducts) centrally between the ribs over the middle half of the beam span but not within 1500mm of the supports, or
- 160mm diameter circular openings but not within 1000mm of the supports.

#### ASB(FE) sections:

Either:

- 160mm deep x 320mm long elongated openings (suitable for 152mm x 318mm flat oval ducts) centrally between the deck ribs but not within 450mm of the supports, or
- 160mm diameter circular openings but not within 450mm of the supports.

The base of all openings should be detailed at a standard dimension of 20mm above the top surface of the bottom flange, independent of their depth. This is done so that the opening avoids the root radius of the section and fits within the deck shape. These detailing rules for openings are summarised in *Figure 2.17*.

### 2.3.3.7 Effect of openings on structural performance

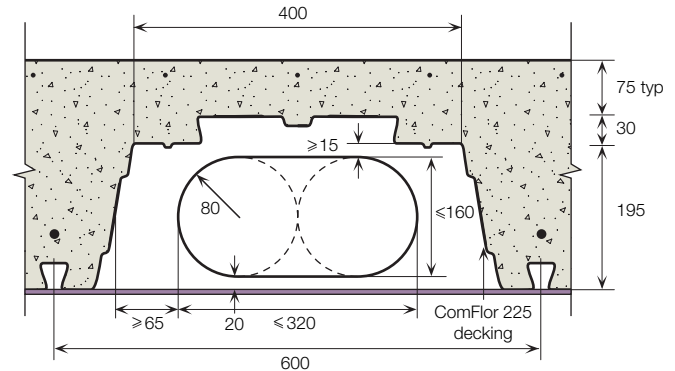
Openings formed in the webs of ASBs affect shear and bending resistance. The transfer of shear across the openings causes local bending effects (or Vierendeel bending) in the remaining web-flange sections. Shear and bending resistance is greatly improved by composite action of the steel section with the concrete encasement.

ASB(FE) sections generally possess more than sufficient shear resistance in normal conditions, even when perforated.

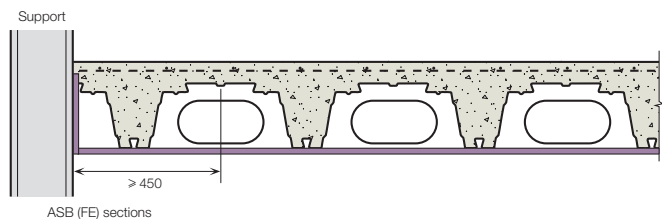
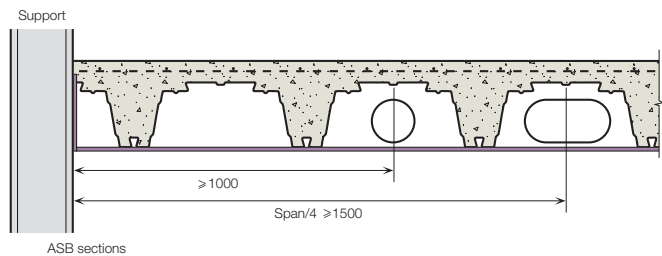
ASB (thinner web) sections may possess insufficient shear resistance when perforated by large elongated openings and should be checked for the particular grid size and loads.

The effect of loss of part of the web on bending resistance and stiffness of ASB and ASB(FE) sections is less significant, as both the plastic and elastic neutral axes of the section are close to the centre of the openings.

The local Vierendeel bending resistance determines the maximum length of elongated openings. The composite properties of the upper web-flange section may be used. Vierendeel action is not critical for circular openings and openings can be located closer to the supports, see *Figure 2.17(b)*.



a) Maximum size of opening in ASB



b) Location of openings along the beam

Figure 2.17 Detailing rules for web openings in ASB

The following empirical formulae have been determined for the effect of openings on the shear and bending resistance and the second moment of area of the ASB sections, acting compositely with the concrete encasement:

$$V_{c,o} = V_c (1 - D_o/D)$$

$$M_{c,o} = M_c (1 - 0.4 D_o/D)$$

$$I_{c,o} = I_c (1 - 0.2 D_o/D)$$

where:

$V_c$  is the shear resistance of the unperforated section

$V_{c,o}$  is the shear resistance of the perforated section

$M_c$  is the bending resistance of the unperforated composite section

$M_{c,o}$  is the bending resistance of the perforated composite section

$I_c$  is the second moment of area of the unperforated composite section

$I_{c,o}$  is the second moment of area of the perforated composite section

$D_o$  is the diameter of the opening

$D$  is the beam depth

N.B. For fire resistance greater than 30 minutes, most ASB and ASB(FE) sections with partial concrete encasement require fire protection to the exposed bottom flange, see Section 2.3.7.

### 2.3.4 Normal stage – Edge beams

There are various alternatives for edge beams in Slimdek construction. These are:

- Conventional downstand beams
- Rectangular Hollow Slimflor beams
- Asymmetric Slimflor beams.

Typical arrangements are shown in Figure 2.18(a) to (d).

Conventional downstand beams are the most cost-effective solution where they can be accommodated within the thickness of the perimeter wall because the loads pass through the shear centre of the beam and there is no torsion in the construction phase.

In situations where the façade of the building will not permit downstand beams the RHSFB is most effective because of its enhanced torsional properties. However, the slab thickness is frequently dictated by internal beam requirements and the design thickness may not be sufficient to accommodate shear studs which are required for edge beams to act compositely (i.e., Figure 2.18(c)).

ASBs may also be used in this situation, provided that the torsional effect due to eccentric loading on the beams is taken into account.

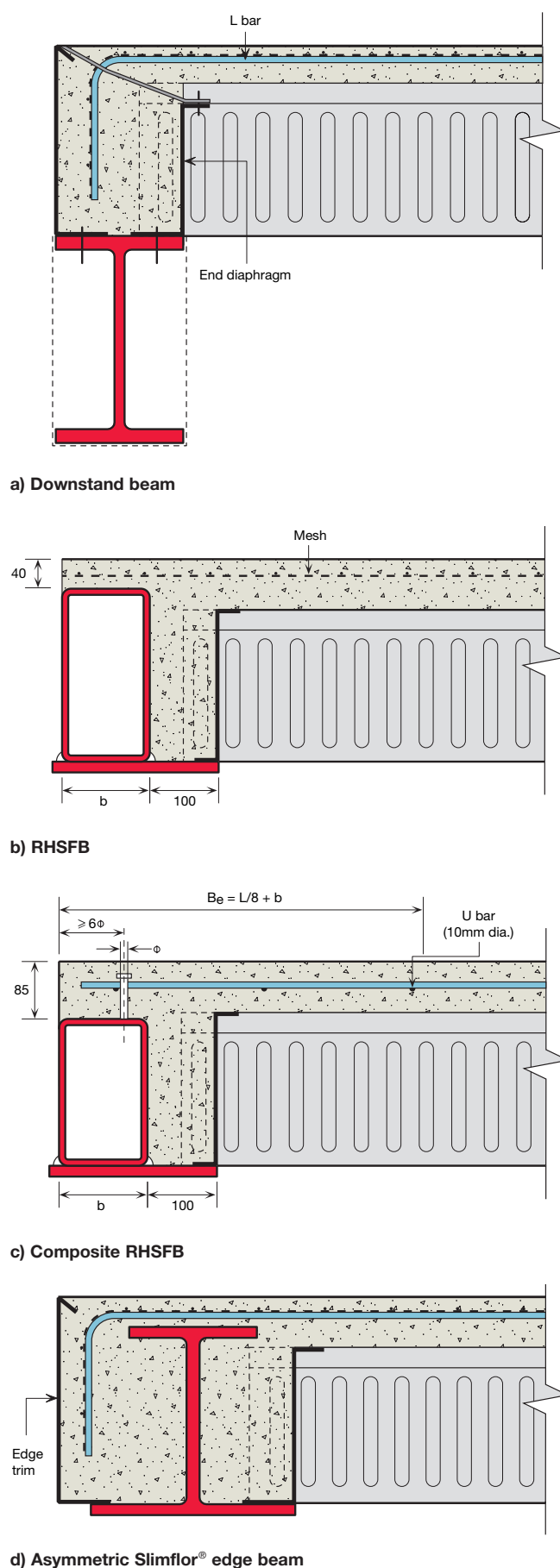
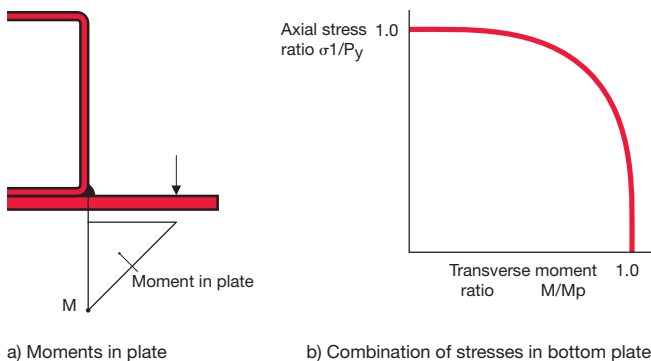


Figure 2.18 Typical edge beams

The designer must also be aware of the design issues that affect edge beams, in particular:

- Extra transverse reinforcement may be required in the form of U bars see *Section 2.3.4.4*.
- Deflection limits may need to be reduced for brittle finishes or cladding, such as glazing.
- Torsional effects of one-side loading have to be considered in combination with lateral torsional buckling.
- Propping of the edge beam may be required in composite applications.
- The beam may project above the slab in longer spanning applications.
- RHSFB provides a neat edge to the slab for attachment of cladding, or in glazed façades.
- RHSFB can be designed as non-composite or composite, as in *Figure 2.18(b) and 2.18(c)*.

Note that the containment of the concrete slab between an internal beam and an RHSFB or an ASB section edge beam will allow diaphragm action. However, when downstand edge beams are used, positive shear connection is required for diaphragm action. When necessary, this could be provided by shear connectors along the edge beam.



$\sigma_1$  = Longitudinal stress

$M$  = Maximum transverse moment applied to plate

$M_p$  = Moment resistance of plate =  $P_y t_p^2 / 6$

$t_p$  = Plate thickness

$P_y$  = Yield stress

**Figure 2.19** Biaxial stresses in flange plates of RHSFB

#### 2.3.4.1 RHSFB – Moment capacity

The moment capacity of the composite RHSFB is dependent on the degree of shear connection between the steel beam and the concrete compressive flange. The effective breadth ( $B_e$ ) of the compression flange is taken as  $(\text{Span}/8 + B/2)$  for edge beams. The majority of beams will be designed for partial shear connection where the number of shear connectors used is less than that required to achieve full shear connection. A full derivation of the moment capacity of these sections is given in SCI-P-169<sup>[18]</sup>.

Shear connectors are usually 19mm diameter x 75mm headed studs giving 70mm as-welded height. Their design shear resistance is typically 70kN for C30 concrete. Sufficient transverse reinforcement looped around the shear connectors is also required to transfer this force into the slab, see *Section 2.3.4.4*. Composite action will increase the load or spanning capabilities and stiffness of a given beam size.

#### 2.3.4.2 RHSFB – Bottom flange loading, torsional effects

It is assumed that all floor loads are transferred to the RHSFB by the bottom flange plate. Consequently, the flange plate is subject to biaxial stresses resulting from the combination of longitudinal stress ( $\sigma_1$ ) due to overall section bending and transverse stress due to the out of balance loads. The capability of the flange plate to resist transverse stress reduces as the longitudinal stress increases and vice versa. The relationship between  $\sigma_1$  and the transverse moment capacity (expressed as a proportion of  $M_p$  the maximum resistance moment of the plate) is illustrated in *Figure 2.19*.

A rigorous method of analysis to combine the longitudinal bending effects with the torsional effects arising from out of balance loading is presented in SCI-P-057<sup>[18]</sup>.

#### 2.3.4.3 RHSFB – Shear resistance

The vertical shear resistance of the steel section should be checked for its influence on the moment capacity of the section. This is only likely to be critical where secondary beams cause high local loads, or where the section is highly perforated for services.

#### 2.3.4.4 RHSFB – Transverse reinforcement

Transverse reinforcement is required in composite construction in order to transfer the longitudinal force generated by the shear connectors into the concrete compression flange without splitting of the concrete.

The concrete, the mesh reinforcement and any additional reinforcing bars that cross each shear plane contribute to the total longitudinal shear resistance. Generally, the use of A142 and A193 mesh is sufficient, except where required for crack control, see *Table 2.4*.

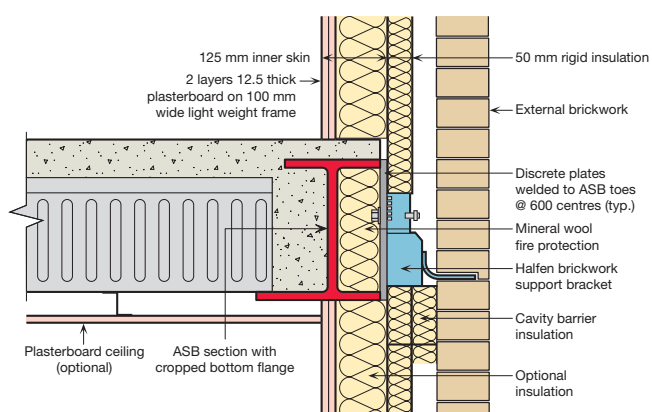
RHSFBs require additional U-bar transverse reinforcement where the slab edge is less than 300mm from the nearest row of shear connectors. The studs should not be less than 6 x stud diameter from the slab edge. Generally this requires studs to be offset from the centre of the RHS. The diameter of the U-bars should be not less than 0.5 x stud diameter (i.e., 10mm). Detailing rules for RHSFB are shown in *Figure 2.18(c)*.

#### 2.3.4.5 RHSFB – Plate and weld design

The normal plate thickness is 15mm with a minimum plate thickness of 12mm. It is recommended that a minimum 6mm continuous fillet weld is used for connection of the flange plate.

#### 2.3.4.6 ASB edge beams with bottom flange cropped

When ASB sections are used as edge beams with a concentric connection to the columns, it may be convenient to crop the bottom flange on one side to prevent it obstructing the cavity. A typical detail through an external wall with a cropped ASB is shown in *Figure 2.20*.



**Figure 2.20** Typical section through an external wall with a cropped Asymmetric Slimflor® edge beam

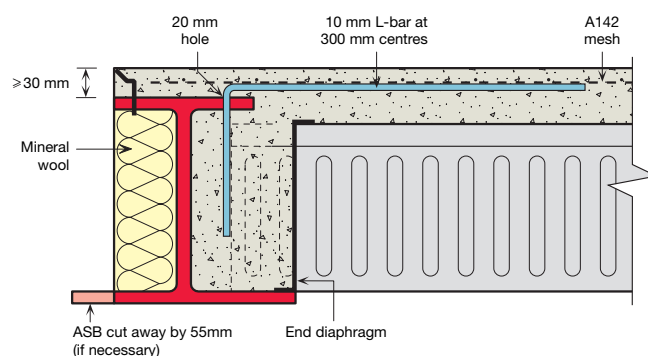
The bare steel properties of a cropped ASB will clearly be slightly less than an uncropped section. Composite action of a cropped ASB may be assumed when there is some form of shear key between the section and the concrete. This may be achieved by inserting 'L' bars bent down through pre-drilled holes in the inner top flange, as shown in *Figure 2.21* or by welding reinforcing bars to the top flange of the section.

Note that when reinforcement is tied to the ASB, it will enhance the torsion resistance, improve the diaphragm action of the floor and improve the robustness of the system, so it is recommended:

- With shear key – composite action may be assumed at both ULS and SLS. The encased concrete, and the concrete above the decking up to a width of span/16 from the beam centre-line, may be assumed to act compositely with the steel section.
- Without a shear key – no composite action may be assumed at the ULS, but composite action with only the encased concrete between the flanges may be assumed at the SLS.

A robust, properly-fitted fire stop is always required between the bottom flange and the outer skin to fill the cavity.

- 60 minutes fire resistance (R60) can be achieved with brickwork or masonry cladding, and the outer face of the ASB does not require any special insulation. Lightweight curtain walling is not suitable with cropped Asymmetric Slimflor edge beams requiring R60.
- 30 minutes fire resistance (R30) can be achieved with normal lightweight curtain walling or any lightweight cladding. However, the external face of the ASB must be insulated, as shown in *Figure 2.21*.



**Figure 2.21** Shear key/tying reinforcement with a cropped Asymmetric Slimflor® edge beam

Cold bridging may be an issue in buildings when supporting external cladding from brackets which connect directly to the steelwork. The thermal transmittance and the internal temperature effects (expressed as a temperature factor) are known to be greatly affected by the cladding bracket spacing. Studies were carried out on brickwork support brackets supported off cropped ASBs and the results showed that:

- No insulation is required for thermal performance on the external face of the ASB, but it may be needed for fire resistance.
- No insulation is needed to the underside of the ASB for thermal performance.
- No special measures are recommended to insulate brickwork support angle brackets from the ASB at bracket spacings of greater than 600mm.
- Heat loss through the edge beam may need to be considered for bracket spacings less than 600mm. Thermal spacers between the bracket and the ASB may be appropriate in such cases, but advice should be sought from the bracket manufacturers.

See also The Avoidance of Thermal Bridging in Steel Construction SCI-P-380<sup>[32]</sup>

### 2.3.5 Temporary propping

Propping of the beams or deck used in the Slimdek system may be required for particular spans of beam and deck. The deck will require propping where its span is greater than 6.4m for lightweight concrete, or 5.9m for normal weight concrete (equal to 6.1m beam centres). These figures will reduce as the concrete depth increases for different design situations, see *Tables 3.1 – 3.4*. It is not normally necessary to prop the beams except for:

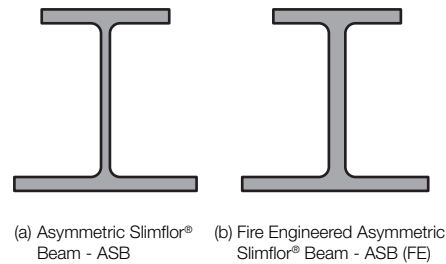
- long span ASBs (> 8m span)
- composite RHSFBs

Where propping is used, there is an increased likelihood of cracking of the slab when the props are removed. Where necessary, crack control reinforcement should be used, see *Section 2.2.5*.

### 2.3.6 Fire resistance

Two types of ASB sections are available:

1. ASB – asymmetric sections with thin webs that require fire protection to the exposed bottom flange to achieve more than 30 minutes or longer fire resistance as shown in *Figure 2.22(a)*.
2. ASB(FE) – asymmetric sections that are designed for optimum characteristics in the normal and fire conditions. The beams have a thick web to achieve a fire resistance up to 60 minutes when unprotected assuming that the integrity of the web is not breached as shown in *Figure 2.22(b)*.

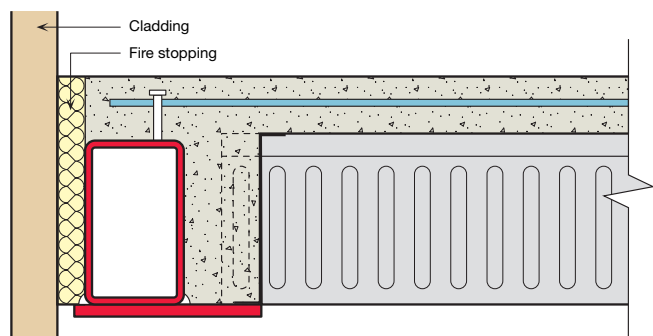


**Figure 2.22** Types of ASB Slimflor® beam

For more than 60 minutes fire resistance, all of the solutions require additional fire protection to the bottom flange only. This requirement also applies if the beam web is heavily perforated for services, see *Section 2.3.7*.

The cost of fire protection has fallen significantly in recent years and, as application is only necessary to the exposed bottom flange of the ASBs, the area to be protected is relatively small. The optimum solution will be different for each project and will depend on the relative cost of steel and fire protection materials. Issues to take into account when considering the best means of providing fire protection are the size of the project and the desirability of having additional trades on site.

RHSFB sections can achieve 60 minutes fire resistance when unprotected, except where the load ratio is very high. The bare outer face of the RHS should be protected by fire stopping for compartmentation as shown in *Figure 2.23*.



**Figure 2.23** Fire compartmentation

The fire resistance of these components has been justified by full-scale fire tests at Efectis (formerly TNO) in The Netherlands. Additional ASB tests have been carried out at Warrington Fire (formerly Warrington Fire Research Station). Details of fire tests are given in *Appendix A*.

Fire resistance of Slimdek components can be assessed using the SIDS software.

### 2.3.7 Effect of web openings

Most perforated ASB and ASB(FE) sections with partial concrete encasement provide at least 30 minutes fire resistance. For more than 30 minutes fire resistance at a high load ratio, perforated ASB and ASB(FE) sections with partial concrete encasement require additional fire protection to the exposed bottom flange. Providing that web shear does not govern the design, ASB (thin web) sections will generally be the most economic solution. Unprotected ASB(FE) sections with perforations outside the middle third of the span may provide up to 60 minutes fire resistance at low load ratios.

The exposed steel perimeter of the web openings does not require additional protection other than that provided by the concrete encasement. This has been confirmed by a fire test, see *Appendix A* and by thermal modelling.

### 2.3.8 Serviceability

#### 2.3.8.1 Deflections

Slimdek construction is relatively slender and may be more controlled by deflection limits than conventional downstand solutions. The second moment of area may be enhanced by taking account of the encasement concrete between the flanges. If the simply supported deflection exceeds the limit chosen, the beam size can be increased or consideration can be given to concrete type, propping and/or partial continuity of the beam. The generally accepted deflection limits are given in *Table 2.7*.

For composite design using unpropped construction, the total deflection is the sum of the construction stage deflection of the steel section, plus the superimposed dead load and the imposed load deflection on the composite section. The deflection of the steel beam (construction stage deflection) should be based on unfactored dead loads only – i.e., construction loads are not considered.

For composite design using propped construction, the total deflection is the sum of the dead load and imposed load deflection of the composite section.

The effect of partial continuity of the slab may increase the load on the first internal ASB support beam in excess of the load consideration for a simply supported condition. This may also occur when additional reinforcement is used over slab supports to satisfy crack control or other requirements. However, as the internal ASB support beams deflect, they provide a ‘sinking support’ that reduces both the moment in the slab over the support and the additional load attracted to the first internal ASB. Grillage analysis has demonstrated that partial continuity of the slab will not increase the load on the first internal beam by more than 10%. Partial continuity is not normally utilised at ultimate limit state, therefore the increase in loading to the first internal ASB need only be considered at the serviceability limit state where partial continuity is assumed or crack control measures are taken.

Beams may be pre-cambered to take account of dead load deflections, but this is only recommended for long spans (> 9m) and special applications.

#### 2.3.8.2 Stiffness of composite section

The stiffness of composite beams is found by taking account of the area of the concrete encasement and the effective breadth of the concrete slab. The calculation of stiffness is based on the uncracked section in which the concrete area is divided by an appropriate modular ratio (ratio of the elastic modulus of steel to concrete) given in *Table 2.6*. For propped beams, the modular ratio should be modified to take account of the long-term nature of the loading. The stiffness of the composite ASB is typically more than 50% greater than that of the parent steel section.

**Table 2.7 Recommended deflection limits for beams**

Design situation	Imposed load deflection	Imposed plus cladding	Total load deflection
General internal beams applications	Span/360	N/A	Span/200 ≤ 40mm
Internal beams supporting more brittle finishes	Span/500	N/A	Span/360 ≤ 25mm
Edge beam supporting cladding	Span/500	Span/360	Span/360 ≤ 25mm

Notes:

- Contract specifications may give other limits
- These limits should be reduced, if necessary, where soffit deflection is considered important, e.g., for service requirements or aesthetics

### 2.3.8.3 Serviceability stresses

When an unpropped beam is loaded during construction, its stresses are locked in when the concrete has gained its strength. The construction stage stresses are added to the normal stage stresses. Combined serviceability stresses should not exceed the value of  $p_y$  (reduced as necessary for transverse plate bending) for the plate in tension and for the section in compression. Similarly, the concrete compressive stress should not exceed  $0.5 f_{cu}$ . These stresses are rarely significant and SIDS will carry out these checks.

These checks are not strictly required for ASB sections as testing has shown that the beam behaviour is essentially elastic until well above working loads.

## 2.4 Vibration control

The sensitivity of floors needs to be checked in most buildings to avoid any vibration problems at the serviceability limit state. This should be carried out by assessing the response of the floor (slab and beams) as a whole. The requirements for special floors, as may be required in laboratories or operating theatres in hospitals, are commonly the most stringent, but the Slimdek system has been shown to perform well in these situations through tests on actual buildings.

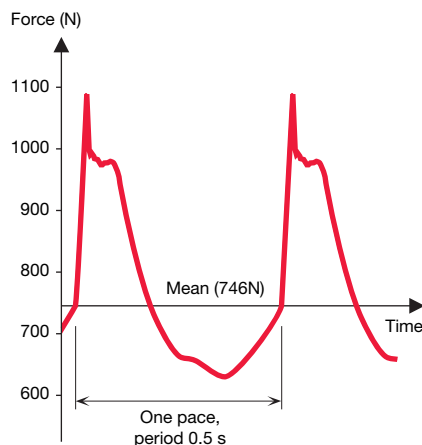


Figure 2.24 Typical force – time function for walking

Sources of vibration range from machinery and traffic to human-induced vibrations such as dancing, but the most common source of vibration in buildings is from walking.

The vibration performance of floors is no longer assessed in terms of a minimum floor frequency, but in terms of the maximum acceleration that is perceived by the occupants of the floor. The requirements are set out in BS 6472<sup>[13]</sup> and ISO/DIS 10137<sup>[14]</sup> generally, and in HTM 2045<sup>[95]</sup> for hospitals.

The magnitude of typical vibrations is very small, but the entire floor area is affected, rather than an individual beam or slab span, as continuity at the supports and across beams will be present, even when elements have been designed to be simply supported. Hand calculation methods can be used to

determine the natural frequencies and other related properties for regular grids, but advanced tools such as finite element modelling may be necessary for more complex structures. Output from an FE package is shown in Figure 2.25.

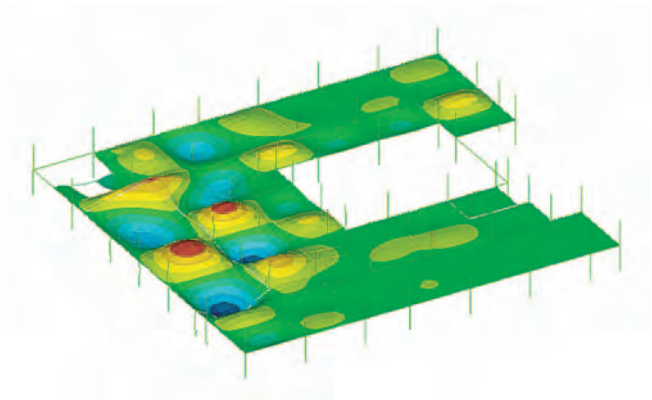


Figure 2.25 Chichester Hospital finite element model (printout)

SCI-P-354: *Design of Floors for Vibration: A New Approach*<sup>[30]</sup> presents two methods for assessing the acceptability of a floor to the British and European Standards. The first method, the general approach, uses finite element analysis to determine the properties of not just the first mode of vibration, but also subsequent modes, and then considers the superposition of the response of each of these modes to the excitation, which is usually walking. The method determines an acceleration that can be compared to the limits in the code for continuous vibration (using Response Factors) which correspond to 'a low probability of adverse comment'. In certain circumstances, however, it can be appropriate to consider the intermittent nature of the vibration through Vibration Dose Values to give an alternative assessment on the acceptability of a floor. The general approach has been calibrated against measurements on a range of buildings that have been collected by the SCI over the last 10 years.

The second method in SCI-P-354<sup>[30]</sup> is a simplified approach; this method is suitable for hand calculations on floors with a regular layout, and again produces Response Factors and Vibration Dose Values that can be compared to the codes. The results from the simplified approach may be conservative compared to the general approach, owing to the simplifying assumptions made.

Vibration performance is determined by the magnitude of the mass and stiffness of a system, and the relationship between them. Slimdek floors have a larger mass of concrete than traditional composite construction with downstand beams, but have a similar or higher stiffness-to-mass ratio due to the deeper decking profile, and this has a major effect on the natural frequency. The increased mass leads to lower accelerations from a given excitation force, and so not only are Slimdek floors generally more than adequate for use in offices, they can also meet the more stringent requirements of specialist floors, such as those required for hospitals.

Table 2.8 Measured response factors on Slimdek floors

Loading	Maximum response factor	Operating Theatre response factor
Laboratory, Cambridge	3.88	-
St. Richards Hospital, Chichester	1.10	0.29
Sunderland Royal Hospital	1.16	0.54

The ability of Slimdek to meet these requirements has been confirmed by testing on a laboratory floor in Cambridge and two hospital floors (St. Richards Hospital in Chichester, and the Sunderland Royal Hospital). As shown in Table 2.8, all three floors easily achieved the limiting Response Factor of 4 for offices or wards.

The response in the operating theatre areas from vibration induced by walking in the adjacent corridors was measured to be well below the threshold of human perception (Response Factor of 1) that is required for operating theatres in HTM 2045<sup>[35]</sup>.

In situations where a floor is found to be unacceptable, the common ways of reducing the response are: to increase the mass of the floor plate, by increasing the slab depth for example; or to introduce extra columns and/or beams into areas which have a high response to restrict the motion of the floor in that area. As an alternative to structural changes, the layout of the building in service can be altered so that corridors and walking paths pass along unresponsive areas of the floor and the effects of the excitation are reduced.

Further information on the design of floors subject to vibration is given in SCI-P-354<sup>[30]</sup>.

## 2.5 Durability

It is now widely recognised that steel is at low risk of corrosion when placed in an internal heated environment. Additionally, ASBs used in Slimdek are encased in concrete on all faces except the underside of the bottom flange. Therefore, it is not necessary to apply corrosion protection to the steel section in internal heated situations. Coatings can be applied to the underside of the bottom flange if desired for aesthetic reasons. ASB sections do not require grit blasting prior to encasement. The ASB must not be painted on faces in contact with the concrete unless designed to act non-compositely.

For RHSFBs used compositely (using shear studs), the sections should not be painted or galvanized where studs are to be welded on site. However, in most cases the studs are welded in the factory and, if necessary, galvanizing or painting could be used. In edge beam locations, where the section is partly built into the cladding system, the edges and the underside of the bottom flange or plate should be painted.

Further guidance on corrosion protection and suggested systems is given in the Corus publication *'The Prevention of Corrosion on Structural Steelwork'*<sup>[55]</sup>. For all other applications, advice should be sought from Corus or an appropriate specialist.

## 2.6 Columns

### 2.6.1 Conventional columns

Slimdek has been developed primarily as a flooring system for braced steel-framed buildings. Typically, the beams and slabs are analysed as simply supported elements. Continuity, which is inherent in the system, is only partially utilised for the serviceability criteria of deflection and vibration. It is possible to use the ASB as part of a sway frame, provided extended end plate connections are used, see Section 4.2.3. In this case, columns must be analysed for combined bending and compression.

*Advance* columns (UKCs) are recommended for internal locations because of their ease of connection. Rectangular Hollow Section (RHS) or Circular Hollow Section (CHS) columns can be used for fire resistance or architectural reasons, although CHS will require the use of welded beam stubs or fin plates. The use of fin plates is not recommended unless additional restraints, such as top and bottom cleats, are provided to resist the torsional loads at the beam-column connection.

Square Hollow Section (SHS) may be used as perimeter columns, especially when they can be located within the perimeter wall. A constant external dimension is achievable that enables the same connection details to be used throughout the height of the building, resulting in considerable savings in cutting and detailing costs.

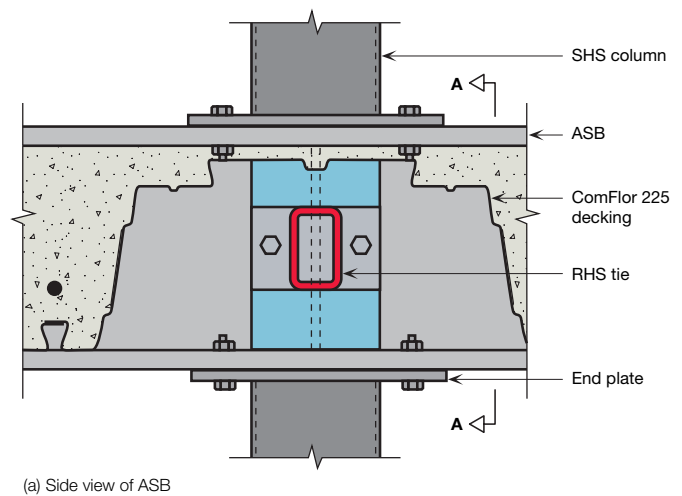
Flowdrill or Holo-Bolt connections may be used to directly bolt end plates and double-angle cleats to columns. For further details, see Corus Tubes publication TD384<sup>[36]</sup>.

### 2.6.2 Discontinuous columns

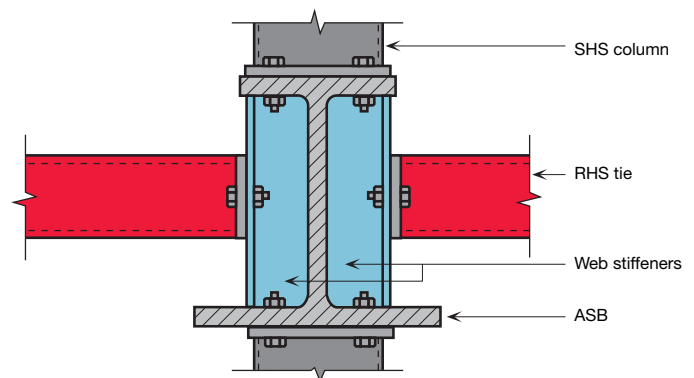
As an alternative to conventional columns, where the columns are continuous over the height of the building and the beams are attached to the faces of the columns, discontinuous storey-high members can be used with continuous floor beams in the Slimdek system. Each column length sits on top of a continuous floor beam and at its upper end the next floor beam sits on an end plate. For this solution, structural hollow sections are the most suitable section for the columns. The detail at a beam–column joint is illustrated in *Figure 2.26*. This structural solution is principally used in the residential sector but it can be suitable in the commercial sector as well. The advantage of this configuration is that, as well as the provision of a shallow overall depth of the floor plate, the clean lines of the structural hollow section columns provide clear floor solutions. The columns may be concrete-filled off site if required for load capacity or fire resistance.

Design of frames using discontinuous columns, and the design of the joints, is outside the scope of traditional design guidance. However, The Steel Construction Institute has produced a series of Advisory Desk Notes covering the use of discontinuous columns in simple construction where the concrete floor plate is contained within the depth of the floor beam, such as ASBs in Slimdek, see advisory notes AD 281<sup>[37]</sup>, AD 283<sup>[38]</sup>, AD 285<sup>[39]</sup>, AD 288<sup>[40]</sup> and AD 292<sup>[41]</sup>. These advisory notes provide guidance on construction and the design of the column, beam, floor plate and the beam–column connection.

Where discontinuous columns are used, it is recommended that full depth stiffeners are provided on the beam at the connection location, as shown in *Figure 2.26*. This provision should be made even when not required by the direct load transfer provisions of BS 5950–1<sup>[6]</sup> for the permanent condition, in order to ensure stability in the temporary condition. Advisory Note AD 288<sup>[40]</sup> provides guidance on load path and structural integrity, and on the verification of the connection components in accordance with BS 5950–1<sup>[6]</sup>. It has been demonstrated by a test programme that the resistances achieved in practice are greater than the capacities determined in accordance with BS 5950–1<sup>[6]</sup> and the design procedures in AD 288<sup>[40]</sup>.



(a) Side view of ASB



(b) Cross section A - A

**Figure 2.26** Continuous ASB and discontinuous (storey high) tubular columns

### 2.7 Robustness

Prior to 2004, the Building Regulations required that buildings having five storeys or more should not be susceptible to disproportionate collapse but this requirement now applies to all buildings. Approved Document A<sup>[42]</sup> states that this can be achieved by “...ensuring that the building is sufficiently robust...” and offers the provision of ties as one method for satisfying this requirement.

In the case of steel-framed buildings, the issue of disproportionate collapse can be addressed by providing tie members between columns which run perpendicular to the main beams, and are designed to provide for:

- Stability during construction.
- Robustness and stability of the completed construction.
- Transfer of forces (e.g., due to wind action).

They can be of various forms, as illustrated in *Figures 2.27 to 2.32*.

Design requirements for ties – which are dependent upon the building class – are given in Clauses 2.4.5.2 and 2.4.5.3 of BS 5950-1<sup>[6]</sup>.

Class 1 and Class 2A buildings, which are low-to-medium consequence structures (e.g., small residential properties and low-rise flats, offices, hotels etc), require columns to be tied in at principal floor levels in two directions and ties to be provided between columns and around the perimeter of the structure which are capable of resisting a factored tensile load of 75kN. The structure also requires ties to be provided at roof level.

For such buildings,

- The recommendations of BS 5950-1<sup>[6]</sup> clause 2.4.5.2 should be applied;
- No modifications to the rules are required for Slimdek floors
- Standard Slimdek construction details will satisfy the requirements.

Class 2B buildings which are high consequence structures (e.g., residential and office buildings between 4 and 15 storeys, hospitals less than four floors etc) require either provision of ties capable of resisting the forces defined in Clauses 2.4.5.2 and 2.4.5.3 of BS 5950-1<sup>[6]</sup> or a check to be carried out to ensure that removal of any single supporting member will not cause a disproportionate area of the structure to fall down.

For such buildings,

- The recommendations of BS 5950-1<sup>[6]</sup> clause 2.4.5.2 and the tying route from clause 2.4.5.3 should be applied with the following modifications:
- Edge tie members and their end connections should be designed for a tie force equal to 25% of the factored dead plus imposed floor load for the slab area (multiplied by  $n$ ) plus any cladding load supported by the edge member.
- Slabs spanning on to an edge beam (but not corner slabs) need not be anchored to the edge beam provided that anchorage is provided on the other three sides of the slab.
- Corner slabs need not be anchored to the edge beam provided that anchorage is provided along the two internal edges of the slab.
- Standard Slimdek construction details will generally satisfy the requirements for tying and anchorage.

Class 3 buildings, which are very high consequence structures (e.g., buildings exceeding 15 storeys, hospitals over three storeys and buildings containing hazardous materials etc) require a systematic risk assessment to be carried out including all normal and abnormal hazards.

A more detailed review of the robustness requirements for Class 2A and 2B structures plus a summary of tying and anchorage requirements are given in Appendix C.

This topic is also discussed further in SCI-P-341<sup>[29]</sup>.

The design of the tie beams is normally controlled by practical issues, e.g., fixing the decking, and practical requirements for minimum slenderness ratio. Connecting decking directly to the bottom flange of I beams is not recommended, *see Figure 2.33* due to the practical difficulties of using fixing tools in the confined space between the web of the decking and the beam flange. Similarly a totally encased tie member is not recommended, *see Figure 2.34*, as the member obstructs the fixing of the decking seam. A structural Tee, ASB or RHS with welded bottom plate is preferred.

### 2.7.1 Tee or RHS members partially encased in the slab

Where decking is unpropped, encased ties should be designed to support the construction loads. For normal situations a 300mm width of slab should be assumed to be acting on the tie member in addition to the construction load for decking mentioned in *Section 2.2.1*. Ties supporting the slab and unrestrained during construction should be designed for lateral torsional buckling. Care should be taken in detailing to ensure that the tie is not overloaded during construction.

Where decking is propped, ties may be propped and designed accordingly. Shelf plates should be welded to the column web to provide local support to the decking.

Tie members providing support to decking should be of sufficient stiffness to eliminate gaps and ensure sound fixing of the side laps.

Bar reinforcement should be used to satisfy the ultimate limit state and serviceability conditions.

### 2.7.2 Tying reinforcement

It is generally recommended that the floor is tied across or to the beams to enable diaphragm action of the floor to resist wind loads, etc. The Slimdek system involves the regular attachment of the decking via self-tapping screws or similar, as well as via reinforcement in the concrete topping at internal beam and edge beam locations, *see Section 2.3.4*.

In unusual cases when no concrete cover is provided above internal beams, so that they are designed 'flush' with the concrete, tie bar reinforcement should be provided through holes in the beam webs, as shown in *Figure 2.35*. However this option involves making holes partially within the root radius, which could be problematic.

For Class 1 and 2A buildings, the slab does not need to be tied to the beams for robustness purposes.

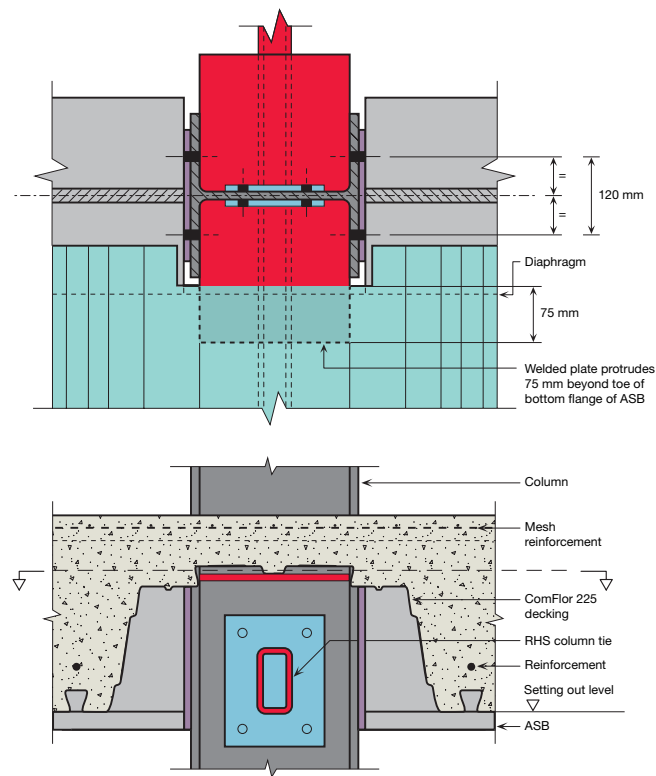
### 2.7.3 Crack control

Where tie beams support or partially support the slab and form a 'hard spot', consideration should be given to the provision of additional crack control reinforcement over and local to the tie beam, see *Table 2.4*.

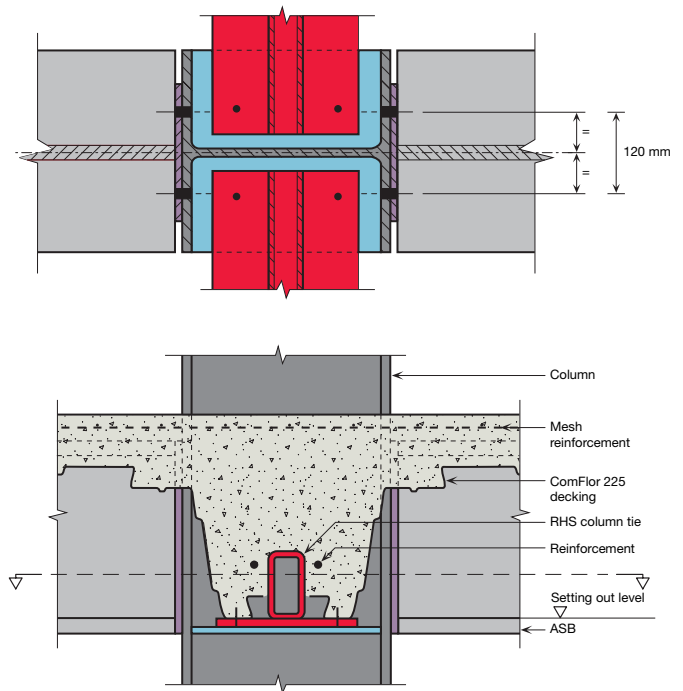
### 2.7.4 Fire resistance

Ties require fire protection if their failure in fire will cause a loss of structural integrity to the building. Ties can be protected by board, spray, mineral fibre wrapping, intumescent coating, or alternatively by embedding them in the slab. Where ties are embedded they should not be positioned above deck laps as this would restrict access for fixing the decking. Minimum cover should not be less than 50mm for C30 grade concrete for ties not less than 15kg/m.

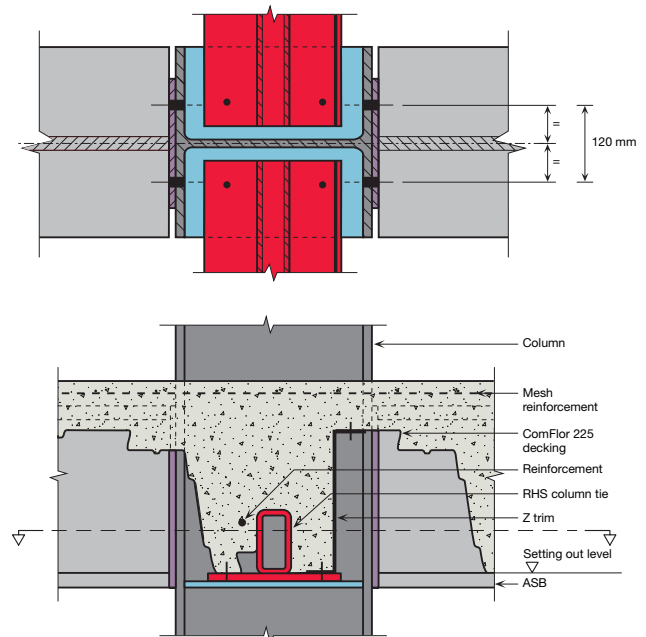
Ties installed solely for the construction phase are not usually fire protected.



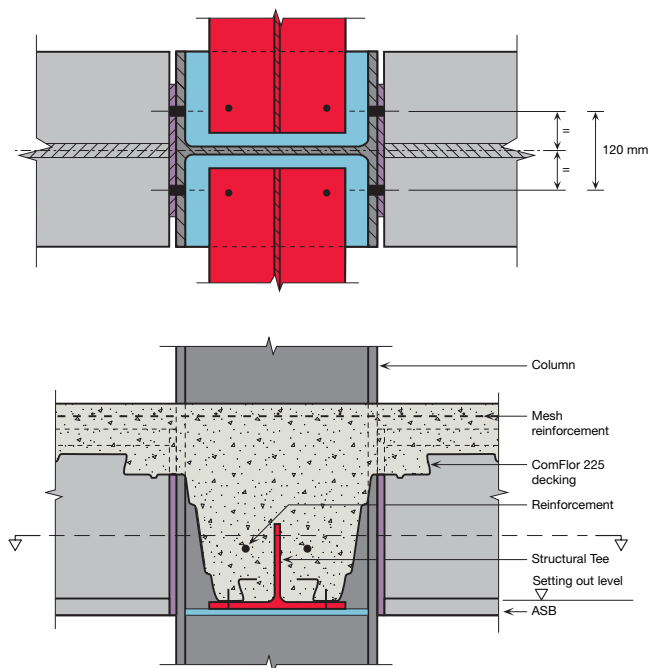
**Figure 2.27** RHS tie between ribs of ComFlor 225



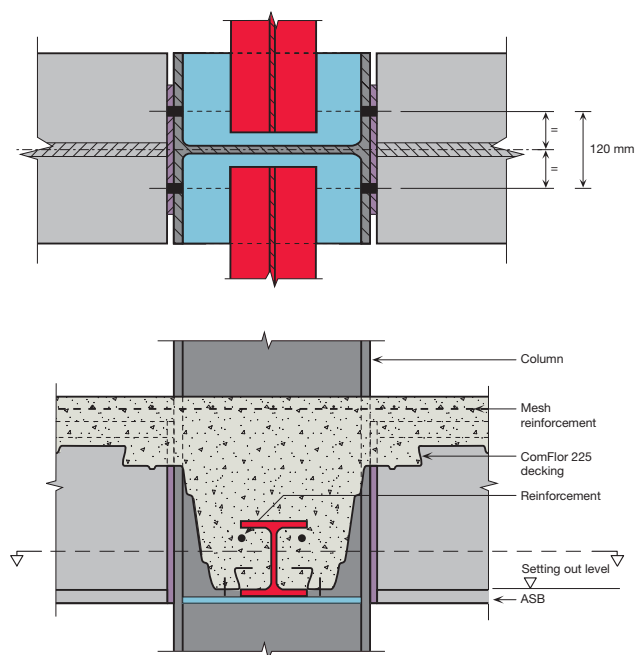
**Figure 2.28** RHS tie with welded deck support plate



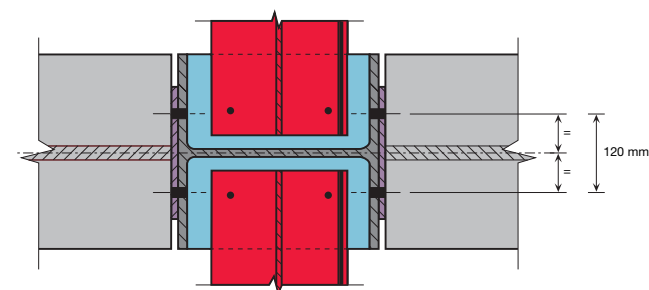
**Figure 2.29** RHS tie with welded deck support plate and cut decking



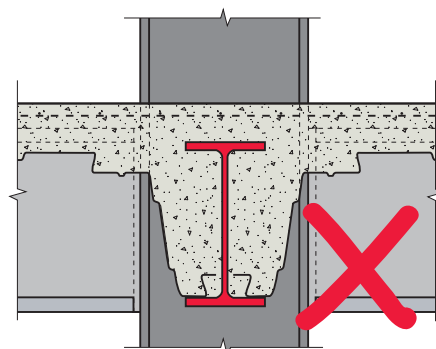
**Figure 2.30** Inverted structural Tee tie



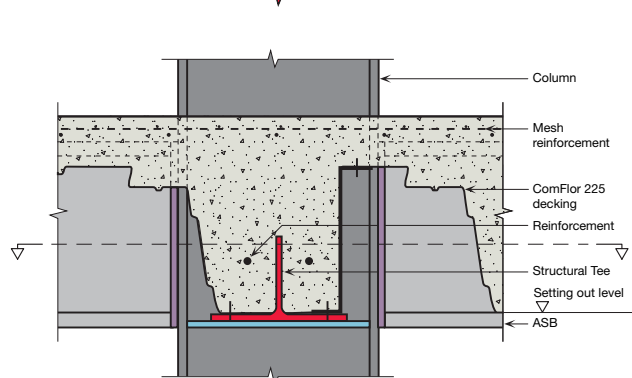
**Figure 2.32** UKB or UKC tie with bottom shelf plate



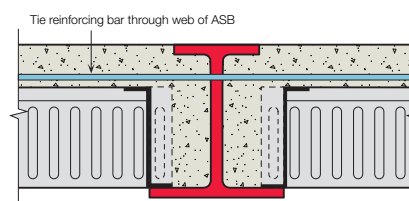
**Figure 2.31** Inverted structural Tee tie with cut decking



**Figure 2.33** Detail using I beam – not recommended



**Figure 2.34** Detail using embedded member – not recommended



**Figure 2.35** Tie bar reinforcement detail for internal beams with no concrete cover



# Deck span and beam selector tables

## The Lighthouse, Cardiff.

The Lighthouse forms part of Bellway Homes' Prospect Place development overlooking Cardiff Bay. The L-shaped structure has a 12-storey main block and a 6-storey wing and comprises 20 studios, 24 one-bedroom and 58 two-bedroom apartments as well as four penthouses. This high-rise residential development was originally conceived and gained planning approval as a reinforced concrete structure, but was ultimately built using steel.

The original planning application included an overall building height based on a floor thickness of 475mm, which Slimdek was easily able to achieve. The use of the Slimdek flooring system required no formwork, and led to quicker construction that allowed follow-on trades to start work much earlier.

The Slimdek system offered:

- speed of construction, allowing follow-on work to start earlier
- quicker lead-in time and no formwork
- the opportunity to build the floors as the frame progressed
- lower weight compared to reinforced concrete, resulting in reduced foundation costs
- a floor depth of 325mm excluding the suspended ceiling
- compliance with Part E of the Building Regulations

Client:	Bellway Homes
Architect:	BMG Architects
Structural engineer:	Bingham Hall O'Hanlon
Main contractor:	Bellway Homes
Steelwork contractor:	Bison Structures
Market sector:	Multi-storey residential



### 3. Deck span and beam selector tables

Selector tables are for use by experienced structural engineers for initial design purposes only. They give approximate sizes of beams that should be verified at the design stage by a more rigorous approach.

The tables indicate the performance of ComFlor 225 decking, Asymmetric Slimflor Beams and RHSFBs for typical building load and span requirements, across the effective range of the Slimdek system.

It is important that the beam design is checked for compatibility with the deck design. The tables are based on the minimum slab depth acceptable in each case. If the slab depth is increased then the beam size may have to be increased above that shown in the tables.

For detailed design guidance refer to the following SCI publications:

SCI-P-169: Design of RHS Slimflor edge beams<sup>[18]</sup>

SCI-P-175: Design of Asymmetric Slimflor Beams using deep composite decking<sup>[19]</sup>

SCI-P-248: Design of Slimflor fabricated beams using deep composite decking<sup>[23]</sup>

The following tables are provided:

Table	Fire resistance	Concrete type	Imposed load
<b>ComFlor 225 decking</b>			
Table 3.1	60 min	Lightweight Concrete (LWC)	2.5 kN/m <sup>2</sup> LL
Table 3.2	60 min	Normal Weight Concrete (NWC)	2.5 kN/m <sup>2</sup> LL
Table 3.3	60 min	Lightweight Concrete (LWC)	4.0 kN/m <sup>2</sup> LL
Table 3.4	60 min	Normal Weight Concrete (NWC)	4.0 kN/m <sup>2</sup> LL
<b>Asymmetric Slimflor Beams with ComFlor 225 decking</b>			
Table 3.5	60 min	Lightweight Concrete (LWC)	2.5 kN/m <sup>2</sup> LL
Table 3.6	60 min	Normal Weight Concrete (NWC)	2.5 kN/m <sup>2</sup> LL
Table 3.7	60 min	Lightweight Concrete (LWC)	4.0 kN/m <sup>2</sup> LL
Table 3.8	60 min	Normal Weight Concrete (NWC)	4.0 kN/m <sup>2</sup> LL
Table 3.9	30 min	Lightweight Concrete (LWC)	2.5 kN/m <sup>2</sup> LL
Table 3.10	30 min	Normal Weight Concrete (NWC)	2.5 kN/m <sup>2</sup> LL
Table 3.11	30 min	Lightweight Concrete (LWC)	4.0 kN/m <sup>2</sup> LL
Table 3.12	30 min	Normal Weight Concrete (NWC)	4.0 kN/m <sup>2</sup> LL
<b>RHSFBs with ComFlor 225 decking</b>			
Table 3.13	60 min	Lightweight Concrete (LWC)	2.5 kN/m <sup>2</sup> LL
Table 3.14	60 min	Normal Weight Concrete (NWC)	2.5 kN/m <sup>2</sup> LL
Table 3.15	60 min	Lightweight Concrete (LWC)	4.0 kN/m <sup>2</sup> LL
Table 3.16	60 min	Normal Weight Concrete (NWC)	4.0 kN/m <sup>2</sup> LL

NB A partition load of 1kN/m<sup>2</sup> has been included in addition to the imposed load in all the above tables

### 3.1 ComFlor 225 span tables

#### General

- Composite design
- ComFlor 225 decking 1.25mm thick in S350 steel
- Decking is propped or unpropped, as noted
- No service openings are provided
- Bar reinforcement grade B500A deformed type 2 conforming to BS 4449

#### Design assumptions

##### Light Weight Concrete (LWC)

Characteristic strength	30 N/mm <sup>2</sup>
Exposure condition	Mild (heated building)
Concrete depth	60mm min above steel deck
Wet density	1900 kg/m <sup>3</sup>
Dry density	1800 kg/m <sup>3</sup>
Modular ratio	15
Mesh reinforcement	A142 (minimum requirement)
Yield strength of mesh reinforcement	500 N/mm <sup>2</sup>

##### Normal Weight Concrete (NWC)

Characteristic Strength	30 N/mm <sup>2</sup>
Exposure condition	Mild (heated building)
Concrete depth	70mm min above steel deck
Wet density	2400 kg/m <sup>3</sup>
Dry density	2350 kg/m <sup>3</sup>
Modular ratio	10
Mesh reinforcement	A142 (minimum requirement)
Yield strength of mesh reinforcement:	500 N/mm <sup>2</sup>

##### Loads Acting On Beam

Occupancy imposed loads	2.5 kN/m <sup>2</sup> or 4.0 kN/m <sup>2</sup> as shown on tables
Partition loads	1.0 kN/m <sup>2</sup>
Ceilings, services and finishes	0.5 kN/m <sup>2</sup>
Construction load	1.5 kN/m <sup>2</sup> over a 3m length: 0.75kN/m <sup>2</sup> elsewhere
Deck weight	0.2 kN/m <sup>2</sup>
Ponding due to deck deflection has been taken into account	

##### Fire Data

Fire resistance period	60 min or 30 min as shown in tables
Proportion of imposed load considered as non-permanent	100%
Additional fire protection	NOT provided or required

##### Partial Safety Factors

Dead (self weight)	1.4
Imposed	1.6
Super imposed dead (partitions & services)	1.4

##### Deflection Limits

Construction stage	L/130 or 30mm maximum
Composite stage: Imposed load	L/350 or 20mm maximum
Composite stage: Superimposed dead load plus imposed load	L/250 or 30mm maximum

##### Frequency Limit

Natural frequency limit (unless noted otherwise on the table)	5 Hz
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## ComFlor 225 span tables

Table 3.1 60 min fire resistance, Light Weight Concrete (LWC), 2.5 kN/m<sup>2</sup> LL

Slab Depth (mm)		290	300	310	320	330	340	350	360
Slab Weight (kN/m <sup>2</sup> )		2.7	2.89	3.08	3.27	3.46	3.65	3.84	4.03
Bar dia (mm)	Props	Span limit (m) for s = simply supported, p = partial continuity							
16	Unpropped	6.3	6.2	6.1	6.0	5.8	5.7	5.6	5.5
	1	7.0 <b>s L</b> 7.6 <b>p L</b>	7.0 <b>s L</b> 7.6 <b>p L</b>	7.1 <b>s L</b> 7.7 <b>p L</b>	7.1 <b>s L</b> 7.8 <b>p L</b>	7.2 <b>s L</b> 7.7 <b>p L</b>	7.2 <b>s L</b> 7.4 <b>p L</b>	7.1 <b>L</b>	6.8 <b>L</b>
	2	7.0 <b>s L</b> 7.6 <b>p L</b>	7.0 <b>s L</b> 7.7 <b>p L</b>	7.1 <b>s L</b> 7.7 <b>p L</b>	7.1 <b>s L</b> 7.8 <b>p L</b>	7.2 <b>s L</b> 7.9 <b>p L</b>	7.2 <b>s L</b> 7.9 <b>p L</b>	7.3 <b>s L</b> 7.9 <b>p L</b>	7.3 <b>s L</b> 8.0 <b>p L</b>
20	Unpropped	6.3	6.2	6.1	6.0	5.8	5.7	5.6	5.5
	1	7.5 <b>s L</b> 7.9 <b>p L</b>	7.6 <b>s L</b> 8.1 <b>p L</b>	7.6 <b>s L</b> 8.2 <b>p L</b>	7.7 <b>s L</b> 8.0 <b>p L</b>	7.7 <b>L</b>	7.4 <b>L</b>	7.1 <b>L</b>	6.8 <b>L</b>
	2	7.5 <b>s L</b> 8.0 <b>p L</b>	7.6 <b>s L</b> 8.1 <b>p L</b>	7.6 <b>s L</b> 8.2 <b>p L</b>	7.7 <b>s L</b> 8.3 <b>p L</b>	7.8 <b>s L</b> 8.5 <b>p L</b>	7.9 <b>s L</b> 8.6 <b>p L</b>	7.9 <b>s L</b> 8.7 <b>p L</b>	8.0 <b>s L</b> 8.7 <b>p L</b>
25	Unpropped	6.3	6.2	6.1	6.0	5.8	5.7	5.6	5.5
	1	7.7 <b>s</b> 8.2 <b>p L</b>	7.8 <b>s</b> 8.3 <b>p L</b>	7.9 <b>s L</b> 8.4 <b>p L</b>	8.0 <b>L</b>	7.7 <b>L</b>	7.4 <b>L</b>	7.1	6.8
	2	7.7 <b>s</b> 8.2 <b>p L</b>	7.8 <b>s</b> 8.3 <b>p L</b>	7.9 <b>s L</b> 8.5 <b>p L</b>	8.0 <b>s L</b> 8.6 <b>p L</b>	8.0 <b>s L</b> 8.7 <b>p L</b>	8.1 <b>s L</b> 8.8 <b>p L</b>	8.2 <b>s L</b> 8.9 <b>p L</b>	8.3 <b>s L</b> 9.0 <b>p L</b>
32	Unpropped	6.3	6.2	6.1	6.0	5.8	5.7	5.6	5.5
	1	8.0 <b>s</b> 8.5 <b>p</b>	8.1 <b>s</b> 8.6 <b>p L</b>	8.2 <b>s</b> 8.4 <b>p L</b>	8.0	7.7	7.4	7.1	6.8
	2	8.0 <b>s</b> 8.5 <b>p</b>	8.2 <b>s</b> 8.7 <b>p L</b>	8.2 <b>s</b> 8.8 <b>p L</b>	8.3 <b>s</b> 8.9 <b>p L</b>	8.4 <b>s L</b> 9.1 <b>p L</b>	8.5 <b>s L</b> 9.2 <b>p L</b>	8.6 <b>s L</b> 9.3 <b>p L</b>	8.6 <b>s L</b> 9.4 <b>p L</b>

Table 3.2 60 min fire resistance, Normal Weight Concrete (NWC), 2.5 kN/m<sup>2</sup> LL

Slab Depth (mm)		290	300	310	320	330	340	350	360
Slab Weight (kN/m <sup>2</sup> )		-	3.51	3.75	3.98	4.22	4.45	4.69	4.93
Bar dia (mm)	Props	Span limit (m) for s = simply supported, p = partial continuity							
16	Unpropped	-	5.7	5.6	5.5	5.4	5.3	5.2	5.1
	1	-	6.7 <b>s</b> 7.2 <b>p L</b>	6.7 <b>s</b> 7.2 <b>p L</b>	6.7 <b>s L</b> 6.8 <b>p L</b>	6.5 <b>L</b>	6.2	6.0	5.7
	2	-	6.7 <b>s</b> 7.2 <b>p L</b>	6.7 <b>s</b> 7.3 <b>p L</b>	6.7 <b>s L</b> 7.3 <b>p L</b>	6.8 <b>s L</b> 7.3 <b>p L</b>	6.8 <b>s L</b> 7.4 <b>p L</b>	6.8 <b>s L</b> 7.4 <b>p L</b>	6.8 <b>s L</b> 7.4 <b>p L</b>
20	Unpropped	-	5.7	5.6	5.5	5.4	5.3	5.2	5.1
	1	-	7.4 <b>s</b> 7.5 <b>p L</b>	7.2	6.8	6.5	6.2	6.0	5.7
	2	-	7.4 <b>s</b> 8.1 <b>p L</b>	7.5 <b>s L</b> 8.2 <b>p L</b>	7.5 <b>s L</b> 8.2 <b>p L</b>	7.6 <b>s L</b> 8.3 <b>p L</b>	7.6 <b>s L</b> 8.4 <b>p L</b>	7.7 <b>s L</b> 8.4 <b>p L</b>	7.8 <b>s L</b> 8.5 <b>p L</b>
25	Unpropped	-	5.7	5.6	5.5	5.4	5.3	5.2	5.1
	1	-	7.5	7.2	6.8	6.5	6.2	6.0	5.7
	2	-	7.6 <b>s</b> 8.3 <b>p L</b>	7.7 <b>s</b> 8.4 <b>p L</b>	7.8 <b>s L</b> 8.5 <b>p L</b>	7.8 <b>s L</b> 8.6 <b>p L</b>	7.9 <b>s L</b> 8.6 <b>p L</b>	8.0 <b>s L</b> 8.7 <b>p L</b>	8.0 <b>s L</b> 8.8 <b>p L</b>
32	Unpropped	-	5.7	5.6	5.5	5.4	5.3	5.2	5.1
	1	-	7.5	7.2	6.8	6.5	6.2	6.0	5.7
	2	-	8.0 <b>s</b> 8.7 <b>p</b>	8.1 <b>s</b> 8.8 <b>p L</b>	8.2 <b>s</b> 9.0 <b>p L</b>	8.2 <b>s</b> 9.0 <b>p L</b>	8.3 <b>s L</b> 9.1 <b>p L</b>	8.4 <b>s L</b> 9.1 <b>p L</b>	8.4 <b>s L</b> 9.0 <b>p L</b>

## Notes:

- Maximum beam centres = span limit + B1 + B2 less 50mm where B1 and B2 are the distances from the centre-line of each support beam to the tip of support respectively.
- Unfactored slab weight = concrete + deck + an allowance for reinforcement.
- Unpropped deck spans are based on the construction stage design.
- L denotes L-bar anchorage.
- U denotes U-bar anchorage.
- p denotes partial end fixity has been assumed to meet serviceability limits (*top reinforcement required at support – see 2.2.3*).
- s denotes simply supported span.

## Deck span and beam selector tables

## ComFlor 225 span tables

Table 3.3 60 min fire resistance, Light Weight Concrete (LWC), 4.0 kN/m<sup>2</sup> LL

Slab Depth (mm)		290	300	310	320	330	340	350	360
Slab Weight (kN/m <sup>2</sup> )		2.70	2.89	3.08	3.27	3.46	3.65	3.84	4.03
Bar dia (mm)	Props	Span limit (m) for s = simply supported, p = partial continuity							
16	Unpropped	6.2 s L 6.3 p L	6.2 L	6.1 L	6.0 L	5.8 L	5.7 L	5.6 L	5.5 L
	1	6.4 s L 6.9 p L	6.4 s L 7.0 p L	6.5 s L 7.1 p L	6.5 s L 7.1 p L	6.6 s L 7.2 p L	6.7 s L 7.3 p L	6.7 s L 7.1 p L	6.7 s L 6.8 p L
	2	6.4 s L 7.0 p L	6.4 s L 7.0 p L	6.5 s L 7.1 p L	6.5 s L 7.2 p L	6.6 s L 7.2 p L	6.7 s L 7.3 p L	6.7 s L 7.3 p L	6.8 s L 7.4 p L
20	Unpropped	6.3 L	6.2 L	6.1 L	6.0 L	5.8 L	5.7 L	5.6 L	5.5 L
	1	7.1 s L 7.6 p L	7.3 s L 7.7 p L	7.4 s L 7.8 p L	7.5 s L 7.9 p L	7.6 s L 7.7 p L	7.4 L	7.1 L	6.8 L
	2	7.2 s L 7.6 p L	7.3 s L 7.7 p L	7.4 s L 7.8 p L	7.5 s L 7.9 p L	7.6 s L 8.1 p L	7.7 s L 8.2 p L	7.9 s L 8.3 p L	7.9 s L 8.4 p L
25	Unpropped	6.3 L	6.2 L	6.1 L	6.0 L	5.8	5.7	5.6	5.5
	1	7.3 s L 7.8 p L	7.5 s L 7.9 p L	7.6 s L 8.0 p L	7.7 s L 8.0 p L	7.7 s L 7.9 p L	7.4 L	7.1 L	6.8 L
	2	7.4 s L 7.8 p L	7.5 s L 7.9 p L	7.6 s L 8.1 p L	7.7 s L 8.2 p L	7.9 s L 8.3 p L	8.0 s L 8.4 p L	8.1 s L 8.6 p L	8.2 s L 8.7 p L
32	Unpropped	6.3	6.2	6.1	6.0	5.8	5.7	5.6	5.5
	1	7.6 s L 8.1 p L	7.8 s L 8.2 p L	7.9 s L 8.4 p L	8.0 L	7.7 L	7.4 L	7.1 L	6.8 L
	2	7.6 s L 8.1 p L	7.8 s L 8.2 p L	7.9 s L 8.4 p L	8.1 s L 8.5 p L	8.2 s L 8.7 p L	8.3 s L 8.8 p L	8.4 s L 8.9 p L	8.6 s L 9.0 p L

Table 3.4 60 min fire resistance, Normal Weight Concrete (NWC), 4.0 kN/m<sup>2</sup> LL

Slab Depth (mm)		290	300	310	320	330	340	350	360
Slab Weight (kN/m <sup>2</sup> )		-	3.51	3.75	3.98	4.22	4.45	4.69	4.93
Bar dia (mm)	Props	Span limit (m) for s = simply supported, p = partial continuity							
16	Unpropped	-	5.7 L	5.6 L	5.5 L	5.4 L	5.3 L	5.2 L	5.1 L
	1	-	6.1 s L 6.6 p L	6.2 s L 6.7 p L	6.2 s L 6.7 p L	6.3 s L 6.5 p L	6.2 L	6.0 L	5.7 L
	2	-	6.1 s L 6.7 p L	6.2 s L 6.7 p L	6.2 s L 6.8 p L	6.3 s L 6.8 p L	6.3 s L 6.8 p L	6.3 s L 6.9 p L	6.4 s L 6.9 p L
20	Unpropped	-	5.7	5.6	5.5	5.4	5.3	5.2	5.1
	1	-	7.3 s L 7.5 p L	7.2 L	6.8 L	6.5 L	6.2 L	6.0 L	5.7 L
	2	-	7.3 s L 7.7 p L	7.4 s L 7.8 p L	7.5 s L 7.9 p L	7.5 s L 8.1 p L	7.6 s L 8.2 p L	7.7 s L 8.3 p L	7.7 s L 8.4 p L
25	Unpropped	-	5.7	5.6	5.5	5.4	5.3	5.2	5.1
	1	-	7.5 L	7.2 L	6.8 L	6.5 L	6.2	6.0	5.7
	2	-	7.5 s L 7.9 p L	7.6 s L 8.1 p L	7.7 s L 8.2 p L	7.8 s L 8.3 p L	7.8 s L 8.4 p L	7.9 s L 8.5 p L	8.0 s L 8.7 p L
32	Unpropped	-	5.7	5.6	5.5	5.4	5.3	5.2	5.1
	1	-	7.5 L	7.2 L	6.8	6.5	6.2	6.0	5.7
	2	-	7.8 s L 8.3 p L	8.0 s L 8.4 p L	8.1 s L 8.6 p L	8.2 s L 8.7 p L	8.2 s L 8.8 p L	8.3 s L 8.9 p L	8.4 s L 9.0 p L

## Notes:

1. Maximum beam centres = span limit + B1 + B2 less 50mm where B1 and B2 are the distances from the centre-line of each support beam to the top of support respectively.
2. Unfactored slab weight = concrete + deck + an allowance for reinforcement.
3. Unpropped deck spans are based on the construction stage design.
4. L denotes L-bar anchorage.
5. U denotes U-bar anchorage.
6. p denotes partial end fixity has been assumed to meet serviceability limits (*top reinforcement required at support – see 2.2.3*).
7. s denotes simply supported span.

## 3.2 ASB selector tables

### General

- Composite design
- All beams have been designed using grade S355 to BS EN 10025–2<sup>(4)</sup>
- All beams are unpropped unless noted by suffix
- All beams have been designed assuming ComFlor 225 deck is used
- Decking is assumed to be propped in construction as necessary
- Decking spans perpendicular to beam on both sides i.e., internal beam
- No service holes are provided in ASB web (refer to design software)

### Design assumptions

#### Light Weight Concrete (LWC)

Characteristic strength	30 N/mm <sup>2</sup>
Exposure condition	Mild (heated building)
Concrete depth	30mm min above ASB or 60mm above steel deck
Wet density	1900 kg/m <sup>3</sup>
Dry density	1800 kg/m <sup>3</sup>
Modular ratio	15
Mesh reinforcement	A142 (minimum requirement)
Yield strength of mesh reinforcement	500 N/mm <sup>2</sup>

#### Normal Weight Concrete (NWC)

Characteristic strength	30 N/mm <sup>2</sup>
Exposure condition	Mild (heated building)
Concrete depth	40mm min above ASB or 70mm min above steel deck
Wet density	2400 kg/m <sup>3</sup>
Dry density	2350 kg/m <sup>3</sup>
Modular ratio	10
Mesh reinforcement	A142 (minimum requirement)
Yield strength of mesh reinforcement	500 N/mm <sup>2</sup>

#### Loads Acting On Beam

Occupancy imposed loads	2.5 kN/m <sup>2</sup> or 4.0 kN/m <sup>2</sup> as shown on tables
Partition loads	1.0 kN/m <sup>2</sup>
Ceilings, services and finishes	0.5 kN/m <sup>2</sup>
Construction load	0.5 kN/m <sup>2</sup>
Deck weight	0.2 kN/m <sup>2</sup>
BS 6399 imposed load reduction has been utilised	
Ponding due to deck deflection has NOT been taken into account	

#### Fire Data

Fire resistance period	60 min or 30 min as shown in tables
Proportion of imposed load considered as non-permanent	100%
Additional fire protection	NOT provided or required

#### Partial Safety Factors

Dead (self weight)	1.4
Imposed	1.6
Super imposed dead (partitions & services)	1.4

#### Deflection Limits (internal beams only)

Imposed load deflection limit	L/360
Total load deflection limit	L/200

#### Frequency Limit

Natural frequency limit (unless noted otherwise on the table)	4 Hz
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## ASB selector tables

Table 3.5 60 min fire resistance, Light Weight Concrete (LWC), 2.5 kN/m<sup>2</sup> LL

Beam spacing (m)	Beam span (m)					
	5.0	6.0	7.0	8.0	9.0	10.0
5.0	280 ASB 74	280 ASB 74	280 ASB(FE) 100	280 ASB(FE) 100 <b>f</b>	300 ASB(FE) 153	300 ASB 155 <b>f</b>
6.0	280 ASB 74	280 ASB(FE) 100	280 ASB(FE) 100	280 ASB(FE) 136	300 ASB 155	300 ASB 196 <b>f</b>
7.0	280 ASB 74 <b>p</b>	280 ASB(FE) 100 <b>p</b>	280 ASB(FE) 100 <b>p</b>	280 ASB(FE) 136 <b>p f</b>	300 ASB(FE) 185 <b>p f</b>	NSA
8.0	280 ASB 74 <b>p</b>	280 ASB(FE) 100 <b>p</b>	280 ASB(FE) 136 <b>p</b>	300 ASB(FE) 153 <b>p</b>	300 ASB 196 <b>p</b>	NSA
9.0	280 ASB(FE) 100 <b>p</b>	280 ASB(FE) 100 <b>p</b>	280 ASB(FE) 136 <b>p</b>	300 ASB(FE) 153 <b>p f</b>	300 ASB(FE) 249 <b>p</b>	NSA

Table 3.6 60 min fire resistance, Normal Weight Concrete (NWC), 2.5 kN/m<sup>2</sup> LL

Beam spacing (m)	Beam span (m)					
	5.0	6.0	7.0	8.0	9.0	10.0
5.0	280 ASB 74	280 ASB 74	280 ASB(FE) 100	280 ASB 124	300 ASB(FE) 153 <b>f</b>	300 ASB 196
6.0	280 ASB 74	280 ASB(FE) 100	280 ASB(FE) 100	280 ASB(FE) 136 <b>f</b>	300 ASB(FE) 153 <b>p f</b>	300 ASB 196 <b>p f</b>
7.0	280 ASB 74 <b>p</b>	280 ASB(FE) 100 <b>p</b>	280 ASB(FE) 136 <b>p</b>	280 ASB(FE) 136 <b>p f</b>	300 ASB(FE) 185 <b>p f</b>	NSA
8.0	280 ASB(FE) 100 <b>p</b>	280 ASB(FE) 100 <b>p</b>	280 ASB(FE) 136 <b>p</b>	300 ASB(FE) 153 <b>p f</b>	300 ASB(FE) 249 <b>p</b>	NSA
9.0	280 ASB(FE) 100 <b>p</b>	280 ASB 124 <b>p</b>	280 ASB(FE) 136 <b>p</b>	300 ASB(FE) 185 <b>p</b>	300 ASB(FE) 249 <b>p f</b>	NSA

Table 3.7 60 min fire resistance, Light Weight Concrete (LWC), 4.0 kN/m<sup>2</sup> LL

Beam spacing (m)	Beam span (m)					
	5.0	6.0	7.0	8.0	9.0	10.0
5.0	280 ASB 74	280 ASB(FE) 100	280 ASB(FE) 100	280 ASB(FE) 136	300 ASB 155	300 ASB 196 <b>f</b>
6.0	280 ASB 74	280 ASB(FE) 100	280 ASB(FE) 100 <b>f</b>	300 ASB(FE) 153	300 ASB(FE) 185 <b>f</b>	300 ASB(FE) 196 <b>pp f</b>
7.0	280 ASB 74 <b>p</b>	280 ASB(FE) 100 <b>p</b>	280 ASB(FE) 136 <b>p</b>	300 ASB(FE) 153 <b>p f</b>	300 ASB 196 <b>p</b>	NSA
8.0	280 ASB(FE) 100 <b>p</b>	280 ASB(FE) 100 <b>p \$</b>	280 ASB(FE) 136 <b>p</b>	300 ASB(FE) 185 <b>p</b>	300 ASB(FE) 249 <b>p</b>	NSA
9.0	280 ASB(FE) 100 <b>p</b>	280 ASB(FE) 136 <b>p</b>	300 ASB(FE) 153 <b>p</b>	300 ASB(FE) 185 <b>p f</b>	300 ASB(FE) 249 <b>pp f</b>	NSA

Table 3.8 60 min fire resistance, Normal Weight Concrete (NWC), 4.0 kN/m<sup>2</sup> LL

Beam spacing (m)	Beam span (m)					
	5.0	6.0	7.0	8.0	9.0	10.0
5.0	280 ASB 74	280 ASB(FE) 100	280 ASB(FE) 100	280 ASB(FE) 136	300 ASB(FE) 153	300 ASB 196 <b>f</b>
6.0	280 ASB 74	280 ASB(FE) 100	280 ASB(FE) 136	300 ASB(FE) 136 <b>p f</b>	300 ASB 185 <b>p f</b>	300 ASB(FE) 249 <b>p f</b>
7.0	280 ASB(FE) 100 <b>p</b>	280 ASB(FE) 100 <b>p</b>	280 ASB(FE) 136 <b>p</b>	300 ASB(FE) 153 <b>p f</b>	300 ASB(FE) 249 <b>p</b>	NSA
8.0	280 ASB(FE) 100 <b>p</b>	280 ASB(FE) 136 <b>p</b>	300 ASB(FE) 153 <b>p</b>	300 ASB(FE) 185 <b>p f</b>	300 ASB(FE) 249 <b>p f</b>	NSA
9.0	280 ASB(FE) 100 <b>p</b>	280 ASB(FE) 136 <b>p</b>	300 ASB(FE) 153 <b>p</b>	300 ASB(FE) 249 <b>p</b>	300 ASB(FE) 249 <b>pp f</b>	NSA

## Notes:

- p** Deck is propped during construction  
**pp** Both deck and beam are propped  
**f** 10% end fixity assumed (Flush Type B or extended end plate required)  
**\$** Reinforcement anchored over or through the beam required for torsion resistance  
**NSA** No Section Available

## ASB selector tables

Table 3.9 30 min fire resistance, Light Weight Concrete (LWC), 2.5 kN/m<sup>2</sup> LL

Beam spacing (m)	Beam span (m)					
	5.0	6.0	7.0	8.0	9.0	10.0
5.0	280 ASB 74	280 ASB 74	280 ASB 74	280 ASB(FE) 100 <b>f</b>	280 ASB 124 <b>f</b>	300 ASB 155 <b>f</b>
6.0	280 ASB 74	280 ASB 74	280 ASB(FE) 100	280 ASB 105 <b>f</b>	300 ASB 155	300 ASB 196 <b>f</b>
7.0	280 ASB 74 <b>p</b>	280 ASB 74 <b>p</b>	280 ASB(FE) 100 <b>p</b>	280 ASB 124 <b>p</b>	300 ASB 155 <b>p f</b>	NSA
8.0	280 ASB 74 <b>p</b>	280 ASB 74 <b>p \$</b>	280 ASB(FE) 100 <b>p f \$</b>	300 ASB(FE) 153 <b>p</b>	300 ASB 196 <b>p</b>	NSA
9.0	280 ASB 74 <b>p</b>	280 ASB 74 <b>p \$</b>	280 ASB 105 <b>p</b>	300 ASB(FE) 153 <b>p f</b>	300 ASB 196 <b>p f</b>	NSA

Table 3.10 30 min fire resistance, Normal Weight Concrete (NWC), 2.5 kN/m<sup>2</sup> LL

Beam spacing (m)	Beam span (m)					
	5.0	6.0	7.0	8.0	9.0	10.0
5.0	280 ASB 74	280 ASB 74	280 ASB(FE) 100	280 ASB 105	300 ASB(FE) 153 <b>f</b>	300 ASB 196
6.0	280 ASB 74	280 ASB 74	280 ASB(FE) 100	280 ASB 124	300 ASB 155 <b>f</b>	300 ASB(FE) 249 <b>f</b>
7.0	280 ASB 74 <b>p</b>	280 ASB 74 <b>p</b>	280 ASB(FE) 100 <b>p</b>	280 ASB 124 <b>p f</b>	300 ASB 155 <b>p f</b>	NSA
8.0	280 ASB 74 <b>p</b>	280 ASB 74 <b>p \$</b>	280 ASB(FE) 100 <b>p f</b>	300 ASB(FE) 153 <b>p f</b>	300 ASB 196 <b>p</b>	NSA
9.0	280 ASB 74 <b>p</b>	280 ASB 74 <b>p \$</b>	280 ASB 105 <b>p f</b>	300 ASB 155 <b>p</b>	300 ASB(FE) 249 <b>p f</b>	NSA

Table 3.11 30 min fire resistance, Light Weight Concrete (LWC), 4.0 kN/m<sup>2</sup> LL

Beam spacing (m)	Beam span (m)					
	5.0	6.0	7.0	8.0	9.0	10.0
5.0	280 ASB 74	280 ASB 74	280 ASB 74 <b>f \$</b>	280 ASB 105 <b>f</b>	300 ASB 155	300 ASB 196 <b>f</b>
6.0	280 ASB 74	280 ASB 74 <b>\$</b>	280 ASB(FE) 100 <b>f</b>	280 ASB 124 <b>f</b>	300 ASB(FE) 185 <b>f</b>	300 ASB(FE) 249 <b>f</b>
7.0	280 ASB 74 <b>p</b>	280 ASB 74 <b>p \$</b>	280 ASB 105 <b>p</b>	300 ASB(FE) 153 <b>p f</b>	300 ASB 196 <b>p</b>	NSA
8.0	280 ASB 74 <b>p</b>	280 ASB 74 <b>p \$</b>	280 ASB 105 <b>p f \$</b>	300 ASB 155 <b>p</b>	300 ASB 196 <b>p f</b>	NSA
9.0	280 ASB 74 <b>p \$</b>	280 ASB(FE) 100 <b>p \$</b>	280 ASB 124 <b>p f</b>	300 ASB(FE) 185 <b>p f</b>	300 ASB(FE) 249 <b>pp f</b>	NSA

Table 3.12 30 min fire resistance, Normal Weight Concrete (NWC), 4.0 kN/m<sup>2</sup> LL

Beam spacing (m)	Beam span (m)					
	5.0	6.0	7.0	8.0	9.0	10.0
5.0	280 ASB 74	280 ASB 74	280 ASB(FE) 100	280 ASB 105 <b>f</b>	300 ASB 155	300 ASB 196 <b>f</b>
6.0	280 ASB 74	280 ASB 74 <b>\$</b>	280 ASB(FE) 100 <b>f</b>	300 ASB(FE) 153	300 ASB 155 <b>p f</b>	300 ASB(FE) 249 <b>p f</b>
7.0	280 ASB 74 <b>p</b>	280 ASB 74 <b>p \$</b>	280 ASB 105 <b>p</b>	300 ASB(FE) 153 <b>p f</b>	300 ASB 196 <b>p</b>	NSA
8.0	280 ASB 74 <b>p \$</b>	280 ASB 74 <b>p \$</b>	280 ASB 105 <b>p f \$</b>	300 ASB 155 <b>p f</b>	300 ASB(FE) 249 <b>p f</b>	NSA
9.0	280 ASB 74 <b>p \$</b>	280 ASB(FE) 100 <b>p \$</b>	280 ASB 124 <b>p f</b>	300 ASB 196 <b>p</b>	300 ASB(FE) 249 <b>pp f</b>	NSA

## Notes:

p Deck is propped during construction

pp Both deck and beam are propped

f 10% end fixity assumed (Flush Type B or extended end plate required)

\$ Reinforcement anchored over or through the beam required for torsion resistance

NSA No Section Available

### 3.3 RHSFB selector tables

#### General

- The non-composite option is based on the lightest RHSFB solution with the minimum slab depth acceptable to achieve the stated floor span. This may result in the RHSFB projecting above the top surface of the slab. A shallower but heavier solution may be possible.
- Composite solutions require adequate cover to the shear studs which will normally govern the slab depth and will generally result in an uneconomic design. As a result, composite solutions are not included in these tables.
- All beams have been designed using a Corus Celsius RHS grade S355 J2H to EN 10210<sup>[6]</sup> (Hot finished 355N/mm<sup>2</sup>) and plate grade S355 to BS EN 10025-2<sup>[4]</sup>.
- All beams are unpropped unless noted by suffix.
- All beams have been designed assuming ComFlor 225 deck is used.
- Decking is assumed to be propped in construction as necessary.
- Decking spans perpendicular to the beam on one side only i.e., edge beam.
- No service holes are provided in the RHSFB web.

## Design assumptions

**Light Weight Concrete (LWC)**

Characteristic strength		30 N/mm <sup>2</sup>
Exposure condition		Mild (heated building)
Concrete depth	Non-Composite RHSFB	40mm min above RHSFB or 60mm above steel deck (except cases shown # which denotes that beam projects above top of slab)
Wet density		1900 kg/m <sup>3</sup>
Dry density		1800 kg/m <sup>3</sup>
Modular ratio		15
Mesh reinforcement		A142 (minimum requirement)
Yield strength of mesh reinforcement		500 N/mm <sup>2</sup>

**Normal Weight Concrete (NWC)**

Characteristic strength		30 N/mm <sup>2</sup>
Exposure condition		Mild (heated building)
Concrete depth	Non-Composite RHSFB	40mm min above RHSFB or 70mm min above steel deck (except cases shown # which denotes that beam projects above top of slab)
Wet density		2400 kg/m <sup>3</sup>
Dry density		2350 kg/m <sup>3</sup>
Modular ratio		10
Mesh reinforcement		A142 (minimum requirement)
Yield strength of mesh reinforcement		500 N/mm <sup>2</sup>

**Loads Acting On Beam**

Occupancy imposed loads	2.5 kN/m <sup>2</sup> or 4.0 kN/m <sup>2</sup> as shown on tables
Partition loads	1.0 kN/m <sup>2</sup>
Ceilings, services and finishes	0.5 kN/m <sup>2</sup>
Construction load	0.5 kN/m <sup>2</sup>
Deck weight	0.2 kN/m <sup>2</sup>
Cladding load	8 kN/m
Cladding eccentricity	200mm
BS 6399 imposed load reduction has been utilised	
Ponding due to deck and beam deflection has NOT been taken into account	

**Fire Data**

Fire resistance period	60 min
Proportion of imposed load considered as non-permanent	100%
Additional fire protection	NOT provided or required

**Partial Safety Factors**

Dead (self weight)	1.4
Imposed	1.6
Super imposed dead (partitions & services)	1.4

**Deflection Limits**

Imposed load deflection limit	L/500
Imposed and cladding deflection limit	L/360
Total load deflection limit	L/250

**Frequency Limit**

Natural frequency limit (unless noted otherwise on the table)	4 Hz
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**Flange Plate Data**

Plate projection on cladding side	10mm
Plate projection on decking side	100mm
Plate thickness	15mm

## RHSFB selector tables

Table 3.13 60 min fire resistance, Light Weight Concrete (LWC), 2.5 kN/m<sup>2</sup> LL

Beam spacing (m)	Beam span (m)						
	5.0	6.0	7.0	8.0	9.0	10.0	
5.0	Non-composite RHSFB 200x150x8	200x150x8	300x200x8 #	400x200x8 #	400x200x8 #	450x250x8 #	
6.0	Non-composite RHSFB 200x150x8	200x150x8	300x200x8 #	400x200x8 #	400x200x10 #	450x250x10 #	
7.0	Non-composite RHSFB 200x150x8 p	200x150x10 p	300x200x8 p #	400x200x8 p #	400x200x10 p #	450x250x12.5 p #	
8.0	Non-composite RHSFB 250x150x8 p	200x150x10 p	300x200x8 p # \$	400x200x8 p #	450x250x8 p #	500x300x8 p #	
9.0	Non-composite RHSFB 250x150x8 p	300x200x8 p #	400x200x8 p #	400x200x10 p #	450x250x8 p #	500x300x8 p #	

Table 3.14 60 min fire resistance, Normal Weight Concrete (NWC), 2.5 kN/m<sup>2</sup> LL

Beam spacing (m)	Beam span (m)						
	5.0	6.0	7.0	8.0	9.0	10.0	
5.0	Non-composite RHSFB 200x150x8	200x150x8	300x200x8 #	400x200x8 #	400x200x10 #	450x250x10 #	
6.0	Non-composite RHSFB 200x150x8	200x150x10	300x200x8 #	400x200x8 #	450x250x8 #	450x250x12.5 #	
7.0	Non-composite RHSFB 200x150x8 p	200x150x10 p	300x200x8 p # \$	300x200x8 p # \$	450x250x8 p #	500x300x8 p #	
8.0	Non-composite RHSFB 250x150x8 p	300x200x8 p #	400x200x8 p #	400x200x10 p #	450x250x8 p #	500x300x10 p #	
9.0	Non-composite RHSFB 250x150x8 p	300x200x8 p #	400x200x8 p #	400x200x10 p #	450x250x10 p #	500x300x10 p #	

Table 3.15 60 min fire resistance, Light Weight Concrete (LWC), 4.0 kN/m<sup>2</sup> LL

Beam spacing (m)	Beam span (m)						
	5.0	6.0	7.0	8.0	9.0	10.0	
5.0	Non-composite RHSFB 250x150x8	250x150x8	300x200x8 #	400x200x8 #	400x200x8 #	450x250x10 #	
6.0	Non-composite RHSFB 250x150x8	200x150x10	300x200x8 #	400x200x8 #	400x200x10 #	500x300x8 #	
7.0	Non-composite RHSFB 200x150x8 p	300x200x8 p #	400x200x8 p #	400x200x8 p #	400x250x8 p #	500x300x8 p #	
8.0	Non-composite RHSFB 250x150x8 p	300x200x8 p #	400x200x8 p #	400x200x10 p #	450x250x10 p #	500x300x8 p #	
9.0	Non-composite RHSFB 250x150x8 p \$	300x200x8 p #	400x200x8 p #	450x250x8 p #	500x300x8 p #	500x300x10 p #	

Table 3.16 60 min fire resistance, Normal Weight Concrete (NWC), 4.0 kN/m<sup>2</sup> LL

Beam spacing (m)	Beam span (m)						
	5.0	6.0	7.0	8.0	9.0	10.0	
5.0	Non-composite RHSFB 200x150x8	250x150x10	300x200x8 #	400x200x8 #	400x200x10 #	450x250x10 #	
6.0	Non-composite RHSFB 250x150x8	300x200x8 #	300x200x10 #	400x200x8 #	450x250x8 #	500x300x8 #	
7.0	Non-composite RHSFB 250x150x8 p	300x200x8 p #	400x200x8 p #	400x200x10 p #	500x300x8 p #	500x300x8 p #	
8.0	Non-composite RHSFB 250x150x8 p \$	300x200x8 p #	400x200x8 p #	400x200x10 p #	500x300x8 p #	500x300x10 p #	
9.0	Non-composite RHSFB 250x150x10 p	400x200x8 p #	400x200x10 p #	450x250x8 p #	500x300x8 p #	500x300x10 p #	

## Notes:

- # Beam projects above the top of the slab
- p Deck is propped during construction
- pp Both deck and beam are propped
- f 10% end fixity assumed (extended end plate required)
- \$ Reinforcement anchored over or through the beam required for torsion resistance
- NSA No Section Available

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# Connection design

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## Sunderland Royal Hospital.

Sunderland Royal Hospital's steel-framed five-storey extension was opened Spring 2006. The 15,000m<sup>2</sup> development incorporates three general and two ultra-clean operating theatres as well as sterilising facilities for the theatres and a mortuary.

The frame comprises 450 tonnes of steelwork, including asymmetrical beams supporting the Slimdek flooring system. The structure was built on a varying grid of 8m in one direction and up to 6m in the opposite direction.

The need to minimise vibrations in the existing structure during construction was paramount. The chosen construction system allowed the new extension to be built as an independent structure and linked to the existing building.

The Slimdek system offered:

- speed of construction, minimising the impact of the development on an already occupied hospital site
- environmental benefits, as an in-situ concrete frame would have involved more wagon movements around the congested city site
- inherent fire resistance, as the beams were cast into the structural topping

Client:

City Hospitals Sunderland NHS Foundation Trust

Structural engineer:

Arup

Design and build contractor:

Kier Northern Limited

Steelwork contractor:

South Durham Structures Ltd

Market sector:

Healthcare



## 4. Connection design

### 4.1 Introduction

The design and detailing of end plate connections to ASB and RHSFB sections in braced frames should take into account:

- The width of beams and column flanges.
- Requirements for torsional resistance (particularly for edge beams).
- Requirements for sufficient bolts to resist shear, as well as those to resist tension.
- The requirements for fillet welding.
- Extension of the end plate above the beam flange (and below for wind moment frames).
- Connections to RHS or CHS columns.

Standard dimensions have been adopted to optimise these requirements. The same details may be used for either:

- Pure shear-resisting connections, or
- Moment-resisting connections.

The difference between the two forms is only in the thickness of the end plate. Some of the lighter columns cannot be used in moment-resisting connections, unless stiffened locally.

Where RHS or SHS columns are used, the use of Hollo-bolt or Flow-drill type connections may be used. For RHSFBs, the end plate should be extended above and below to facilitate the connection to the column. Other connections may be used to suit the particular construction method but the designer must take into account the need for torsional rigidity of the connection. Detailed guidance on connection design is given in SCI/BCSA publications on connections:

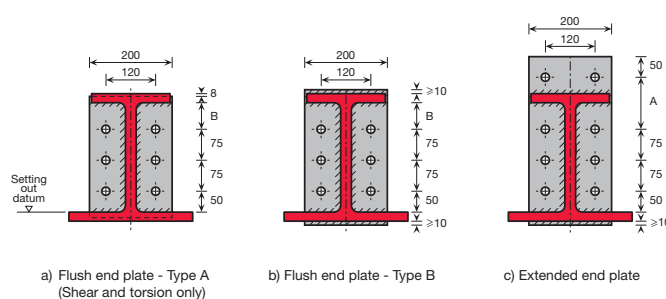
SCI-P-207: *Joints in steel construction: Moment connections*<sup>[21]</sup>.

SCI-P-212: *Joints in steel construction: Simple connections*<sup>[22]</sup>.

### 4.2 ASB sections

#### 4.2.1 Detailing

The setting out point for detailing of the connections is taken as the top of the bottom flange. This is done so that this level is the surface level of the slab minus the slab depth, and is consistent for all ASB sizes. The lower bolts are positioned at 50mm above the bottom flange. The recommended bolt-detailing rules are given in *Figure 4.1*.



Dimension	A	B
ASB280	110	44
ASB300	140	62

**Figure 4.1** Detailing rules for end plate connections to ASBs

The end plate may be taken as a standard width of 200mm for all ASB sections, which allows connections to 203 UKC and larger columns. The bolt cross-spacing is taken as 120mm, in order that the bolts are efficient in both tension and torsion. The vertical distance between the bolts is 75mm for 3-bolt rows, and 150mm for 2-bolt rows. These detailing rules differ from the SCI/BCSA 'Moment Connections' publication because of the thicker flanges of ASB sections in comparison to UKB sections, and because of the shallower depth of section. These detailing rules provide connections that achieve sufficient shear resistance, bending resistance and stiffness. The recommended bolt size and end plate thickness for ASB connections are given in *Table 4.1*.

**Table 4.1** Recommended bolt sizes and end plate thicknesses for ASB connections

	Grade 8.8 Bolt Diameter mm	End Plate Thickness (mm)	
		Shear-Resisting Connections	Moment-Resisting Connections
Spans ≤ 6m	20	10	12
Spans > 6m	24	10	15

#### 4.2.2 Shear-resisting connections with torsional resistance – Flush type A end plates to ASB sections

The normal method of connecting ASBs to columns is to use a 4 or 6 bolt, full depth flush end plate connection, see Figure 4.1(a). These connections possess excellent shear and torsional resistance which is utilised at the construction stage, or where the beam is subjected to high out-of-balance forces (e.g., edge beams). Shear resistances (simplified) based on the standard end plate details are given in Table 4.2. For internal beams at ultimate limit state with equal slab spans, use the pure shear resistance value. For internal beams at construction state and for all edge beams, use the shear resistance for combined shear and torsion.

#### 4.2.3 Moment-resisting connections – Flush type B or extended end plates to ASB sections

Moment-resisting connections generally require the use of a thicker end plate fully welded to the ASB flanges. Extended end plates using 8 bolts can develop end moments of at least 10% of the moment capacity of the beam. The advantage of end fixity is not taken into account in the ASB design software, except by a nominal reduction in deflections. Moment resistance and shear and torsion capacities based on the standard end plate details and minimum weld sizes are given in Tables 4.3 to 4.10. The tables have been developed from the guidance given in the SCI/BCSA publication SCI-P-207<sup>[21]</sup>, and the assumptions made in developing the resistances are explained in Section 4.2.4.

#### 4.2.4 Assumptions made in producing connection resistance tables

The shear resistance of the bolt group is calculated from:

- 40% of the shear resistance of the top pair of bolts
- 40% of the shear resistance any other bolts acting in tension
- The full shear resistance of the remaining bolts.

The torsional resistance of the bolt group is calculated from:

- The shear resistance of the bolts x their distance from the centre of rotation of the bolt group.

The analysis for torsion acting on the bolt group assumes that:

- The upper bolts resist tension and have a shear resistance of 0.4 times their normal shear resistance
- The number of bolts resisting the pure shear component of force is taken as  $N - 2$ , where  $N$  is the number of bolts between the flanges
- The shear forces due to pure torsion acting on the upper bolt group are limited to 0.4 times the normal shear resistance
- The eccentricity of force is taken as 0.5 x bottom flange or plate width minus 25mm, which is half the assumed bearing length (50mm) of the decking on the support.

The torsional resistance of the bolt group is also dependent on the spacing of the bolts, as illustrated in Figure 4.2. Extended plated connections have greater torsional resistance than flush end-plated connections.

The bending resistance of the end plate connection is calculated on the basis of:

- A yield-line failure pattern in the end plate which is used to calculate the effective tensile resistance of the bolt group
- The moment contribution of these bolts is obtained by taking moments about the centre of the bottom flange.

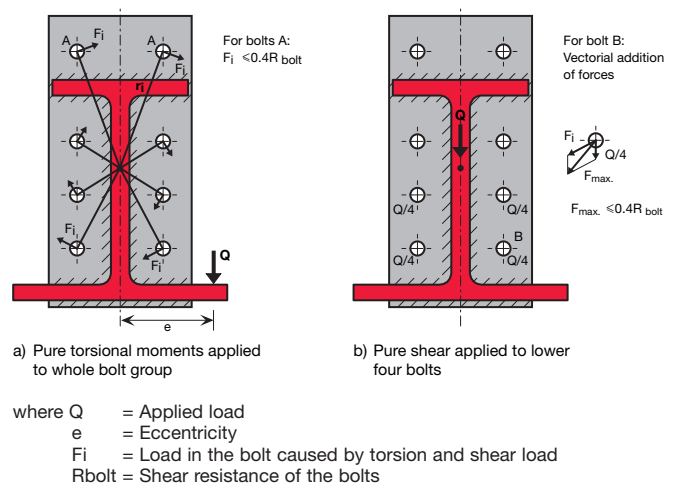


Figure 4.2 Torsional analysis of bolt group

Table 4.2 Shear resistance (simplified) for flush type A end plate connections to ASB sections										
Bolt diameter mm	End plate thickness mm	280 ASB (2-bolt rows)			280 ASB (3-bolt rows)			300 ASB (3-bolt rows)		
		Fillet weld size mm	Pure shear kN	Combined shear and torsion kN	Fillet weld size mm	Pure shear kN	Combined shear and torsion kN	Fillet weld size mm	Pure shear kN	Combined shear and torsion kN
20	10	6	257	83	6	441	169	6	441	161
20	12	6	257	83	6	441	169	6	441	161
24	12	6	371	120	6	635	244	6	635	232
24	15	6	371	120	6	635	244	6	635	232

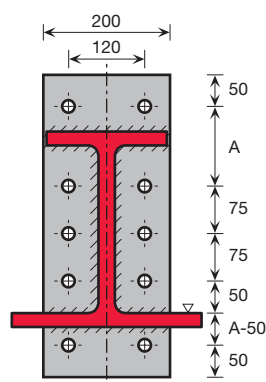
Notes:

1. "Bolt rows" refer to rows of bolts between flanges.
2. Flush type A connections are suitable for shear and torsion only.
3. Flush type B or extended connections must be used where moment resistance is required.

### 4.2.5 Wind moment connections

Connections with extended end plates above and below the section may be used in 'wind moment' frames (sway frames). At present, this design approach is limited to buildings of up to four storeys in height in order to limit sway deflections. The stiffness of ASB connections has not yet been determined by test and designers should make their own assessment for frame analysis.

Wind moment connections may be designed by extending the end plate above and below the beam in order to resist negative and positive moments when the unbraced frame is subject to horizontal wind forces. In this case, 8 or 10 bolts are used based on the detailing dimensions in *Figure 4.3* and the preferred end plate thickness is 15mm in grade S275. The vertical pitch of the lower bolt pair may be taken as the dimension A so that the bolt group is symmetric. The moment capacity of the connection may be taken from *Table 4.8* applied to resist both negative and positive moments.



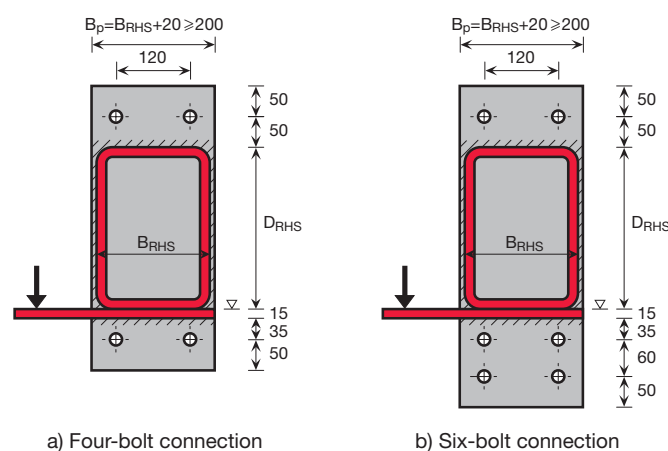
Dimension	A
ASB280	110
ASB300	140

**Figure 4.3** Wind moment extended end plate

### 4.3 RHSFB sections

For RHSFB connections, the end plate width is variable depending on the RHS section. However, it is reasonable to use a minimum dimension of 200mm, increasing in 50mm increments depending on the RHS size. The connections are extended above and below the RHS. For detailing purposes, the bolts are set out at a standard distance of 50mm from the top and bottom edges of the RHS (i.e., again using the top of the bottom flange plate as a setting out level). Four M20 or M24 bolts would normally be used (two above and two below), which should be checked for their shear and torsional resistance. These connections are relatively stiff for serviceability design. A typical detail is shown in *Figure 4.4(a)*.

Heavily loaded connections may require the use of additional bolts by further extending the end plate below the beam, as shown in *Figure 4.4(b)*.



**Figure 4.4** Connection to RHSF beam

Table 4.3 Moment capacity (kNm)

M20 8.8 bolts 12mm S275 flush type B end plate											
Column size	280 ASB No. of bolts = 6					300 ASB No. of bolts = 6					Web panel shear capacity (kN)
S355	74	100	105	124	136	153	155	185	196	249	
203 x 203 x	46	38 (184)	38 (184)	39 (184)	39 (184)	39 (184)	40 (184)	39 (184)	40 (184)	40 (184)	312
	52	39 (186)	40 (191)	39 (187)	40 (188)	41 (193)	41 (192)	40 (185)	42 (195)	41 (187)	347
	60	39 (186)	40 (191)	39 (187)	40 (188)	41 (193)	41 (192)	40 (185)	42 (195)	41 (187)	420
	71	39 (186)	40 (191)	39 (187)	40 (188)	41 (193)	41 (192)	40 (185)	42 (195)	41 (187)	447
	86	39 (186)	40 (191)	39 (187)	40 (188)	41 (193)	41 (192)	40 (185)	42 (195)	41 (187)	584
254 x 254 x	73	39 (186)	40 (191)	39 (187)	40 (188)	41 (193)	41 (192)	40 (185)	42 (195)	41 (187)	465
	89	39 (186)	40 (191)	39 (187)	40 (188)	41 (193)	41 (192)	40 (185)	42 (195)	41 (187)	555
	107	39 (186)	40 (191)	39 (187)	40 (188)	41 (193)	41 (192)	40 (185)	42 (195)	41 (187)	707
	132	39 (186)	40 (191)	39 (187)	40 (188)	41 (193)	41 (192)	40 (185)	42 (195)	41 (187)	875
	167	39 (186)	40 (191)	39 (187)	40 (188)	41 (193)	41 (192)	40 (185)	42 (195)	41 (187)	1149
305 x 305 x	97	39 (186)	40 (191)	39 (187)	40 (188)	41 (193)	41 (192)	40 (185)	42 (195)	41 (187)	649
	118	39 (186)	40 (191)	39 (187)	40 (188)	41 (193)	41 (192)	40 (185)	42 (195)	41 (187)	781
	137	39 (186)	40 (191)	39 (187)	40 (188)	41 (193)	41 (192)	40 (185)	42 (195)	41 (187)	916
	158	39 (186)	40 (191)	39 (187)	40 (188)	41 (193)	41 (192)	40 (185)	42 (195)	41 (187)	1070
	198	39 (186)	40 (191)	39 (187)	40 (188)	41 (193)	41 (192)	40 (185)	42 (195)	41 (187)	1344
	240	39 (186)	40 (191)	39 (187)	40 (188)	41 (193)	41 (192)	40 (185)	42 (195)	41 (187)	1678
	283	39 (186)	40 (191)	39 (187)	40 (188)	41 (193)	41 (192)	40 (185)	42 (195)	41 (187)	1968
356 x 368 x	129	39 (186)	40 (191)	39 (187)	40 (188)	41 (193)	41 (192)	40 (185)	42 (195)	41 (187)	766
	153	39 (186)	40 (191)	39 (187)	40 (188)	41 (193)	41 (192)	40 (185)	42 (195)	41 (187)	922
	177	39 (186)	40 (191)	39 (187)	40 (188)	41 (193)	41 (192)	40 (185)	42 (195)	41 (187)	1098
	202	39 (186)	40 (191)	39 (187)	40 (188)	41 (193)	41 (192)	40 (185)	42 (195)	41 (187)	1279
<b>Shear capacity (kN)</b>											
No torsion	441	441	441	441	441	441	441	441	441	441	
Shear & torsion	180	174	179	178	169	169	177	166	174	161	
<b>Welds (mm)</b>											
Flange	8F	8F	8F	8F	8F	8F	8F	8F	8F	8F	
Web	8F	8F	8F	8F	8F	8F	8F	8F	8F	8F	

## Notes:

1. Normal type = Mode 1 failure; *Italic type* = Mode 2 failure.
2. Figure in brackets gives the sum of the maximum available bolt row forces, in tension (kN), on the beam side when no axial forces are present.
3. Flange welds are full strength or 1.4 times over strength of the connection (F = Fillet, P = Penetration).
4. Web welds are a nominal 8mm fillet.

Table 4.4 Moment capacity (kNm)

Column size		280 ASB No. of bolts = 8					300 ASB No. of bolts = 8					Web panel shear capacity (kN)
		74	100	105	124	136	153	155	185	196	249	
203 x 203 x	46	66 (271)	67 (275)	73 (289)	77 (302)	73 (289)	68 (267)	75 (284)	72 (277)	86 (309)	86 (309)	312
	52	67 (279)	70 (286)	74 (298)	81 (317)	76 (304)	71 (282)	78 (294)	76 (294)	91 (328)	90 (327)	347
	60	67 (279)	70 (286)	74 (298)	81 (317)	76 (304)	71 (282)	78 (294)	76 (294)	91 (328)	90 (331)	420
	71	67 (279)	70 (286)	74 (298)	81 (317)	76 (304)	71 (282)	78 (294)	76 (294)	91 (328)	90 (331)	447
	86	67 (279)	70 (286)	74 (298)	81 (317)	76 (304)	71 (282)	78 (294)	76 (294)	91 (328)	90 (331)	584
254 x 254 x	73	67 (279)	70 (286)	74 (298)	81 (317)	76 (304)	71 (282)	78 (294)	76 (294)	91 (328)	90 (331)	465
	89	67 (279)	70 (286)	74 (298)	81 (317)	76 (304)	71 (282)	78 (294)	76 (294)	91 (328)	90 (331)	555
	107	67 (279)	70 (286)	74 (298)	81 (317)	76 (304)	71 (282)	78 (294)	76 (294)	91 (328)	90 (331)	707
	132	67 (279)	70 (286)	74 (298)	81 (317)	76 (304)	71 (282)	78 (294)	76 (294)	91 (328)	90 (331)	875
	167	67 (279)	70 (286)	74 (298)	81 (317)	76 (304)	71 (282)	78 (294)	76 (294)	91 (328)	90 (331)	1149
305 x 305 x	97	67 (279)	70 (286)	74 (298)	81 (317)	76 (304)	71 (282)	78 (294)	76 (294)	91 (328)	90 (331)	649
	118	67 (279)	70 (286)	74 (298)	81 (317)	76 (304)	71 (282)	78 (294)	76 (294)	91 (328)	90 (331)	781
	137	67 (279)	70 (286)	74 (298)	81 (317)	76 (304)	71 (282)	78 (294)	76 (294)	91 (328)	90 (331)	916
	158	67 (279)	70 (286)	74 (298)	81 (317)	76 (304)	71 (282)	78 (294)	76 (294)	91 (328)	90 (331)	1070
	198	67 (279)	70 (286)	74 (298)	81 (317)	76 (304)	71 (282)	78 (294)	76 (294)	91 (328)	90 (331)	1344
	240	67 (279)	70 (286)	74 (298)	81 (317)	76 (304)	71 (282)	78 (294)	76 (294)	91 (328)	90 (331)	1678
	283	67 (279)	70 (286)	74 (298)	81 (317)	76 (304)	71 (282)	78 (294)	76 (294)	91 (328)	90 (331)	1968
356 x 368 x	129	67 (279)	70 (286)	74 (298)	81 (317)	76 (304)	71 (282)	78 (294)	76 (294)	91 (328)	90 (331)	766
	153	67 (279)	70 (286)	74 (298)	81 (317)	76 (304)	71 (282)	78 (294)	76 (294)	91 (328)	90 (331)	922
	177	67 (279)	70 (286)	74 (298)	81 (317)	76 (304)	71 (282)	78 (294)	76 (294)	91 (328)	90 (331)	1098
	202	67 (279)	70 (286)	74 (298)	81 (317)	76 (304)	71 (282)	78 (294)	76 (294)	91 (328)	90 (331)	1279
<b>Shear capacity (kN)</b>												
No torsion		515	515	515	515	515	515	515	515	515	515	
Shear & torsion		212	205	211	210	200	206	216	202	212	196	
<b>Welds (mm)</b>												
Flange		8F	8F	8F	10F	8F	8F	8F	8F	10F	8F	
Web		8F	8F	8F	8F	8F	8F	8F	8F	8F	8F	

Notes:

1. Normal type = Mode 1 failure; *Italic type* = Mode 2 failure.
2. Figure in brackets gives the sum of the maximum available bolt row forces, in tension (kN), on the beam side when no axial forces are present.
3. Flange welds are full strength or 1.4 times over strength of the connection (F = Fillet, P = Penetration).
4. Web welds are a nominal 8mm fillet.

Table 4.5 Moment capacity (kNm)

M20 8.8 bolts 15mm S275 flush type B end plate											
Column size	280 ASB No. of bolts = 6					300 ASB No. of bolts = 6					Web panel shear capacity (kN)
S355	74	100	105	124	136	153	155	185	196	249	
203 x 203 x	46	38 (184)	38 (184)	39 (184)	39 (184)	39 (184)	40 (184)	39 (184)	40 (184)	40 (184)	312
	52	42 (202)	42 (202)	43 (202)	43 (202)	43 (202)	44 (202)	43 (202)	44 (202)	44 (202)	347
	60	46 (221)	47 (225)	47 (222)	47 (222)	48 (225)	47 (223)	47 (217)	48 (225)	48 (219)	420
	71	46 (221)	47 (225)	47 (222)	47 (222)	48 (226)	47 (223)	47 (217)	48 (225)	48 (219)	447
	86	46 (221)	47 (225)	47 (222)	47 (222)	48 (226)	47 (223)	47 (217)	48 (225)	48 (219)	584
254 x 254 x	73	46 (221)	47 (225)	47 (222)	47 (222)	48 (226)	47 (223)	47 (217)	48 (225)	48 (219)	465
	89	46 (221)	47 (225)	47 (222)	47 (222)	48 (226)	47 (223)	47 (217)	48 (225)	48 (219)	555
	107	46 (221)	47 (225)	47 (222)	47 (222)	48 (226)	47 (223)	47 (217)	48 (225)	48 (219)	707
	132	46 (221)	47 (225)	47 (222)	47 (222)	48 (226)	47 (223)	47 (217)	48 (225)	48 (219)	875
	167	46 (221)	47 (225)	47 (222)	47 (222)	48 (226)	47 (223)	47 (217)	48 (225)	48 (219)	1149
305 x 305 x	97	46 (221)	47 (225)	47 (222)	47 (222)	48 (226)	47 (223)	47 (217)	48 (225)	48 (219)	649
	118	46 (221)	47 (225)	47 (222)	47 (222)	48 (226)	47 (223)	47 (217)	48 (225)	48 (219)	781
	137	46 (221)	47 (225)	47 (222)	47 (222)	48 (226)	47 (223)	47 (217)	48 (225)	48 (219)	916
	158	46 (221)	47 (225)	47 (222)	47 (222)	48 (226)	47 (223)	47 (217)	48 (225)	48 (219)	1070
	198	46 (221)	47 (225)	47 (222)	47 (222)	48 (226)	47 (223)	47 (217)	48 (225)	48 (219)	1344
	240	46 (221)	47 (225)	47 (222)	47 (222)	48 (226)	47 (223)	47 (217)	48 (225)	48 (219)	1678
	283	46 (221)	47 (225)	47 (222)	47 (222)	48 (226)	47 (223)	47 (217)	48 (225)	48 (219)	1968
356 x 368 x	129	46 (221)	47 (225)	47 (222)	47 (222)	48 (226)	47 (223)	47 (217)	48 (225)	48 (219)	766
	153	46 (221)	47 (225)	47 (222)	47 (222)	48 (226)	47 (223)	47 (217)	48 (225)	48 (219)	922
	177	46 (221)	47 (225)	47 (222)	47 (222)	48 (226)	47 (223)	47 (217)	48 (225)	48 (219)	1098
	202	46 (221)	47 (225)	47 (222)	47 (222)	48 (226)	47 (223)	47 (217)	48 (225)	48 (219)	1279
<b>Shear capacity (kN)</b>											
No torsion	441	441	441	441	441	441	441	441	441	441	
Shear & torsion	180	174	179	178	169	169	177	166	174	161	
<b>Welds (mm)</b>											
Flange	8F	8F	8F	8F	8F	8F	8F	8F	8F	8F	
Web	8F	8F	8F	8F	8F	8F	8F	8F	8F	8F	

## Notes:

1. Normal type = Mode 1 failure; *Italic type* = Mode 2 failure.
2. Figure in brackets gives the sum of the maximum available bolt row forces, in tension (kN), on the beam side when no axial forces are present.
3. Flange welds are full strength or 1.4 times over strength of the connection (F = Fillet, P = Penetration).
4. Web welds are a nominal 8mm fillet.

Table 4.6 Moment capacity (kNm)

Column size		280 ASB No. of bolts = 8					300 ASB No. of bolts = 8					M20 8.8 bolts 15mm S275 extended end plate	
		S355										Web panel shear capacity	(kN)
		74	100	105	124	136	153	155	185	196	249		
203 x 203 x	46	81 (316)	81 (316)	86 (316)	88 (316)	85 (316)	85 (314)	97 (342)	91 (329)	101 (342)	101 (342)	312	
	52	86 (342)	89 (349)	95 (361)	98 (361)	95 (361)	90 (336)	102 (365)	97 (353)	109 (371)	109 (371)	347	
	60	91 (366)	94 (373)	103 (396)	106 (396)	103 (396)	95 (360)	109 (395)	102 (376)	118 (409)	118 (409)	420	
	71	92 (371)	95 (380)	106 (410)	113 (428)	104 (406)	97 (369)	111 (402)	104 (386)	123 (433)	125 (438)	447	
	86	92 (371)	95 (380)	106 (410)	113 (428)	104 (406)	97 (369)	111 (402)	104 (386)	123 (433)	125 (438)	584	
254 x 254 x	73	92 (371)	95 (380)	106 (410)	110 (414)	104 (406)	97 (369)	111 (402)	104 (386)	122 (426)	122 (426)	465	
	89	92 (371)	95 (380)	106 (410)	113 (428)	104 (406)	97 (369)	111 (402)	104 (386)	123 (433)	125 (438)	555	
	107	92 (371)	95 (380)	106 (410)	113 (428)	104 (406)	97 (369)	111 (402)	104 (386)	123 (433)	125 (438)	707	
	132	92 (371)	95 (380)	106 (410)	113 (428)	104 (406)	97 (369)	111 (402)	104 (386)	123 (433)	125 (438)	875	
	167	92 (371)	95 (380)	106 (410)	113 (428)	104 (406)	97 (369)	111 (402)	104 (386)	123 (433)	125 (438)	1149	
305 x 305 x	97	92 (371)	95 (380)	106 (410)	113 (428)	104 (406)	97 (369)	111 (402)	104 (386)	123 (433)	125 (438)	649	
	118	92 (371)	95 (380)	106 (410)	113 (428)	104 (406)	97 (369)	111 (402)	104 (386)	123 (433)	125 (438)	781	
	137	92 (371)	95 (380)	106 (410)	113 (428)	104 (406)	97 (369)	111 (402)	104 (386)	123 (433)	125 (438)	916	
	158	92 (371)	95 (380)	106 (410)	113 (428)	104 (406)	97 (369)	111 (402)	104 (386)	123 (433)	125 (438)	1070	
	198	92 (371)	95 (380)	106 (410)	113 (428)	104 (406)	97 (369)	111 (402)	104 (386)	123 (433)	125 (438)	1344	
	240	92 (371)	95 (380)	106 (410)	113 (428)	104 (406)	97 (369)	111 (402)	104 (386)	123 (433)	125 (438)	1678	
	283	92 (371)	95 (380)	106 (410)	113 (428)	104 (406)	97 (369)	111 (402)	104 (386)	123 (433)	125 (438)	1968	
356 x 368 x	129	92 (371)	95 (380)	106 (410)	113 (428)	104 (406)	97 (369)	111 (402)	104 (386)	123 (433)	125 (438)	766	
	153	92 (371)	95 (380)	106 (410)	113 (428)	104 (406)	97 (369)	111 (402)	104 (386)	123 (433)	125 (438)	922	
	177	92 (371)	95 (380)	106 (410)	113 (428)	104 (406)	97 (369)	111 (402)	104 (386)	123 (433)	125 (438)	1098	
	202	92 (371)	95 (380)	106 (410)	113 (428)	104 (406)	97 (369)	111 (402)	104 (386)	123 (433)	125 (438)	1279	
<b>Shear capacity (kN)</b>													
No torsion		515	515	515	515	515	515	515	515	515	515		
Shear & torsion		212	205	211	210	200	206	216	202	212	196		
<b>Welds (mm)</b>													
Flange		10F	10F	12F	12F	10F	10F	12F	10F	12F	12F		
Web		8F	8F	8F	8F	8F	8F	8F	8F	8F	8F		

Notes:

1. Normal type = Mode 1 failure; *Italic type* = Mode 2 failure.
2. Figure in brackets gives the sum of the maximum available bolt row forces, in tension (kN), on the beam side when no axial forces are present.
3. Flange welds are full strength or 1.4 times over strength of the connection (F = Fillet, P = Penetration).
4. Web welds are a nominal 8mm fillet.

Table 4.7 Moment capacity (kNm)

M24 8.8 bolts												
15mm S275 flush type B end plate												
Column size S355		280 ASB No. of bolts = 6					300 ASB No. of bolts = 6				Web panel shear capacity (kN)	
		74	100	105	124	136	153	155	185	196		249
203 x 203 x	46	45 (218)	45 (218)	46 (218)	47 (218)	46 (218)	46 (218)	47 (218)	47 (218)	48 (218)	48 (218)	312
	52	53 (257)	54 (257)	54 (257)	55 (257)	54 (257)	55 (257)	56 (257)	55 (257)	57 (257)	57 (257)	347
	60	57 (276)	58 (281)	58 (277)	59 (278)	59 (281)	60 (281)	59 (274)	60 (281)	61 (277)	62 (281)	420
	71	57 (276)	59 (283)	58 (277)	59 (278)	60 (286)	60 (283)	59 (274)	62 (288)	61 (277)	65 (296)	447
	86	57 (276)	59 (283)	58 (277)	59 (278)	60 (286)	60 (283)	59 (274)	62 (288)	61 (277)	65 (296)	584
254 x 254 x	73	57 (276)	59 (283)	58 (277)	59 (278)	60 (286)	60 (283)	59 (274)	62 (288)	61 (277)	65 (296)	465
	89	57 (276)	59 (283)	58 (277)	59 (278)	60 (286)	60 (283)	59 (274)	62 (288)	61 (277)	65 (296)	555
	107	57 (276)	59 (283)	58 (277)	59 (278)	60 (286)	60 (283)	59 (274)	62 (288)	61 (277)	65 (296)	707
	132	57 (276)	59 (283)	58 (277)	59 (278)	60 (286)	60 (283)	59 (274)	62 (288)	61 (277)	65 (296)	875
	167	57 (276)	59 (283)	58 (277)	59 (278)	60 (286)	60 (283)	59 (274)	62 (288)	61 (277)	65 (296)	1149
305 x 305 x	97	57 (276)	59 (283)	58 (277)	59 (278)	60 (286)	60 (283)	59 (274)	62 (288)	61 (277)	65 (296)	649
	118	57 (276)	59 (283)	58 (277)	59 (278)	60 (286)	60 (283)	59 (274)	62 (288)	61 (277)	65 (296)	781
	137	57 (276)	59 (283)	58 (277)	59 (278)	60 (286)	60 (283)	59 (274)	62 (288)	61 (277)	65 (296)	916
	158	57 (276)	59 (283)	58 (277)	59 (278)	60 (286)	60 (283)	59 (274)	62 (288)	61 (277)	65 (296)	1070
	198	57 (276)	59 (283)	58 (277)	59 (278)	60 (286)	60 (283)	59 (274)	62 (288)	61 (277)	65 (296)	1344
	240	57 (276)	59 (283)	58 (277)	59 (278)	60 (286)	60 (283)	59 (274)	62 (288)	61 (277)	65 (296)	1678
	283	57 (276)	59 (283)	58 (277)	59 (278)	60 (286)	60 (283)	59 (274)	62 (288)	61 (277)	65 (296)	1968
356 x 368 x	129	57 (276)	59 (283)	58 (277)	59 (278)	60 (286)	60 (283)	59 (274)	62 (288)	61 (277)	65 (296)	766
	153	57 (276)	59 (283)	58 (277)	59 (278)	60 (286)	60 (283)	59 (274)	62 (288)	61 (277)	65 (296)	922
	177	57 (276)	59 (283)	58 (277)	59 (278)	60 (286)	60 (283)	59 (274)	62 (288)	61 (277)	65 (296)	1098
	202	57 (276)	59 (283)	58 (277)	59 (278)	60 (286)	60 (283)	59 (274)	62 (288)	61 (277)	65 (296)	1279
Shear capacity (kN)												
No torsion		635	635	635	635	635	635	635	635	635	635	
Shear & torsion		260	250	259	256	244	244	255	239	251	232	
Welds (mm)												
Flange		8F	8F	8F	8F	8F	8F	8F	8F	8F	8F	
Web		8F	8F	8F	8F	8F	8F	8F	8F	8F	8F	

## Notes:

1. Normal type = Mode 1 failure; *Italic type* = Mode 2 failure.
2. Figure in brackets gives the sum of the maximum available bolt row forces, in tension (kN), on the beam side when no axial forces are present.
3. Flange welds are full strength or 1.4 times over strength of the connection (F = Fillet, P = Penetration).
4. Web welds are a nominal 8mm fillet.

Table 4.8 Moment capacity (kNm)

Column size		280 ASB No. of bolts = 8					300 ASB No. of bolts = 8					M24 8.8 bolts 15mm S275 extended end plate	
		S355										Web panel shear capacity	(kN)
		74	100	105	124	136	153	155	185	196	249		
203 x 203 x	46	81 (316)	81 (316)	86 (316)	89 (316)	85 (316)	92 (343)	97 (343)	95 (343)	104 (343)	103 (343)	312	
	52	100 (403)	102 (410)	106 (410)	110 (410)	106 (410)	102 (392)	116 (427)	109 (408)	128 (445)	128 (445)	347	
	60	104 (427)	107 (434)	117 (461)	126 (485)	117 (461)	109 (421)	121 (451)	116 (438)	140 (499)	140 (499)	420	
	71	105 (431)	109 (443)	118 (466)	127 (491)	120 (474)	112 (434)	123 (459)	120 (455)	142 (509)	145 (522)	447	
	86	105 (431)	109 (443)	118 (466)	127 (491)	120 (474)	112 (434)	123 (459)	120 (455)	142 (509)	145 (522)	584	
254 x 254 x	73	105 (431)	109 (443)	118 (466)	127 (491)	120 (474)	112 (434)	123 (459)	119 (454)	142 (509)	144 (514)	465	
	89	105 (431)	109 (443)	118 (466)	127 (491)	120 (474)	112 (434)	123 (459)	120 (455)	142 (509)	145 (522)	555	
	107	105 (431)	109 (443)	118 (466)	127 (491)	120 (474)	112 (434)	123 (459)	120 (455)	142 (509)	145 (522)	707	
	132	105 (431)	109 (443)	118 (466)	127 (491)	120 (474)	112 (434)	123 (459)	120 (455)	142 (509)	145 (522)	875	
	167	105 (431)	109 (443)	118 (466)	127 (491)	120 (474)	112 (434)	123 (459)	120 (455)	142 (509)	145 (522)	1149	
305 x 305 x	97	105 (431)	109 (443)	118 (466)	127 (491)	120 (474)	112 (434)	123 (459)	120 (455)	142 (509)	145 (522)	649	
	118	105 (431)	109 (443)	118 (466)	127 (491)	120 (474)	112 (434)	123 (459)	120 (455)	142 (509)	145 (522)	781	
	137	105 (431)	109 (443)	118 (466)	127 (491)	120 (474)	112 (434)	123 (459)	120 (455)	142 (509)	145 (522)	916	
	158	105 (431)	109 (443)	118 (466)	127 (491)	120 (474)	112 (434)	123 (459)	120 (455)	142 (509)	145 (522)	1070	
	198	105 (431)	109 (443)	118 (466)	127 (491)	120 (474)	112 (434)	123 (459)	120 (455)	142 (509)	145 (522)	1344	
	240	105 (431)	109 (443)	118 (466)	127 (491)	120 (474)	112 (434)	123 (459)	120 (455)	142 (509)	145 (522)	1678	
	283	105 (431)	109 (443)	118 (466)	127 (491)	120 (474)	112 (434)	123 (459)	120 (455)	142 (509)	145 (522)	1968	
356 x 368 x	129	105 (431)	109 (443)	118 (466)	127 (491)	120 (474)	112 (434)	123 (459)	120 (455)	142 (509)	145 (522)	766	
	153	105 (431)	109 (443)	118 (466)	127 (491)	120 (474)	112 (434)	123 (459)	120 (455)	142 (509)	145 (522)	922	
	177	105 (431)	109 (443)	118 (466)	127 (491)	120 (474)	112 (434)	123 (459)	120 (455)	142 (509)	145 (522)	1098	
	202	105 (431)	109 (443)	118 (466)	127 (491)	120 (474)	112 (434)	123 (459)	120 (455)	142 (509)	145 (522)	1279	
<b>Shear capacity (kN)</b>													
No torsion		741	741	741	741	741	741	741	741	741	741		
Shear & torsion		306	295	305	302	288	297	311	291	306	282		
<b>Welds (mm)</b>													
Flange		12F	12F	12F (4P)	12F (4P)	12F	12F	12F	12F	12F (4P)	12F (4P)		
Web		8F	8F	8F	8F	8F	8F	8F	8F	8F	8F		

Notes:

1. Normal type = Mode 1 failure; *Italic type* = Mode 2 failure.
2. Figure in brackets gives the sum of the maximum available bolt row forces, in tension (kN), on the beam side when no axial forces are present.
3. Flange welds are full strength or 1.4 times over strength of the connection (F = Fillet, P = Penetration).
4. Web welds are a nominal 8mm fillet.

Table 4.9 Moment capacity (kNm)

M24 8.8 bolts												
15mm S355 flush type B end plate												
Column size S355		280 ASB No. of bolts = 6					300 ASB No. of bolts = 6				Web panel shear capacity (kN)	
		74	100	105	124	136	153	155	185	196		249
203 x 203 x	46	45 (218)	45 (218)	46 (218)	47 (218)	46 (218)	46 (218)	47 (218)	47 (218)	48 (218)	48 (218)	312
	52	53 (257)	54 (257)	54 (257)	55 (257)	54 (257)	55 (257)	56 (257)	55 (257)	57 (257)	57 (257)	347
	60	58 (281)	58 (281)	59 (281)	60 (281)	59 (281)	60 (281)	61 (281)	60 (281)	62 (281)	62 (281)	420
	71	63 (306)	64 (310)	65 (306)	65 (306)	66 (313)	65 (308)	65 (300)	67 (312)	67 (303)	70 (319)	447
	86	63 (306)	64 (310)	65 (306)	65 (306)	66 (313)	65 (308)	65 (300)	67 (312)	67 (303)	70 (319)	584
254 x 254 x	73	61 (297)	62 (297)	63 (297)	63 (297)	63 (297)	63 (297)	64 (297)	64 (297)	65 (297)	65 (297)	465
	89	63 (306)	64 (310)	65 (306)	65 (306)	66 (313)	65 (308)	65 (300)	67 (312)	67 (303)	70 (319)	555
	107	63 (306)	64 (310)	65 (306)	65 (306)	66 (313)	65 (308)	65 (300)	67 (312)	67 (303)	70 (319)	707
	132	63 (306)	64 (310)	65 (306)	65 (306)	66 (313)	65 (308)	65 (300)	67 (312)	67 (303)	70 (319)	875
	167	63 (306)	64 (310)	65 (306)	65 (306)	66 (313)	65 (308)	65 (300)	67 (312)	67 (303)	70 (319)	1149
305 x 305 x	97	63 (306)	64 (310)	65 (306)	65 (306)	66 (313)	65 (308)	65 (300)	67 (312)	67 (303)	70 (319)	649
	118	63 (306)	64 (310)	65 (306)	65 (306)	66 (313)	65 (308)	65 (300)	67 (312)	67 (303)	70 (319)	781
	137	63 (306)	64 (310)	65 (306)	65 (306)	66 (313)	65 (308)	65 (300)	67 (312)	67 (303)	70 (319)	916
	158	63 (306)	64 (310)	65 (306)	65 (306)	66 (313)	65 (308)	65 (300)	67 (312)	67 (303)	70 (319)	1070
	198	63 (306)	64 (310)	65 (306)	65 (306)	66 (313)	65 (308)	65 (300)	67 (312)	67 (303)	70 (319)	1344
	240	63 (306)	64 (310)	65 (306)	65 (306)	66 (313)	65 (308)	65 (300)	67 (312)	67 (303)	70 (319)	1678
	283	63 (306)	64 (310)	65 (306)	65 (306)	66 (313)	65 (308)	65 (300)	67 (312)	67 (303)	70 (319)	1968
356 x 368 x	129	63 (306)	64 (310)	65 (306)	65 (306)	66 (313)	65 (308)	65 (300)	67 (312)	67 (303)	70 (319)	766
	153	63 (306)	64 (310)	65 (306)	65 (306)	66 (313)	65 (308)	65 (300)	67 (312)	67 (303)	70 (319)	922
	177	63 (306)	64 (310)	65 (306)	65 (306)	66 (313)	65 (308)	65 (300)	67 (312)	67 (303)	70 (319)	1098
	202	63 (306)	64 (310)	65 (306)	65 (306)	66 (313)	65 (308)	65 (300)	67 (312)	67 (303)	70 (319)	1279
Shear capacity (kN)												
No torsion		635	635	635	635	635	635	635	635	635	635	
Shear & torsion		260	250	259	256	244	244	255	239	251	232	
Welds (mm)												
Flange		10F	8F	10F	8F	8F	8F	8F	8F	8F	8F	
Web		8F	8F	8F	8F	8F	8F	8F	8F	8F	8F	

## Notes:

1. Normal type = Mode 1 failure; *Italic type* = Mode 2 failure.
2. Figure in brackets gives the sum of the maximum available bolt row forces, in tension (kN), on the beam side when no axial forces are present.
3. Flange welds are full strength or 1.4 times over strength of the connection (F = Fillet, P = Penetration).
4. Web welds are a nominal 8mm fillet.

Table 4.10 Moment capacity (kNm)

Column size		280 ASB No. of bolts = 8					300 ASB No. of bolts = 8					M24 8.8 bolts 15mm S355 extended end plate	
		S355										Web panel shear capacity	(kN)
		74	100	105	124	136	153	155	185	196	249		
203 x 203 x	46	85 (316)	86 (316)	91 (316)	91 (316)	90 (316)	97 (343)	104 (343)	101 (343)	106 (343)	106 (343)	312	
	52	106 (410)	107 (410)	112 (410)	116 (410)	112 (410)	118 (437)	127 (445)	124 (445)	134 (445)	134 (445)	347	
	60	118 (470)	121 (479)	133 (508)	137 (508)	133 (508)	123 (461)	139 (501)	132 (484)	154 (521)	154 (521)	420	
	71	125 (505)	129 (518)	141 (549)	151 (574)	143 (555)	132 (503)	147 (538)	142 (528)	170 (592)	170 (592)	447	
	86	125 (505)	129 (518)	141 (549)	153 (580)	143 (555)	132 (503)	147 (538)	142 (528)	172 (602)	174 (611)	584	
254 x 254 x	73	121 (485)	125 (494)	137 (528)	141 (528)	137 (528)	126 (476)	142 (516)	135 (499)	158 (540)	158 (540)	465	
	89	125 (505)	129 (518)	141 (549)	153 (580)	143 (555)	132 (503)	147 (538)	142 (528)	172 (602)	174 (611)	555	
	107	125 (505)	129 (518)	141 (549)	153 (580)	143 (555)	132 (503)	147 (538)	142 (528)	172 (602)	174 (611)	707	
	132	125 (505)	129 (518)	141 (549)	153 (580)	143 (555)	132 (503)	147 (538)	142 (528)	172 (602)	174 (611)	875	
	167	125 (505)	129 (518)	141 (549)	153 (580)	143 (555)	132 (503)	147 (538)	142 (528)	172 (602)	174 (611)	1149	
305 x 305 x	97	125 (505)	129 (518)	141 (549)	153 (580)	143 (555)	132 (503)	147 (538)	142 (528)	171 (599)	171 (599)	649	
	118	125 (505)	129 (518)	141 (549)	153 (580)	143 (555)	132 (503)	147 (538)	142 (528)	172 (602)	174 (611)	781	
	137	125 (505)	129 (518)	141 (549)	153 (580)	143 (555)	132 (503)	147 (538)	142 (528)	172 (602)	174 (611)	916	
	158	125 (505)	129 (518)	141 (549)	153 (580)	143 (555)	132 (503)	147 (538)	142 (528)	172 (602)	174 (611)	1070	
	198	125 (505)	129 (518)	141 (549)	153 (580)	143 (555)	132 (503)	147 (538)	142 (528)	172 (602)	174 (611)	1344	
	240	125 (505)	129 (518)	141 (549)	153 (580)	143 (555)	132 (503)	147 (538)	142 (528)	172 (602)	174 (611)	1678	
	283	125 (505)	129 (518)	141 (549)	153 (580)	143 (555)	132 (503)	147 (538)	142 (528)	172 (602)	174 (611)	1968	
356 x 368 x	129	125 (505)	129 (518)	141 (549)	153 (580)	143 (555)	132 (503)	147 (538)	142 (528)	172 (602)	174 (611)	766	
	153	125 (505)	129 (518)	141 (549)	153 (580)	143 (555)	132 (503)	147 (538)	142 (528)	172 (602)	174 (611)	922	
	177	125 (505)	129 (518)	141 (549)	153 (580)	143 (555)	132 (503)	147 (538)	142 (528)	172 (602)	174 (611)	1098	
	202	125 (505)	129 (518)	141 (549)	153 (580)	143 (555)	132 (503)	147 (538)	142 (528)	172 (602)	174 (611)	1279	
<b>Shear capacity (kN)</b>													
No torsion		741	741	741	741	741	741	741	741	741	741		
Shear & torsion		306	295	305	302	288	297	311	291	306	282		
<b>Welds (mm)</b>													
Flange		12F (4P)	12F (5P)	12F (6P)	12F (7P)	12F (5P)	12F (4P)	12F (5P)	12F (4P)	12F (7P)	12F (6P)		
Web		8F	8F	8F	8F	8F	8F	8F	8F	8F	8F		

Notes:

1. Normal type = Mode 1 failure; *Italic type* = Mode 2 failure.
2. Figure in brackets gives the sum of the maximum available bolt row forces, in tension (kN), on the beam side when no axial forces are present.
3. Flange welds are full strength or 1.4 times over strength of the connection (F = Fillet, P = Penetration).
4. Web welds are a nominal 8mm fillet.



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# Services design

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## Platinum Point, Leith.

The Platinum Point development at Leith is a luxury residential regeneration project that comprises five interconnecting 11 and 13-storey blocks containing apartments and penthouses on Edinburgh's waterfront. The Slimdek flooring system was adopted to allow higher ceilings as well as maximising the number of floor levels within the building height.

The development included apartments over car parking, and involved long spans and a complicated layout. Slimdek's flexibility allowed these requirements to be achieved with just one unobtrusive column in each flat.

The Slimdek system offered:

- long spans with internal column free spaces, giving greater flexibility to the internal layout
- maximum slab depth of 325mm, allowing clear floor to ceiling height of 2.55m to be maintained
- reduced need for any special fire protection measures, as the steel floor beams were encased in the concrete slab
- clear soffits, making it much easier for follow-on trades to fix services

Developer:

Gregor Shore

Architect:

Gilbert Associates

Structural engineer:

Goodson Associates

Steelwork contractor:

Conder Structures Ltd

Market sector:

Multi-storey residential



## 5. Services design

### 5.1 Services integration

There are three opportunities for service integration using Slimdek:

- Partial integration: pass major services below the slab and beams and use the space between the ribs for small pipes and fitments, such as lighting units. This allows for cross-overs of ducts or pipes. The elimination of downstands provides for greater flexibility of service distribution and reduces the depth of the structure-services zone.
- Full integration: form circular or elongated openings in the webs of the ASB so that ducts and pipes located between and within the depth of the ribs can pass through the beams. (Alternatively, the space between the ribs can act as a duct in itself, which again continues through the openings in the beams).
- Slab penetrations: Slimdek offers the services engineer flexibility in the provision of service openings within the floor slab. However, careful co-ordination with the structural engineer is required so that openings can be provided without any, or with minimal, additional strengthening. See also *Section 2.2.6*.

The ComFlor 225 decking has a re-entrant portion in the crest of its profile that can provide a suspension point for services and ceilings without need for additional fixings, see *Section 5.4*.

In certain areas where spans are relatively short ( $< 3.5\text{m}$ ), shallower floors may be created locally, using a composite slab of 120 to 150mm depth comprising more traditional decking of 50 to 60mm depth. This is particularly useful in, or adjacent to, core areas where duct cross-overs and horizontal bends are required without deepening the ceiling-floor zone excessively.

### 5.2 Openings

#### 5.2.1 Openings in ASB sections

Full integration of services can be achieved by providing openings through the ASB midway between the ribs of the deep decking. During fabrication, an opening (usually circular or oval) is cut in the web of the ASB. The same sized openings are also cut in the diaphragms that fit between the ribs and a sleeve is placed through the beam and diaphragms before the concrete is placed. The elements that form the opening in an ASB are shown in *Figure 5.1*. Flat, oval or circular ducts may be placed inside the sleeve and sealed externally.

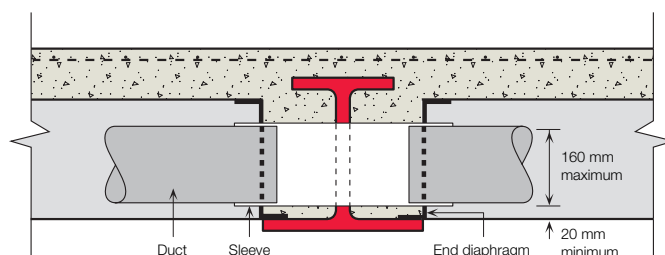


Figure 5.1 Forming openings through ASB

#### 5.2.2 Maximum sizes of openings

Information on the size of openings that can be formed in an ASB is given in *Section 2.3.3.6*.

#### 5.2.3 Openings in the slab

Generally, openings can be introduced without significant limitations as shown in *Figure 5.2*. Structural requirements around openings are given in *Section 2*.

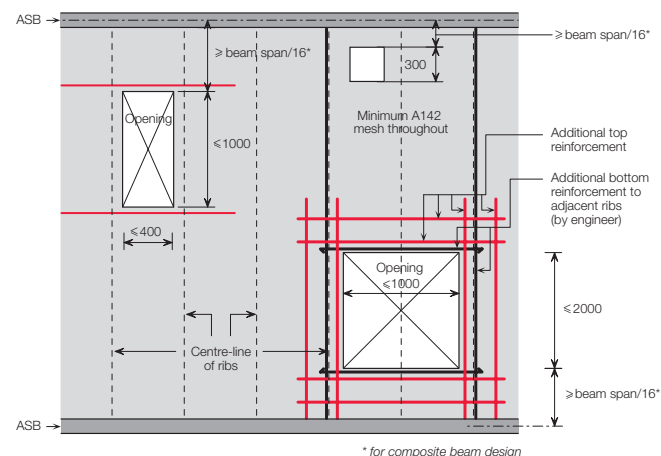
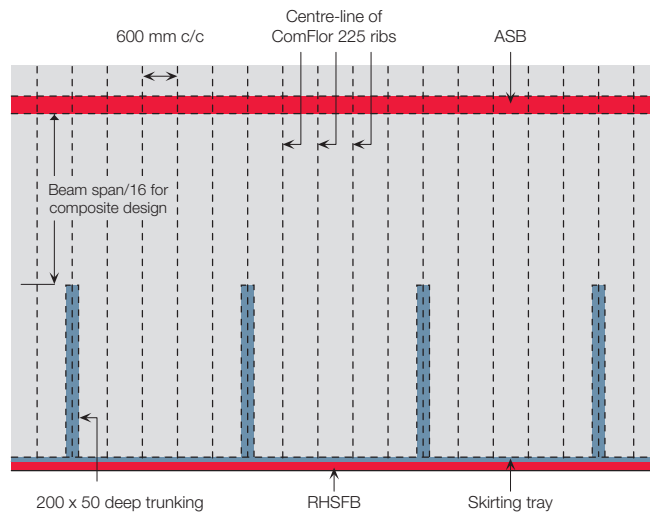
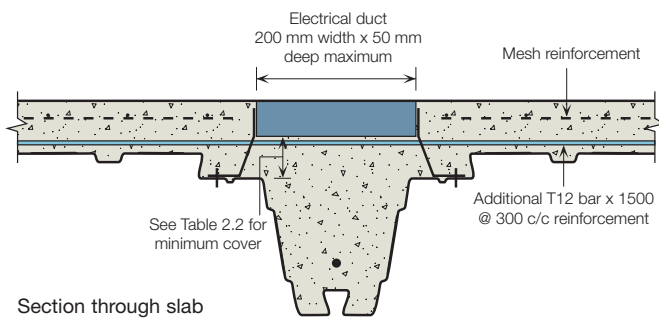


Figure 5.2 Details of openings in the slab

### 5.3 Electrical trunking and ducts

It is also possible to create routes for electrical trunking and ducts which can be located either within the structural slab or within a structural or non-structural screed, see *Figure 5.3*. Further information is given in *Section 2.2.7*.



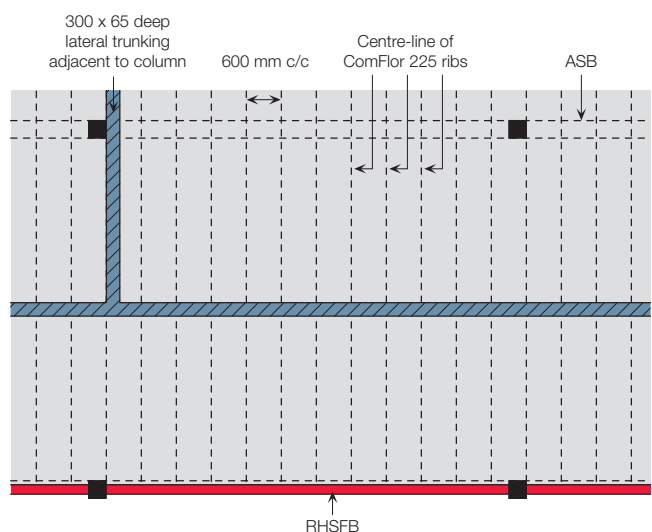
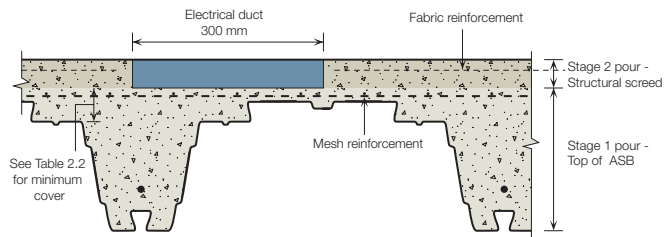
**Figure 5.3** Electrical duct within composite slab

### 5.4 Service attachments

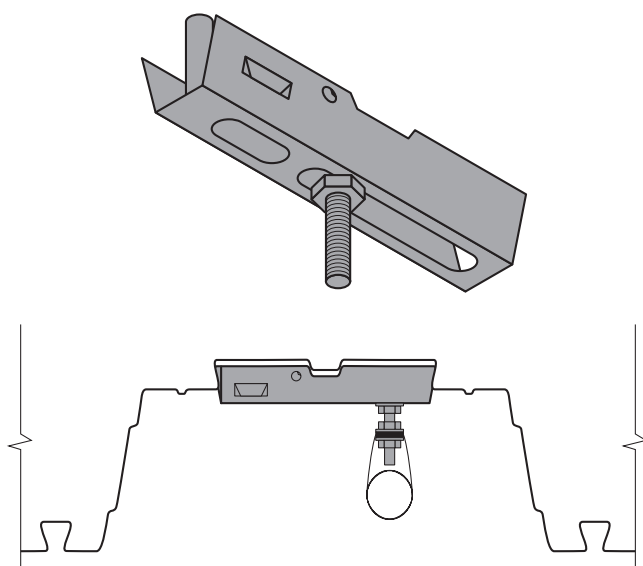
The ComFlor 225 decking makes the fixing of services and ceilings much easier. Service hangers can be suspended from the crest dovetail and utilised for services running parallel or perpendicular to the deck span. In addition, ceiling systems may be hung from the top crest hanger or attached directly to the underside of the rib using self-drilling self-tapping screws after the concrete has been placed.

The adjustable Lindapter Slimdek 2 fixing, see *Figure 5.5*, is designed for use with ComFlor 225 to accommodate variances encountered on site and enable secure suspension of services directly from the underside of the ComFlor 225 composite floor decking. It clips into the crest dovetail of the decking and achieves a safe working load of 1.0kN per fixing with a built-in factor-of-safety of 3. Minimum spacing of crest fixings is 500mm at full working load provided the overall design load for the slab is not compromised. Installation of Lindapter Slimdek 2 is fast and accurate every time and is carried out without specialist tools or skills because the product slots easily into the re-entrant channel and is locked mechanically with a 180° turn of a spanner. Variable drop rod position and lateral adjustability along the re-entrant channel permit unhindered alignment of service runs, whilst the shallow fixing depth enables pipework, ducting, electrical equipment and cable trays to run within the structural floor space.

Alternatively self-drilling self-tapping screws may be used to attach hangers to the decking after the concrete has been placed. Care must be taken not to compromise the integrity of the decking and the use of non-percussive equipment is advised.



**Figure 5.4** Electrical duct within structural screed (second stage pour)



**Figure 5.5** Lindapter Slimdek 2 fixing

## 5.5 Advanced energy efficient systems

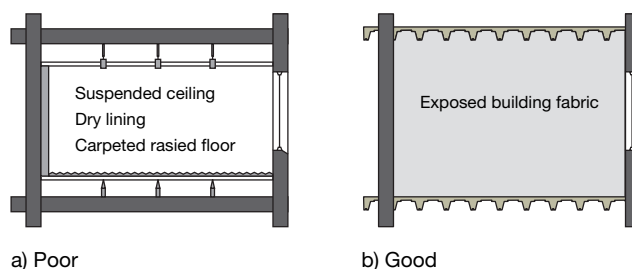
The Slimdek system can be designed to incorporate the latest technology in energy efficient services principles. Research and development into enhanced forms of passive service systems and optimising the contribution of the structure to the operation of the building environment have led to new methods of air distribution on and through the floor construction.

Enhanced systems may be created through the use of a combination of suspended ceilings, raised floor voids and supply and extract ventilation. Some of these systems are described in the SCI-P-181: *Environmental Floor Systems*<sup>[20]</sup>, and in SCI-P-273: *Service Integration in Slimdek*<sup>[24]</sup>.

For applications where increased cooling capacity is necessary, radiant cooling ceiling panels (chilled ceilings and beams) can be integrated into Slimdek construction. The above SCI publications give more detail and a worked example of the use of chilled beam systems.

### 5.5.1 Fabric energy storage

Floor slabs provide for regulation of internal temperatures by their thermal capacity which may be used to control ambient heat gains within a building. Surface finishes such as raised floors or direct finishes such as carpets generally restrict the efficient transfer of heat between the space and the top surface of the floor slab as shown in *Figure 5.6(a)*. However, the underside of the floor slab is considered an ideal surface to utilise the inherent thermal capacity of the slab, provided the ceiling is designed not to impede this action as shown in *Figure 5.6(b)*. This technique can reduce internal temperatures by 2 to 3°C, which is equivalent to a cooling effect of approximately 20W/m<sup>2</sup>°C. If advantage can be taken of the energy storage capabilities of the floor slab, then the need for air conditioning may be reduced or, in some cases, eliminated, which will generate considerable savings in capital and running costs.



**Figure 5.6** Fabric thermal regulation

In a typical building, with diurnal temperature variation, only a relatively thin depth of concrete (typically 75mm to 100mm) is effective for efficient heat transfer and storage to take place. For depths of 100mm and over, no more heat is stored in the slab. The thermal admittance to concrete slabs is shown in *Figure 5.7*. The high surface area of the ComFlor 225 decking used for Slimdek is ideally suited for this daily heat transfer.

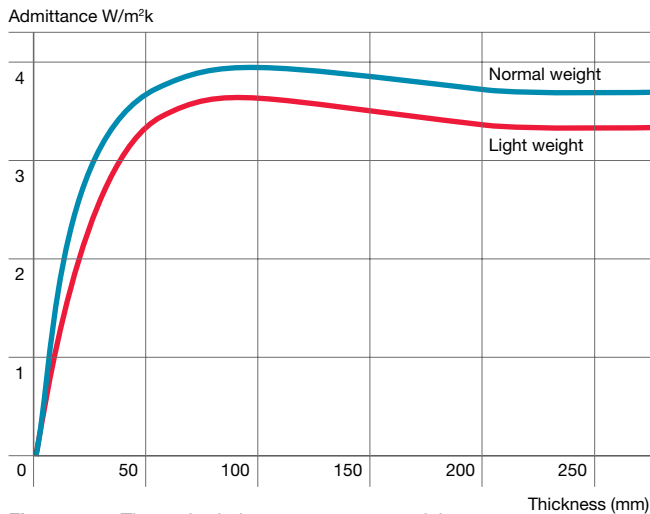


Figure 5.7 Thermal admittance to concrete slabs

In the daytime, heat is generated by the activities within the building, and by the external conditions and solar gain (depending on the fenestration and shading). At night, heat is lost from the building as temperatures fall externally. There is often a need to cool the internal environment during the summer, whilst in winter additional heat may be needed for comfort of the occupants. A typical summer day/night temperature cycle is shown in Figure 5.8. Studies show that, in a typical modern office building, equilibrium is reached with the external environment at around zero degrees Celsius. Thus the main requirement is for cooling.

Harnessing the fabric energy storage (FES) potential of Slimdek during the day will reduce the maximum air temperature, whilst at night the heat stored in the slab can be released through ventilation via external louvres.

For passive cooling, it is necessary to expose the soffit of the slab, and the ComFlor 225 decking and bottom flange of the supporting beam ensure good radiative heat transfer. The high surface area of the ComFlor 225 decking provides a 20% better heat transfer than an exposed flat soffit whilst minimising the weight of the floor slab. Where soffits are required to have an improved architectural visual appearance, careful attention to detailing the decking and fixing of fasteners is required. Alternatively, where it is necessary to conceal the slab from below for aesthetic reasons, a perforated ceiling as described in Section 5.6 may be used.

As with all passive design strategies, success relies upon careful design and implementation as part of an overall low energy philosophy in which heat gain reduction and effective ventilation play key roles.

### Modern office building: warm summer conditions

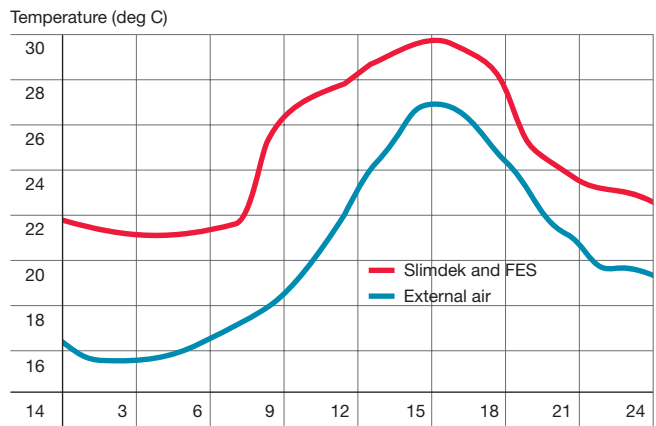


Figure 5.8 Day/night cooling/heating cycle

### 5.5.2 Integrated services between the ribs

The space between the ribs may be used as ducts for natural ventilation to internal zones as shown in Figure 5.9.

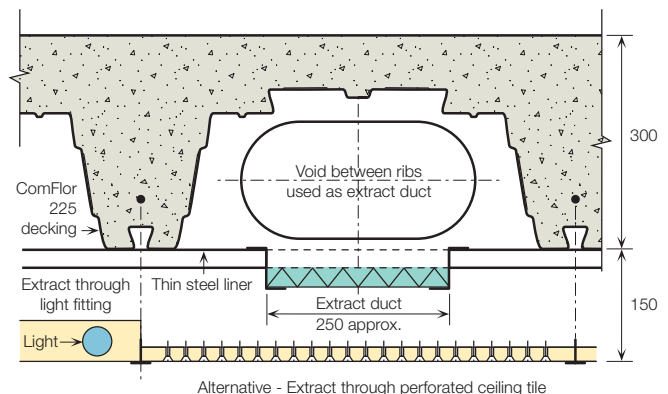


Figure 5.9 Use of space between ComFlor 225 ribs for service duct

In some advanced systems of service integration, it is possible to use the space between the deck ribs as a duct for mechanical ventilation to internal zones as shown in Figure 5.10. These details are covered in the SCI publication 'Service Integration in Slimdek' (SCI-P-273)<sup>[24]</sup>.

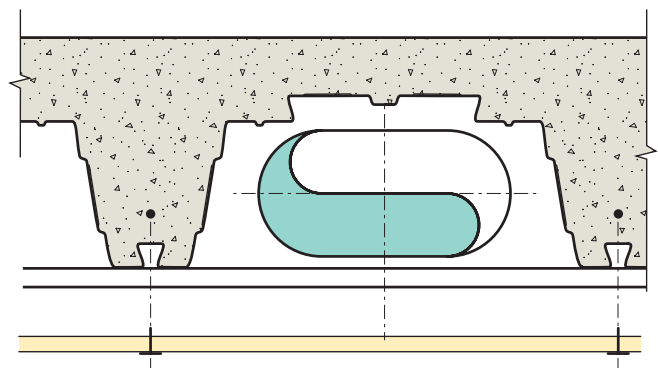


Figure 5.10 Ventilation duct to internal zones

## 5.6 Perforated ceilings

Perforated ceilings may be used, and tests have shown that ceilings with perforations of approximately 20% of the surface area allowed 85% of the cooling effect of a fully exposed soffit to be achieved by creating sufficient air movement through the ceiling. Thin tiles (preferably metallic) should be used because they present the least resistance to air flow.

Tests have been carried out at The Steel Construction Institute to assess the effect of different ceiling configurations on the passive cooling performance of composite slabs. A test room was monitored over twenty-four-hour cycles when subjected to heat gains replicating a typical office. The major findings are summarised below.

- Conventional closed suspended ceilings reduce the cooling effect of a floor slab by at least 70%.
- Open areas greater than 22% fail to conceal the soffit fully unless the decking and any services are painted in dark colours.
- Thin steel tiles (0.7mm) should be used because they present the least resistance to air flow and allow significant radiative heat transfer to and from the soffit. Mineral board tiles are thicker, providing more resistance to air flow and reduced radiative heat transfer. They are also less robust and are liable to shed material when handled or cleaned.
- Tiles with an open area of 20% allowed about 50% of the air to flow through to the ceiling void under test room conditions when compared with an exposed slab. However, the tiles attain a temperature close to the air temperature at that height, and radiate this heat to the slab during occupied hours. Thus, although convective heat transfer is reduced, radiative heat transfer is increased, resulting in overall heat transfer close to that achieved by a fully exposed slab.
- In test room conditions, perforated ceilings of 15% and 20% open area respectively allowed 81% and 86% of the cooling effect to be achieved, based upon measurements of the soffit heat flux.

The Slimdek profile has been designed to be compatible with the standard 600mm square ceiling tile available from most manufacturers. Perforated steel tiles are usually equipped with a backing of acoustic fleece to moderate the acoustic environment in offices, particularly to reduce reverberation time and enhance acoustic privacy. These tiles can be supplied without the backing so as to be open to allow air to flow through. If possible, the larger hole sizes should be chosen because these have a lower air flow resistance for a given open area. Typical perforation patterns available are shown in *Table 5.1*.

**Table 5.1 Typical patterns in perforated ceiling tiles**

Open area	Perforation size	Pitch
14%	2.4mm diameter	Diagonal
16%	2.5mm diameter	5.5mm square
20%	1.8mm diameter	3.6mm square
22%	1.5mm diameter	Diagonal
22%	3.0mm diameter	Diagonal

Acoustic performance must be considered when passive fabric thermal storage is used. The exposure of hard reflective slab surfaces can lead to unacceptably long reverberation times and lack of acoustic privacy. With the use of perforated tiles, however, the problem can be more easily overcome, either by use of acoustic absorbent batts suspended in the troughs or by including some tiles with acoustic backing at intervals. The main open areas should always be concentrated near the façade and at the opposite wall because the dominant convection currents during the day will facilitate flow through the ceiling void with this configuration. The overall effect should remain uniform when viewed from below.

The soffit finish will affect the performance. The standard galvanised finish of the soffit will have an emissivity of around 0.3, a value which is significantly lower than that of most building materials (0.8–0.9). Radiative heat transfer can be improved significantly by painting the soffit. Most standard paint finishes will raise the emissivity to 0.8 or above and at the longer wavelengths, the chosen colour has little effect.



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# Acoustic design

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## St Richard's Hospital, Chichester.

The Slimdek flooring system proved to be the healthy choice for the state-of-the-art Diagnosis and Treatment Centre at St Richard's Hospital, Chichester.

This is predominantly a three-storey steel structure with 'clean air' operating theatres and wards on the first floor, plant room on the second floor, and a spectacular central full height atrium. The upper floors consist of varying sizes of Asymmetric Slimflor Beam sections supporting CornFlor 225 deep decking, creating clear spans of over 8 metres between columns.

Careful vibration analysis was carried out by the Steel Construction Institute and led to significant savings in steelwork. It also confirmed that the structure met the stringent requirements of Health Technical Memorandum 2045 (HTM2045) and BS 6472 for operating theatre environments.

The Slimdek system offered:

- shallow structural depth, maximising useable floor depth for services
- vibration response within HTM2045 limits
- flexibility for openings
- good acoustic performance
- rapid construction system
- future flexibility for new installations and refurbishment

Client:

Royal West Sussex NHS Trust,  
St Richard's Hospital, Chichester

Architect:

Nightingale Associates

Structural engineer:

Gyoury Self Partnership

Main contractor:

Henry Jones, Kier Group

Steelwork contractor:

FH Dale Ltd

Market sector:

Healthcare



## 6. Acoustic design

### 6.1 General

Limiting the amount of sound that is transmitted between rooms is an important consideration for the serviceability of buildings especially for residential, educational and healthcare buildings. For separating constructions between dwellings, Part E of the Building Regulations requires that both airborne and impact sound transmission is addressed.

Acoustic insulation is often associated with high mass constructions. However, this is constructionally and economically inefficient and inappropriate for dry assembled construction. Acoustic insulation is best provided by a combination of mass, isolation of separate layers and sealing of joints. Furthermore, there is a need for resilient layers to be introduced to deal with the effect of impact sound, even in concrete floors.

All of these attributes are encompassed within the Slimdek system which, combined with appropriate interface details, can easily provide a suitable solution for acoustic insulation as has been proven by numerous acoustic tests conducted both on and off site.

The mass is provided by the composite slab: acoustic testing in buildings has shown that generally the effective mass of the slab per m<sup>2</sup> of floor area can be used to predict performance. Additional mass is provided by the floor finish (screed or board) and ceiling plasterboard. The resilience is provided by mounting the plasterboard on resilient bars or on a proprietary metal frame ceiling. This decouples the ceiling from the slab and reduces sound transfer. In addition, a variety of acoustic floors can be used on top of the slab to decouple the floor finish from the slab. Examples of floor and ceiling details which more than meet the acoustic requirements of the Building Regulations are given later in this section.

Further information on acoustic performance of steel structures is given in SCI-P-372<sup>[31]</sup>.

#### 6.1.1 Sound paths

Clearly this manual is concerned with the construction of Slimdek floors but sound can travel between rooms either by direct transmission (i.e., through the separating structure) or by flanking transmission (around the separating structure through adjacent building elements).

Direct transmission depends upon the separating floor construction. Flanking transmission depends on the detailing of the interfaces between adjacent walls and floors. Both direct and flanking sound paths must be considered to achieve the required acoustic performance.

### 6.2 Regulations

The acoustic requirements for dwellings and rooms for residential purposes are specified in Approved Document E<sup>[43]</sup> of the Building Regulations for England and Wales. The equivalent document in Scotland is Section 5 of the Domestic Technical Handbook<sup>[44]</sup>, in Northern Ireland it is Technical Booklet G<sup>[45]</sup>. For hospitals, Health Technical Memorandum 2045 'Acoustic design considerations'<sup>[35]</sup>, produced by NHS Estates, specifies the requirements. For schools, Building Bulletin 93 'Acoustic design of schools'<sup>[46]</sup> produced by the Department for Education and Skills, should be adopted.

The acoustic requirements detailed in the documents stated above are expressed using different terms and methods as appropriate to the different building types. Therefore, a direct comparison of requirements is not straightforward. However, the principles of good acoustic detailing are consistent.

The acoustic requirements from Approved Document E<sup>[43]</sup> for separating floors are given in *Table 6.1*.

**Table 6.1 Required acoustic performance from Approved Document E<sup>[43]</sup>**

Building type	Separating floors	
	Airborne sound	Impact sound
	$D_{nT,w} + C_{tr}$	$L'_{nT,w}$
Purpose-built dwellings	≥ 45 dB	≤ 62 dB
Dwellings formed by material change of use	≥ 43 dB	≤ 64 dB
Purpose-built rooms for residential purposes	≥ 45 dB	≤ 62 dB
Rooms for residential purposes formed by material change of use	≥ 43 dB	≤ 64 dB

It can be seen from *Table 6.1* that both airborne sound and impact sound need to be considered for separating floors.

The  $C_{tr}$  term is a spectrum adaptation term, which is generally negative and adjusts the airborne performance to take additional account of the low-frequency sounds that often cause problems in residential buildings.  $C_{tr}$  is generally in the region of -6 dB to -9 dB for Slimdek floors.

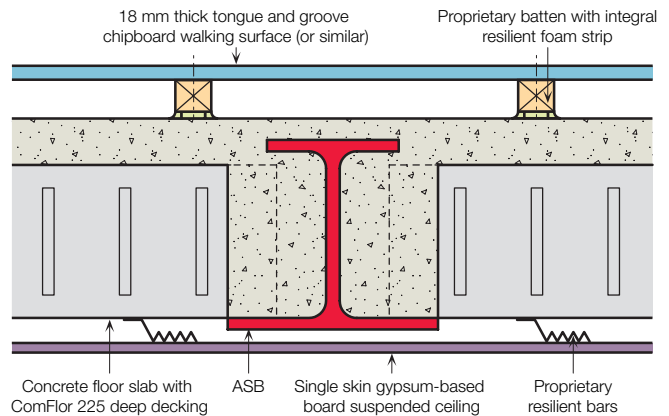
Approved Document E explains that there are two methods of demonstrating compliance with the regulation:

- a) Pre-completion testing (PCT) i.e., carry out on-site tests to measure the acoustic performance of separating walls and floors, to confirm that the performance standards in Approved Document E<sup>[43]</sup> are met.
- b) Construction using Robust Details (RDs), as published in the Robust Details Handbook<sup>[47]</sup>. Before construction the developer must also register the site with Robust Details Limited, who administer the RD scheme.

Both methods of compliance can be used for Slimdek floors although pre-completion testing is probably the most appropriate because it allows more flexibility in the design and detailing.

### 6.3 Floor construction

The acoustic insulation of Slimdek floors is provided by the combination of the structural floor, a ceiling and a floor treatment, see *Figure 6.1*.



**Figure 6.1** Slimdek separating floor

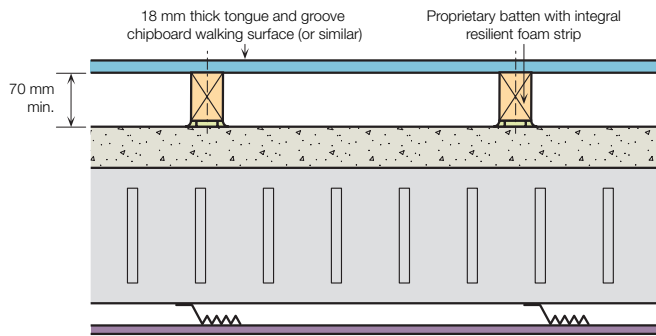
For Slimdek floors to be compliant with the relevant Robust Detail (E-FS-1) the following must be satisfied:

- a) Concrete thickness must be at least 80mm at the shallowest point i.e., a slab depth of 305mm is required with ComFlor 225 decking.
- b) Concrete density must be at least 2200kg/m<sup>3</sup>.
- c) A ceiling must be provided of at least 8kg/m<sup>2</sup> of gypsum-based board.
- d) One of the floating floor treatments described in the Robust Details Handbook<sup>[47]</sup> must be applied. An isolated screed floor treatment is not applicable for an RD compliant Slimdek floor.

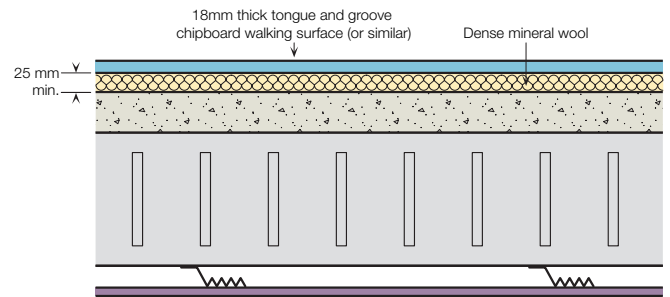
For Slimdek floors that are not designed to be RD compliant the options are less restrictive. All of the floor treatments shown in *Figures 6.2 to 6.7* may be used and thickness of concrete at the shallowest point may be less than 80mm depending on the type of floor treatment and ceiling used.

All separating floors should have a ceiling treatment of at least one layer of nominal 8kg/m<sup>2</sup> of gypsum-based board. Ceiling boards may be supported by resilient bars, timber battens or a propriety metal frame system.

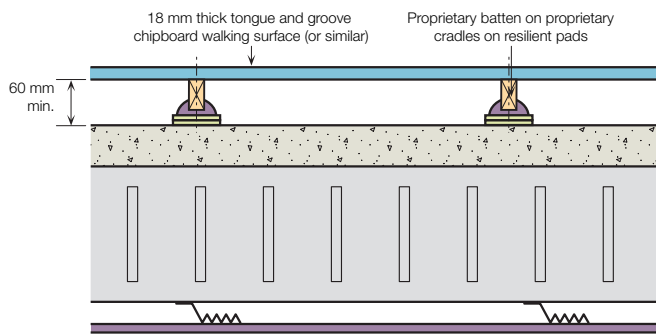
The expected acoustic performances for Slimdek floors with different floor treatments are provided in *Table 6.2*.



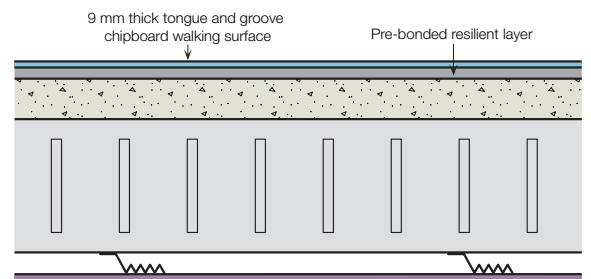
**Figure 6.2** Deep batten floor treatment



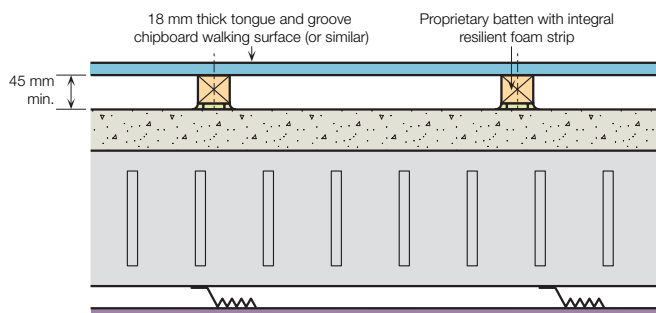
**Figure 6.5** Platform floor treatment



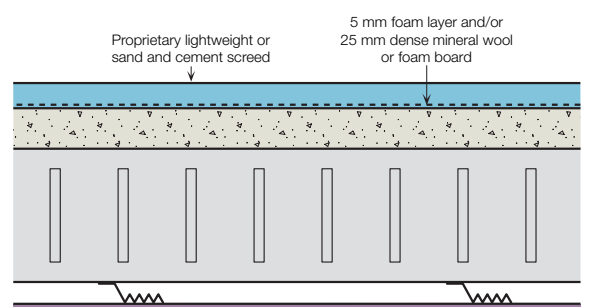
**Figure 6.3** Cradle and batten floor treatment



**Figure 6.6** Shallow Platform floor treatment



**Figure 6.4** Standard batten floor treatment



**Figure 6.7** Isolated screed floor treatment

Table 6.2 Expected acoustic performance of Slimdek floors

Structural slab	Ceiling	Floor treatment	Expected acoustic performance	
			Airborne sound	Impact sound
			$D_{nT,w} + C_{tr}$	$L'_{nT,w}$
Slab depth of 300mm on ComFlor 225 decking. Normal weight concrete (at least 2200 kg/m <sup>3</sup> ).	Gypsum-based board at least 8kg/m <sup>2</sup> supported on resilient bars.	Deep batten	54 – 58 dB	38 – 48 dB
		Cradle and batten	54 – 58 dB	38 – 48 dB
		Standard batten	54 – 58 dB	38 – 48 dB
		Platform	52 – 57 dB	40 – 45 dB
		Shallow platform	50 – 55 dB	40 – 45 dB
		Isolated screed	50 – 57 dB	40 – 50 dB

Note: Actual acoustic performance is dependent on the junction details as well as the floor construction.

Acoustic performance of the floor system can be enhanced by increasing the depth of concrete, increasing the mass of the ceiling, providing mineral wool insulation within the ceiling void or for a battened floor treatment, including insulation between the battens.

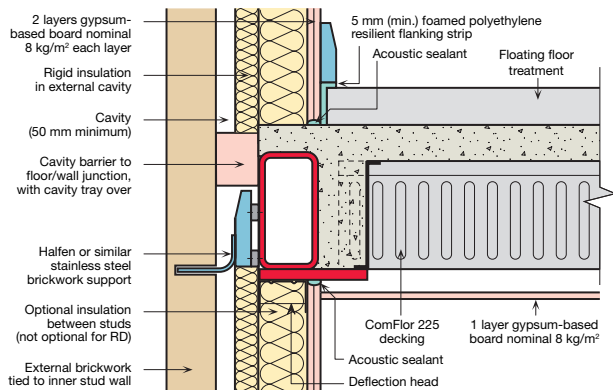
## 6.4 Separating walls

Either light steel or masonry separating walls can be used with Slimdek floors. Light steel walls are generally recommended because of the ease and speed of construction and the elimination of wet trades on site. Typically, light steel separating walls comprise twin frames of studs separated by mineral wool insulation. The outer faces of the studs are lined with two layers of gypsum-based board giving an overall thickness of around 250–300mm. Alternative, single frame stud wall solutions are available. These usually include the use of resilient bars or specially designed acoustic studs. The overall thickness of single stud wall solutions is around 150–220mm. Specialist manufacturers have produced a number of proprietary wall and detail solutions.

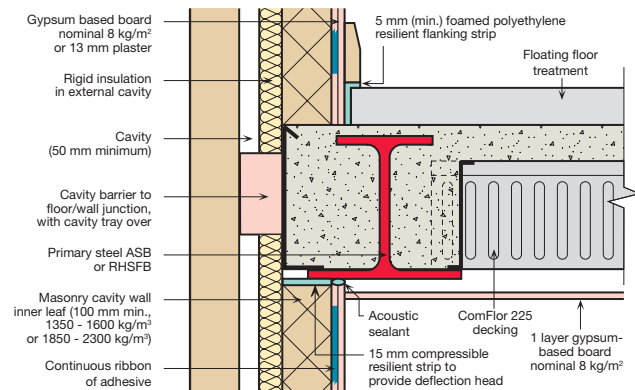
### 6.4.1 Detailing of joints

Junctions between walls and floors must be detailed appropriately to limit the amount of flanking sound that is transmitted. *Figures 6.8 to 6.11* show detailing that is required between Slimdek separating floors and walls. Specific points to note are:

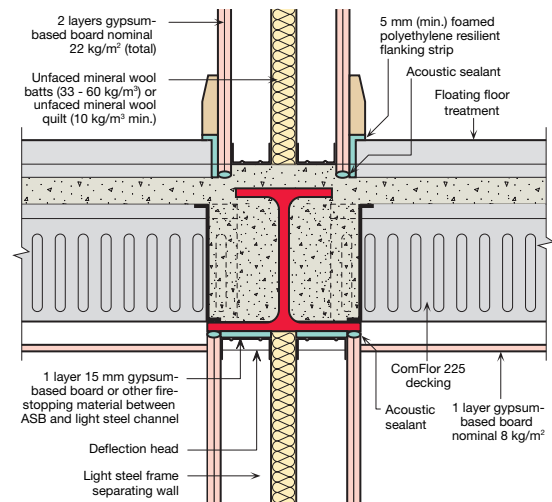
- Floor treatments should not be continuous underneath separating walls.
- Floor treatments should be isolated from the walls with a resilient flanking strip.
- In some situations it is necessary to install additional mineral wool insulation in the wall in the zone of the junction.
- Primary steelwork should not be in direct contact with wall or ceiling lining boards.
- Wall linings should be stopped about 5mm above the floor slab. The gap should be filled with acoustic sealant.
- A deflection head is provided at the top of the light steel separating wall to allow for relative movement between the Slimdek floor and the wall.



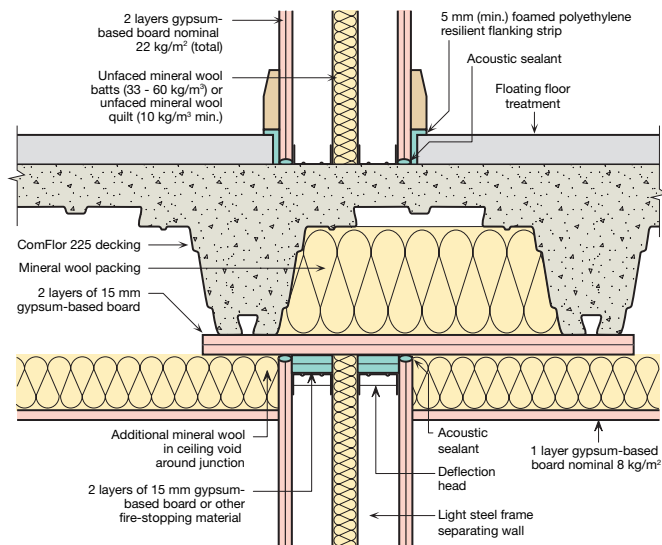
**Figure 6.8** Acoustic detailing at junction between floor and external wall with RHSFB



**Figure 6.9** Acoustic detailing at junction between floor and external wall with ASB edge



**Figure 6.10** Acoustic detailing at junction between floor and separating wall with ASB



**Figure 6.11** Acoustic detailing at junction between floor and separating wall without ASB



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# Construction

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## Central Park, Dublin.

Central Park, Dublin, comprises ten multi-storey buildings providing commercial office space with the largest building, Block E, having a plan area of 24,435m<sup>2</sup>. Originally the design was conceived as a reinforced concrete flat slab, but Slimdek's ability to compete on speed of construction resulted in the change to steel. The Slimdek flooring system was able to provide the same construction depth as a flat slab, but crucially saved 22 weeks from the construction programme.

A novel lifting system was adopted on the project, allowing 7.5m square deck panels to be prefabricated and lifted into place with maximum safety. This project won a European Steel Design Award.

The Slimdek system offered:

- speed of construction
- lighter weight and better service distribution
- prefabrication of deck panels at ground floor level, reducing the need for safety netting and access to the frame

Client:

Clyde Road Partnership

Architect:

Henry Lyons & Partners

Structural engineer:

T J O'Connor & Partners

Main contractor:

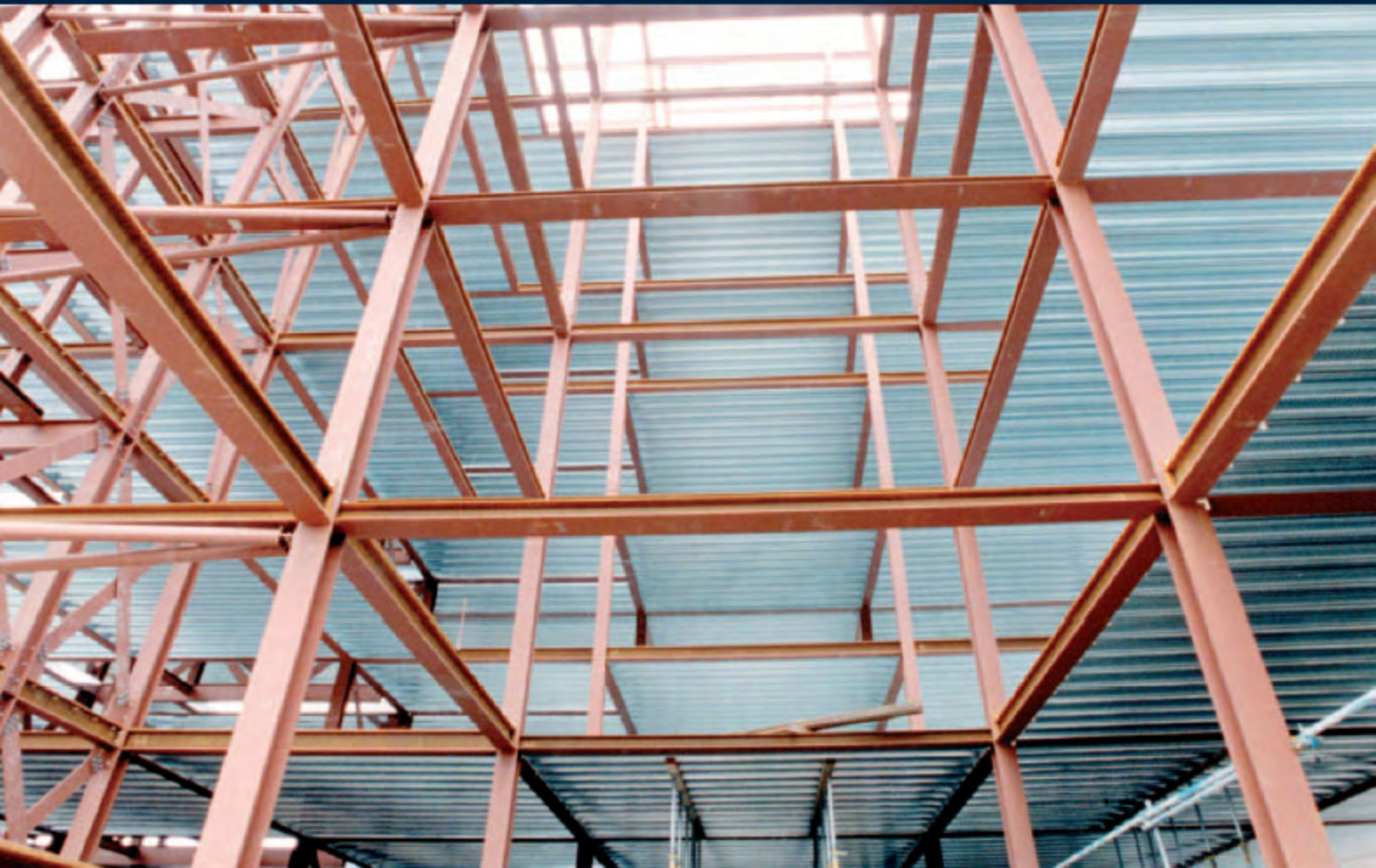
G & T Crampton Ltd

Steelwork contractor:

Fisher Engineering Ltd

Market sector:

Commercial



# 7. Construction

## 7.1 Forward planning

The CDM 2007 regulations<sup>[33]</sup> are of particular importance during the construction phase of a project. Coordination between the designer and contractor will ensure that the construction work proceeds safely and with maximum efficiency.

Important mistakes to be avoided in the planning and execution of Slimdek projects:

- Design details that are impossible to install. Please refer to *Section 2, Figures 2.33 and 2.34* of this guide for examples of such design details (both of which have been derived from actual projects). Correct details are shown both in this book and are available on [www.corusconstruction.com](http://www.corusconstruction.com)
- Installation of handrails directly above edge beams. This frequent occurrence makes it very difficult to install the deck and edge trims safely. The handrails should cantilever outside the edge beams as provided by 'Easy Edge' and other proprietary systems.
- Incorrect placement of deck bundles. The correct placement positions are shown on the deck installation contractor's drawings. However, the crane operator is employed by the main contractor and sometimes places the deck bundles in the wrong place. It is essential for safe installation that the deck bundles are not only placed in the correct bay but also in the correct place in the bay. This ensures that the decking operatives can access the bundle from the safest place, reducing the need to access the beam flange.

Avoiding these errors (which are all outside of the control of the deck installation contractor) is the first step towards safe and efficient installation. Safe working is also promoted by eliminating difficult details which need to be assembled at height and speed of construction may sometimes be achieved by preparing elements for construction at ground level and lifting them into place in one piece.

Pre-assembly of ComFlor 225 decking is possible when sufficient space is available on site to enable decking bays to be assembled at ground level prior to lifting into place and a suitable crane is regularly available for lifting. Pre-assembly removes the need to pre-fix the end diaphragms and enables most of the work to be carried out without working at height. However, if this process is to be considered then early detailed discussions must be held between the principal steelwork and decking contractors, as pre-assembly requires a radical change to the planning, programming and construction process.

In most cases pre-assembly will not be possible and other measures should therefore be taken to reduce, as far as is reasonably practicable, the amount of work to be carried out at height. Work should be planned and programmed to allow the end diaphragms to be installed from Mobile Elevated Work Platforms (MEWPs) or mobile scaffold platforms wherever reasonably practicable.

Work should be planned so that no other trades or stored materials are permitted onto the deck and no other works are carried out immediately above or below the working area until it has been completed and handed over.

Safety netting is the preferred method of fall protection for the installation of deep decks as it provides passive and collective protection for operatives working above. Where safety nets are used they act as primary fall protection and there will be no requirement for the decking contractor to use additional measures such as running lines. The safety net system should be installed directly onto the bottom flange of the steelwork to which the decking is fixed minimising the height of any potential fall.

Consideration must be given to the sequence of installation of safety netting as the safety net anchors ('Grippa' clamps or similar) will need to be installed in advance of the end diaphragms, as once the diaphragm is fitted the clamps cannot be installed. This method of work will require close cooperation and coordination between the deckers and the safety net riggers.

## 7.2 Steelwork erection

The erection of steelwork using ASB or RHSFB is similar in principle to other steel frames and should not cause any unusual difficulty. As with all steelwork erection, clear communication and allocation of responsibility is the key to speed and safety in erection. Designers and contractors should be aware of the guidance, recommendations and obligations contained in the following:

- The Construction (Design and Management) Regulations 2007 (CDM 2007)<sup>[33]</sup>
- British Constructional Steelwork Association (BCSA) publication: *Guide to the installation of deep decking*<sup>[48]</sup>
- British Constructional Steelwork Association (BCSA) Guidance Notes: *Safer Erection of Steel-Framed Buildings*<sup>[49]</sup>
- BS 5531: *Code of practice for safety in erecting structural frames*<sup>[15]</sup>
- Health & Safety Executive Approved Code of Practice (AcoP) in support of CDM 2007<sup>[50]</sup>

Designers are not legally required to keep records of the process through which they achieve a safe design, but it can be useful to record why certain key decisions are made. Brief records of the points considered, the conclusions reached and the basis for those conclusions can be very helpful when designs are passed from one designer to another. This will reduce the likelihood of important decisions being altered by those who may not fully understand the implications of doing so.

The designer should clearly state all design assumptions, including design loading considered and any allowances for temporary construction loads. It is particularly important that the designer identifies how the structural stability of the steel frame is to be achieved, and any assumptions made with regard to temporary stability during erection. It is helpful to the project if the designer identifies his recommended erection sequence.

The requirement for 'ledge angles' to be fitted in the appropriate places for side support is easier to address during the steelwork fabrication stage than on site. Some details also make it impossible to access with the low-velocity powder actuated fastener tools used to secure the edge of the sections. This is also a factor when hollow sections are positioned within the area to be concreted.

The steelwork contractor should prepare a detailed erection method statement. This should emphasise the sequence of erection and how stability is to be maintained throughout construction. The designer's assumptions and the steelwork contractor's erection method statement should be discussed with the principal contractor.

The designer and contractor should recognise that in a Slimdek system it is common for the beam to span in only one direction, with more flexible tie beams in the orthogonal direction. Therefore, there are limitations with regard to access onto tie beams. This will influence the erection method.

It is recommended that ASBs are hoisted into position using a 'Dawson Ratchet Release Beam Sling'. This lifting device avoids the need to access the sling after erection as the sling hooks are removed by remote operation. The usual range of access methods is available for the bolted end connections.

Every site is unique and poses individual challenges. Prior planning and individual attention is essential to overcoming these challenges and ensure the smooth running of any project.

### 7.3 Floor deck erection and fixing

All designers and installation contractors should be familiar with the *BCSA Guide to the Installation of Deep Decking*<sup>[48]</sup> available from The British Constructional Steelwork Association. The guidance in this manual is compatible with the BCSA publication and the joint MCRMA/SCI publication on *Composite Slabs and Beams using Steel Decking: Best Practice for Design and Construction* SCI-P-300<sup>[26]</sup>. As with the erection of the steel frame, clear communication and allocation of responsibilities is the key to the safe installation and subsequent use of the ComFlor 225 decking.

The designer should clearly state all relevant design assumptions including the span, the direction of span, the overall slab thickness, the design loading, the fire rating required and any constraints on deck erection which are perceived.

The decking contractor should normally prepare detailed decking drawings and an erection method statement which should identify methods of and restrictions to adjacent working. These should be discussed and agreed with the principal contractor. The structural engineer should also identify any temporary propping requirements, including prop locations and types, which should be shown on the decking drawings.

Decking installation will only commence once the end diaphragms and safety net fall arrest system are in place. As it is often not possible to straddle the steelwork once the end diaphragms are in place, decking operatives will usually stand on the top flange of the beam at either end of the first bundle of decking to cut open the steel banding and lift the first decking sheet out onto the steelwork and over the pre-fitted diaphragms. Decking sheets should always be positioned by a minimum of two operatives where sheets are longer than 6m, unless mechanical lifting systems are an option. Where practical, decking should be positioned by two operatives at each end of the sheet using an extended handlebar lifting device. Decking installation should always cease a minimum of two metres back from any unprotected edge, such as the end of a safety netted work zone.

#### 7.3.1 Delivery

The decking is delivered in bundles, sized to suit the project and placed by crane at the positions shown on the deck layout drawings. Each bundle normally contains sufficient deck pieces to cover one bay of the floor. The bottom sheet of each bundle is 200mm longer than those above to give sufficient bearing onto the top flange of the beams.

#### 7.3.2 Deck cut-outs

The decking elements are delivered to site cut to length with approximately 50mm x 240mm wide cut-outs in the crest at each end. These cut-outs are required in order to facilitate concreting around the steel section, see *Figure 7.4*. If sheets are cut to length on site, these cut-outs should be made on site.

### 7.3.3 Fasteners

Recommended fasteners for installation of the ComFlor 225 decking are given in *Table 7.1*.

### 7.3.4 Diaphragms

The diaphragms are fixed first, as shown in *Figure 7.1*. Diaphragms are manufactured from galvanised steel in the following sizes:

- without openings 1.6mm thick x 1800mm long
- with openings 2.0mm thick x 1800mm long

The diaphragms are fixed to the edges of the lower flanges of the beams on both sides (except for edge beam situations) using two fixings at pre-marked positions for each length. The 1800 length equates to three sections of ComFlor 225 decking. Each length should be positioned and abutted accurately so that the 600mm pitch of decking sections is located as shown on the layout drawings.

### 7.3.5 Very short lengths of deck

Where the beam spacing demands a length of deck of 1m or less, especially where the beams are not parallel, it is far more efficient to use a shallow composite floor deck. This eliminates the need for close spaced diaphragms and complicated cutting of very short pieces of ComFlor 225.

### 7.3.6 Decking to steelwork

Steelwork must be adequately restrained with support for the decking around columns and openings.

The individual deck sheets are then manually lowered onto the end diaphragms that provide both location and alignment. The nominal end bearing of the decking is 50mm, although this may reduce to an absolute minimum of 35mm on site, allowing for all constructional tolerances. When the sheets for the whole bay are in place, they are fixed through the trough of the profile to the beam lower flanges using steel/self-tapping screws or heavy duty shot-fired pins. Two main fixings per trough (one each side of the central dovetail) are required. The decking is then fixed to the top of the diaphragms using two self-drilling self-tapping screws. Side laps should be fixed at 500mm centres through the top flat of the dovetail using self-drilling fasteners.

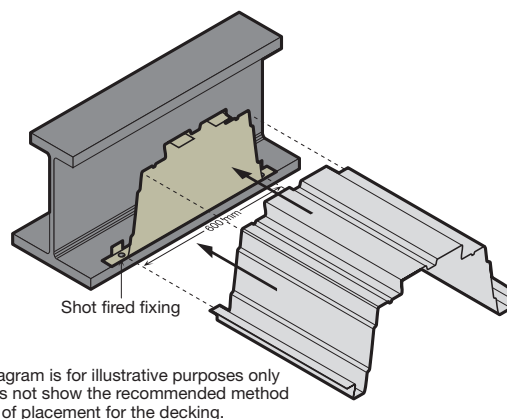
### 7.3.7 Edge and inside trim

A light steel edge trim is used for retention of the wet concrete and to form the vertical edges of the slab. Each length should be fixed to the perimeter beams with edge restraint straps fixed at 500mm centres. The decking installers should take care to ensure that the restraint straps are tight preventing the trim from bowing out during concrete placement. Edge trim positions are generally set out from the centre line of the supporting steelwork and should be fixed with a tolerance of  $\pm 10$ mm in accordance with the National Structural Steelwork Specification for Building Construction<sup>[51]</sup>. Edge trim should be set to the correct level taking account of beam and/or deck deflections at the deck floor perimeters. Internally

Z trim is used to close the decking where the 600mm profile of the deck does not align with the parallel supports and has to be cut longitudinally. Trims with butted joints should be propped where they are not adequately supported.

Generally, edge and Z trims are custom manufactured for each project from 2.0mm galvanised steel (up to 450mm deep) in 3m lengths. They should be fixed to the deck and/or support beams at approximate 500mm centres using fixings described in *Table 7.1*. The location of each type of fixing should be shown on the layout drawing.

Straps restraining deep edge trim are manufactured from 50mm x 1.25mm thick galvanised steel.



**Figure 7.1** Fixing of end diaphragms at ASB

### 7.3.8 Sealing joints

Normally, providing the decking installation has been carried out with reasonable accuracy, joints between decking and closures should be fairly tight fitting and do not require sealing. Small gaps tend to close and seal when the weight of concrete is applied, but some grout loss should be expected. In situations where larger gaps occur or where grout loss has to be minimised for visual reasons, a foam sealant can be used. It is easier to carry out this operation prior to fixing the reinforcement.

Seam stitching between decking panels is required to hold the lips of the decking together during concreting operations. Seam stitching should be carried out as soon as practical following the installation of the decking.

### 7.3.9 Working platform

The fixed deck is a safe and secure working platform for both the deck fixers and the following trades. The deck is capable of supporting foot traffic and loads from small tools carried onto the floor. Larger pieces of equipment, steelwork, bundles of mesh, etc. should be supported directly by the steel frame.

The main contractor completes the floor slab by fixing the reinforcement, provides any temporary propping and finally places the concrete.

Table 7.1 Schedule of decking fasteners

Location	No. of fixings	Type of Fastener		
		SFS Stadler	Hilti	ITW Construction Products
Diaphragm to Corus ASB or RHSFB flange plate	2	<b>Up to 9 mm flange thickness:</b> SFS self-drilling self-tapping SD12-5.5 x 36 mm screws.	Heavy duty shot-fired pins ref. Hilti ENP2 X-ENP-19 L 15	<b>Up to 9 mm flange thickness:</b> Heavy duty shot-fired pin ref. Spit SBR14FD or; self-drilling, self-tapping Buildex Teks Screw TL32.
ComFlor 225 decking rib to Corus ASB or RHSFB flange plate	2 each end			
Trims, closures and straps to Corus ASB, RHSFB or RHSFB flange plate	500mm centres	<b>Up to 40 mm flange thickness:</b> SFS self-tapping TDC-T-6.3 x 25 mm fixed through a pre-drilled 5.9 mm diameter pilot hole.		<b>Up to 40 mm flange thickness:</b> Heavy duty shot-fired pin ref. Spit SBR14FD** or; self-drilling self-tapping Buildex Teks Screw T25B with pre-drilled 5.8 mm diameter pilot hole to both part and substrate.
Diaphragms, decking, trims, closures and straps to other carbon steel support beams.				
ComFlor 225 decking crest to diaphragm	2 each end	SFS self-drilling self-tapping SD3-5.5 x 25 mm screws	N/A	Self-drilling self-tapping Buildex Teks Screw TC25
ComFlor 225 side laps (through top flat of dovetail)	500mm centres			
Trims, closures and straps to ComFlor 225 decking and to each other	500mm centres			

## Notes:

- Other fasteners of equivalent performance may be used.
- \*\* Site test for suitability recommended - contact manufacturer's technical support.

## 7.4 Reinforcement

The deck forms a part of the slab reinforcement, with the remainder being supplied by a bar in each trough of the decking and mesh placed near to the top of the slab. The deck reinforcement is fixed in accordance with the requirements of the deck designer. For low shear and up to 60 minutes fire resistance, spacers at an axis distance of 70mm from the base of the trough support individual straight bars. For high shear (spans greater than approx 6m) additional end anchorage in the form of L-bar or U-bar is required. The axis distance is increased to 90 and 120mm respectively for 90 and 120 minutes fire resistance. See *Table 2.3 for reinforcement requirements*.

Table 7.2 Minimum laps for mesh

Mesh type	Lap (mm)
A142	300
A193	400
A252	400

The top mesh is typically A142 to A252 mesh, with minimum cover as necessary for durability considerations – minimum laps are given in *Table 7.2*. At mesh overlaps, the support stools should be positioned more frequently.

There may be requirements for additional mesh or bar reinforcement adjacent to supports or edge beams for crack control or other purposes.

## 7.5 Propping

Where temporary supports are specified by the structural engineer it is important that:

- The props are of adequate strength and construction to carry the slab.
- Props will either be at half-span or one-third span position (or other specified by engineer).
- Bearers should be a minimum of 100mm wide.
- Propping should not be removed until the slab has reached at least 75% of its characteristic strength.

It is necessary to prop the ComFlor 225 decking where spans exceed the limit of the bending resistance of the decking in the construction stage (typically beam-beam centres of 6m).

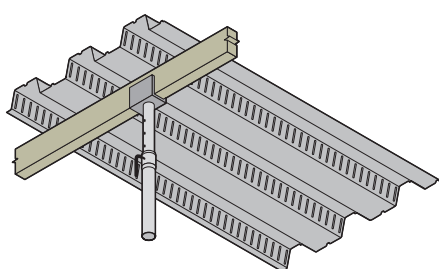
The decking should be propped by a continuous line of 'Acrow' or similar props with a timber spreader below the decking extending the full width of the bay and supporting each decking rib and Z trims if present, as illustrated in *Figure 7.2*. These requirements are presented in *Table 7.3* as a function of the slab depth (and hence weight). The width of the spreader should be at least 100mm to avoid local deformation of the decking. Spreaders should be of sufficient strength to span between props and support each rib.

For spans less than 7.5m, props may be positioned after the decking is placed. Props should be firmly wedged to level prior to concreting.

Props should be stable without relying on friction with the deck for lateral stability. The end props in a row should be self-supporting and braced to the internal props.

For spans greater than 7.5m, props should be in place prior to the decking being placed and should be adequately laterally restrained. Mobile towers may be used and the supporting floor must be checked for the applied concentrated construction loads.

For slabs subject to low construction loads, the props should not be removed until the concrete has achieved 75% of its specified strength (normally after seven days). The slab design should also be checked for cases of high construction loading (e.g., where pallets of bricks or plasterboard are stored on the slab). In this case and for cases where crack control is required, props should not be removed until the concrete has achieved its specified cube strength.



**Figure 7.2** Propping of slab during construction

It is not normally necessary to prop to more than one floor below the level concerned, because the imposed load resistance of the supporting slab should exceed the loading transferred through the props. However, the designer should ensure that the supporting slab has reached adequate strength at this stage. In cold weather, curing times may be extended.

### 7.5.1 Propping of partially encased ties

Where decking is unpropped, encased ties should normally be designed to support a 300mm width of slab in addition to the construction loads. Where cutting of the decking results in slab thickening over a width greater than 300mm, advice should be sought from the designer, or alternatively the tie member should be adequately propped.

Where decking is propped during construction, partially encased ties should normally be adequately propped.

### 7.5.2 Propping of beams

The structural engineer should specify requirements for the propping of beams. Propping will normally only be required for:

- long span ASBs (> 8m span)
- composite RHSFBs

Where propping is specified, the props used to support beams should be much more robust than those used to support decking. Generally, a 'Tri-shore' or braced propping system is required, which should have a load capacity of over 10 tonnes. These props should be placed at a maximum of 3m spacing along the beams. In this case, it will often be necessary to prop to two levels below the supported beam to avoid creep-induced deflections of the supporting beam.

## 7.6 Cladding attachments

Attachment to various forms of cladding may be considered in Slimdek construction:

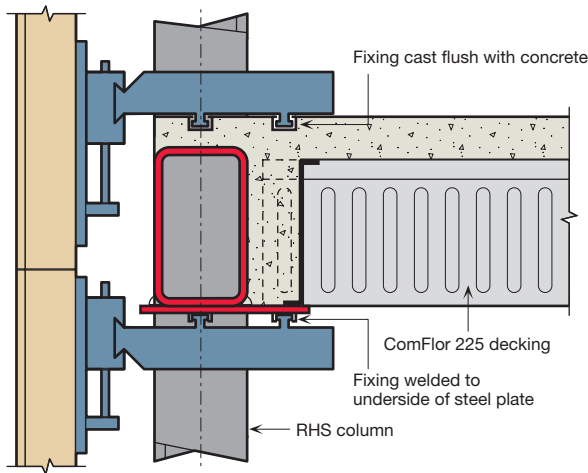
- Masonry
- Concrete panels
- Lightweight cladding
- Glazing

Special details have been developed for masonry cladding. Brackets can be welded to the side of RHSFBs or embedded in the concrete encasement to ASB sections.

For lightweight cladding and curtain walling, special adjustable brackets have been developed which can be attached to channels cast into the slab and attached to the bottom plate. A typical detail is shown in *Figure 7.3*.

**Table 7.3** Recommended maximum spacing of temporary props to slabs (in metres)

Concrete type	Slab depth			
	Up to 300	320	340	360
NWC	3.3	3.0	2.6	2.2
LWC	3.6	3.3	2.9	2.5



**Figure 7.3** Attachment for lightweight cladding to RHSFB

## 7.7 Concreting operation

### 7.7.1 Construction loads

The deck is designed in accordance with the requirements of BS EN 1994-1-1<sup>[11]</sup> for the dead load of the concrete and a construction stage imposed load. This imposed construction load is  $1.5\text{kN/m}^2$  over the centre 3m of the deck span and  $0.75\text{kN/m}^2$  for the remainder. This load is reduced to an average of  $0.5\text{kN/m}^2$  when considered over the supported area for design of the beams. These represent the load imposed by the operatives during placing of the concrete and allow for slight heaping of the concrete. If loads greater than these are expected, they must be taken into account in the design of the deck and supporting beams.

Similarly, high loads applied to the slab after construction should be checked carefully with respect to the design loads in normal conditions, as should punching shear due to point loads which may occur during construction.

### 7.7.2 Inspection

The decking should be checked prior to concrete placement to ensure it has been correctly fixed. Any damaged decking must be replaced. If the deck span requires propping, this should be in place and adequate. The deck and beam surfaces should be clean and free from dirt and grease that would affect the performance of the slab. Oil left from the roll-forming process will not affect the bond of the concrete to the galvanised steel or the performance of the slab.

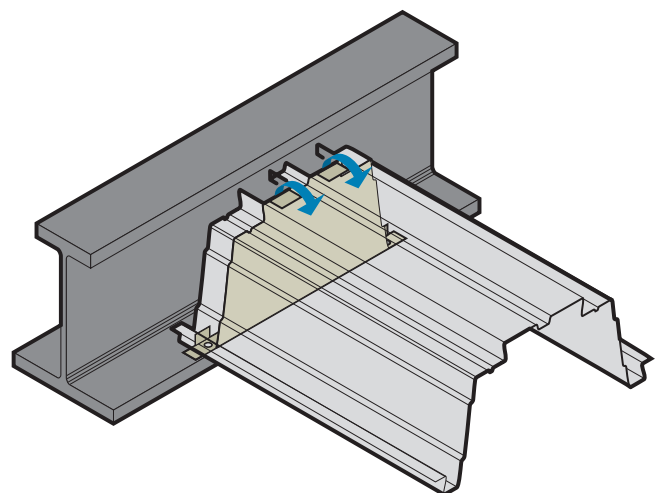
### 7.7.3 Placement

The normal practice for concreting a floor slab is either by pumping or by placement from a skip. Limitations on pour sizes need not be made because the decking acts as reinforcement and helps to distribute shrinkage stresses and early age thermal strains. C30 concrete is normally specified for normal and lightweight concrete in mild exposure conditions, see Table 2.1.

Lightweight aggregate is less than 12mm and should be used where the cover to the beam is less than 35mm, see Table 2.1. For other cases, either a 12mm lightweight aggregate or a 20mm normal weight aggregate may be used. Both lightweight and normal weight concrete are suitable for pumping.

Concrete placement by pump is the most popular method of construction. Flow rates of  $0.5\text{m}^3$  to  $1\text{m}^3$  per minute can be achieved. Long pump lines and greater pumping distances will slow the rates below these levels. Pumps are normally used for 'lifts' of up to 30m. Secondary pumps on intermediate levels may be necessary for higher lifts. The pump manufacturer or supplier should be consulted for further advice.

The pipes used to carry the wet concrete are normally approximately 150mm diameter and are connected in segments. The pipes should be supported on timber blocks at intervals of 2m to 3m. The force exerted at bends may be significant, therefore, straight-line pumping is preferred. The outlet pipe should be moved frequently and carefully so that heaping of the concrete is minimised and excessive deflection prevented. Two operatives are often needed to shovel away excess concrete. Despite the fluidity of the mix, good vibration is important, particularly around the beam to ensure adequate compaction. The deck cut-outs and clearances to the top flange give adequate space for placement and observation of concrete filling around the steel section, as shown in Figure 7.4.



**Figure 7.4** ComFlor 225 decking showing cut-outs for placement of concrete

Using a skip from a crane to place concrete may be difficult because of obstruction from decking on levels above. However, it is sometimes necessary to use a skip and barrow method for small infill bays. Skips should have a controlled outlet and should not be discharged at more than 0.5m above the deck or barrow. Barrows should be placed on a 20mm thick plywood board placed on top of the ComFlor 225 decking or finished slab, in order to limit point and impact loads. Barrows should be run over scaffolding boards placed on the mesh, which should be supported locally to prevent it being depressed or displaced.

The concrete should be placed to a level dictated by deflection of the support beams. Therefore, the slab thickness at the centre of the deck span will be slightly greater than at the beams, which should be taken into account in design. If a level surface is required, beam deflection should be taken into account in setting the level of tamping rails positioned above the beams.

#### 7.7.4 Curing

Concrete gains strength relatively quickly and is able to resist damage due to low temperature, if kept above 5°C for at least three days after placement. Similarly, in warm or windy weather, it may be necessary to prevent excessive moisture loss by the use of wet hessian on the slab surface or similar procedure.

#### 7.7.5 Screed or raised floor

There is no structural requirement for a screed, although a levelling screed may be specified. Furthermore, the achievable quality of surface to the slab should be considered before specifying the floor finish. Power floating and power trowelling of the slab may be used to allow direct application of surface coverings but allowance for movement or deflection of the structural members should be made. If a raised floor is specified, a wood-floated surface finish is generally adequate. All standard raised floor systems are suitable for use with Slimdek.

### 7.8 Ceiling finishes

Slimdek accommodates a standard ceiling panel based on the 600mm grid. The ceiling manufacturer's guidance on support systems should be sought to ensure suitable construction details.

Exposed floor slabs can be used for advanced energy systems using fabric energy storage, see *Section 5.6*. A range of perforated ceilings is available that allows for air movement over the deck soffit and for heat exchange.

### 7.9 Refurbishment

The Slimdek system allows for extensive alteration to internal layout and servicing after construction. The clear space between the ribs and below the slab allows new service or partition layout. Small service ducts may be cut in the crest of the ComFlor 225 deck using appropriate concrete-coring equipment. Should large service duct openings be required in a refurbishment scheme, it is recommended that new steel trimming members be provided.

If service ducts are to be cut through the slab or beam after concreting, care should be taken to avoid loss of the shear bond between the steel and concrete by using a non-percussion drill.

Where flying shores retained facades require penetrations through the decking, special temporary/permanent trimming may be required to support the decking. The designer should make clear notes on the drawings so that the contractor and temporary works designers can carry out a design risk assessment.

### 7.10 Removal of waste

A skip should be available at or adjacent to the working level for the disposal of waste materials. Care should be taken to ensure that a skip is located securely over the structural steelwork and not loaded directly onto the decking as all ComFlor 225 is single span. If a skip is not available, the decking contractor will be required to gather the scrap together in one neat pile at each level for disposal by others once suitable means of removal are available. The skips should be arranged to arrive as soon as possible following the start of the decking work.

On single storey structures, a controlled drop may be permissible, subject to assessment of the risks and the inclusion of a suitable procedure in the method statement. It should be noted that off-cuts of decking and edge trims are extremely sharp and can be hazardous to move long distances. The distance to the skip should therefore be kept to a minimum. Scrap should never be carried down ladders or over long distances across the site. Protective gloves must be worn at all times whilst handling decking or edge trim.



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# Supplier information

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## The Gateway, Dublin.

The Gateway development in Dublin comprises three office blocks and two apartment blocks on a 3.75 acre site with a large single storey basement beneath, which has space for 300 cars.

The design brief was to create a floor to ceiling height of 2.625m and an overall services depth of 765mm whilst keeping storey heights to the minimum. With a structural depth of 350mm, adoption of Slimdek kept the storey height to 3.74m. Based on a grid sized to suit the car park layout in the basement, Slimdek spanned 7.75m to provide large open spaces with a flat soffit. The minimal depth of the structural slab allowed an uninterrupted services zone to be created.

The Slimdek system offered:

- speed of construction
- a flat clear soffit minimising storey heights
- an uninterrupted services zone, making service installation easier
- maximum use of offsite manufacture

Client:	Collen Group
Architect:	CPM Architecture
Structural engineer:	CPM Engineering
Main contractor:	Collen Construction Limited
Steelwork contractor:	Duggan Steel
Market sector:	Mixed development



## 8. Supplier information

### Design and budget costing

Corus provides free technical and commercial advice to help specifiers acquire best practice in design and construction using steel. Experts with in-depth knowledge and experience in civil, structural and building design are available to help speed the design process, facilitate innovation and encourage an integrated approach. The Corus Construction Services & Development department is directly linked to Corus offices around the world to give rapid sales and technical support at a local level.

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Web [www.corusconstruction.com](http://www.corusconstruction.com)

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Email [library@steel-sci.com](mailto:library@steel-sci.com)  
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Web [www.corusconstruction.com](http://www.corusconstruction.com)

### ComFlor 225 Deep Decking

Corus Panels and Profiles  
Severn Drive  
Tewkesbury Business Park  
Tewkesbury  
Gloucestershire  
GL20 8TX United Kingdom  
Telephone +44 (0) 1684 856600  
Facsimile +44 (0) 1684 856601  
Email [technical@coruspanelsandprofiles.co.uk](mailto:technical@coruspanelsandprofiles.co.uk)  
Web [www.coruspanelsandprofiles.co.uk](http://www.coruspanelsandprofiles.co.uk)

### Tegral

Tegral Metal Forming Ltd  
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Facsimile +353 (0) 507 38156  
Email [metalinfo@tegral.ie](mailto:metalinfo@tegral.ie)  
Web [www.tegral.ie](http://www.tegral.ie)

**Other suppliers and services Steelwork Fabricators**

For listing contact the British Constructional Steelwork Association at [www.steelconstruction.org](http://www.steelconstruction.org)

**Accredited ComFlor 225 installers**

For listing contact Corus Panels and Profiles.

**Perforated ceilings**

SAS International  
Unit 31 Suttons Business Park  
London Road  
Reading  
Berkshire  
RG6 1AZ United Kingdom  
Telephone +44 (0) 118 949 1092  
Facsimile +44 (0) 118 935 1219  
Email [enquiries@sasintco.uk](mailto:enquiries@sasintco.uk)  
Web [www.sasint.co.uk](http://www.sasint.co.uk)

**Service and ceiling fixings**

Lindapter International  
Lindsay House  
Brackenbeck Road  
Bradford  
West Yorks  
BD7 2NF United Kingdom  
Telephone +44 (0) 1274 521444  
Facsimile +44 (0) 1274 521130  
Email [enquiries@lindapter.com](mailto:enquiries@lindapter.com)

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Idsall House  
High Street  
Prestbury  
Cheltenham  
GL52 3AX United Kingdom  
Telephone +44 (0) 1242 585400  
Facsimile +44 (0) 1242 520682  
Email [sani@sfsintec.biz](mailto:sani@sfsintec.biz) (Technical support)  
[uk.leeds@sfsintec.biz](mailto:uk.leeds@sfsintec.biz)  
Web [www.sfs-online.com/construction](http://www.sfs-online.com/construction)

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Email [GBSales@hilti.com](mailto:GBSales@hilti.com)  
Web [www.hilti.com](http://www.hilti.com)

ITW Construction Products  
Crompton Way  
Crawley  
West Sussex  
RH10 2QR United Kingdom  
Telephone +44 (0) 1293 523372  
Facsimile +44 (0) 1293 515186  
Email [sales@itwspit.co.uk](mailto:sales@itwspit.co.uk)  
Web [www.itwspit.co.uk](http://www.itwspit.co.uk)

**Technical support for Spit products**

Telephone +44 (0) 141 764 2700  
Facsimile +44 (0) 141 774 5802  
Email [support@itwspit.co.uk](mailto:support@itwspit.co.uk)

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Facsimile +44 (0) 1189 926 8568  
Email [customerservice@itwbuildex.co.uk](mailto:customerservice@itwbuildex.co.uk)

**Reinforcement spacers and stools**

Hy-Ten Reinforcement Company  
12 The Green  
Richmond  
Surrey  
TW9 1PX United Kingdom  
Telephone +44 (0) 20 8940 7578  
Facsimile +44 (0) 20 8332 1757  
Email [sales@hy-ten.co.uk](mailto:sales@hy-ten.co.uk)

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# References

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## House of Fraser, Guildford.

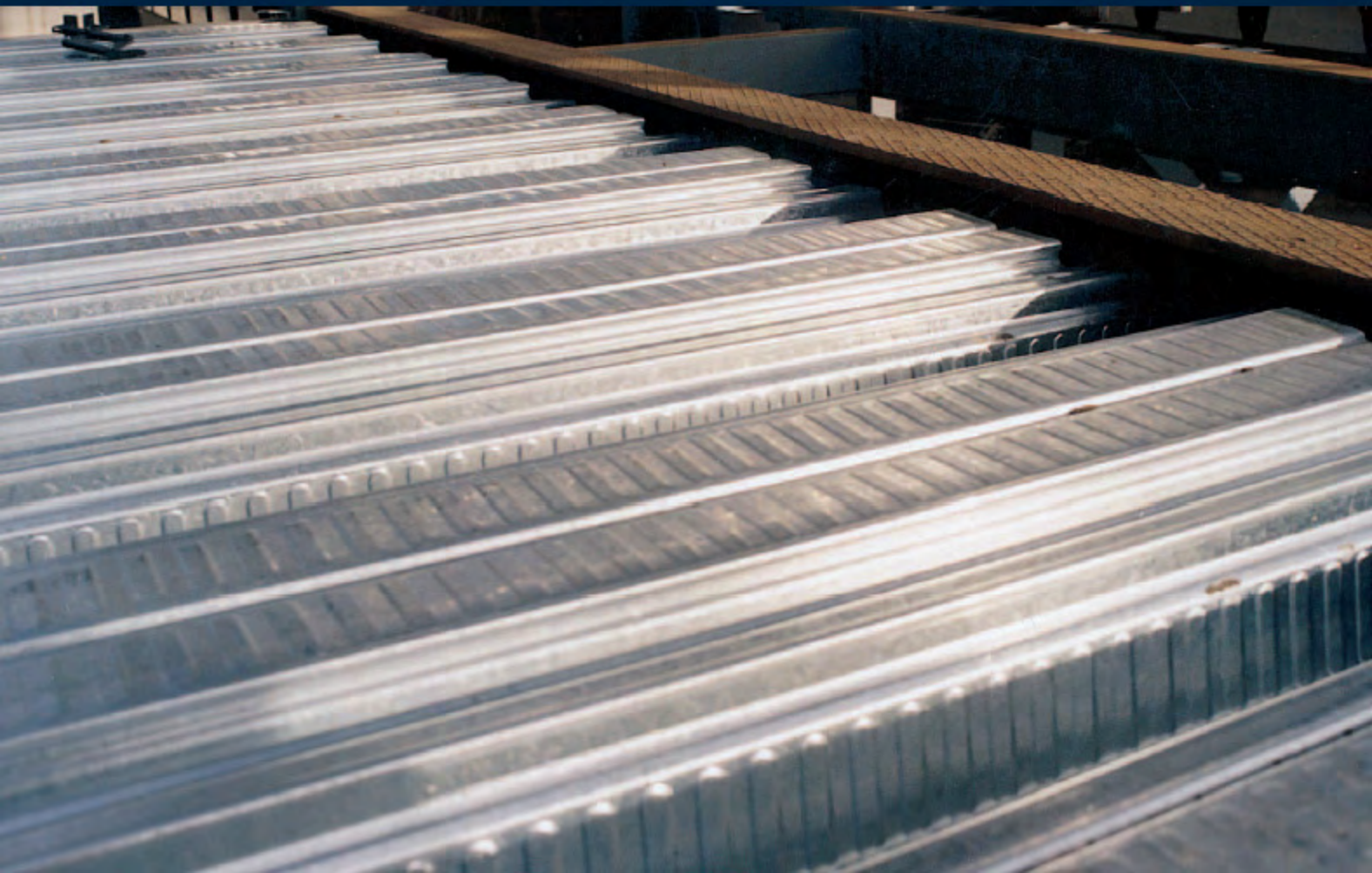
Slimdek was the first choice for renovation of the House of Fraser's retail store in Guildford. The building was extended by 2-storeys whilst satisfying strict planning requirements on building height, and the opportunity was created to remodel the store, providing a 5-storey atrium topped with a curved tubular steel roof.

The Slimdek flooring system was able to match the shallow depth of the existing flat slab structure and its speed of construction meant the project could be completed within the tight timescale required. The whole building was completely reserviced by a central air delivery system from roof-mounted modular units, with ducts to each floor.

The Slimdek system offered:

- speed of installation within a tight delivery timescale
- a lightweight structure, minimising piling requirements in chalky ground conditions
- shallow construction - essential to suit a floor to floor height of 3.46m
- no temporary propping, which was essential to speed of construction
- opportunities for services integration
- inherent fire protection

Client:	Westpoint Homes Ltd
Architect:	Maxwell Design Consultants
Structural engineer:	Walton Garden & Partners
Main contractor:	Beechwood Developments Ltd
Steelwork contractor:	Bone Steel Ltd
Market sector:	Residential



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## Appendices

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# Appendix A: Summary of tests on the components of Slimdek construction

The following tables summarise the testing carried out in the development of Slimdek. The background test reports are also identified.

## ASB tests

Beam size	Beam span (m)	Test configuration	Failure load* (kN/m <sup>2</sup> )	Test report
280 ASB prototype	7.5	Single beam	18	City University SCI RT 554
280 ASB prototype	7.5	Single beam	23	City University SCI RT 592
280 ASB 100 with openings	7.5	Single beam – transverse bending	16	City University SCI RT 645
280 ASB 100 with openings	6	2 x 6m square floor bays	30	City University SCI RT 740
280 ASB 100	5.4	Single beam – transverse bending	30	University of Kaiserslautern (G)

\*Failure load equivalent to a 6m bay width

## ComFlor 225 Composite slab tests

Slab depth/concrete type	Slab span (m)	Failure load (kN/m <sup>2</sup> )	Test report
285mm LWC	3	> 90	University of Salford
350mm LWC	8	15	University of Salford
300mm NWC	4	66	University of Kaiserslautern (G)
300mm NWC	6	34	University of Kaiserslautern (G)

All reviewed in SCI-RT-703

## Fire tests

Beam size	Beam span (m)	Slab span (m)	Fire resistance	Test report
280 ASB 100	4.5	-	107	Warrington Fire Research
300 ASB 153	4.5	-	75	Warrington Fire Research
280 ASB prototype with openings	Unloaded	NA	Protected with intumescent to exposed bottom flange only	Warrington Fire Research*
Slimflor Beam	4.3	3.8	90 (beam) 120 (slab)	TNO (NL)
RHSFB	4.3	6	90 (beam) 120 (slab)	TNO (NL)
ASB and RHSFB	6	6	> 90	BRE Natural Fire Test*

\*All reviewed in SCI-P-175, except BRE and Warrington tests

## ComFlor 225 deck tests

Deck span (m)	Test configuration	Failure load (kNm)	Test report
5.5	Unpropped span	8.3	Welsh Technology Centre RT703
6.0	Unpropped span	6.9	Welsh Technology Centre RT703
8.0	Single prop	11	Welsh Technology Centre RT703
-	End diaphragm test	38 kN/m	Welsh Technology Centre RT703
-	Local web crushing test	36 kN/m	Welsh Technology Centre RT703

## Appendix B: Worked example

The following example has been created to illustrate how a particular ASB can be shown to be suitable for a given loading situation. The calculations have been taken to a basic level to show how the various structural properties and analysis results are achieved. The layout of the calculations follows the format of the output from the SIDS (Slim floor Integrated Design System) software package which can be downloaded from [www.corusconstruction.com](http://www.corusconstruction.com).

In the main, the hand calculations match the computer-generated results quite closely but there are occasions where the hand-generated numbers and those from the computer vary albeit by a small amount. This is due to a combination of number rounding and the effect that has on subsequent computations, simplifying assumptions and the use of algorithms – which are built into the software to generate some values – rather than graphs or look-up tables when performing hand calculations. It should be noted that when the values differ, it is generally by a small percentage that has little or no effect on the conclusion.

This design example is for an internal beam forming part of the support structure for a floor on a 7m x 6m grid.

The design check is to be carried out for a 280 ASB 100 section with 30mm of cover above the top flange of the beam giving a deck thickness of 290mm. The concrete cover is sufficient to permit the beam to be designed for composite action.

The beam is analysed for both the construction and normal stages. Additional checks are made to determine the performance of the section in fire and to ensure that the serviceability limit state stresses are not excessive.

As can be seen the chosen beam section is shown to be adequate for the anticipated loading conditions as the unity factors are all less than 1.0.

## FULL OUTPUT

Beam: 280 ASB 100

Construction Stage:

Normal Stage:

Fire design:

Serviceability Limit State:

Deflections:

PASS

PASS

PASS

SATISFACTORY

SATISFACTORY

\*\*\* SECTION ADEQUATE \*\*\*

Max Unity Factor = 0.68

Max Unity Factor = 0.87

Max Unity Factor = 0.94

Max Unity Factor = 0.75

## INPUT DATA

### MAIN DATA:

Construction with SD 225

Beam will NOT be propped in construction

Decking PERPENDICULAR to beam (SIDE 1)

Design type COMPOSITE

Decking will be NOT propped in construction

Decking PERPENDICULAR to beam (SIDE 2)

### FLOOR PLAN DATA: ( Unpropped Internal Beam )

Beam span 7.00m

Beam spacing (SIDE 1)

6.00m

Beam spacing (SIDE 2) 6.00m

\*\*\* No secondary beams provided within the span of the beam

### PROFILE DATA: ( SD225 )

Depth 225mm

Pitch of deck ribs

600mm

Trough width 100mm

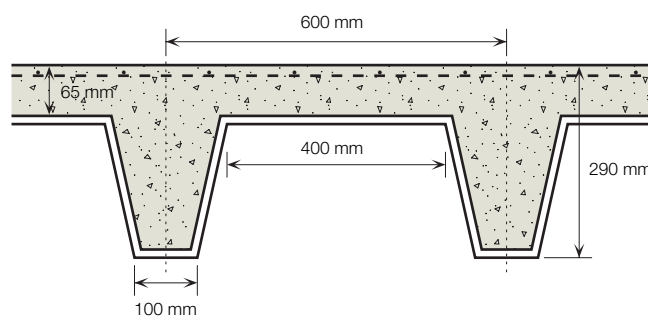
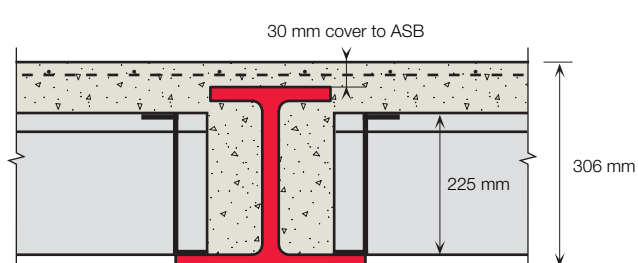
Crest width

400mm

Deck weight 0.20 kN/m<sup>2</sup>

\*\*\*NOTE : 1. SD225 decking is available in 1.1mm and 1.25mm sheet thickness ONLY

\*\*\*NOTE : 2. A deck bearing width of 50mm (on bottom flange) is assumed



### CONCRETE SLAB: (290 mm overall slab depth in Light Weight Concrete - LWC)

Characteristic strength 30 N/mm<sup>2</sup>

Concrete depth above beam flange

30mm

Wet density 1900 kg/m<sup>3</sup>

Dry density

1800 kg/m<sup>3</sup>

Modular ratio 15

Mesh reinforcement

A142

Overall slab depth 290 mm

Yield strength of mesh reinf't

460.0 N/mm<sup>2</sup>

Non-structural screeds 0mm

### LOADS ACTING ON BEAM:

Occupancy imposed loads 3.5 kN/m<sup>2</sup>

Partition loads

1.0 kN/m<sup>2</sup>

Ceilings, services and finishes 0.5 kN/m

Construction load

0.5 kN/m<sup>2</sup>

Natural frequency limit 4.0 Hz

Screed self-weight

0.0 kN/m<sup>2</sup>

BS 6399 imposed load reduction is NOT considered

\*\*\* NOTE: Beam subjected to uniformly distributed loads (UDL) ONLY

### ADDITIONAL POINT LOADS:

None

**ADDITIONAL UNIFORMLY DISTRIBUTED LOADS:**

None

**FIRE DATA:**

Fire resistance period	60 mins	Non-permanent imposed loads	3.5kN/m <sup>2</sup>
Fire protection is NOT provided		Fire partial safety factor	0.80
Occupancy loads considered non-permanent	100%	Permanent imposed loads	0.0 kN/m <sup>2</sup>

**PARTIAL SAFETY FACTORS:**

Dead (self-weight)	1.4	Imposed	1.6
Super imposed dead	1.4		

**SHEAR CONNECTORS DATA (Composite Slimflor only)**

Diameter	19mm	Height	70.0 mm
Cost (per stud)	£ 0.0		

**BEAM DATA: (280 ASB 100. Steel Grade Advance 355)**

Depth	276mm	Mass	100.3 kg/m
Top flange width	184mm	Bottom flange width	294mm
Top flange thickness	16mm	Bottom flange thickness	16mm
Web thickness	19mm	Root radius	24mm
Steel grade	Advance 355	Design strength	345.0 N/mm <sup>2</sup>
Steel cost (per tonne)	£ 0	Percentage allowed for fittings, etc	0.0 %

**STEEL SECTION PROPERTIES:**

Section classification is PLASTIC

Elastic neutral axis is in WEB (155.9mm from beam top flange)

Plastic neutral axis is in WEB (184.3mm from beam top flange)

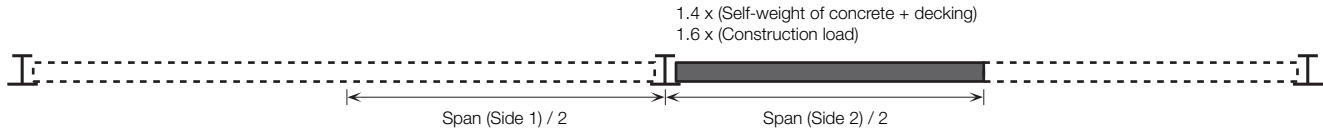
2nd moment of area	Ixx	= 15506cm <sup>4</sup>	Iyy	= 4245cm <sup>4</sup>
Elastic Modulus (top)	Zt	= 995cm <sup>3</sup>	(bottom)	Zb = 1291cm <sup>3</sup>
Radius of gyration	rx	= 11cm	ry	= 5.8cm
Plastic Modulus	Sx	= 1294cm <sup>3</sup>	Elastic modulus (minor axis)	Zy = 289cm <sup>3</sup>
Torsional constant	J	= 160cm <sup>4</sup>	Cross section area	A = 127.8cm <sup>2</sup>
Warping constant	Iw	= 450943cm <sup>6</sup>	Shear centre (from top flange)	ys = 216.8mm

Elastic modulus about yy axis used in NS torsion check = Iyy/(Top flange width/2) = 4245/(18.4/2) = 461.4cm<sup>3</sup>**CONSTRUCTION STAGE – ULTIMATE LIMIT STATE CHECKS****FLOOR LOADS: (unfactored)**

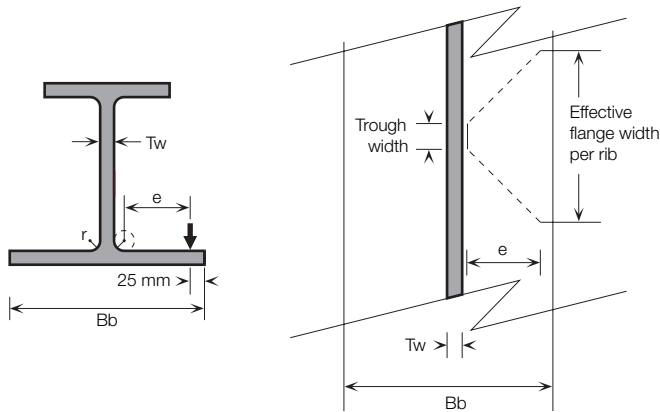
Self-weight of beam	= ASB weight /m spread over supported floor width of 6m = 100.3 x 9.81 / 6.0 / 1000 = 0.16 kN/m <sup>2</sup>
Self-weight of in-situ concrete	= Cross section area x wet density of concrete = 2.39 kN/m <sup>2</sup>
Construction load	= 0.5 kN/m <sup>2</sup>
Self-weight of steel decking	= 0.20 kN/m <sup>2</sup>
Total load (factored)	= (1.4 x (0.16 + 2.39+ 0.20) + 1.6 x 0.5) x 7.0 x 6.0 = 195.6 kN

**CS: BOTTOM FLANGE PLATE BENDING CHECK:**

NOTE: This check considers the loading on the bottom flange, during construction. If the decking is propped (and the beam is not propped) the load is reduced by half.



Total applied loading on flange  $= (1.4 \times (2.39 + 0.2) + 1.6 \times 0.5) \times 3 \times 7 = 92.98 \text{ kN}$



Eccentricity  $= (Bb - Tw) / 2 - r / 2 - 25$   
 $= (294.0 - 19.0) / 2 - 24.0 / 2 - 25.0$   
 $= 100.5 \text{ mm}$

Bottom flange transverse moment  $= 92.98 \times 100.5 / 1000$   
 $= 9.34 \text{ kNm}$

Effective flange width per rib  $= \text{Minimum of pitch of decking ribs or } (Trough \text{ width} + 2 \times \text{eccentricity})$   
 $= 600 \text{ mm or } 100 + 2 \times 100.5$   
 $= 301 \text{ mm}$

From Clause 4.2.5.1 of BS 5950-1:2000, the total flange bending capacity is limited to  $1.2 \times P_y \times Z$  in which  $Z = b \times d^2 / 6$  and  $b$  is the effective flange width for the full beam length ie  $7000 \times 301 / 600$

Total flange bending capacity  $= 1.2 \times 7.0 \times 1000 \times 301 / 600 \times 16.0^2 / 6 \times 345.0 / 10^6$   
 $= 62.03 \text{ kNm}$

UNITY FACTOR  $= 9.34 / 62.03$   
 $= 0.15$

**PASS**

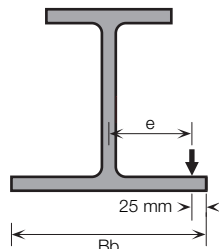
if the ratio  $M_t / M_{pb} > 0.3$  the design bending stress will need to be reduced to take account of the interaction between axial tension and transverse bending.

The reduction factor is calculated from  $\frac{\sigma_t}{p_y} = \left[ 1 - 0.52 \left( \frac{M_t}{M_{pb}} \right) - 0.48 \left( \frac{M_t}{M_{pb}} \right)^2 \right]^{0.5}$  (See section 4.1.3 of SCI-P-175)

However if the ratio is less than 0.3 – as is the case in this example – then the reduction can be ignored.

**CS: WEB BENDING CHECK:**

Total applied loading on web = 92.98 kN



Eccentricity =  $294.0 / 2 - 25.0$

= 122.0 mm

Web moment =  $92.98 \times 122.0 / 1000$

= 11.34 kNm

Effective web width per rib =  $600\text{mm or } 100 + 2 \times 122$

= 344mm

To avoid irreversible deformations under serviceability loads, Clause 4.2.5.1 of BS 5950-1:2000 recommends that the moment capacity is limited to  $1.2 p_y Z$ .

Hence total web bending capacity =  $1.2 \times 7.0 \times 1000 \times 344 / 600 \times 19.0^2 / 6 \times 345.0 / 10^6$

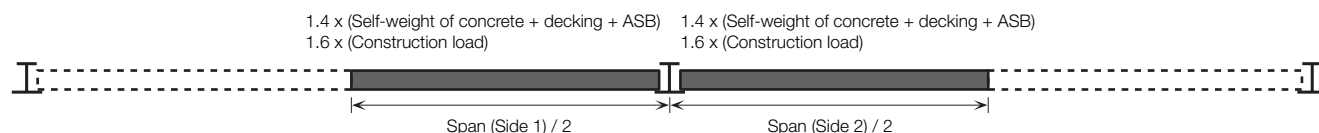
= 99.97 kNm

UNITY FACTOR =  $11.34 / 99.97$

= 0.11

**PASS**

As the ratio  $M_t / M_{pb} < 0.3$  the design bending stress will not need to be reduced.

**CS: LTB CHECK:**

Uniformly distributed loading = 195.6 kN

Point loading = 0.00 kN

Applied moment  $M_{bar}$  =  $UDL \times L / 8$  where  $L = 7\text{m}$

= 171.15 kNm

The buckling capacity of the beam is determined using Clause 4.3.6 of BS 5950-1:2000

$\lambda_{LT} = u \times v \times \lambda \times (\beta_w)^{0.5}$

where

$\lambda = L_e / r_y = 7000 / 57.6 = 121.45$

$u$  = Buckling parameter from section data tables = 0.815

$\beta_w = 1.0$  for compact sections

$\eta = \frac{I_{yc}}{I_{yc} + I_{yt}} = 0.1969$

$[I_{yc} = B_t \times T_f^3 / 12 = 18.4 \times 1.6^3 / 12 = 830.6\text{cm}^4 \text{ and } I_{yt} = B_b \times T_f^3 / 12 = 29.4 \times 1.6^3 / 12 = 3388.3\text{cm}^4]$

$x$  = Torsional index from section data tables = 13.1

$\lambda/x = 121.45 / 13.1 = 9.25$

From Cl 4.3.6.7 of BS 5950-1:2000, 
$$v = \frac{1}{[(4\eta(1 - \eta) + 0.05(\lambda/\chi)^2 + \psi)^2 + \psi]^0.5}$$

Where  $\psi = -0.638$  using the method from Annex B of BS 5950-1:2000  
 $v = 0.7745$

Hence  $\lambda_{LT} = 0.815 \times 0.7745 \times 121.45 \times (1)^{0.5} = 76.63$

From Table 16, BS 5950-1:2000,  $p_b = 196.9 \text{ N/mm}^2$  for  $\lambda_{LT} = 76.63$  and  $p_y = 345$

From Cl 4.3.6.4 BS 5950-1:2000 Buckling capacity  $M_b$  
$$= p_b S_x$$

$$= 196.9 \times 1294 \times 10^{-3}$$

Buckling capacity 
$$= 254.8 \text{ kNm}$$

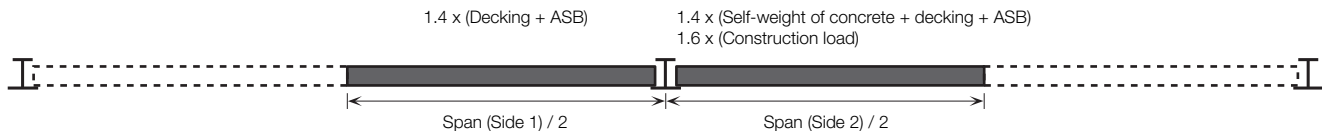
[Note: The value calculated by SIDS is 251.14 kNm. The variance from the hand-calculated value is due to the precision of constants, rounding of numbers and use of equations rather than look-up tables.]

UNITY FACTOR 
$$= 171.15 / 254.8$$

$$= 0.67$$

**PASS****CS: TORSION CHECK:**

NOTE: The torsion check considers concrete and construction loads applied to one side of the beam.



Uniformly distributed loading 
$$= 108.50 \text{ kN}$$

Point loading 
$$= 0.00 \text{ kN}$$

Maximum moment (major axis) 
$$= \text{UDL} \times L / 8$$

$$= 94.94 \text{ kNm (at midspan)}$$

Stress  $\sigma_{bx}$  
$$= \text{Maximum moment} / \text{Elastic modulus (top)}$$

$$= 94.94 / 994.5 \times 1000$$

$$= 95.5 \text{ N/mm}^2$$

Torque 
$$= \text{Out of balance load} \times \text{eccentricity from beam centre line}$$

$$= \{(1.4 \times 2.39 + 1.6 \times 0.5) \times 3 \times 7\} \times 0.122$$

$$= 10.63 \text{ kNm}$$

To determine the rotation, it is necessary to calculate  $L/a$  and  $GJ/Tqa$

$$a = (EH/GJ)^{0.5}$$

Where  $H = \text{Warping constant} = 450943 \text{ cm}^6$   
 $J = \text{Torsional constant} = 160.45 \text{ cm}^4$   
 $E = \text{Elastic modulus} = 205000 \text{ N/mm}^2$   
 $G = \text{Shear modulus} = E / \{2(1 + \nu)\}$  where  $\nu = 0.3$   
 $a = (2.6 \times 450943 / 160.45)^{0.5} = 85.5 \text{ cm}$

Hence  $L/a = 7000 / 85.5 = 8.189$

$\phi$  is determined using Graph 1 at the end of Appendix B, (see also Appendix B5 of SCI-P-175)

$$\phi GJ/Tqa = 0.903 \text{ so } \phi = (0.903 \times 10.63 \times 10^6 \times 85.5) / (7900 \times 160.5 \times 10^4)$$

Rotation 
$$= 0.065 \text{ rad}$$

Max. moment (minor axis) 
$$= \text{Rotation} \times \text{Maximum major axis moment}$$

$$= 0.065 \times 94.94$$

$$= 6.16 \text{ kNm}$$

Stress  $\sigma_{by}$  
$$= \text{Max. minor axis moment} / \text{Elastic modulus about yy axis}$$

$$= 6.16 / 461.4 \times 1000$$

$$= 13.3 \text{ N/mm}^2$$

$\phi''$  is determined using Graph 2 at the end of Appendix B where  $-\phi'' GJ_a/Tq = 0.118$  or  $-\phi'' = 0.118 \times Tq / GJ_a$

Consequently,  $-\phi'' = (0.118 \times 10.63 \times 10^6) / (7900 \times 160 \times 10^4 \times 856)$

$$\phi'' = 1.16 \times 10^{-8}$$

The stress associated with this rotation is calculated using  $\sigma_w = E \times W_{NO} \times \phi''$  where  $W_{NO} = (Bt \times ht) / 2$  and  $ht$  is the distance from the shear centre to the middle of the top flange of the ASB

$$W_{NO} = (184 \times (216.8 - 8)) / 2 = 19210$$

$$\text{Stress } \phi_w = 205000 \times 19210 \times 1.16 \times 10^{-8} = 45.7 \text{ N/mm}^2$$

Buckling check:

The unity factor for this check is derived from 
$$\frac{M_x}{M_b} + \left( \frac{\sigma_{byt} + \sigma_w}{p_y} \right) \chi \left( 1 + \frac{0.5 \chi M_x}{M_b} \right)$$

where  $M_x$  = Max major axis moment,  $M_b$  is the buckling capacity of the ASB,

$$\text{UNITY FACTOR} = 94.9 / 251.1 + (13.4 + 45.7) / 345.0 \times (1 + 0.5 \times 94.9 / 251.1) = 0.58$$

**PASS**

Local capacity check:

$$\text{Total stress } \sigma = 95.5 + 13.4 + 45.7 = 154.5$$

$$\text{UNITY FACTOR} = 154.5 / 345.00 = 0.45$$

**PASS**

## NORMAL STAGE – ULTIMATE LIMIT STATE CHECKS

### FLOOR LOADS: (unfactored)

#### Dead: (see construction stage load calculations for details)

$$\begin{aligned} \text{Self weight of beam} &= 0.16 \text{ kN/m}^2 \\ \text{Self weight of in-situ concrete} &= 2.39 \times 1800.00 / 1900.00 \\ &= 2.27 \text{ kN/m}^2 \\ \text{Self weight of steel decking} &= 0.20 \text{ kN/m}^2 \end{aligned}$$

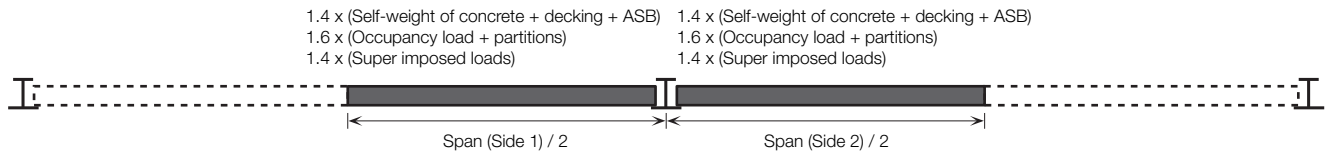
#### Live:

$$\begin{aligned} \text{Occupancy load} &= 3.50 \text{ kN/m}^2 \\ \text{Partitions} &= 1.00 \text{ kN/m}^2 \\ \text{Total imposed load} &= 4.50 \text{ kN/m}^2 \text{ (*** no BS6399 imposed load reduction)} \end{aligned}$$

#### Super-imposed dead:

$$\begin{aligned} \text{Ceilings and services} &= 0.50 \text{ kN/m}^2 \\ \text{Screeds} &= 0.00 \text{ kN/m}^2 \end{aligned}$$

$$\begin{aligned} \text{Total load (factored)} &= (1.4 \times (0.16 + 2.27 + 0.2) + 1.6 \times 4.5 + 1.4 \times (0.5 + 0)) \times 6 \times 7 + 1.4 \times 7 \times 0 \\ &= 486.4 \text{ kN} \end{aligned}$$

**NS: SECTION SHEAR CHECK:**

Applied shear = Total load / 2  
 = 486.4 / 2 = 243.20 kN

Shear capacity = 0.6 x steel stress x area of web  
 = 0.6 x 345 x 19 x 276 / 1000  
 = 1085.5 kN

UNITY FACTOR = 243.20 / 1085.5  
 = 0.22

**PASS****NS: SECTION BENDING CHECK:**

Uniformly distributed loading = 486.4 kN  
 Point loading = 0.00 kN  
 Maximum applied moment = UDL x Beam span / 8  
 = 425.60 kNm ( at midspan )

Concrete in compression = Depth of concrete above decking  
 = (290 – 225) = 65mm

Effective width of slab = Beam span / 8  
 = 875. mm

To calculate the plastic moment capacity and plastic section modulus of the combined section, the position of the plastic neutral axis needs to be determined. Practically there are three locations to consider:

- in the top flange of the steel section  
occurs when  $R_w + R_b + R_t \geq R_c$  ( $D_c/D_s \geq R_w + R_b + R_t$ )
- in the steel web within the solid concrete slab  
occurs when  $D_c \leq y_c \leq D_s$  where  $y_c = (R_b + R_w + R_t + 2(D_c + T_j)R_w/d) / (2R_w/d + R_c/d)$
- in the steel web below the solid slab  
occurs when  $R_t + R_c < R_b + R_w(d - 2(D_s + D_c + T_j))/d$

where  $R_b$  and  $R_t$  are the resistance of the bottom and top flanges of the ASB,  $R_w$  is the total resistance of the web ( $R_{wt}$  and  $R_{wb}$  representing the top and bottom parts of the web respectively) and  $R_c$  is the resistance provided by the concrete section.

Resistance of btm flange ( $R_b$ ) =  $B_b \times T_b \times P_y / 1000 = 294 \times 16 \times 345 / 1000 = 1622.88$  kN  
 Resistance of top flange ( $R_t$ ) =  $B_t \times T_t \times P_y / 1000 = 184 \times 16 \times 345 / 1000 = 1015.68$  kN  
 Resistance of web ( $R_w$ ) =  $\{A_{tot} - (B_b \times T_b + B_t \times T_t)\} \times P_y / 1000$   
 =  $\{12780 - (294 \times 16 + 184 \times 16)\} \times 345 / 1000 = 1770.0$  kN  
 Resistance of concrete =  $0.45 \times f_{cu} \times B_e \times D_s / 1000 = 0.45 \times 30 \times 875 \times 65 / 1000 = 767.81$  kN

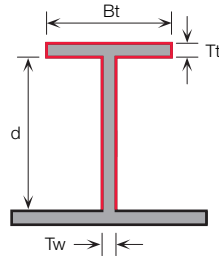
In this instance, case c) applies and the plastic neutral axis falls within the web of the ASB below the solid concrete slab. Full resistance of the concrete can be assumed provided that the calculated value for  $R_c$  is exceeded by the available longitudinal shear resistance, i.e  $F_{sb} > R_c$

Available shear resistance is the lesser of that resulting from bond ( $F_{sbb}$ ) between the concrete and the ASB section and the resistance provided by the concrete cross section and reinforcing mesh ( $F_{sbs}$ ).

Bond resistance ( $F_{sbb}$ ) = Design bond strength  $\times$  bonded perimeter of ASB  $\times$  span / 4

Tests on ASBs have shown that a design bond strength of  $0.6 \text{ N/mm}^2$  can be adopted.

Bonded perimeter =  $2 \times (B_e + d + T_t) - T_w = 2 \times (184 + 244 + 16) - 19 = 869 \text{ mm}$



$$F_{sbb} = 0.6 \times 869 \times 7000 / 4 / 1000 = 912.5 \text{ kN}$$

Longitudinal shear resistance is defined in BS 5950:Part 3, Section 3.1, Clause 5.6.3 as

$$F_{sbs} = 0.7 \times A_{sv} \times f_{ys} + 0.03 \times \eta \times A_{cv} \times f_{cu} = 0.8 \times \eta \times A_{cv} \times f_{cu}^{0.5} = 449.75 \text{ kN}$$

[This is limited by the code to

where

$A_{sv}$  = Area of transverse reinforcement ( $142 \text{ mm}^2/\text{m}$ )

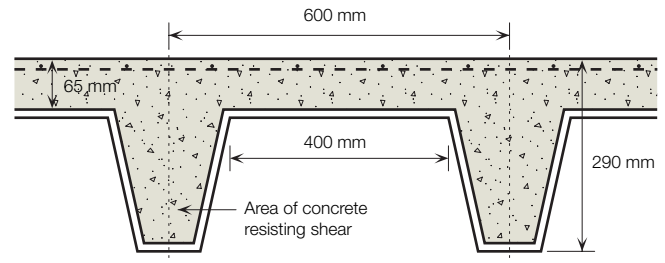
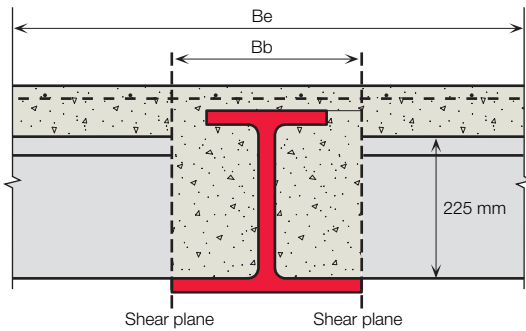
$f_{ys}$  = Yield strength of reinforcement ( $460 \text{ N/mm}^2$ )

$\eta$  = Factor, 0.8 for lightweight concrete

$A_{cv}$  = Area of concrete resisting shear ( $128300 \text{ mm}^2/\text{m}$ )

$f_{cu}$  = Concrete strength ( $30 \text{ N/mm}^2$ )

$$F_{sbs} = 0.7 \times 142 \times 460 \times 10^{-3} + 0.03 \times 0.8 \times 128300 \times 30 \times 10^{-3} = 138.1 \text{ kN}$$



As the concrete force  $R_c$  is assumed to act at the centre of the ASB, the value for  $F_{sbs}$  must also be assessed at the beam centre line and it is assumed that the value of  $F_{sbs}$  is linearly distributed over the effective slab width from zero at each extremity to a peak over the ASB. Hence the value of  $F_{sbs}$  calculated above must be increased by the factor  $B_e/(B_e - B_b)$ .

The resistance for 2 shear planes and an elastic distribution over the length of the beam is

$$F_{sbs} = 2 \times 138.1 \times 875 / (875 - 294) \times 7000 / 4 = 727.94 \text{ kN}$$

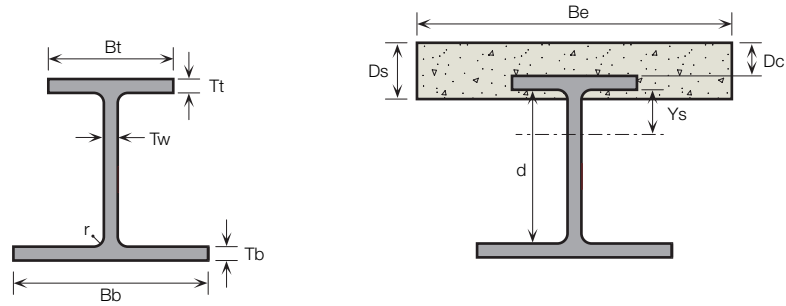
End effects are taken into account by the  $L / (L - 2 \text{ trough widths})$  so that the total resistance due to concrete and reinforcement shear

$$= 7 / 5.8 \times 727.94 = 878.54 \text{ kN}$$

$F_{sbs} < F_{sbb}$  so

$$F_{sb} = 878.54 \text{ kN}$$

$F_{sb} > R_c$  so full shear connection can be assumed and the maximum concrete force used to determine the plastic moment capacity of the combined section.



The plastic neutral axis of the composite section occurs in the web of the ASB below the concrete 'flange', and is located where the tensile resistances below the line equal the compressive resistances above.

$$R_b + R_{wb} = R_t + R_{wt} + R_c$$

$$R_{wb} = R_w \times \left( \frac{d - y_s}{d} \right) \text{ and } R_{wt} = R_w \times \frac{y_s}{d}$$

$$R_b + R_w \times \left( \frac{d - y_s}{d} \right) = R_t + R_w \times \frac{y_s}{d} + R_c$$

$$\frac{y_s}{d} = \left( \frac{R_b + R_w - R_t - R_c}{2 R_w} \right)$$

Therefore,

$$\frac{y_s}{d} = \left( \frac{1622.88 + 1770.54 - 1015.68 - 767.81}{2 \times 1770.54} \right)$$

$$d = 24.4\text{cm so } y_s = 11.093\text{cm}$$

The plastic neutral axis occurs at a distance  $11.093 + 1.6 + 3 = 15.693\text{cm}$  below the top of the slab.

The plastic resistance moment for the composite section  $M_p = M_{\text{top flange}} + M_{\text{botm flange}} + M_{\text{concrete}} + M_{\text{web}}$

Where

$$M_{\text{top flange}} = R_t \times (y_s + T_t/2)$$

$$M_{\text{botm flange}} = R_b \times (d - y_s + T_b/2)$$

$$M_{\text{concrete}} = R_c \times (y_s + T_t + D_c - D_s/2)$$

$$M_{\text{web}} = R_w \times d \times \left\{ \left( \frac{y_s}{d} \right)^2 - \left( \frac{y_s}{d} \right) + 0.5 \right\}$$

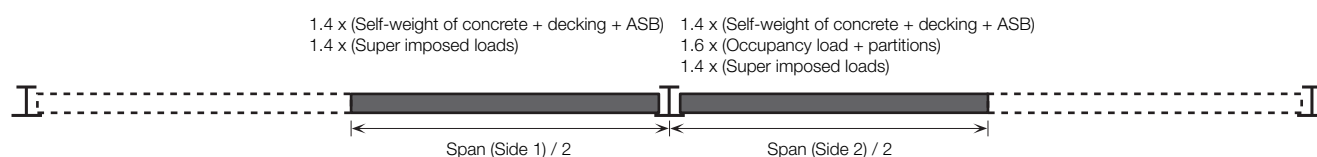
$$\begin{aligned} M_p &= 1015.68 \times (11.093 + 1.6 / 2) \\ &+ 1622.88 \times (24.4 - 11.093 + 1.6 / 2) \\ &+ 767.81 \times (11.093 + 1.6 + 3 - 6.5 / 2) \\ &+ 1770.54 \times 24.4 \times \left\{ \left( \frac{11.093}{24.4} \right)^2 - \left( \frac{11.093}{24.4} \right) + 0.5 \right\} \\ &= 554.13 \text{ kNm} \end{aligned}$$

In steel units, the plastic modulus  $S_x = M / P_y = 554.13 \times 1000 / 345 = 1606.2\text{cm}^3$

[Note: The plastic modulus for the composite section as calculated by SIDS is  $1593.2\text{cm}^3$ . The difference between the numbers is due in part to a simplifying assumption made in the hand calculation where the steel in the root radii is assumed to be 'smeared' over the height of the ASB web rather than being dealt with rigorously.]

$$\begin{aligned} \text{UNITY FACTOR} &= 425.6 / 554.13 \\ &= 0.77 \end{aligned}$$

**PASS**

**NS: TORSION CHECK:**

Uniformly distributed loading	= 335.20 kN
Point loading	= 0.00 kN
Maximum moment (major axis)	= 293.30 kNm (at midspan )
Stress $\sigma_{bx}$	= $293.30 / 1948.3 \times 1000$ = 150.5 N/mm <sup>2</sup>

NOTE: The torsion check considers torsion load due to the difference between the loads on the two sides of the beam.

Net torsional load	= $4.5 \times 1.6 \times 7 \times 3 = 151.20$ kN
Torque	= $151.2 \times 0.122 = 18.45$ kNm
From previous calculations, $L/a = 7000 / 856 = 8.189$	
Rotation $\phi$	= 0.11287 rad
Max. moment (minor axis)	= $0.11287 \times 293.30$ = 33.10 kNm
Stress $\sigma_{by}$	= Max. minor axis moment / Elastic modulus about yy axis = $33.10 / 461.4 \times 1000$ = 71.8 N/mm <sup>2</sup>
	$\phi'' = 2.0136 \times 10^{-8}$
Stress $\sigma_w$	= $E \times W_{NO} \times \phi'' = 205000 \times 19210.7 \times 2.0136 \times 10^{-8}$ = 79.3 N/mm <sup>2</sup>
Local capacity check:	
Total stress $\sigma$	= $150.5 + 71.8 + 79.3$
UNITY FACTOR	= $301.59 / 345.00$ = 0.87

**PASS****FORCES FOR THE DESIGN OF END CONNECTIONS:**

Load Case No	Vertical Shear (kN)	Torque (kNm)	
1	54.3	5.3	(CS : torsion case)
2	243.2	-	(NS : maximum loading)
3	167.6	9.2	(NS : torsion case)

NOTE: If properly anchored rebars are provided over or through the beams, Load Case 3 may be ignored in the design of the connections (as out of balance moment is resisted by the slab).

**FIRE DESIGN :**

Fire resistance period	= 60 mins
Uniformly distributed loading in fire	= Slab self weight + ceilings and services + partitions + 80% imposed = $(0.16 + 2.27 + 0.2 + 0.8 \times 3.5) \times 7 \times 6$ = 291.03 kN
Point loading in fire	= 0.00 kN
NOTE: Loading at the fire limit state is calculated using reduced partial factors to BS 5950:Part 8.	
Check at maximum moment position:	
Maximum applied moment	= 254.65 kNm (at midspan)

The fire moment capacity for the composite beam can be determined from the detailed output in the table below. The sum of the forces in elements 1 to 15 multiplied by the relevant lever arm ( $y_c$  in the table) gives the moment capacity.

Fire moment capacity = 269.54 kNm

Concrete force = Sum of concrete forces for elements 9 to 12 and 15 from table below

$$= 669.4 + 256.7 = 926.04 \text{ kN}$$

Bond Stress = Concrete force / (bond perimeter x span / 4)

$$= 926.04 \times 10^3 / (869 \times 7000 / 4)$$

$$= 0.61 \text{ N/mm}^2 \text{ (Bond stress limit} = 0.9 \text{ N/mm}^2 \text{)}$$

UNITY FACTOR = 254.65 / 269.54

$$= 0.94$$

**PASS**

#### DETAILED OUTPUT AT MAXIMUM MOMENT POSITION :

Element No	Material	Width mm	Depth (mm)	Position (mm)	Temperature °C	$y_c$ mm	Strength $\text{N/mm}^2$	Stress $\text{N/mm}^2$	Force kN
1	1	294	16	290	756	298	345	56.1	-263.9
2	1	44.7	6.4	283.6	677	286.8	345	98.1	-28.2
3	1	31.8	6.4	277.2	642	280.4	345	127.1	-26
4	1	19	7.2	270	605	273.6	345	157.1	-21.4
5	1	19	20	250	532	260	345	235.3	-89.4
6	1	19	20	230	422	240	345	328.3	-124.8
7	1	19	184	46	*	138	345	345	-1206.1
8	1	184	16	30	*	38	345	Element,split	
9	2	275	225	65	*	177.5	30	Element,split	
10	2	856	19	46	*	55.5	30	Element,split	
11	2	691	16	30	*	38	30	Element,split	
12	2	875	30	0	*	15	30	25.5	669.4
13	1	184	14.6	30	*	37.3	345	345	924.7
14	1	184	1.4	44.6	*	45.3	345	345	-91
15	2	691	14.6	30	*	37.3	30	25.5	256.7

NOTE: Fire moment resistance is determined from plastic cross-section analysis using cross-section temperatures shown above. Temperatures shown as (\*) are less than 400°C and do not cause a strength reduction.

This table identifies the size of the elements, their normal strength, the temperature and the reduced strength of each element. The plastic neutral axis is calculated by equating compression and tension forces. Where the plastic neutral axis is midway in an element, the element is split. The forces in the concrete may be limited by shear bond action in fire. If there is insufficient shear connection, the width of some of the concrete elements is set to zero. The moment resistance in fire is obtained by taking the force in each element times its distance from the plastic neutral axis.

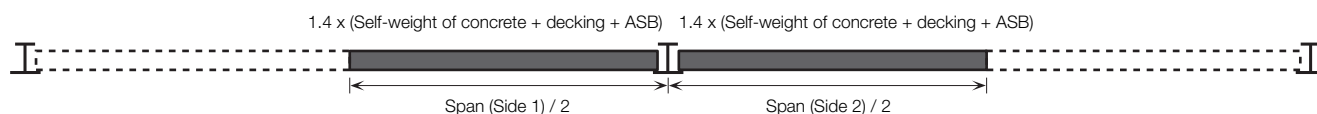
## SERVICEABILITY LIMIT STATE CHECKS

### DEFLECTION CHECKS

#### CONSTRUCTION STAGE:

##### SELF-WEIGHT DEFLECTION

In the following calculations, SIDS uses the dry weight of concrete – which would appear to be illogical as this calculation relates to the construction stage. The self-weight deflection at this stage is not normally critical and this calculation is used to build up the total deflection for in-service conditions when the concrete will be dry. If a check on ponding is to be carried out for the decking then the 'wet' weight of concrete should be used.



Uniformly distributed loading  $= (0.2 + 2.27 + 0.16) \times 6 \times 7 = 110.43 \text{ kN}$

Deflection  $= \left( \frac{5WL^3}{384EI} \right) = \left( \frac{5 \times 110.43 \times 10^3 \times 7000^3}{384 \times 205000 \times 15506 \times 10^4} \right)$

$= 15.52 \text{ mm}$

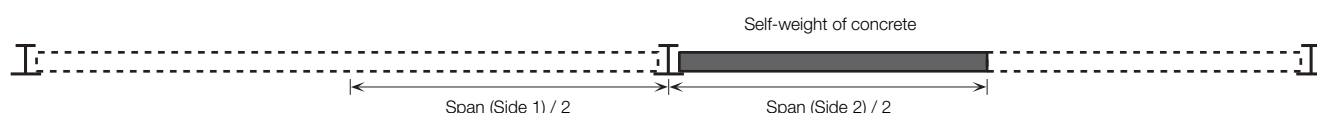
Point loading  $= 0.00 \text{ kN}$

Deflection  $= 0.00 \text{ mm}$

Total deflection  $= 15.52 + 0.00$

$= 15.52 \text{ mm}$

##### HORIZONTAL DEFLECTION



Net torsional loading  $= 2.27 \times 3 \times 7 = 47.57 \text{ kN}$

Torque  $= 47.57 \times (294 / 2 - 25) / 1000$

$= 5.8 \text{ kNm}$

Distance to shear centre  $= 216.8 \text{ mm}$  (from beam top flange)

From previous calculations,  $L/a = 7000 / 856 = 8.189$

Rotation  $= 0.03538 \text{ rad}$

Deflection  $= 0.03551 \times 216.8$

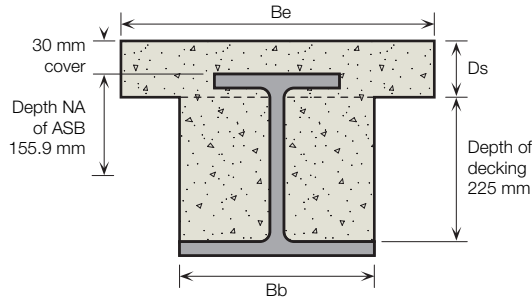
$= 7.70 \text{ mm} (< \text{SPAN} / 500 = 14.0 \text{ mm})$

**SATISFACTORY**

## NORMAL DESIGN STAGE:

### DEFLECTION DUE TO IMPOSED LOADS

The neutral axis of the composite section is found by taking moments of the contributing areas about the top of the ASB. For compatibility, concrete areas are converted to 'steel units' by dividing by the modular ratio of 15.



ASB:	Area = $12780\text{mm}^2$ Lever arm = 155.9mm
Concrete flange:	Area = $B_e \times D_s = 875 \times 65 = 56875\text{mm}^2 = 3791.667\text{mm}^2$ in steel units Lever arm = $D_s / 2 - \text{cover} = 65 / 2 - 30 = 2.5\text{mm}$
Concrete encasement	Area = $B_b \times 225 = 294 \times 225 = 66150\text{mm}^2 = 4410\text{mm}^2$ in steel units Lever arm = $\text{Depth of ASB} - T_b - \text{Depth of decking} / 2 = 276 - 16 - 225 / 2 = 147.5\text{mm}$
Depth to composite NA	$= (12780 \times 155.9 + 3791.667 \times 2.5 + 4410 \times 147.5) / (12780 + 3791.667 + 4410)$ $= 126.41\text{mm}$ below the top of the ASB.

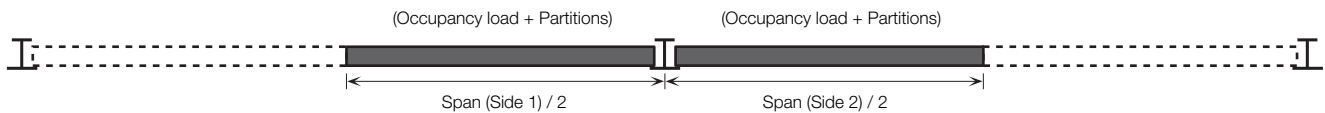
The composite inertia is determined by taking moments of areas about the composite neutral axis depth.

$$\text{Inertia of ASB} = 15506 + (12780 \times (155.9 - 126.41)^2) / 10000 = 16617.52\text{cm}^4$$

$$\text{Inertia of flange} = \left\{ \left( \frac{875 \times 65^3}{12} \right) + (875 \times 65) \times (126.41 - 2.5)^2 \right\} / (15 \times 10000) = 5955.76$$

$$\text{Inertia of encasement} = \left\{ \left( \frac{294 \times 225^3}{12} \right) + (294 \times 225) \times \left( 276 - 16 - \frac{225}{2} - 126.41 \right)^2 \right\} / (15 \times 10000) = 2056.49$$

$$\text{Inertia (composite section)} = 16617.52 + 5955.76 + 2056.49 = 24630\text{cm}^4 \text{ (In steel units)}$$



$$\text{Uniformly distributed loading} = (3.5 + 1) \times 6 \times 7 = 189.00 \text{ kN}$$

$$\text{Deflection} = \left( \frac{5 \times 189 \times 10^3 \times 7000^3}{384 \times 205000 \times 24630 \times 10^4} \right) = 16.72\text{mm}$$

$$\text{Point loading} = 0.00 \text{ kN}$$

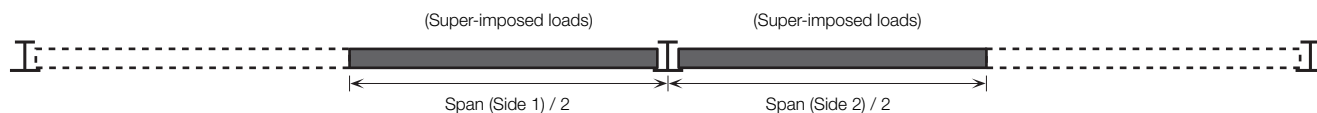
$$\text{Deflection} = 0\text{mm}$$

$$\begin{aligned} \text{Total deflection} &= 16.72 + 0 \\ &= 16.72\text{mm} (< \text{SPAN} / 360 \text{ i.e } 19.4\text{mm}) \end{aligned}$$

**SATISFACTORY**

**DEFLECTION DUE TO SUPER-IMPOSED DEAD LOADS**

Inertia (composite section) = 24630cm<sup>4</sup>



Uniformly distributed loading = 0.5 x 6 x 7 = 21.00 kN

Deflection =  $\left( \frac{5 \times 21 \times 10^3 \times 7000^3}{384 \times 205000 \times 24630 \times 10^4} \right) = 1.86 \text{ mm}$

Point loading = 0 kN

Deflection = 0mm

Total deflection = 1.86 + 0

= 1.86mm

**TOTAL DEFLECTION CHECK**

Total deflection = 15.52 + 16.72 + 1.86  
= 34.09mm (< SPAN / 200 = 35.0mm)

**SATISFACTORY****VIBRATION CHECK:****DYNAMIC SENSITIVITY**

The calculations performed above to calculate the composite inertia with a modular ratio of 15 need to be repeated with a reduced modular ratio to represent the short term modulus of concrete. In this example, a value of 10 has been used which results in a revised depth to the neutral axis of 118.9mm and a revised inertia.

Inertia (composite section) = 28494 cm<sup>4</sup> \*\* based on short-term modulus of concrete of 10 \*\*

For this check, the loads are dead + super-imposed dead + 10% imposed

Uniformly distributed loading = 150.33 kN

Deflection =  $\left( \frac{5 \times 150.3 \times 10^3 \times 7000^3}{384 \times 205000 \times 28494 \times 10^4} \right) = 11.49 \text{ mm}$

Point loading = 0.00 kN

Deflection = 0.00mm

Total deflection = 11.49 + 0.00

= 11.49mm

Frequency = 18 / sqrt( 11.49 )

= 5.31 Hz (greater than 4.00 Hz)

**SATISFACTORY**

## STRESS CHECKS:

### CONSTRUCTION STAGE

For this check, unfactored dead loads as derived for the self-weight deflection check are used.

Uniformly distributed loading	= 110.43 kN
Point loading	= 0.00 kN
Maximum applied moment	= 96.63 kNm ( at midspan )
Elastic neutral axis in WEB	155.9 mm from beam top flange
Moment of inertia (steel section)	= 15506cm <sup>4</sup>
Steel modulus (top)	= 995cm <sup>2</sup>
Compression	= 96.6 x 1000 / 995
	= 97.2 N/mm <sup>2</sup>
Steel modulus (btm)	= 1291cm <sup>3</sup>
Tension	= 96.6 x 1000 / 1291
	= 74.8 N/mm <sup>2</sup>

### NORMAL STAGE

For this check, unfactored imposed loads are used.

Uniformly distributed loading	= (3.5 + 1 + 0.5) x 6 x 7 = 210.00 kN
Point loading	= 0.00 kN
Maximum moment (major axis)	= 183.75 kNm (at midspan )
Calculate position of elastic NA	

The depth to composite NA was calculated previously as 126.41mm below the top of the ASB. As the concrete cover to the ASB is 30mm, the distance from the NA to the top of the slab is 156.41mm.

Therefore the elastic neutral axis occurs in the WEB  $306 - 156.4 = 149.6$ mm from the soffit of the ASB beam

Moment of inertia (composite section)	= 24630cm <sup>4</sup> (expressed in steel units)
This modulus value assumed a modular ratio of 15, so the equivalent inertia in concrete units is	369450cm <sup>4</sup>
The modulus of the composite section in concrete units = Inertia / distance from NA to top of slab	
Concrete modulus	= 369450 / 15.64 = 23620cm <sup>3</sup>
Concrete compressive stress	= 183.8 x 1000 / 23620
	= 7.8 N/mm <sup>2</sup>

UNITY FACTOR	= 7.78 / (1 * 30.0)	
	= 0.26	<b>PASS</b>

Steel modulus (top)	= 24630 / 12.641 = 1948cm <sup>3</sup>
Steel stress (top - compression)	= 183.75 x 1000 / 1948
	= 94.3 N/mm <sup>2</sup>
Steel modulus (btm)	= 24630 / (27.6 - 12.641) = 1646cm <sup>3</sup>
Steel stress (btm - tension)	= 183.8 / 1646 x 1000
	= 111.6 N/mm <sup>2</sup>

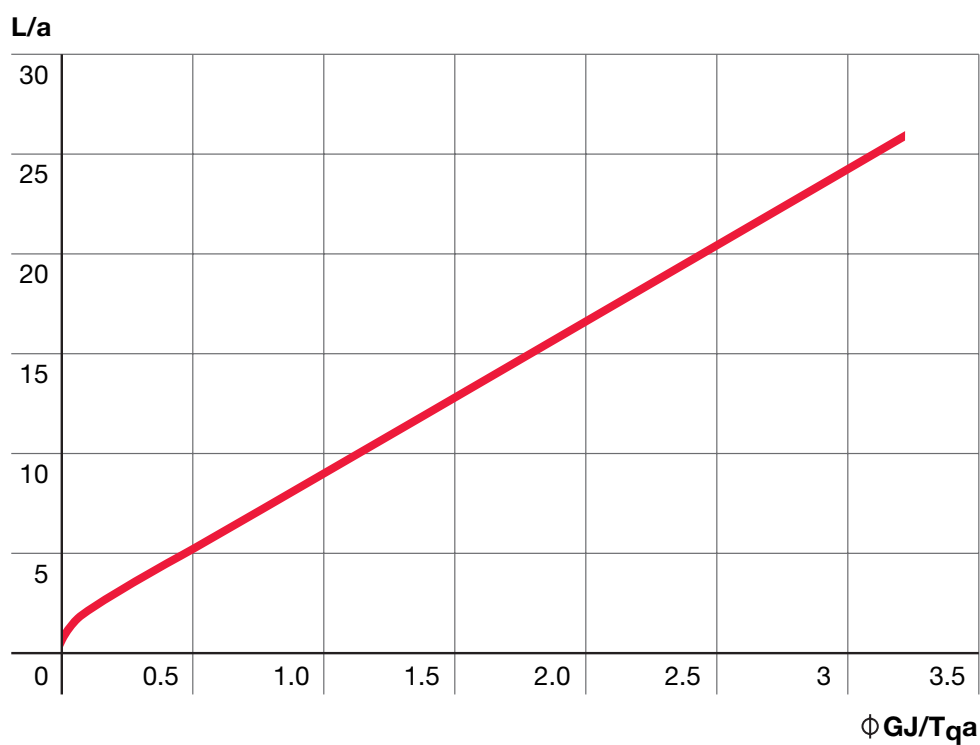
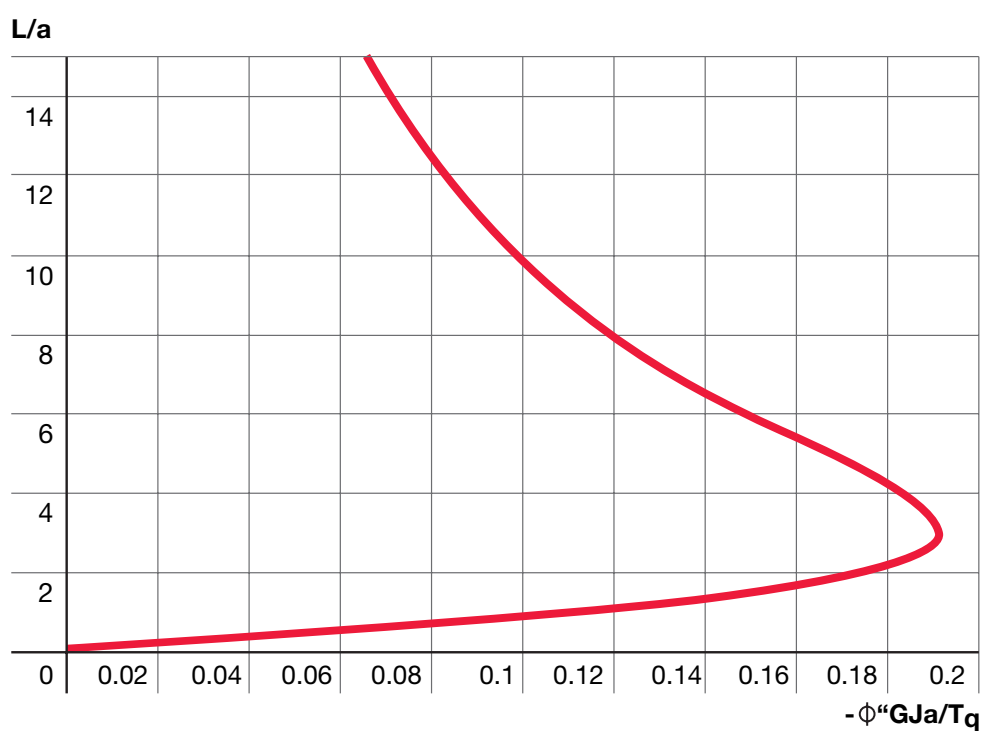
### STEEL STRESS CHECK (total stresses)

Compression (at top of beam)	= 97.16 + 94.31	
	= 191.47 N/mm <sup>2</sup>	
UNITY FACTOR	= 191.47 / 345.0	
	= 0.55	<b>PASS</b>
Tension (at bottom of beam)	= 74.84 + 111.60	
	= 186.44 N/mm <sup>2</sup>	
UNITY FACTOR	= 186.44 / 345.0	
	= 0.54	<b>PASS</b>

**BOTTOM FLANGE COMBINED STRESS CHECK**

\*\*\* No direct loads on ASB beam bottom flange \*\*\*

\*\*\* check not required \*\*\*

**Graph 1: Determination of the angle of rotation  $\phi$** **Graph 2: Determination of second derivative of  $\phi$** 



# Appendix C: Robustness requirements for Class 2 buildings

This topic is discussed at greater length in SCI Report RT1215 which can be accessed via [www.corusconstruction.com](http://www.corusconstruction.com)

## Class 2A buildings

No special robustness rules or considerations are required for Class 2A buildings with Slimdek floors. The general rules for Class 2A buildings from BS 5950-1 should be followed. All beams and ties and their end connections should have a minimum tying capacity of 75 kN which can be achieved by adopting conventional connection details such as those illustrated in the Connection Design section of this manual. The floor bearing requirement is satisfied using the standard Slimdek construction details. The floor slab is not required to be anchored to its supports apart from satisfying the bearing conditions.

## Class 2B buildings

Three approaches exist to demonstrate compliance with the requirements of Clause 2.4.5.3 of BS 5950-1 for Class 2B structures namely the tying route, the notional element removal method and key element design. As the tying route is the most commonly used, because it is normally the easiest one to adopt for design and construction, the other methods are not taken further in this manual.

## Tie requirements

### 1. Members supporting floor loads

#### ASBs and RHSFBs

These members and their end connections should have a tie capacity at least equal to the beam end reactions under factored loading multiplied by  $n$  (a factor dependent upon the number of storeys).

### 2. Members not supporting floor loads

#### General requirement

Tie members and their end connections should have a tie capacity at least equal to 75 kN.

#### Tying of edge columns.

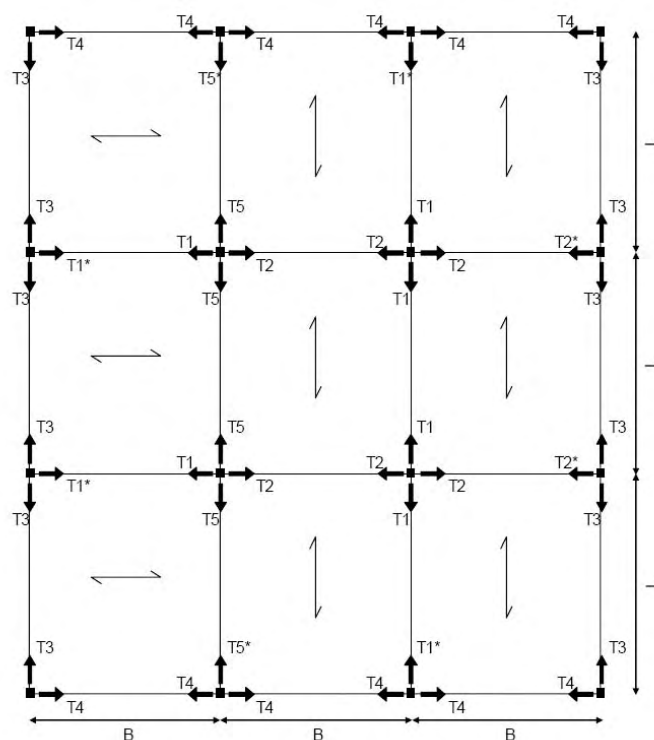
Sub-clause 2.4.5.3 b) of BS 5950-1 requires that horizontal ties anchoring edge columns should be capable of resisting a factored tensile load (acting perpendicular to the edge) at least equal to 1% of the maximum factored vertical dead and imposed load in the column at that level. Therefore, for this to exceed the general requirement, the axial load in the column must exceed 7500 kN – which is unlikely unless there are at least 10 storeys.

#### Edge ties

Edge ties and their end connections should be designed for a tie capacity equal to their end shear reactions (multiplied by  $n$ ) but not less than the general requirement of 75kN – which will normally govern.

However, a tie force of 75 kN will not be capable of forming a catenary if, for example, a central edge column was to be damaged. It is therefore recommended that edge tie members (and their end connections) should be designed for a minimum tie force equal to 25% of the factored dead and imposed floor load (multiplied by  $n$ ) of the slab plus any load contribution from the cladding that is supported by the edge member. Edge tie members are usually a structural section because stiffness, bending capacity and tying capacity is required. These are all substantial members and are capable of having end connections with the necessary tying capacities.

The various tie requirements are summarised in the diagram below:



T1 = 75kN

T2 =  $(wLB/2).n$  but > 75kN

T3 =  $(wLB/4 + cL/2).n$  but > 75kN

T4 =  $(wLB/4 + cB/2).n$  but > 75kN

T5 =  $(wLB/4).n$  but > 75kN

where:

$w$  = factored floor load per  $m^2$

$c$  = factored cladding load per  $m$

$n$  = reduction factor from BS 5950-1

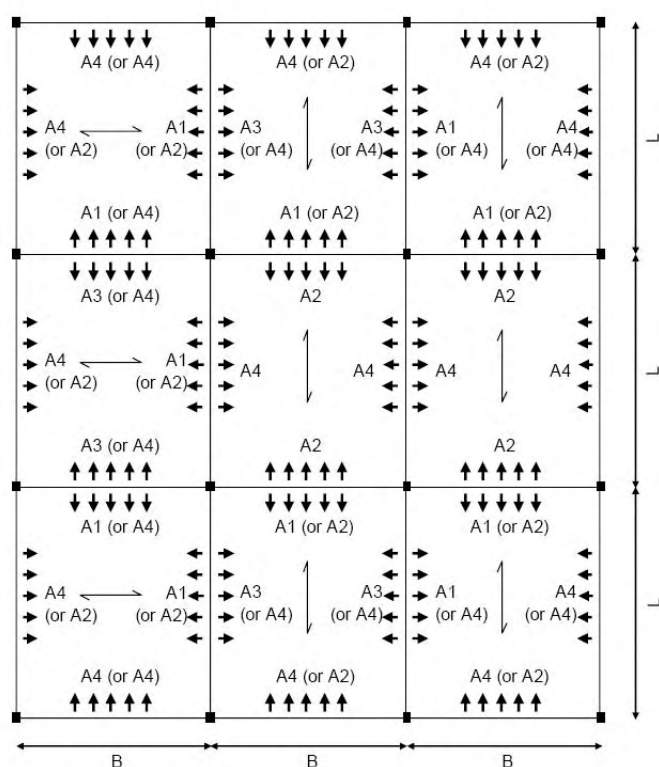
\* tie force > 1% of axial load in column

Standard connection details for main and edge members are given in Section 4 of this manual. Good practice requires that a minimum of four bolts (20mm dia. Grade 8.8) are used to make structural connections for Slimdek components and this arrangement will provide sufficient tie capacity for most situations.

### Anchorage requirements

Sub-clause 2.4.5.3 e) of BS 5950-1 requires that floor slabs are anchored in the direction of their span either to an adjacent slab or to a support to prevent disproportionate collapse in the event that columns or beams are damaged. Where the slab spans on to an edge tie or occurs at a corner, the anchorage requirements vary. The reinforcement provided must be capable of supporting the weight of the slab in the event of a collapse but it can be the same reinforcement as that used to prevent cracking in the slabs, provided the reinforcement is continuous over the beam or tie member. A142 mesh is usually provided in Slimdek slabs as a minimum which is adequate for the majority of situations.

Anchorage requirements are summarised in the diagram below:



A1 = dLB  
A2 = dLB/2  
A3 = dLB/3  
A4 = No anchorage

where:  
d = weight of floor slab per m<sup>2</sup>

#### Notes:

Where there are two alternative anchorage values presented, the selection must be consistent within each slab (i.e., if the first option is selected for one side of the slab the first option must be selected for all other sides of the slab).

Where the slab anchorage requirements are different on either side of the interface between two slabs, the anchorage provided across the interface must be capable of resisting the larger of the two anchorage forces.