

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^[34] and BS 5950-1^[5]. 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. Detailed design methods for all the Slimdek components are presented in SCI publications listed in the References.

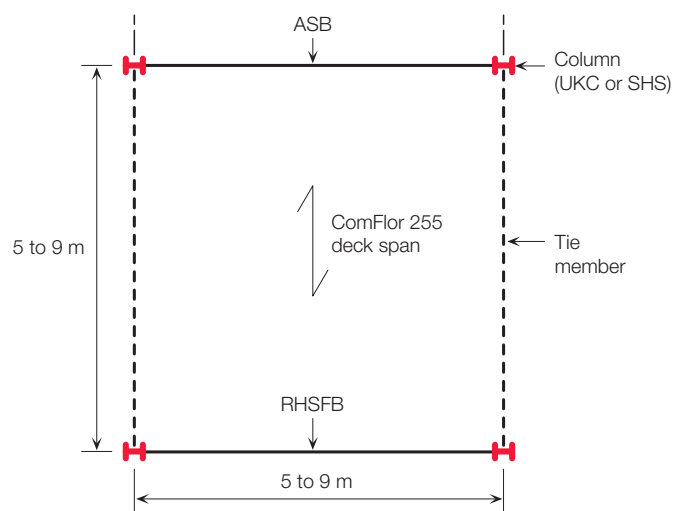


Figure 2.1 Slimdek – beam layout

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^[7] and capable of being designed plastically whereas non-composite beams are designed using BS 5950-1^[5].

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 0.6N/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^[7] 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^[6] and 2^[9]. 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.

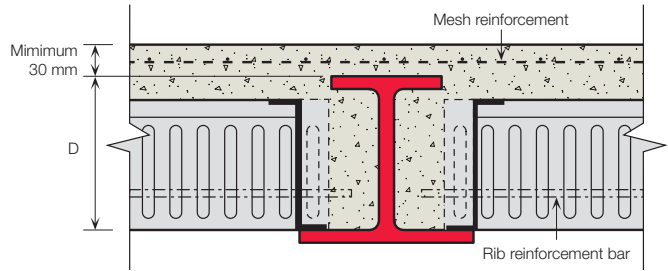


Figure 2.2 Composite arrangement

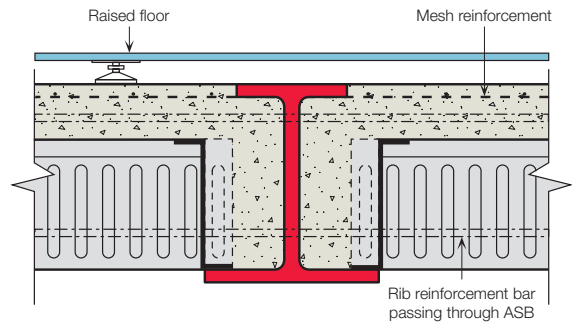


Figure 2.3 Non-composite arrangement

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 mm²/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^[3], BS 5950-6^[2] and BS 5950-8^[10]. Construction loading for long span applications is taken in accordance with BS EN 1994-1-1^[11] which states that a construction load of 1.5kN/m^2 should be applied over a 3m square area of the decking with a reduced construction load of 0.75kN/m^2 elsewhere.

BS 5950:Part 4^[3] 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 20mm 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 3m) is shown graphically in *Figure 2.4*.

The nominal end bearing distance of the decking on its supports is 50mm , 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 35mm .

Deck span tables are given in *Section 3*.

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

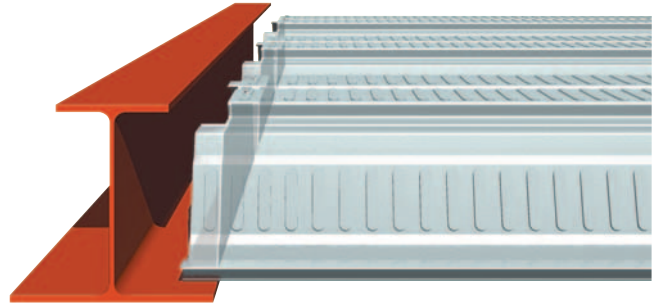


Figure 2.5 End bearing and cut-outs of decking

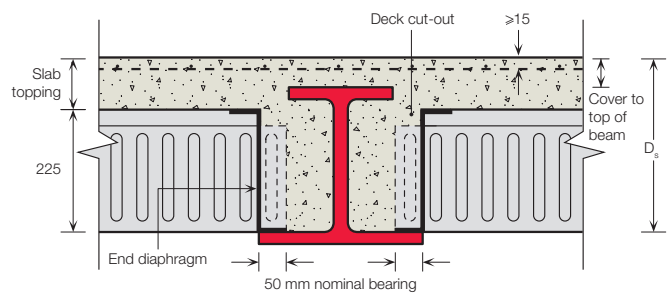


Figure 2.6 Geometry for ASB with ComFlor 225

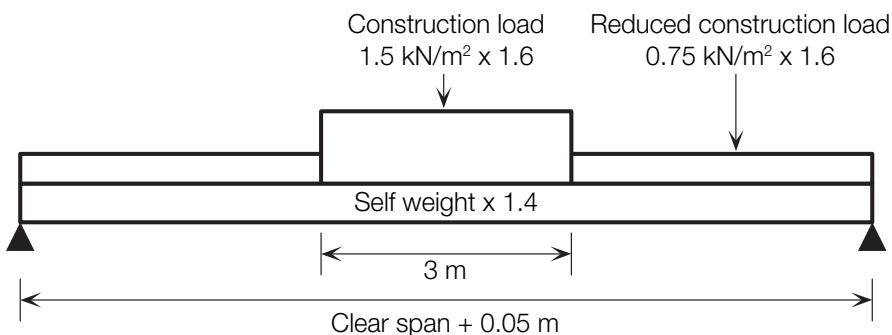


Figure 2.4 Construction stage loading

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/m³ against 2400kg/m³ 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⁽¹⁰⁾) less 10mm to account for the insulating effect of the adjacent ribs, see *Figure 2.7*.

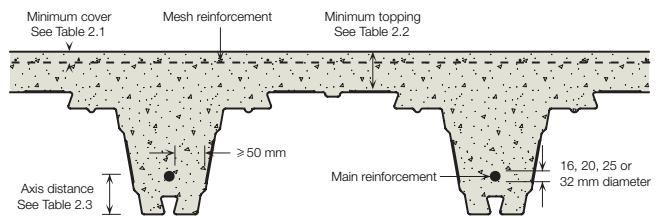


Figure 2.7 Details of reinforcement and minimum slab depth

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 = 296mm for the beam which is greater than slab requirement of 225 + 60 = 285mm (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^[8] 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^[8], 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^[8] 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 12ϕ in accordance with BS 8110-1^[8]. 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^[8], 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 ϕ (mm) – unpropped slab – propped slab		16 ($\phi_L=12\text{mm}$)		20 ($\phi_L=16\text{mm}$)		25 ($\phi_L=16\text{mm}$)							
		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								
		$\phi_L=12\text{mm}$	L-bar		$\phi_L=20\text{mm}$	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		$\phi_L=20\text{mm}$	U-bar*							
End shear resistance	Concrete type	NWC		LWC		NWC		LWC					
	Anchorage bar diameter ϕ_L	0	12	16	0	12	16	16	16	20	16	20	
	Resistance (kN/m)	55**	67	73	51**	61	66	72	65	71	77	63	68
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.

ϕ = diameter of main bar reinforcement, in mm.

ϕ_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.

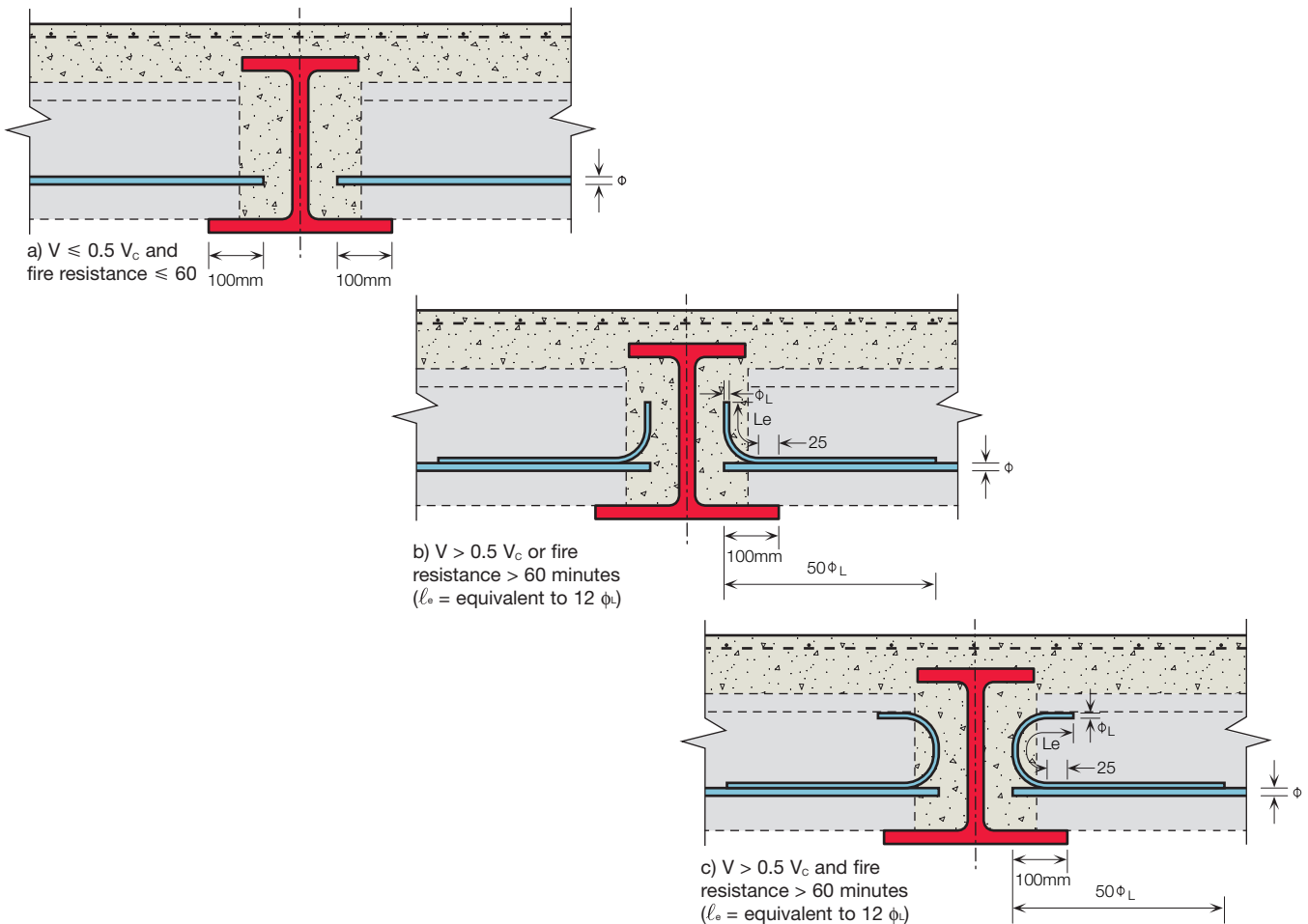


Figure 2.8 Detailing of bar reinforcement in slabs for end anchorage

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^[8].

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^[9] 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^[3] 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 x beam span to each side of the column. The width of crack control reinforcement at columns should extend 0.15 x 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*.

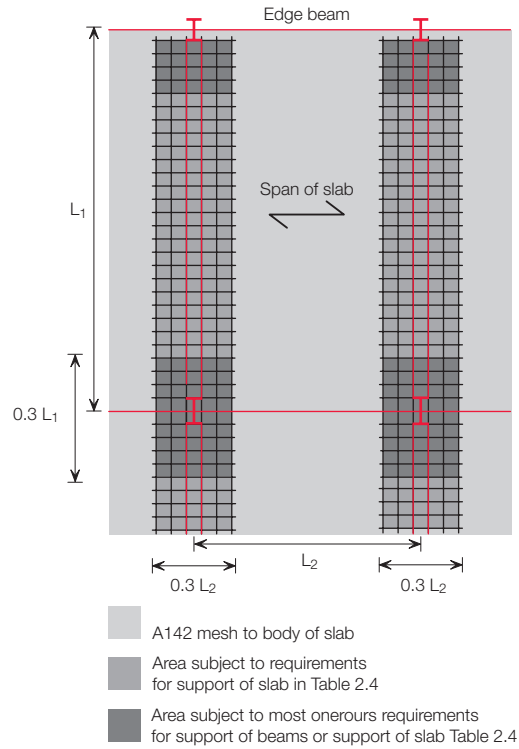


Figure 2.9 Crack control mesh in the slab

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		A193		A142	
	Slab span up to 8m		A252		A193	
	Slab span up to 9m		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:

- The minimum percentage is based on the cross-sectional area of the slab topping (above the ribs of the decking).
- 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 4kN/m² (+ 1kN/m² partition allowance).
- A393 mesh is required for 120 minutes fire resistance.
- Reinforcement should extend 0.15 x slab span on each side of supports.
- Reinforcement at columns should extend along the beam 0.15 x beam span to each side of the column. The width of reinforcement at columns should extend 0.15 x 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:

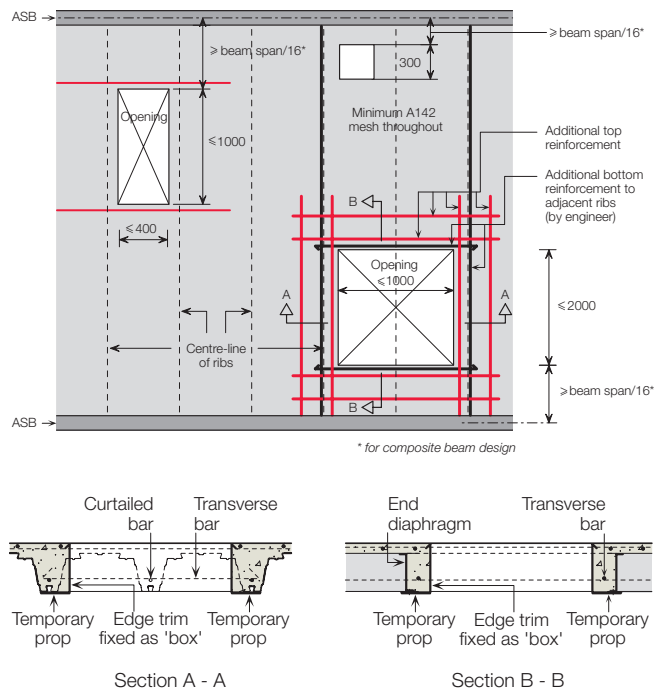


Figure 2.10 Details of openings in the slab

- 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^[8]. 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^[8] 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^[8], 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).

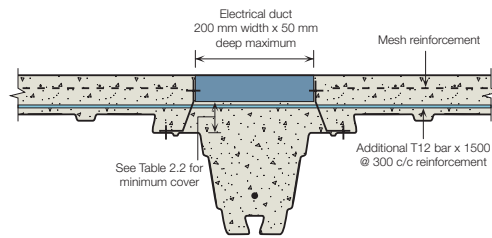


Figure 2.11 Electrical duct within structural slab

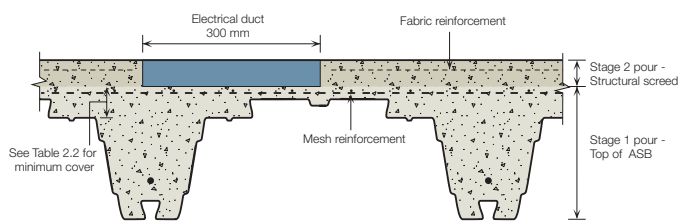


Figure 2.12 Electrical duct within structural screed (two stage pour)

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:

- Contract specifications may give other limits.
- These limits may be varied where greater deflection will not impair the strength or efficiency of the floor.
- Where soffit deflection is considered important (e.g., for service requirements or aesthetics) it may be necessary to reduce these limits.
- 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^[6]. 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.

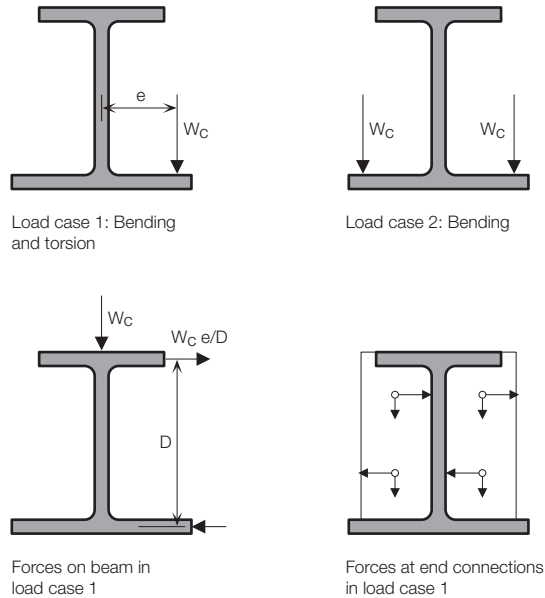


Figure 2.13 Combined bending and torsion on ASB during construction

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^[7]. The plastic stress blocks of the composite ASB section are shown in *Figure 2.14*.

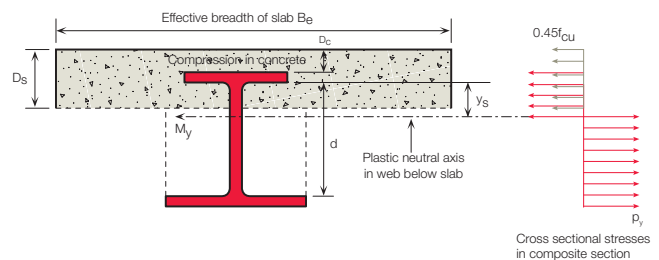
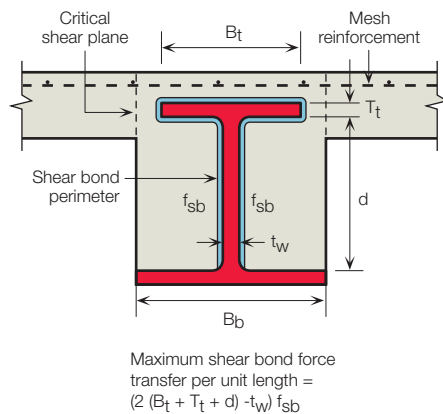
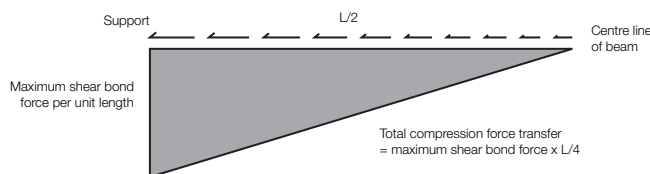


Figure 2.14 Composite design using ASB

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 $span/8$, 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.

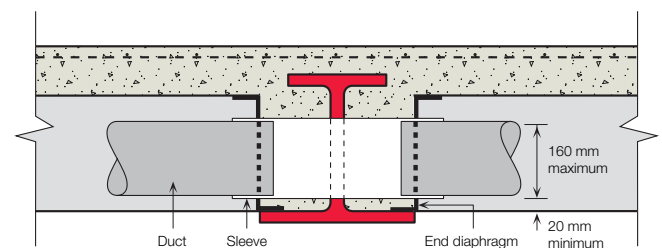


Figure 2.16 Forming openings through ASB

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

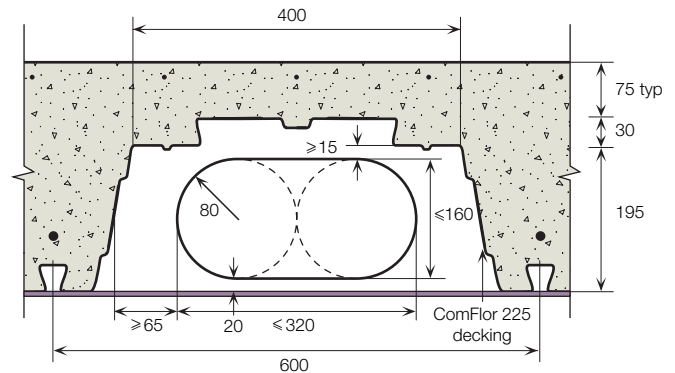
Openings formed in the webs of ASBs affect shear and bending resistance. The transfer of shear across the openings causes local bending effects (or *Vierendeel bending*) in the remaining web-flange sections. Shear and bending resistance is greatly improved by composite action of the steel section with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

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

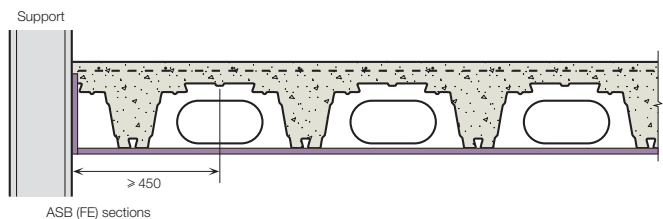
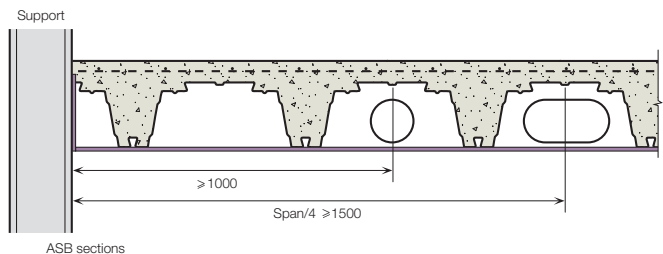
ASB (thinner web) sections may possess insufficient shear resistance when perforated by large elongated openings and should be checked for the particular grid size and loads.

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.

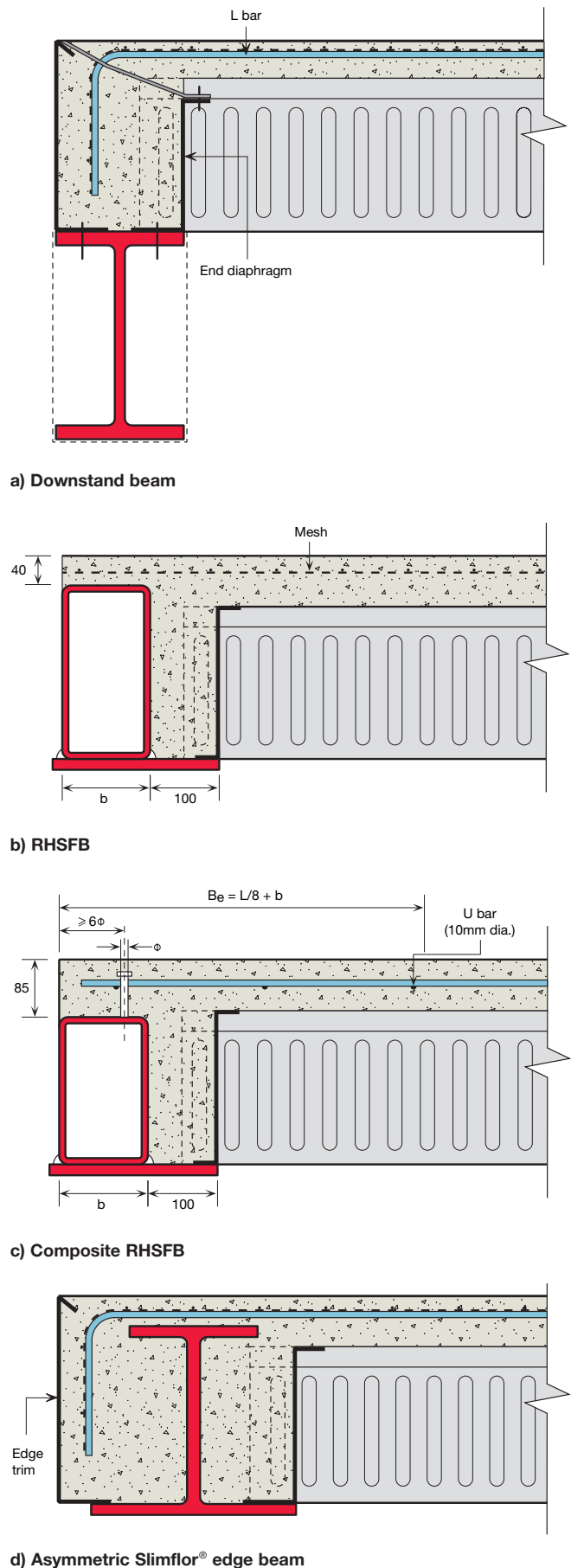
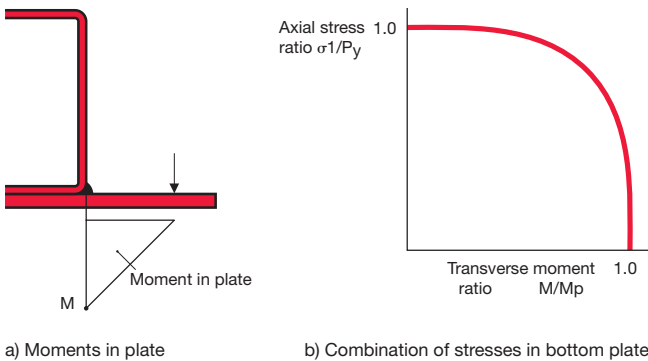


Figure 2.18 Typical edge beams

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.



- σ_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 $(Span/8 + B/2)$ 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^[18].

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 (σ_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 σ_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^[18].

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*.

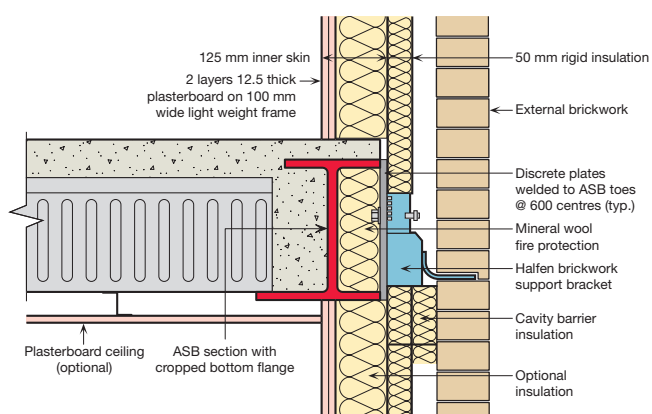


Figure 2.20 Typical section through an external wall with a cropped Asymmetric Slimflor® edge beam

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*.

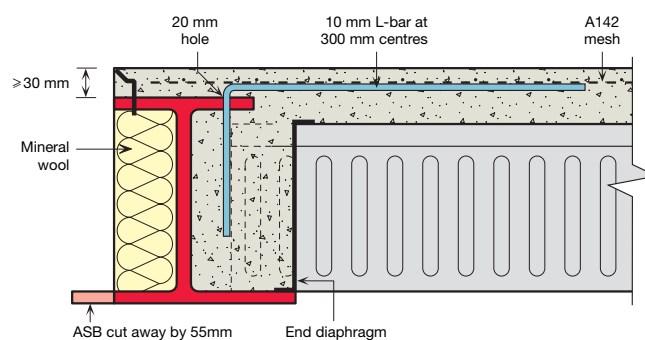


Figure 2.21 Shear key/tying reinforcement with a cropped Asymmetric Slimflor® edge beam

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^[32]

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).

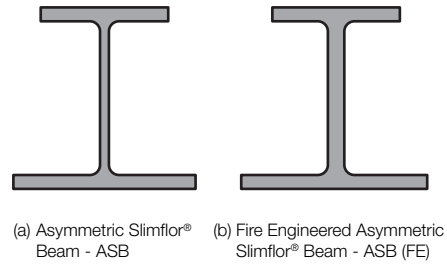


Figure 2.22 Types of ASB Slimflor® beam

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.

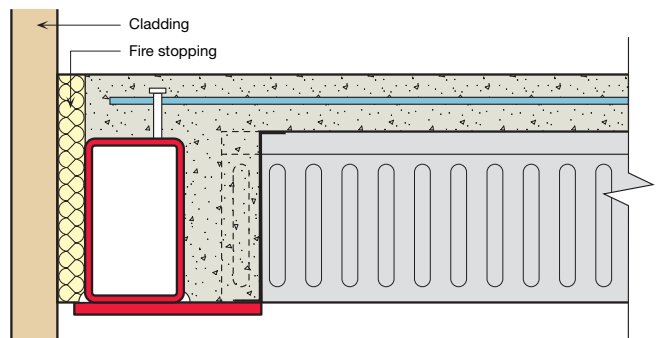


Figure 2.23 Fire compartmentation

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.

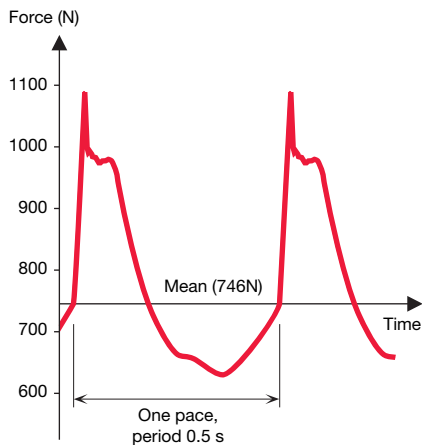


Figure 2.24 Typical force – time function for walking

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^[13] and ISO/DIS 10137^[14] generally, and in HTM 2045^[95] 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*.

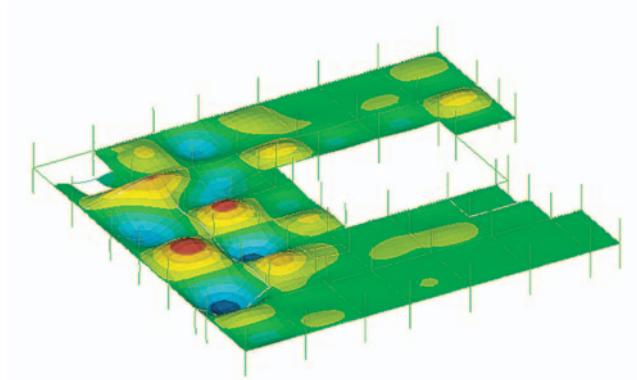


Figure 2.25 Chichester Hospital finite element model (printout)

SCI-P-354: *Design of Floors for Vibration: A New Approach*^[30] 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^[30] 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^[35].

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^[30].

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'*^[55]. 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.

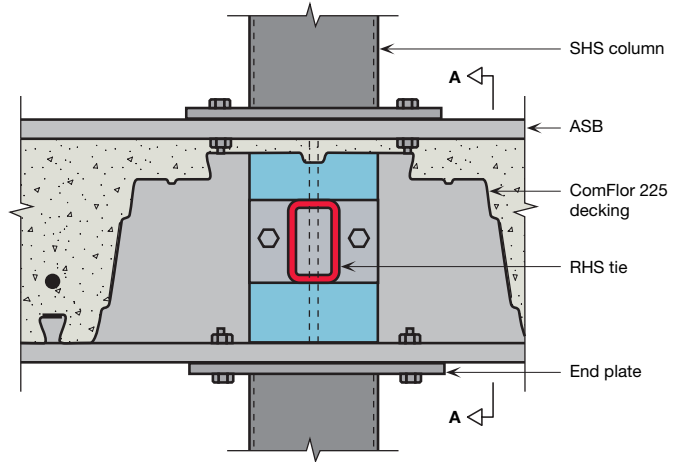
Flowdrill or Holo-Bolt connections may be used to directly bolt end plates and double-angle cleats to columns. For further details, see Corus Tubes publication TD384^[36].

2.6.2 Discontinuous columns

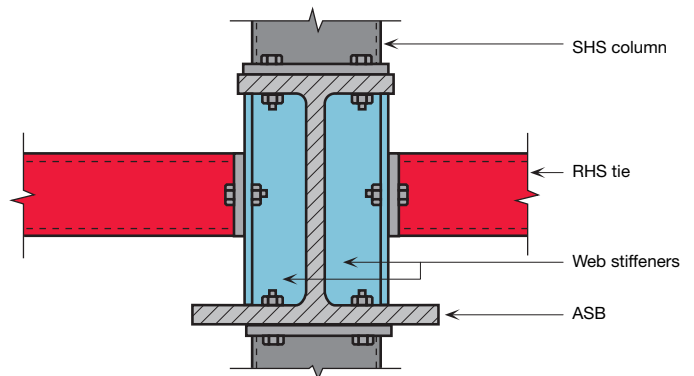
As an alternative to conventional columns, where the columns are continuous over the height of the building and the beams are attached to the faces of the columns, discontinuous storey-high members can be used with continuous floor beams in the Slimdek system. Each column length sits on top of a continuous 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^[37], AD 283^[38], AD 285^[39], AD 288^[40] and AD 292^[41]. 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^[6] for the permanent condition, in order to ensure stability in the temporary condition. Advisory Note AD 288^[40] provides guidance on load path and structural integrity, and on the verification of the connection components in accordance with BS 5950-1^[6]. It has been demonstrated by a test programme that the resistances achieved in practice are greater than the capacities determined in accordance with of BS 5950-1^[6] and the design procedures in AD 288^[40].



(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^[42] 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^[6].

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^[6] 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^[6] 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^[6] 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^[29].

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.

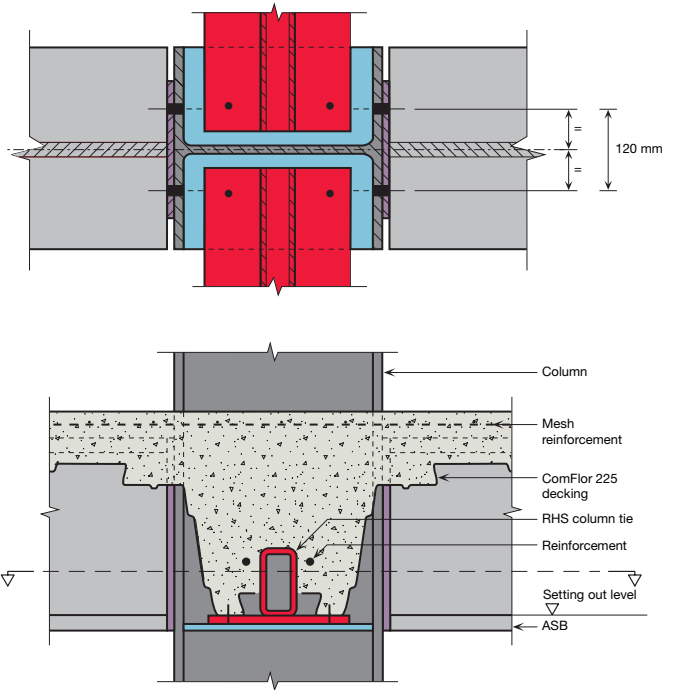


Figure 2.28 RHS tie with welded deck support plate

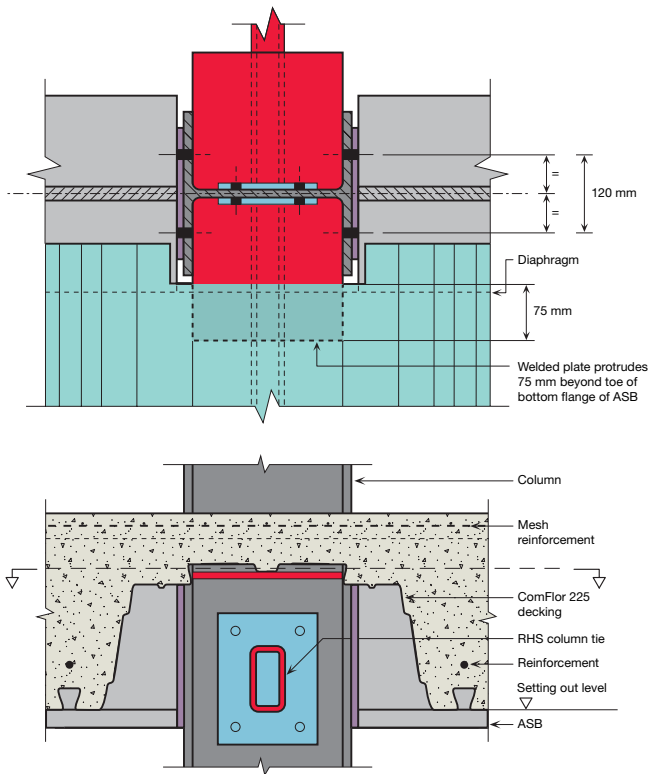


Figure 2.27 RHS tie between ribs of ComFlor 225

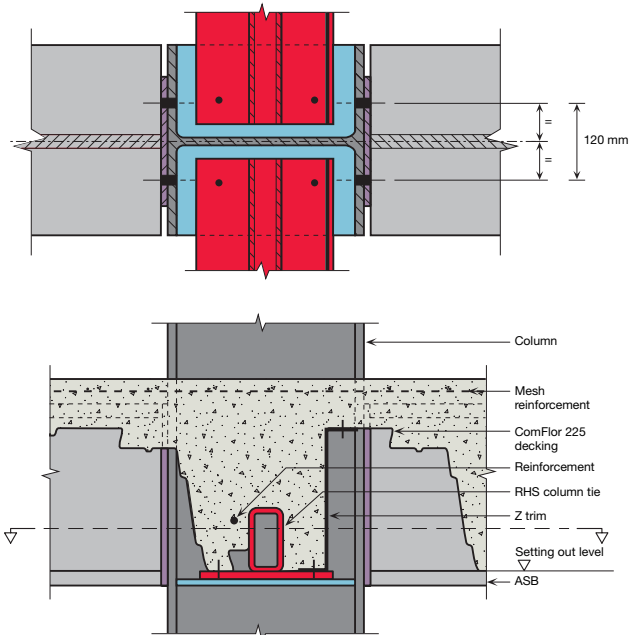


Figure 2.29 RHS tie with welded deck support plate and cut decking

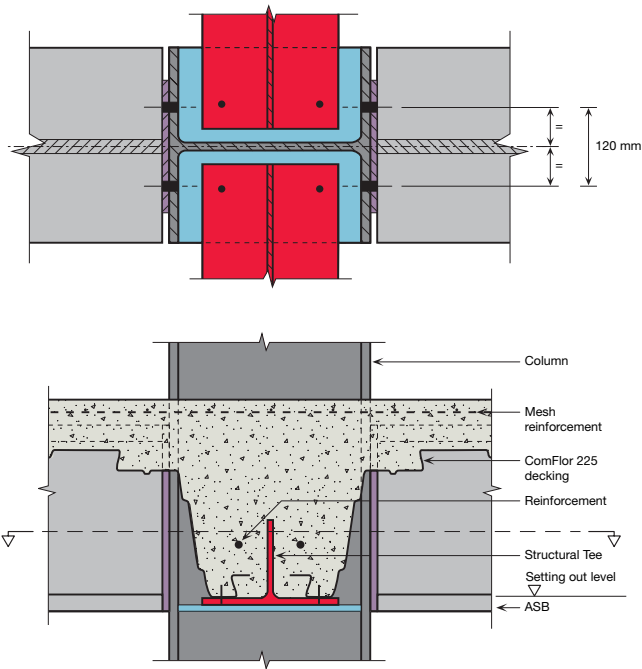


Figure 2.30 Inverted structural Tee tie

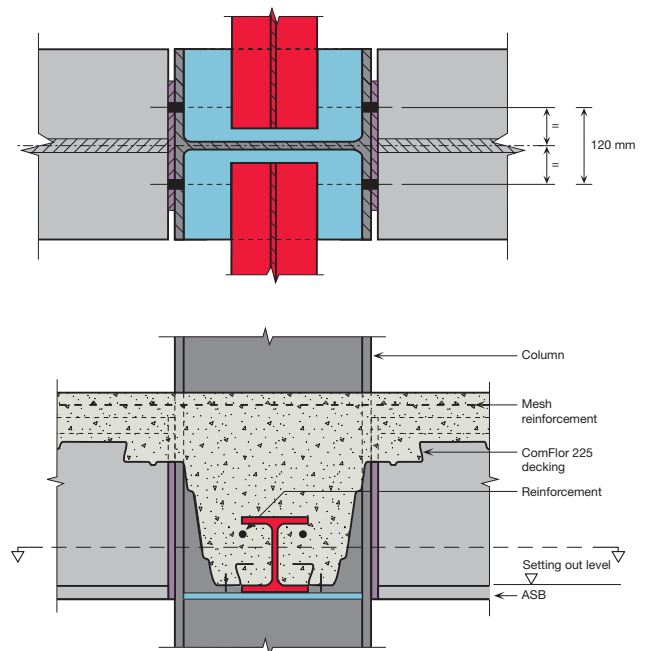


Figure 2.32 UKB or UKC tie with bottom shelf plate

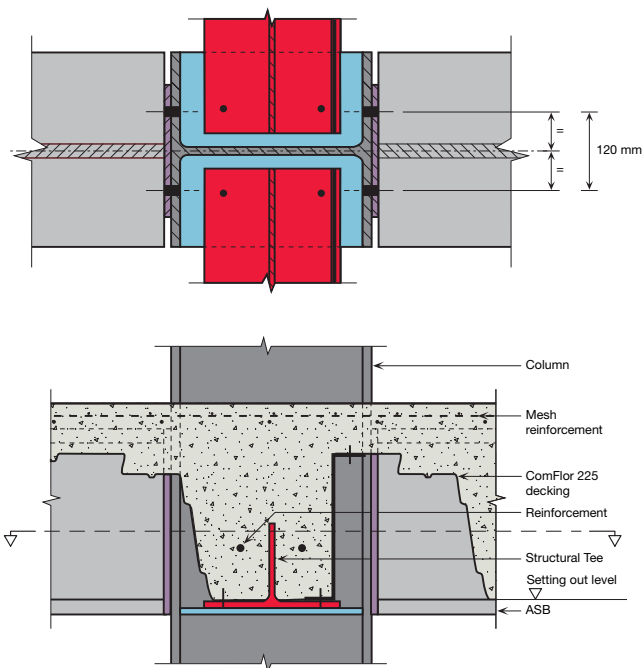


Figure 2.31 Inverted structural Tee tie with cut decking

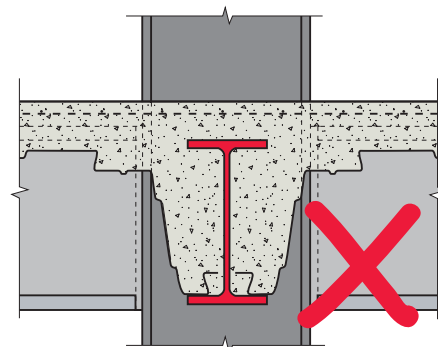


Figure 2.33 Detail using I beam – not recommended

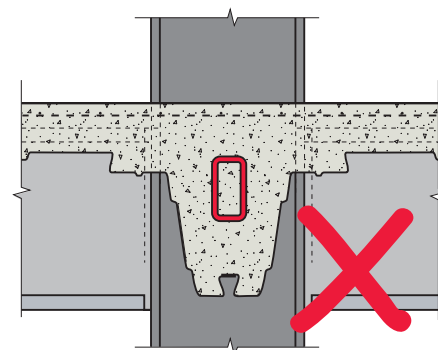


Figure 2.34 Detail using embedded member – not recommended

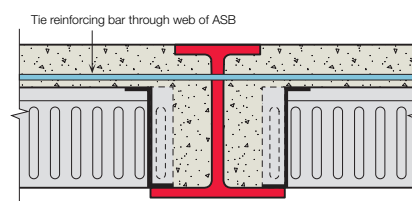


Figure 2.35 Tie bar reinforcement detail for internal beams with no concrete cover

