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.

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:	**PASS**	**Max Unity Factor = 0.68**
Normal Stage:	**PASS**	**Max Unity Factor = 0.87**
Fire design:	**PASS**	**Max Unity Factor = 0.94**
Serviceability Limit State:	**SATISFACTORY**	**Max Unity Factor = 0.75**
Deflections:	**SATISFACTORY**	
	***** SECTION ADEQUATE *****	

INPUT DATA

MAIN DATA:

Construction with SD 225	Design type COMPOSITE
Beam will NOT be propped in construction	Decking will be NOT propped in construction
Decking PERPENDICULAR to beam (SIDE 1)	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		

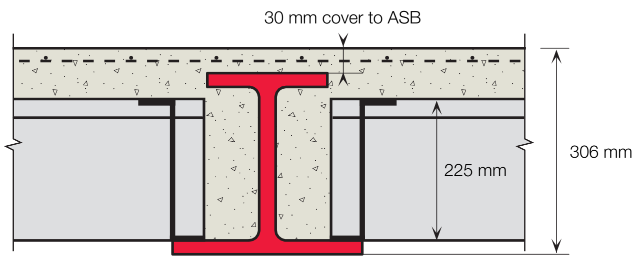
*** No secondary beams provided within the span of the beam

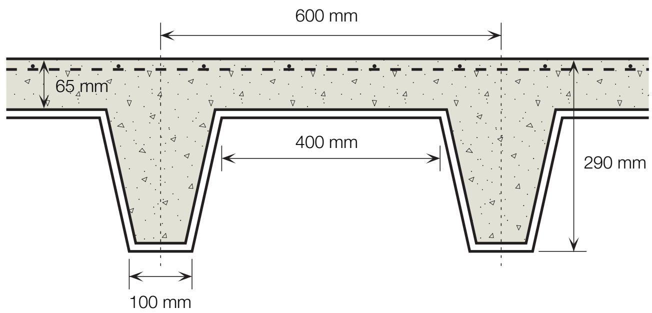
PROFILE DATA: (SD225)

Depth	225mm	Pitch of deck ribs	600mm
Trough width	100mm	Crest width	400mm
Deck weight	0.20 kN/m^2		

***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^2	Concrete depth above beam flange	30mm
Wet density	1900 kg/m^3	Dry density	1800 kg/m^3
Modular ratio	15	Mesh reinforcement	A142
Overall slab depth	290 mm	Yield strength of mesh reinf't	460.0 N/mm^2

Non-structural screeds 0mm

LOADS ACTING ON BEAM:

Occupancy imposed loads	3.5 kN/m^2	Partition loads	1.0 kN/m^2
Ceilings, services and finishes	0.5 kN/m	Construction load	0.5 kN/m^2
Natural frequency limit	4.0 Hz	Screed self-weight	0.0 kN/m^2

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^2
Fire protection is NOT provided		Fire partial safety factor	0.80
Occupancy loads considered non-permanent	100%	Permanent imposed loads	0.0 kN/m^2

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^2
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^4		Iyy	= 4245cm^4
Elastic Modulus (top)	Zt	= 995cm^3	(bottom)	Zb	= 1291cm^3
Radius of gyration	rx	= 11cm		ry	= 5.8cm
Plastic Modulus	Sx	= 1294cm^3	Elastic modulus (minor axis)	Zy	= 289cm^3
Torsional constant	J	= 160cm^4	Cross section area	A	= 127.8cm^2
Warping constant	Iw	= 450943cm^6	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^3

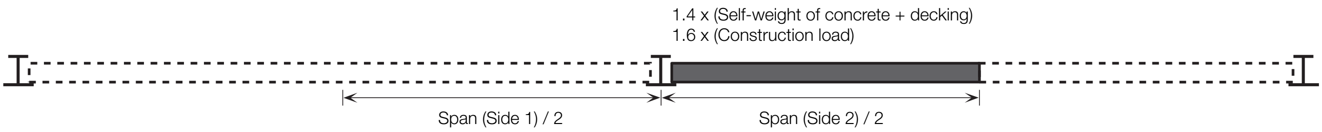
CONSTRUCTION STAGE – ULTIMATE LIMIT STATE CHECKS

FLOOR LOADS: (unfactored)

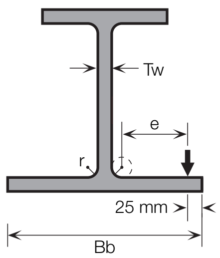
Self-weight of beam	= ASB weight /m spread over supported floor width of 6m
	= 100.3 x 9.81 / 6.0 / 1000
	= 0.16 kN/m^2
Self-weight of in-situ concrete	= Cross section area x wet density of concrete
	= 2.39 kN/m^2
Construction load	= 0.5 kN/m^2
Self-weight of steel decking	= 0.20 kN/m^2
Total load (factored)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

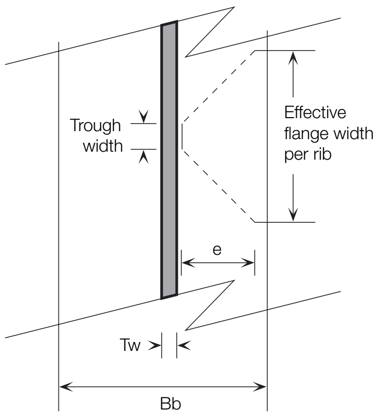
CS: BOTTOM FLANGE PLATE BENDING CHECK:

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 x (2.39 + 0.2) + 1.6 x 0.5) x 3 x 7 = 92.98 kN





Eccentricity	= (Bb - Tw) / 2 – r / 2 – 25
	= (294.0 - 19.0) / 2 - 24.0 / 2 - 25.0
	= 100.5mm
Bottom flange transverse moment	= 92.98 * 100.5 / 1000
	= 9.34 kNm
Effective flange width per rib	= Minimum of pitch of decking ribs or (Trough width + 2 x eccentricity)
	= 600mm or 100 + 2 x 100.5
	= 301mm

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

Total flange bending capacity	= 1.2 x 7.0 x 1000 x 301 / 600 x 16.0^2 / 6 x 345.0 / 10^6	
	= 62.03 kNm	
UNITY FACTOR	= 9.34 / 62.03	
	= 0.15	**PASS**

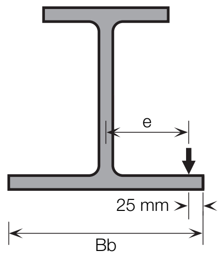
if the ratio M_t / M_{pb} >0.3 the design bending stress will need to be reduced to take account of the interaction between axial tension and transverse bending.

The reduction factor is calculated from $\frac{\sigma_1}{p_y} = \left[1 - 0.52\left(\frac{M_t}{M_{pb}}\right) - 0.48\left(\frac{M_t}{M_{pb}}\right)^2\right]^{0.5}$ (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



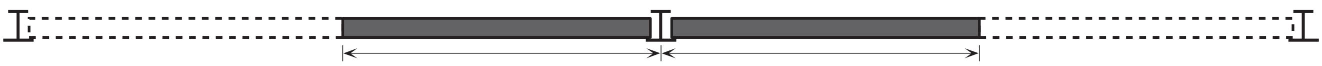
Eccentricity = 294.0 / 2 - 25.0
= 122.0 mm
Web moment = 92.98 x 122.0 / 1000
= 11.34 kNm
Effective web width per rib = 600mm or 100 + 2 x 122
= 344mm

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 pyZ.

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 Mbar = UDL x L / 8 where L = 7m
= 171.15 kNm

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

$\lambda_{LT} = u \times \upsilon \times \lambda \times (\beta_w)^{0.5}$ where $\lambda = L_e/r_y$ = 7000 / 57.6 = 121.45
u = Buckling parameter from section data tables = 0.815
β_w = 1.0 for compact sections

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

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

x = Torsional index from section data tables = 13.1
λ/x = 121.45 / 13.1 = 9.25

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

Where ψ = -0.638 using the method from Annex B of BS 5950–1:2000

ν = 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 = 196.9N/mm^2 for λ_{LT} = 76.63 and py = 345

From Cl 4.3.6.4 BS 5950–1:2000 Buckling capacity M_b = p_bS_x

= 196.9 x 1294 x 10^{-3}

Buckling capacity = 254.8 kNm

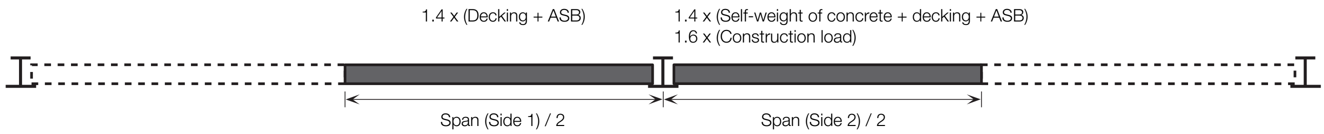
[Note: The value calculated by SIDS is 251.14 kNm. The variance from the hand-calculated value is due to the precision of constants, rounding of numbers and use of equations rather than look-up tables.]

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 kN

Point loading = 0.00 kN

Maximum moment (major axis) = UDL x L / 8

= 94.94 kNm (at midspan)

Stress σ_{bx} = Maximum moment / Elastic modulus (top)

= 94.94 / 994.5 * 1000

= 95.5 N/mm^2

Torque = Out of balance load x eccentricity from beam centre line

= {(1.4 x 2.39 + 1.6 x 0.5) x 3 x 7} x 0.122

= 10.63 kNm

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

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

Where H = Warping constant = 450943 cm^6

J = Torsional constant = 160.45 cm^4

E = Elastic modulus = 205000 N/mm^2

G = Shear modulus = E / {2(1 + ν)} where ν = 0.3

$a = (2.6 \times 450943/160.45)^{0.5}$ = 85.5cm

Hence L/a = 7000 / 855 = 8.189

ϕ is determined using Graph 1 at the end of Appendix B, (see also Appendix B5 of SCI–P–175)

ϕGJ/Tqa = 0.903 so $\phi = (0.903 \times 10.63 \times 10^6 \times 85.5) / (7900 \times 160.5 \times 10^4)$

Rotation = 0.065 rad

Max. moment (minor axis) = Rotation x Maximum major axis moment

= 0.065 x 94.94

= 6.16 kNm

Stress σ_{by} = Max. minor axis moment / Elastic modulus about yy axis

= 6.16 / 461.4 x 1000

= 13.3 N/mm^2

ϕ'' is determined using Graph 2 at the end of Appendix B where $-\phi''GJa/Tq = 0.118$ or $-\phi'' = 0.118 \times Tq / GJa$
Consequently, $-\phi'' = (0.118 \times 10.63 \times 10^6) / (7900 \times 160 \times 10^4 \times 856)$
ϕ'' $= 1.16 \times 10^{-8}$
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} $= (184 \times (216.8 - 8)) / 2 = 19210$
Stress ϕ_w $= 205000 \times 19210 \times 1.16 \times 10^{-8}$
$= 45.7\ N/mm^2$
Buckling check:
The unity factor for this check is derived from $$\frac{M_x}{M_b} + \left(\frac{\sigma_{byt} + \sigma_w}{p_y}\right) \chi \left(1 + \frac{0.5\chi M_\chi}{M_b}\right)$$

where Mx = Max major axis moment, Mb is the buckling capacity of the ASB,

UNITY FACTOR = 94.9 / 251.1 + (13.4 + 45.7) /345.0 * (1 + 0.5 *94.9 / 251.1)
= 0.58 **PASS**

Local capacity check:
Total stress σ = 95.5 + 13.4 + 45.7 = 154.5

UNITY FACTOR = 154.5/ 345.00
= 0.45 **PASS**

NORMAL STAGE – ULTIMATE LIMIT STATE CHECKS

FLOOR LOADS: (unfactored)

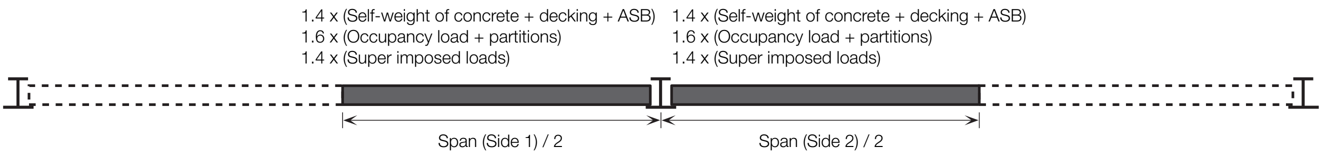
Dead: (see construction stage load calculations for details)
Self weight of beam = 0.16 kN/m^2
Self weight of in-situ concrete = 2.39 x 1800.00 / 1900.00
= 2.27 kN/m^2
Self weight of steel decking = 0.20 kN/m^2

Live:
Occupancy load = 3.50 kN/m^2
Partitions = 1.00 kN/m^2
Total imposed load = 4.50 kN/m^2 (*** no BS6399 imposed load reduction)

Super-imposed dead:
Ceilings and services = 0.50 kN/m^2
Screeds = 0.00 kN/m^2

Total load (factored) = (1.4 x (0.16+2.27+0.2) + 1.6 x 4.5 + 1.4 x (0.5+0)) x 6 x 7 +1.4 x 7 x 0
= 486.4 kN

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:

a) in the top flange of the steel section
occurs when $R_w + R_b + R_t \ge R_c (D_c/D_s) \ge R_w + R_b + R_t$

b) in the steel web within the solid concrete slab
occurs when $D_c \le y_c \le D_s$ where $y_c = (R_b + R_w + R_t + 2(D_c + T_t)R_w/d)/(2R_w/d + R_c/d)$

c) in the steel web below the solid slab
occurs when $R_t + R_c < R_b + R_w(d-2(D_s + D_c + T_t))/d$

where Rb and Rt are the resistance of the bottom and top flanges of the ASB, Rw is the total resistance of the web (Rwt and Rwb representing the top and bottom parts of the web respectively) and Rc is the resistance provided by the concrete section.

Resistance of btm flange (Rb)	= Bb x Tb x Py / 1000 = 294 x 16 x 345 / 1000 = 1622.88 kN
Resistance of top flange (Rt)	= Bt x Tt x Py / 1000 = 184 x 16 x 345 / 1000 = 1015.68 kN
Resistance of web (Rw)	= {Atot – (Bb x Tb + Bt x Tt)} x Py / 1000
	= {12780 – (294 x 16 + 184 x 16)} x 345 / 1000 = 1770.0 kN
Resistance of concrete	= 0.45 x fcu x Be x Ds / 1000 = 0.45 x 30 x 875 x 65 / 1000 = 767.81 kN

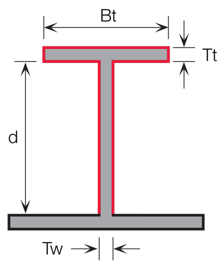
In this instance, case c) applies and the plastic neutral axis falls within the web of the ASB below the solid concrete slab. Full resistance of the concrete can be assumed provided that the calculated value for Rc is exceeded by the available longitudinal shear resistance, i.e $F_{sb} > R_c$

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 x bonded perimeter of ASB x span / 4

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

Bonded perimeter = 2 x (Be +d + Tt) – Tw = 2 x (184 + 244 + 16) – 19 = 869mm



F_{sbb} = 0.6 x 869 x 7000 / 4 / 1000 = 912.5 kN

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

F_{sbs} = $0.7 \times A_{sv} \times f_{ys} + 0.03 \times \eta \times A_{cv} \times f_{cu}$

[This is limited by the code to = $0.8 \times \eta \times A_{cv} \times f_{cu}^{0.5}$ = 449.75kN]

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

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

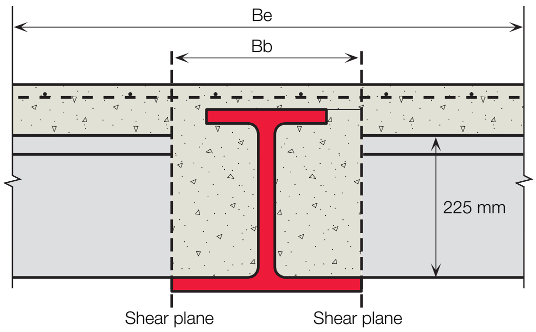
η = Factor, 0.8 for lightweight concrete

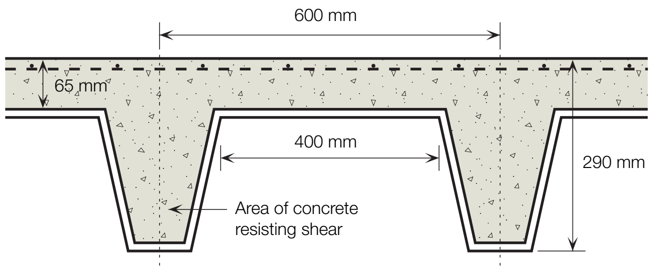
A_{cv} = Area of concrete resisting shear (128300mm^2/m)

f_{cu} = Concrete strength (30N/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 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 $B_e/(B_e-B_b)$.

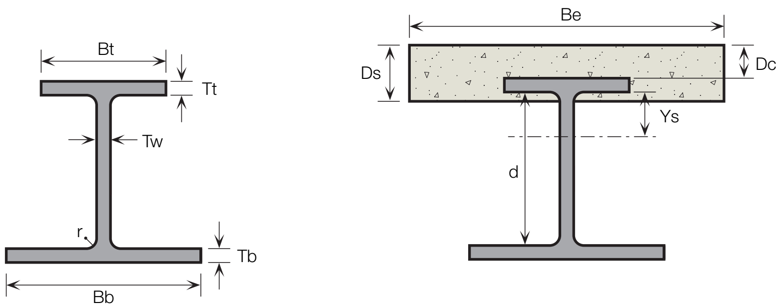
The resistance for 2 shear planes and an elastic distribution over the length of the beam is

F_{sbs} = 2 x 138.1 x 875 / (875-294) x 7000 / 4 = 727.94 kN

End effects are taken into account by the L / (L – 2 trough widths) so that the total resistance due to concrete and reinforcement shear = 7 / 5.8 x 727.94 = 878.54 kN

$F_{sbs} < F_{sbb}$ so F_{sb} = 878.54 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.

$$Rb + Rwb = Rt + Rwt + Rc$$

$$Rwb = Rw \text{ x } \left(\frac{d - ys}{d}\right) \text{ and } Rwt = Rw \text{ x } \frac{ys}{d}$$

$$Rb + Rw \text{ x } \left(\frac{d - ys}{d}\right) = Rt + Rw \text{ x } \frac{ys}{d} + Rc$$

$$\frac{ys}{d} = \left(\frac{Rb + Rw - Rt - Rc}{2\,Rw}\right)$$

Therefore,

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

d = 24.4cm so ys = 11.093cm

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

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

Where
$M_{top\ flange}$ = Rt x (ys + Tt/2)
$M_{btm\ flange}$ = Rb x (d - ys + Tb/2)
$M_{concrete}$ = Rc x (ys + Tt + Dc –Ds/2)

$$M_{web} = Rw \text{ x } d \text{ x } \left\{\left(\frac{ys}{d}\right)^2 - \left(\frac{ys}{d}\right) + 0.5\right\}$$

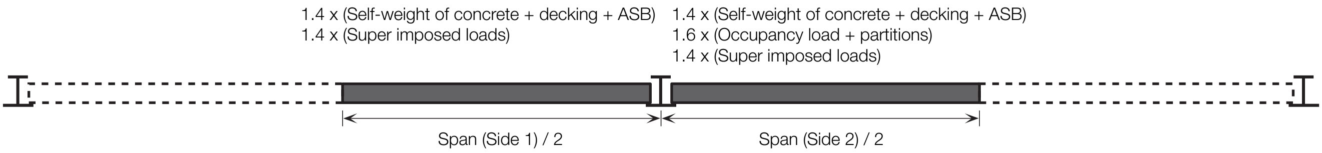
$$\begin{aligned} Mp &= 1015.68 \text{ x } (11.093 + 1.6 / 2) \\ &+ 1622.88 \text{ x } (24.4 - 11.093 + 1.6 / 2) \\ &+ 767.81 \text{ x } (11.093 + 1.6 + 3 - 6.5 / 2) \\ &+ 1770.54 \text{ x } 24.4 \text{ x } \left\{\left(\frac{11.093}{24.4}\right)^2 - \left(\frac{11.093}{24.4}\right) + 0.5\right\} \\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

In steel units, the plastic modulus Sx = M / Py = 554.13 x 1000 / 345 = 1606.2cm^3

[Note: The plastic modulus for the composite section as calculated by SIDS is 1593.2cm^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.]

UNITY FACTOR = 425.6 / 554.13
= 0.77 **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 σ_{bx} = 293.30 / 1948.3 x 1000
= 150.5 N/mm^2
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 x 1.6 x 7 x 3 = 151.20 kN
Torque = 151.2 x 0.122 = 18.45 kNm
From previous calculations, L/a = 7000 / 856 = 8.189
Rotation ϕ = 0.11287 rad
Max. moment (minor axis) = 0.11287 x 293.30
= 33.10 kNm
Stress σ_{by} = Max. minor axis moment / Elastic modulus about yy axis
= 33.10 / 461.4 x 1000
= 71.8 N/mm^2

ϕ'' = 2.0136 x 10^{-8}
Stress σ_w = E x W_{NO} x ϕ'' = 205000 x 19210.7 x 2.0136 x 10^{-8}
= 79.3 N/mm^2
Local capacity check:
Total stress σ = 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 x 3.5) x 7 x 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 (yc 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 kN
Bond Stress	= Concrete force / (bond perimeter x span / 4)
	= 926.04 x 10^3 / (869 x 7000 / 4)
	= 0.61 N/mm^2 (Bond stress limit = 0.9 N/mm^2)
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 oC	yc mm	Strength N/mm^2	Stress 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.

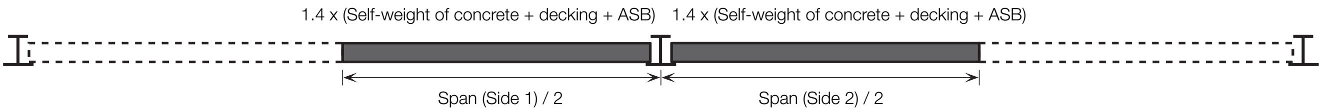
SERVICEABILITY LIMIT STATE CHECKS

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.



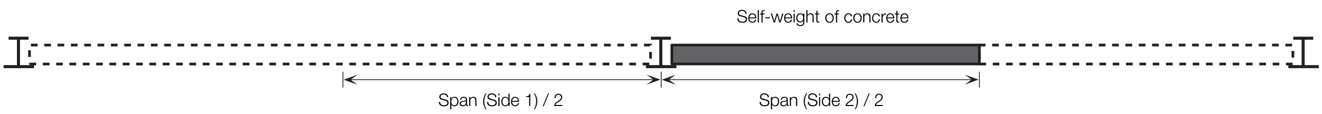
Uniformly distributed loading = (0.2 + 2.27 + 0.16) x 6 x 7 = 110.43 kN

Deflection $= \left(\dfrac{5WL^3}{384EI}\right) = \left(\dfrac{5x110.43x10^3\ x7000^3}{384x205000x15506x10^4}\right)$

= 15.52mm

Point loading = 0.00 kN
Deflection = 0.00 mm
Total deflection = 15.52 + 0.00
= 15.52mm

HORIZONTAL DEFLECTION

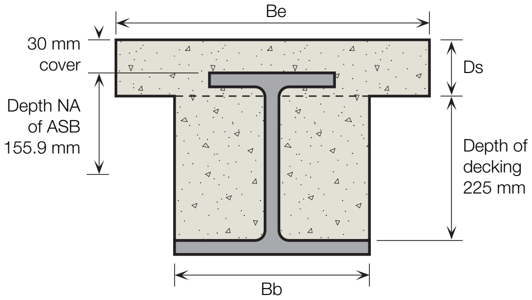


Net torsional loading = 2.27 x 3 x 7 = 47.57 kN
Torque = 47.57 x (294 / 2 - 25) / 1000
= 5.8 kNm
Distance to shear centre = 216.8mm (from beam top flange)
From previous calculations, L/a = 7000 / 856 = 8.189
Rotation = 0.03538 rad
Deflection = 0.03551 x 216.8
= 7.70mm (< SPAN / 500 = 14.0mm) **SATISFACTORY**

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 = 12780mm^2
	Lever arm = 155.9mm
Concrete flange:	Area = Be x Ds = 875 x 65 = 56875mm^2 = 3791.667mm^2 in steel units
	Lever arm = Ds / 2 - cover = 65 / 2 - 30 = 2.5mm
Concrete encasement	Area = Bb x 225 = 294 x 225 = 66150mm^2 = 4410mm^2 in steel units
	Lever arm = Depth of ASB – Tb – Depth of decking / 2 = 276 – 16 –225 / 2 = 147.5mm
Depth to composite NA	= (12780 x 155.9 + 3791.667 x 2.5 + 4410 x 147.5) / (12780 + 3791.667 + 4410)
	= 126.41mm below the top of the ASB.

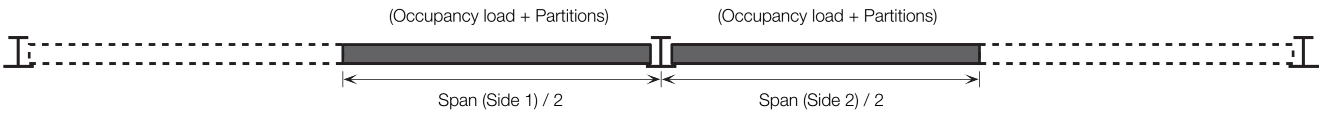
The composite inertia is determined by taking moments of areas about the composite neutral axis depth.

Inertia of ASB $= 15506 + (12780 \times (155.9 - 126.41)^2) / 10000 = 16617.52\text{cm}^4$

Inertia of flange $$= \left\{\left(\frac{875 \times 65^3}{12}\right) + (875 \times 65) \times (126.41 - 2.5)^2\right\} / (15 \times 10000) = 5955.76$$

Inertia of encasement $$= \left\{\left(\frac{294 \times 225^3}{12}\right) + (294 \times 225) \times \left(276 - 16 - \frac{225}{2} - 126.41\right)^2\right\} / (15 \times 10000) = 2056.49$$

Inertia (composite section) $= 16617.52 + 5955.76 + 2056.49 = 24630\text{cm}^4$ (In steel units)



Uniformly distributed loading = (3.5 + 1) x 6 x 7 = 189.00 kN

Deflection $$= \left(\frac{5 \times 189 \times 10^3 \times 7000^3}{384 \times 205000 \times 24630 \times 10^4}\right) = 16.72\text{mm}$$

Point loading = 0.00 kN

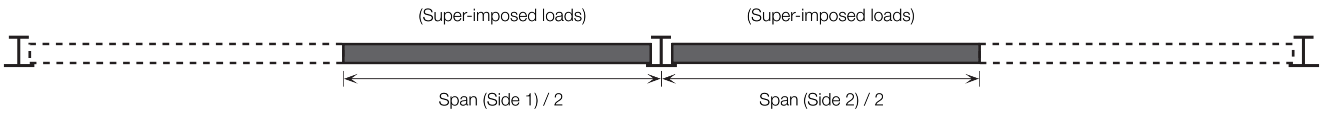
Deflection = 0mm

Total deflection = 16.72 + 0

= 16.72mm (< SPAN / 360 i.e 19.4mm) **SATISFACTORY**

DEFLECTION DUE TO SUPER-IMPOSED DEAD LOADS

Inertia (composite section) = 24630cm4



Uniformly distributed loading = 0.5 x 6 x 7 = 21.00 kN

Deflection $= \left(\dfrac{5x21x10^3\ x7000^3}{384x205000x24630x10^4} \right) = 1.86\text{mm}$

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^4 ** 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 $= \left(\dfrac{5x150.3x10^3\ x7000^3}{384x205000x28494x10^4} \right) = 11.49\text{mm}$

Point loading = 0.00 kN

Deflection = 0.00mm

Total deflection = 11.49 + 0.00

= 11.49mm

Frequency = 18 / sqr(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^4
Steel modulus (top)	= 995cm^2
Compression	= 96.6 x 1000 / 995
	= 97.2 N/mm^2
Steel modulus (btm)	= 1291cm^3
Tension	= 96.6 x 1000 / 1291
	= 74.8 N/mm^2

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 = 149.6mm from the soffit of the ASB beam

Moment of inertia (composite section) = 24630cm^4 (expressed in steel units)

This modulus value assumed a modular ratio of 15, so the equivalent inertia in concrete units is 369450cm^4

The modulus of the composite section in concrete units = Inertia / distance from NA to top of slab

Concrete modulus	= 369450 / 15.64 = 23620cm^3	
Concrete compressive stress	= 183.8 x 1000 / 23620	
	= 7.8 N/mm^2	
UNITY FACTOR	= 7.78 / (1 * 30.0)	
	= 0.26	**PASS**
Steel modulus (top)	= 24630 / 12.641 = 1948cm^3	
Steel stress (top - compression)	= 183.75 x 1000 / 1948	
	= 94.3 N/mm^2	
Steel modulus (btm)	= 24630 / (27.6 - 12.641) = 1646cm^3	
Steel stress (btm - tension)	= 183.8 / 1646 x 1000	
	= 111.6 N/mm^2	

STEEL STRESS CHECK (total stresses)

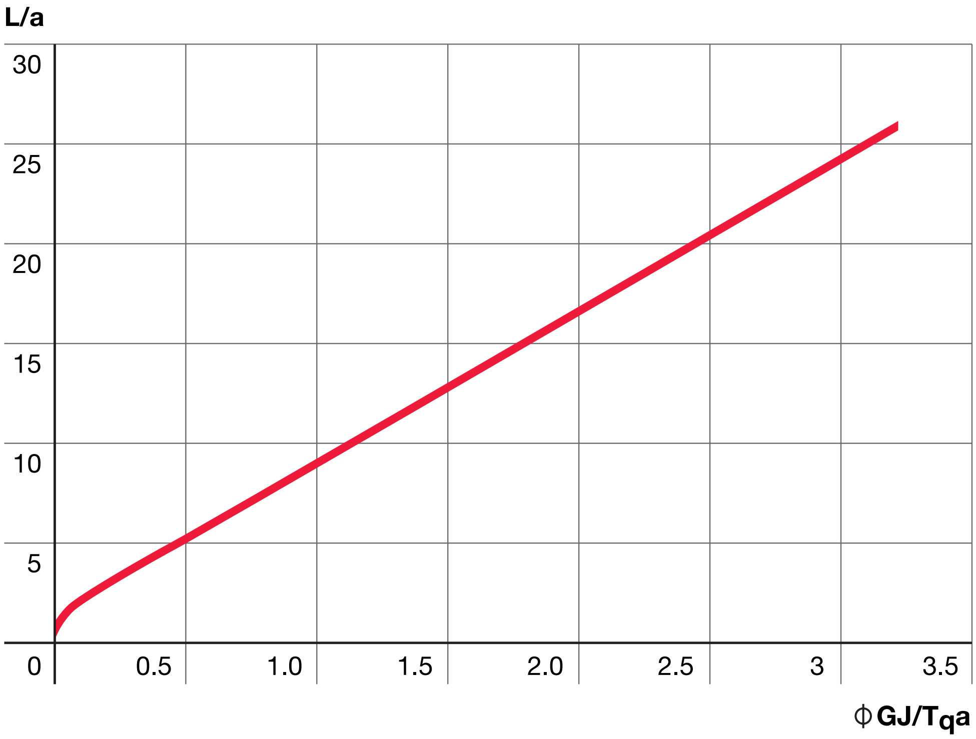
Compression (at top of beam)	= 97.16 + 94.31	
	= 191.47 N/mm^2	
UNITY FACTOR	= 191.47 / 345.0	
	= 0.55	**PASS**
Tension (at bottom of beam)	= 74.84 + 111.60	
	= 186.44 N/mm^2	
UNITY FACTOR	= 186.44 / 345.0	
	= 0.54	**PASS**

BOTTOM FLANGE COMBINED STRESS CHECK

*** No direct loads on ASB beam bottom flange ***

*** check not required ***

Graph 1: Determination of the angle of rotation ϕ



Graph 2: Determination of second derivative of ϕ