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Introduction

The report details the structural design of a sub-structure within a proposed 6-storey student residence at Quarryfield Rd, Gateshead NE8 3BE, featuring a steel-framed construction with a single-level basement car park, central atrium, and four core areas for vertical circulation and services. The building, designed in accordance with Eurocodes 3 and 4, has a square footprint with curtain wall glazing on all elevations, 3.5-meter floor-to-floor height, and requires careful consideration of the structural grid for both aesthetic and functional purposes. The detailed design specifically focuses on three critical components extracted from the global structural model: a primary vertical member (column), a horizontal member (beam), and the connection detail between these elements, all developed through collaborative group work during the concept and developed design phases.

1.1. The sub-structure

The substructure selected is located on the ground floor of the building along grid 4 between grid A and B as shown in Figure 1. It was chosen based on its numerous instances across the overall structural system. It entails a single span 10 m 457 × 191 × 67 UKB secondary beam loaded by a 3.75 m strip (140 mm thick) of Tata Steel ComFlor® 60 profiled steel decking slab and simply supported on either side by two 305 × 305 × 97 UKC columns. The support conditions are nominally pinned the beam connected to the columns on the web. Universal beams and universal columns are grade S355 while the endplate and bolts which are of grade S275. As the building is a simply supported non-sway frame, only gravity loads are considered in the design of the beam and columns. The beam acts compositely, its design moments obtained from SCI P394 rather than the model to incorporate changes outlined in SCI AD 346.

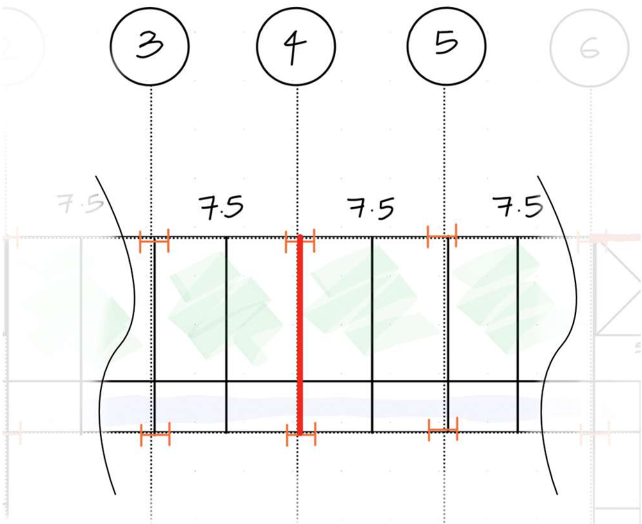


Figure 1: Sub-structure selection

1.2. Standards

The following standards were used for the analysis and design of the sub-structure:

- BS EN 1991−1−1:2002: Eurocode 1. Actions on structures: General actions. Densities, self−weight, imposed loads for buildings
    - NA to BS EN 1991−1−1:2002: UK National Annex to Eurocode 1. Actions on structures. General actions. Densities, self−weight, imposed loads for buildings. Densities, self−weight, imposed loads for buildings
- BS EN 1993−1−1:2005+A1:2014: Eurocode 3. Design of steel structures. General rules and rules for buildings

- NA+A1:2014 to BS EN 1993−1−1:2005+A1:14: UK National Annex to Eurocode 3. Design of steel structures. General rules and rules for buildings

1.3. Design forces and deformations

Design forces and deformations from the finite element model are summarised in Table 1.

Element	Load Case	Symbol	(kN/m^2)	Reference
Beam	Shear force, kN	V_{Ed}	1.50	Figure 2
	Bending moment, kNm	M_{Ed}	3.00	Figure 3
	Deflection, mm		47.60	Figure 4
Column	Axial load, kN	N_{Ed}	2759.40	Figure 5

Table 1: Design loads and deformations

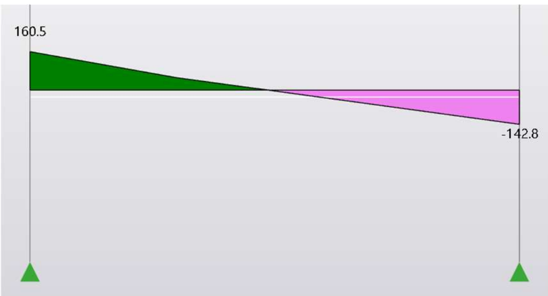


Figure 2: Beam shear forces

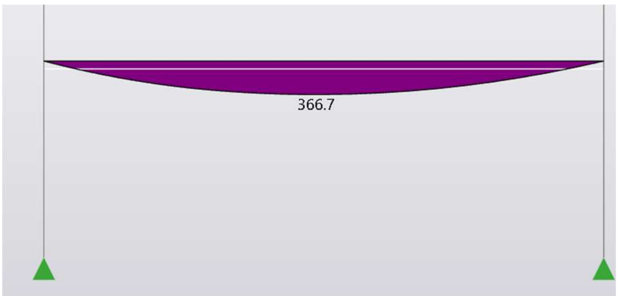


Figure 3: Beam bending moments

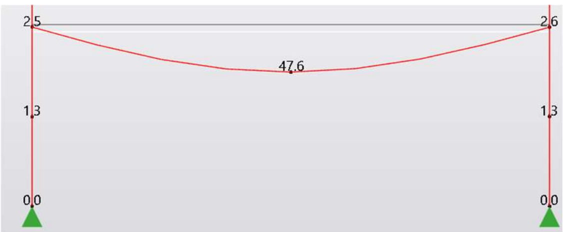


Figure 4: Beam deflection

Figure 5: Column axial loads

2. Composite Beam Design

In designing composite floors, adjustments are necessary to account for differences in material properties and loading conditions. The Steel Construction Institute notes that the 1 kN/m^3 increase in concrete density due to reinforcement, as per BS EN 1991-1-1, is suitable for reinforced concrete but not for composite floors using lighter mesh reinforcement (SCI, 2010). Fresh concrete weight is treated as a variable action with partial factor γ_Q, while reinforcement weight is treated as a permanent action. SCI recommends load values of 24 kN/m^3 for dry normal weight concrete, 19 kN/m^3 for dry lightweight aggregate concrete, 25 kN/m^3 for wet normal weight concrete, and 20 kN/m^3 for wet lightweight aggregate concrete, with reinforcement weight determined case-by-case (SCI, 2010). BS EN 1991-1-6 adopts a construction load factor $\psi_0 = 1.0$, making expressions 6.10 and 6.10a identical but requiring stricter evaluation of expression 6.10a using $\xi = 0.925$ from the UK National Annex (SCI, 2010).

During concreting, self-weight components (excluding concrete) are factored by 1.35, and variable actions by 1.5; design effects at the ultimate limit state are calculated using specific expressions that account for construction loads (e.g., personnel, heaping of concrete, ponding), decking self-weight, and temporary states as defined in BS EN 1991-1-6 (SCI, 2010). Loads relevant to the substructure chosen are listed in Table 2. Information regarding the composite slab has been obtained from TATA steel catalogue (TATA, 2024).

Area	Load Case	Load	Value (kN/m^2)
Internal floor	Imposed	A1 i.e. All usages within self–contained dwelling units	1.50
		C31 Corridors, hallways, aisles in institutional type buildings	3.00
	Permanent	Services	0.10
		Suspended ceiling	0.20
		140 mm ComFlor® 60 Composite slab (dry)	2.71
		140 mm ComFlor® 60 Composite slab (wet)	2.60
		0.9 mm ComFlor® 60 (S350)	0.10
		A193 mesh	0.03

Table 2: Loads for analysis and design of composite beam

2.1. Floor details

Beam span	L	= 10 m
Beam spacing	B	= 3.75 m
Profiled steel sheeting	Tata Steel ComFlor® 60	
Sheeting profile height to shoulder,	h_p	= 60 mm
Overall height of sheeting profile,	h_d	= 75 mm
Depth of concrete above profile,	h_c	= 65 mm
Slab depth,	h_s	$= h_c + h_d = 140$ mm
h_c based on overall height of profile.		

2.2. Shear connectors

Single 19 mm dia. × 120 mm high at 300 mm centres

Connector spacing,	$s_{conn.}$	= 300 mm
Shear connector diameter,	d_{conn}	= 19 mm
Overall (as-welded) height,	h_{sc}	= 95 mm
Ultimate tensile strength,	f_u	= 450 N/mm^2

2.3. Concrete

Normal weight concrete grade,	C25/30	
Characteristic cylinder strength,	f_{ck}	= 25 N/mm^2
Characteristic cube strength,	$f_{ck,cube}$	= 30 N/mm^2
Secant modulus of elasticity of concrete,	E_{cm}	= 31 kN/mm^2
Density of wet concrete,	ρ_{wet}	= 25 kN/m^3
Density of dry concrete,	ρ_{dry}	= 24 kN/m^3
Tata steel data sheet		
Concrete volume,	V_{CF60}	= 0.108 m^3/m^2

2.4. Reinforcement

A193 mesh

Reinforcement bar diameter,	ϕ_{reinf}	= 7 mm
Spacing of bars,	s_{reinf}	= 200 mm
Area of steel reinforcement per unit width,	A_{reinf}	= 193 mm^2/m
Self–weight of the mesh per unit area,	g_{mesh}	= 3.03 kg/m^2
Yield strength,	f_{sd}	= 500 N/mm^2

2.5. Actions

2.5.1. Construction stage

Permanent actions

Sheeting self–weight,	$g_{k,1}$	= 0.10 kN/m^2

For a 457 × 191 × 67 UKB in S355, the beam self-weight is 67.1 kg/m. Using 9.81 m/s^2 as the gravitational acceleration, its self-weight is given as 67.1 kg/m × 9.81 m/s^2 = 0.66 kN/m

Beam self–weight,	$g_{k,2}$	= 0.66 kN/m

The self-weight allowance of 1 kN/m³ for reinforced concrete but is not applicable to composite floors, which feature a comparatively lightweight mesh.

Allowance for mesh,	$g_{k,3}$	= g_{mesh} × 9.81 m/s^2 = 0.03 kN/m^2

Variable actions

The weight of fresh concrete should be considered a variable action. Partial factor γ_Q is applied, rather than γ_G.

Wet concrete self–weight,	$q_{k,1}$	= $V_{CF60} \times \rho_{wet}$ = 2.7 kN/m^2

SCI P394 recommends that designers take advantage of *1-1-6/N.A.2.13* to use “values of Q_{ca} and Q_{cc} determined for the individual project”. Thus $Q_{k,1a}$ = 0 and $q_{k,1b}$ = 0.75 kN/m^2 for the design of beams.

2.5.2. Composite stage

Permanent actions

Dry slab,	$g_{k,1}$	= $V_{CF60} \times \rho_{dry}$ = 2.592 kN/m^2
Sheeting self-weight,	$g_{k,2}$	= 0.1 kN/m^2
Allowance for mesh,	$g_{k,3}$	= 0.03 kN/m^2
Beam self-weight,	$g_{k,4}$	= 0.66 kN/m
Ceiling and services,	$g_{k,5}$	= 0.3 kN/m^2

Variable actions

Imposed floor load (residential),	$q_{k,1}$	= 1.5 kN/m^2
Corridor,	$q_{k,2}$	= 3.0 kN/m^2

2.6. Partial factors for actions

Partial factor for permanent actions,	γ_G	= 1.35
Partial factor for variable actions,	γ_Q	= 1.5
Reduction factor,	ζ	= 0.925
Partial coefficient in construction stage,	$\psi_{0,1}$	= 1.0
Partial coefficient in composite stage,	$\psi_{0,2}$	= 0.5

2.7. Construction stage at ULS

$F_{d,1}$ $= 1.35 \times 0.66 + (1.35(0.1 + 0.03) + 1.5(2.7 + 0.75)) \times 3.75$
$= 20.951$

2.8. Composite stage at ULS

To form a uniformly distributed load, the corridor load which acts over 2.5 m of the beam is converted to act over the whole beam using the relationship below,

$q_{k,2}$ $= 1.5 \text{ kN/m}^2 \times 2.5 \text{ m} / 10 \text{ m}$
$= 0.375 \text{ kN/m}^2$

Modified corridor load,
$q_{k,2}$ $= 0.375 \text{ kN/m}^2$

UDL for the beam at composite stage is calculated as;

F_d $= \zeta \gamma_G g_{k,4} + [\zeta \gamma_G (g_{k,1} + g_{k,2} + g_{k,3} + g_{k,5}) + \gamma_Q (q_{k,1} + q_{k,2})]$
$= 0.925\times(1.35)(0.66)+[0.925\times1.35\times(2.592+0.1+0.03+0.3)+1.5(1.5+0.375)]\times3.75$
$= 25.519 \text{ kN/m}$

2.9. Composite stage at SLS

The characteristic load combination will be used. Therefore, the actions for calculation of deflections are:

Permanent actions applied to steel beam

Self-weight of the slab + mesh + sheeting + steel section
g_1 $= (2.592 + 0.1 + 0.03) \times 3.75 + 0.66$
$= 10.8675 \text{ kN/m}$

Permanent actions applied to composite beam

Ceiling and services
g_2 $= 0.3 \times 3.75$
$= 1.125 \text{ kN/m}$

Variable actions applied to composite beam

q_1 $= (1.5 + 0.375) \times 3.75$
$= 7.031 \text{ kN/m}$

2.10. Design bending moments & shear forces

Construction stage

M_{Ed} $= F_{d,1} \times L^2 / 8$
$= 20.95 \text{ kN / m} \times (10 \text{ m})^2 / 8$
$= 261.891 \text{ kNm}$
V_{Ed} $= F_{d,1} \times L / 2$
$= 20.95 \text{ kN/m} \times 10 \text{ m} / 2$
$= 104.756 \text{ kN}$

Composite stage

$M_{Ed,comp}$ $= F_d \times L^2 / 8$
$= 25.52 \text{ kN/m} \times (10 \text{ m})^2 / 8$
$= 318.984 \text{ kNm}$
$V_{Ed,comp}$ $= F_d \times L / 2$

= 25.52 kN/m × 10 m / 2
= 127.594 kN

The shear force diagram and bending moment diagram are illustrated in Figure 6.

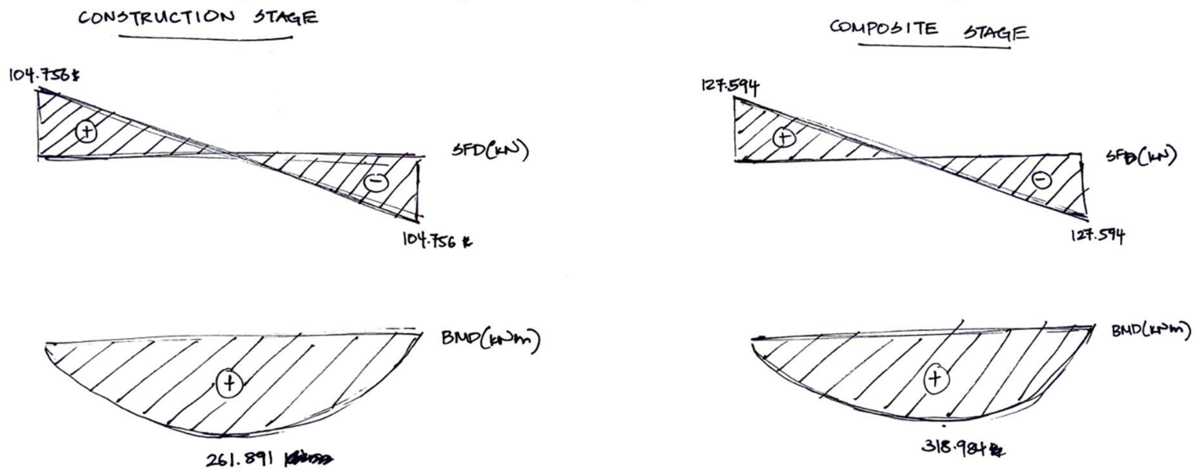


Figure 6: Composite beam design forces for construction and composite stage

2.11. Section properties

For a 457 × 191 × 67 UKB in S355 steel
From section property tables:

Depth,	h_a	= 453.4 mm
Width,	b	= 189.9 mm
Web thickness,	t_w	= 8.5 mm
Flange thickness,	t_f	= 12.7 mm
Root radius,	r	= 10.2 mm
Depth between fillets,	d	= 407.6 mm
Second moment of area y axis,	I_y	= 29400 cm^4
Elastic modulus y axis,	$W_{el,y}$	= 1300 cm^3
Plastic modulus y axis,	$W_{pl,y}$	= 1470 cm^3
Area,	A_a	= 85.5 cm^2
Modulus of elasticity,	E	= 210 kN/mm^2
Mass per metre,		= 67.1 kg/m
For S355 steel and $t \leq 16$ mm,	f_y	= 355 N/mm^2

2.12. Cross-section classification

3-1-1/Table 5.2 ε $= \sqrt{(235\ N/mm^2 / f_y)}$
$= 0.814$

Outstand of compression flange
3-1-1/Table 5.2 c_{flange} $= (b - t_w - 2 \times r)/2$
$= 80.5$ mm
c_{flange}/t_f $= 6.339$

The limiting value for Class 1 is $c / t_f \leq 9 \times \varepsilon = 7.323$

Therefore, the flange in compression is Class 1
Web subject to bending
3-1-1/Table 5.2 c_{web} $= d = 407.6$ mm
c_{web} / t_w $= 47.953$

The limiting value for Class 1 is $c / t_w \leq 72 \times \varepsilon = 58.580$

The web in bending is Class 1. Therefore, the cross-section in bending at the construction stage is Class 1. At the composite stage, the cross-section will also be Class 1.

2.13. Partial factors for resistance

Steel section

3-1-1/NA.2.15 $\gamma_{M0} = 1.00$

$\gamma_{M1} = 1.00$

2.14. Shear connector

For the resistance of a shear connector, the *UK NA to 4-1-1/2.3* adopts the recommended value of $\gamma_v = 1.25$ given in *4-1-1/6.6.3.1*: “unless stud resistances given in non-contradictory complementary information would justify the use of an alternative value” Hence, $\gamma_v = 1.25$

Concrete

For persistent and transient design situations $\gamma_c = 1.5$

Reinforcement

For persistent and transient design situations $\gamma_s = 1.15$

2.15. Design resistance for the construction stage

2.15.1. Cross-sectional resistance of the steel beam

Shear buckling

3-1-5/5.1(2) $h_w / t_w > 72 \times \varepsilon / \eta$

3-1-5/NA.2.4 $\eta = 1$

$h_w = h_a - 2 \times t_f = 428$ mm

$h_w / t_w = 50.353$

$72 \times \varepsilon / \eta = 58.580$

Shear buckling of the web does not need to be verified.

Vertical shear resistance

Verify that:

3-1-1/6.2.6(1) Eq (6.17) $V_{Ed}/V_{c,Rd} < 1.0$

For plastic design, $V_{c,Rd}$ is the design plastic shear resistance ($V_{pl,Rd}$)

3-1-1/6.2.6(2) Eq (6.18) $V_{c,Rd} = V_{pl,a,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$

A_v is the shear area and is determined as follows for rolled I and H sections with the load applied parallel to the web.

3-1-1/6.2.6(2) Eq (6.2.6(3) $A_v = A_a - 2 \times b \times t_f + t_f \times (t_w + 2 \times r)$

$= 40.936 \text{ cm}^2$

but not less than,

$\eta \times h_w \times t_w = 36.38 \text{ cm}^2$

Therefore, $A_v = 40.936 \text{ cm}^2$

Plastic shear resistance $V_{pl,a,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$

$= 839.015$ kN

Max design shear at construction stage $V_{Ed} = 104.756$ kN

$V_{Ed}/V_{pl,a,Rd} = 0.125$

Shear resistance of the cross-section is adequate.

2.15.2. Bending Resistance of Steel Beam

Verify that:

3-1-1/6.2.5(1) Eq (6.12) $M_{Ed}/M_{c,Rd} < 1.0$

$V_{pl,a,Rd} / 2 = 419.508$ kN $> V_{Ed} = 104.756$ kN

No reduction in the bending moment resistance of the steel section need be accounted for at any point along the beam.

The design resistance to bending moment for Class 1 and 2 cross sections is:

3-1-1/6.2.5(2) Eq (6.13) $M_{c,Rd} = M_{pl,a,Rd} = W_{pl,y}f_y / \gamma_{M0}$

$M_{pl,a,Rd} = W_{pl,y}f_y / \gamma_{M0}$

$= 521.85$ kNm

$M_{c,Rd} = M_{pl,a,Rd}$

$= 521.85$ kNm

$M_{Ed}/M_{c,Rd} = 0.502 < 1.0$

Therefore, the bending moment resistance is adequate.

2.15.3. Buckling resistance

The steel sheeting provides continuous restraint to the top flange of the steel beam, so the beam is not susceptible to lateral torsional buckling.

2.16. Shear connection

2.16.1. Design resistance of shear connectors

Shear connector in a solid slab

The design resistance of a single headed shear connector in a solid concrete slab, automatically welded in accordance with *BS EN 14555*, should be determined as the smaller of:

4-1-1/6.6.3.1(1) Eq (6.18) and 4-1-1/Eq (6.19)

$P_{Rd,solid} = \mathrm{Min}(0.8 \times f_u \times \pi \times (d_{conn}^2) / 4\gamma_v\ ,\ 0.29 \times \alpha \times d_\phi^2 \times \sqrt{}(f_{ck} \times E_{cm})/\gamma_v)$

4-1-1/Eq (6.21) $\alpha = 1.0$ as $h_{sc} / d_{conn} = 5 > 4$

$P_{Rd,solid} = \mathrm{Min}(0.8 \times f_u \times \pi \times (d_{conn}^2) / (4\gamma_v), 0.29 \times \alpha \times d_{conn}^2 \times \sqrt{}(f_{ck} \times E_{cm}) / \gamma_v)$

$= 73.730$ kN

Shear connectors in profiled steel sheeting

For profiled sheeting with ribs running transverse to the supporting beams $P_{Rd,solid}$ should be multiplied by the following reduction factor,

4-1-1/6.6.4.2(1) Eq (6.23)

$k_t = 0.7 \times b_0 \times (h_{sc} / h_p - 1)/(\sqrt{}(n_r) \times h_p)$

For a single stud per trough with the mesh positioned above the head of the studs.

k_{mod} = 1.0 *(Table 2.1 NCCI PN001a−GB) PN001a-GB*

Dimensions that influence calculation of stud resistance

b_0 = 145 mm
h_{sc} = 95 mm
h_p = 60 mm
n_r = 1, for one shear connector per rib

$k_t = 0.7 \times b_0 \times (h_{sc} / h_p - 1)/(\sqrt{(n_r)} \times h_p) = 0.987$

But k_t should not be taken greater than the appropriate value $k_{t,max}$ given in *4-1-1/Table 6.2*.

For shear connectors welded through the profiled sheeting, $t \leq 1.0$ mm and $n_r = 1$

$k_{t,max} = 0.85$

Therefore,
$k_{t,limit} = 0.85$

Hence, the design resistance / shear connector in a rib where there is one connecter per rib is:

$P_{Rd} = k_{t,limit} \times P_{Rd,solid} = 62.671$ kN

And the design resistance per rib is:

$n_r \times P_{Rd} = 62.671$ kN

Degree of shear connection

For composite beams in buildings, the headed shear connectors may be considered as ductile when the minimum degree of shear connection given in *4-1-1/6.6.1.2* is achieved.
For headed shear connectors with:

$h_{sc} \geq 4d$ and 16 mm $\leq d \leq$ 25 mm

The degree of shear connection may be determined from:

$\eta = N_c / N_{c,f}$ where

N_c is the reduced value of the compressive force in the concrete flange (i.e. the force transferred by the shear connectors)
$N_{c,f}$ is the compressive force in the concrete flange at full shear connection (i.e. the minimum of the axial resistance of the concrete and the axial resistance of the steel).

For steel sections with equal flanges and $L_e < 25$ m

$\eta \geq 1 - (355 / f_y) \times (0.75 - 0.03 \times L_e)$, $\eta = 0.4$

where L_e is the distance between points of zero bending moment.

For a simply supported beam:

L_e = L = 10 m

$\eta \geq 1 - ((355\ N/mm^2) / f_y) \times (0.75 - 0.03 /1\ m \times L_e)$
= 0.55, therefore η = 0.4

Degree of shear connection present

To determine the degree of shear connection present in the beam first the axial resistances of the steel and concrete are required ($N_{pl,a}$ and $N_{c,f}$ respectively).

Determine the effective width of the concrete flange

At the mid−span the effective width of the concrete flange is determined from:

$b_{eff} = b_0 + (b_{e1} + b_{e2})$

For $n_r = 1$, $b_{0,stud} = 0$ mm

$b_{ei} = L_e / 8$, but not greater than b_i
where L_e is the distance between points of zero bending moment.

Therefore, for a simply supported beam:
L_e = 10 m

b_i is the distance from the outside shear connector to a point mid-way between adjacent webs, therefore:

b_1 = B / 2 = 1.875 m
b_2 = B / 2 = 1.875 m
$b_{e1} = L_e / 8$ = 1.25 m
$b_{e2} = L_e / 8$ = 1.25 m

Hence at the mid−span the effective width of the concrete flange is:

$b_{eff} = b_{0,stud} + b_{e1} + b_{e2}$ = 2.5 m

Compressive resistance of the concrete flange

For compressive resistance of the concrete flange the depth of concrete considered is that above the top of the re-entrant top flange stiffener present. The CF60 profile has a 15 mm deep re-entrant stiffener above the top flange making the overall profile depth, h_d = 75 mm.

Compressive resistance of the concrete flange, $N_{c,f} = 0.85 \times f_{cd} \times b_{eff} \times h_c$
= 2302.083 kN

Tensile resistance of the steel member

$N_{pl,a} = f_y \times A_a$
= 3035.25 kN

Compressive force in the concrete flange

The compressive force in the concrete at full shear connection is the lesser of $N_{c,f}$ and $N_{pl,a}$.

= Min($N_{c,f}$, $N_{pl,a}$)
$N_{c,f}$ = 2302.083 kN

Resistance of the shear connectors

n is the number of shear connectors present to the point of maximum bending moment. In this example there are 18 ribs available for positioning shear connectors, per half span

n_{conn} = L / (2 × $s_{conn.}$) = L / (2 × 300) = 18

Where there is less than full shear connection, the reduced value of the compressive force in the concrete flange, N_c, is the combined resistance of the shear connectors in each half-span. Thus,

N_c = n_{conn} × P_{Rd} = 1128.074 kN

Shear connection present

The degree of shear connection, η, is the ratio of the reduced value of the compressive force, N_c, to the concrete compressive force at full shear connection, $N_{c,f}$.

η_{degree} = N_c / $N_{c,f}$ = 0.490 > 0.40

2.17. Design resistances of the cross-section for the composite stage

Shear buckling

As noted for the construction stage, the top flange is restrained laterally and therefore only cross-sectional resistances need to be verified.

Plastic resistance to vertical shear

The resistance to vertical shear ($V_{pl,Rd}$) should be taken as the resistance of the structural steel section ($V_{pl,a,Rd}$).

$V_{pl,a,Rd}$ = 839.015 kN

Max design shear for the composite stage V_{Ed} = $V_{Ed,comp}$ = 127.594 kN

$V_{Ed}/V_{pl,a,Rd}$ = 127.594 kN / 839.015 kN = 0.152

Therefore, the vertical shear resistance of the section is adequate.

2.17.1. Resistance to bending

$V_{pl,a,Rd}/2$ = 839.015 kN / 2
= 419.508 kN > V_{Ed}

No reduction in the bending resistance of the steel section need be accounted for at any point along the beam. For one connector per trough, rigid plastic theory from *4-1-1/(6.2.1.2)* may be used as shown in Figure 7.

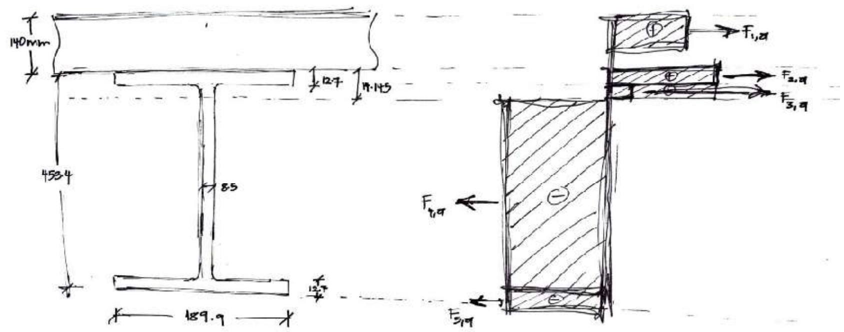


Figure 7: Stress distribution of composite section

With partial shear connection, the axial force in the concrete flange N_c is less than $N_{pl,a}$ (939 kN < 1612 kN). Therefore, the plastic neutral axis lies within the steel section. Assuming that the plastic neutral axis lies a distance x_{pl} below the top of the top flange of the section, where

$$x_{pl} = (N_{pl,a} - N_c) / (2 \times f_y \times b)$$
$$= (3035.25\ \text{kN} - 1128.074\ \text{kN}) / (2 \times 355\ \text{N/mm}^2 \times 189.9\ \text{mm})$$
$$= 14.145\ \text{mm}$$

As t_f = 12.7 mm, the plastic neutral axis lies in the web.

$$F_{1,a} = N_c$$
$$= 1128.074\ \text{kN}$$
$$F_{2,a} = f_y \times b \times t_f$$
$$= 355\ \text{N/mm}^2 \times 189.9\ \text{mm} \times 12.7\ \text{mm}$$
$$= 856.164\ \text{kN}$$
$$F_{3,a} = f_y \times t_w \times (x_{pl} - t_f)$$
$$= 355\ \text{N/mm}^2 \times 8.5\ \text{mm} \times (14.145\ \text{mm} - 12.7\ \text{mm})$$
$$= 4.361\ \text{kN}$$
$$F_{4,a} = f_y \times t_w \times (h_a - x_{pl} - t_f)$$
$$= 355\ \text{N/mm}^2 \times 8.5\ \text{mm} \times (453.4\ \text{mm} - 14.145\ \text{mm} - 12.7\ \text{mm})$$
$$= 1287.129\ \text{kN}$$
$$F_{5,a} = F_{2,a}$$
$$= 856.164\ \text{kN}$$
$$z_{F,4,a} = h_s + x_{pl} + (h_a - (h_a + x_{pl} + t_f)) / 2$$
$$= 140\ \text{mm} + 14.145\ \text{mm} + (453.4\ \text{mm} - (453.4\ \text{mm} + 14.145\ \text{mm} + 12.7\ \text{mm}))/2$$
$$= 140.723\ \text{mm}$$
$$z_{F,1,a} = 33.7\text{mm} / 2$$
$$= 16.85\ \text{mm}$$
$$z_{f,2,a} = h_s + t_f / 2$$
$$= 140\ \text{mm} + 12.7\ \text{mm} / 2$$
$$= 146.35\ \text{mm}$$
$$Z_{f,3,a} = h_s + t_f + (x_{pl} - t_f) / 2$$
$$= 140\ \text{mm} + 12.7\ \text{mm} + (14.145\ \text{mm} - 12.7\ \text{mm}) / 2$$
$$= 153.423\ \text{mm}$$
$$Z_{f,4,a} = h_s + x_{pl} + (h_a - h_s - x_{pl} - t_f) / 2$$
$$= 140\ \text{mm} + 14.145\ \text{mm} + (453.4\ \text{mm} - 140\ \text{mm} - 14.145\ \text{mm} - 12.7\ \text{mm})/2$$
$$= 297.423\ \text{mm}$$
$$z_{f,5,a} = h_a - t_f / 2$$
$$= 453.4\ \text{mm} - 12.7\ \text{mm} / 2$$
$$= 447.05\ \text{mm}$$

$$M_A = F_{1,a} \times z_{F,1,a} + F_{2,a} \times z_{f,2,a} + F_{3,a} \times z_{f,3,a} - F_{4,a} \times z_{f,4,a} - F_{5,a} \times z_{f,5,a}$$
$$= 1128.074\ \text{kN} \times 16.85\ \text{mm} + 856.164\ \text{kN} \times 146.35\ \text{mm} + 4.361\ \text{kN} \times 153.423\ \text{mm} - 1287.129\ \text{kN} \times 297.423\ \text{mm} - 856.164\ \text{kN} \times 447.05\ \text{mm}$$
$$= -620.593\ \text{kNm}$$

$$M_{Rd} = \text{Abs}(M_A) = \text{Abs}(-620.593\ \text{kNm}) = 620.593\ \text{kNm}$$

$$M_{Ed} = M_{Ed,comp} = 318.984\ \text{kNm}$$

M_{Ed}/M_{Rd} = 318.984 kNm / 620.593 kNm
= 0.514

Design bending resistance is adequate.

2.18. Longitudinal shear resistance of the slab

Transverse reinforcement

As the profiled steel sheeting has its ribs transverse to the beam, is continuous over the beam and has mechanical interlocking, its contribution to the transverse reinforcement for the shear surface shown above may be allowed for by replacing expression *(6.21)* in *2−1−1/6.2.4(4)* by:

$(A_{sf} \times f_{sd} / s_f) + (A_{pe} \times f_{yp,d}) > v_{Ed} \times h_f / \mathrm{Cot}(\theta)$

However, in practice it is usual to neglect the contribution of the steel sheeting.
Therefore, verify that:
$(A_{sf} \times f_{sd} / s_f) > v_{Ed} \times h_f / \mathrm{Cot}(\theta)$ where
v_{Ed} is the design longitudinal shear stress in the concrete slab
f_{sd} is the design yield strength of the reinforcing mesh
f_{sd} $= f_{sd} / \gamma_s = 500\ \mathrm{N/mm^2} / 1.15 = 434.783\ \mathrm{N/mm^2}$
f_{yd} $= f_{sd} = 434.783\ \mathrm{N/mm^2}$
h_f is the depth of concrete above the profiled sheeting
h_f $= h_c = 65$ mm

$\theta = \theta_f$ given in *BS EN 1992−1−1* as the angle of failure.

$45° \leq \theta_f \leq 26.5°$

To minimise the amount of reinforcement,
θ_f = 26.5°

A_{sf} / s_f $= A_t$ (for the failure plane shown in Figure B.11 as section a-a)
A_t is the cross−sectional area of transverse reinforcement ($\mathrm{mm^2}$ / m)

Therefore, the verification becomes:

$A_t \times f_{yd} > v_{Ed} \times h_f / \mathrm{Cot}(\theta)$

And the required area of tensile reinforcement (A_t) must satisfy the following

$A_t > v_{Ed} \times h_f / (\mathrm{Cot}(\theta) \times f_{yd})$
The longitudinal shear stress is given by:

$v_{Ed} = \Delta F_d / (h_f \times \Delta x)$ where
Δx is the critical length under consideration, which for this example is the distance between the maximum bending moment and the support.

Δx $= L / 2 = 10\ \mathrm{m} / 2 = 5$ m
ΔF_d $= N_c / 2 = 1128.074\ \mathrm{kN} / 2 = 564.037$ kN
h_f = 65 mm
v_{Ed} $= \Delta F_d / (h_f \times \Delta x) = 564.037\ \mathrm{kN} / (65\ \mathrm{mm} \times 5\ \mathrm{m}) = 1.735\ \mathrm{N/mm^2}$

A_t $= v_{Ed} \times h_f / (\mathrm{Cot}(\theta_f) \times f_{sd})$
$= 1.735\ \mathrm{N/mm^2} \times 65\ \mathrm{mm} / (\mathrm{Cot}(26.5^o) \times 434.783\ \mathrm{N/mm^2})$
$= 0.129\ \mathrm{mm}$

The reinforcement provided is A193 mesh, for which:

A_t $= 193\ \mathrm{mm^2/m} > 129\ \mathrm{mm^2/m}$

Therefore, an A193 mesh is adequate.

Crushing of the concrete flange

Verify that:

v_{Ed} $< v \times f_{cd} \times \sin(\theta_f) \times \cos(\theta_f)$ where

ν $= 0.6 \times (1 - f_{ck} / (250\ \mathrm{N/mm^2}))$
$= 0.6 \times (1 - 25\ \mathrm{N/mm^2} / (250\ \mathrm{N/mm^2})) = 0.54$

θ_f $= 26.5^o$

v_{Ed} $= \nu \times f_{cd} \times \sin(\theta_f) \times \cos(\theta_f)$
$= 0.54 \times 16.667\ \mathrm{N/mm^2} \times \sin(26.5^o) \times \cos(26.5^o)$
$= 3.594\ \mathrm{N/mm^2}$

Therefore, the crushing resistance of the concrete is adequate.

2.19. Verification at SLS

2.19.1. Modular ratios

For short term loading, the secant modulus of elasticity should be used.

n_0 $= E / E_{cm}$
$= 210\ \mathrm{kN/mm^2}/31\ \mathrm{kN/mm^2}$
$= 6.774$

For long term loading, the modular ratio should be calculated from:

n_L $= n_0 \times (1 + \psi_L \times \varphi_t)$

ψ_L $= 1.1$ is the creep multiplier, taken as 1.1 for permanent loads

φ_t $= 3$ is the creep coefficient, taken as 3 in this case

n_L $= n_0 \times (1 + \psi_L \times \varphi_t)$
$= 6.774 \times (1 + 1.1 \times 3) = 29.129$

When calculating deflections due to variable actions the modular ratio is taken as follows:

n $= 1 / 3 \times n_L + 2 / 3 \times n_0 = 1/3 \times 29.129 + 2 / 3 \times 6.774 = 14.226$

For dynamic conditions (i.e. natural frequency calculations), the value of E_c should be determined according to SCI publication P354, which gives $E_c = 38\ \mathrm{kN/mm^2}$, and so the dynamic modular ratio is:

n_d $= E / E_c$
$= 210\ \mathrm{kN/mm^2} / 38\ \mathrm{kN/mm^2} = 5.526$

2.19.2. Second moment of area of composite section

b_{eff} = 2.25 m

r = A_a / (b_{eff} × h_c)
= 85.5 cm^2 / (2.5 m × 65 mm) = 0.0526

$I_{c,n0}$ = A_a × (h_c + 2 × h_d + h_a)^2 / (4 ×(1 + n × r)) + b_{eff} × h_c^3 / (12 × n) + I_y
= 85.5 cm^2 × (65 mm + 2 × 75 mm + 453.4 mm)^2 / (4 × (1 + 14.226 × 0.0526)) + 2.5 m × (65 mm)^3 / (12 × 14.226) + 29400 cm^4
= 84417.478 cm^4

A_c = b_{eff} × h_c
= 2.5 m × 65 mm
= 0.163 m^2

z_{el} = (A_a × (h_s + h_a / 2) + (A_c / n) × h_c / 2) / (A_a + A_c / n)
= (85.5 cm^2 × (140 mm + 453.4 mm / 2) + (0.1625 m^2 / 14.226) × 65 mm / 2) / (85.5 cm^2 + 0.1625 m^2 / 14.226)
= 175.564 mm

Modular ratio	n	I_c (cm^4)	z_{el} (mm)
n_0	6.774194	100646.199	333.082
n_L	29.12903	67302.058	218.657
n_d	5.526316	104418.031	345.615
n	14.22581	84417.478	277.836

Table 3: Second moment of area of composite section

2.19.3. Beam deflection

Deflection due to actions on the steel section at the construction stage

δ_{G1} = 5 × $g_{1,sls}$ × L^4 / (384 × E × I_y)
= 5×10.864 kN/m × (10 m)^4 / (384×210 kN/mm^2 × 29400 cm^4)
= 22.913 mm

Deflection due to permanent actions on the composite beam

δ_{G2} = 5 × $g_{2,sls}$ × L^4 / (384 × E × $I_{y,nL}$)
= 5 × 1.125 kN/m × (10 m)^4 / (384 × 210 kN/mm^2 × 67302.058 cm^4)
= 1.036 mm

Deflection due to variable actions on the composite beam

δ_{Q1} = 5 × $q_{1,sls}$ × L^4 / (384 × E × $I_{y,n}$)
= 5×7.03125 kN/m × (10 m)^4 / (384 × 210 kN/mm^2 × 84417.478 cm^4)
= 5.164 mm

δ_{TOT} = δ_{G1} + δ_{G2} + δ_{Q1}
= 22.913 mm + 1.036 mm + 5.164 mm
= 29.114 mm

Deflection due to variable actions is

δ_{Q1} = 5.164 mm < limit = L / 360 to mm = 10 m / 360 to mm = 27.778 mm OK

2.19.4. SLS stress verification

Stresses in the steel section due to actions on the steel section at the construction stage

$\sigma_{G1,a}$ $= g_{1,sls} \times L^2 \times (h_a / 2) / (8 \times I_y)$
$= 10.864\ \text{kN/m} \times (10\ \text{m})^2 \times (453.4\ \text{mm}/2) / (8 \times 29400\ \text{cm}^4)$
$= 104.718\ \text{N/mm}^2$

Stresses in the steel section due to permanent actions on the composite beam

$\sigma_{G2,a}$ $= g_{2,sls} \times L^2 \times (z_{el,nL} / 2) / (8 \times I_{y,nL})$
$= 1.125\ \text{kN/m} \times (10\ \text{m})^2 \times (218.657\ \text{mm} / 2) / (8 \times 67302.058\ \text{cm}^4)$
= 2.284 N/mm^2

Stresses in the steel section due to variable actions on the composite beam

$\sigma_{Q1,a}$ $= q_{1,sls} \times L^2 \times (z_{el,n} / 2) / (8 \times I_{y,n})$
$= 7.03125\ \text{kN/m} \times (10\ \text{m})^2 \times (277.836\ \text{mm} / 2) / (8 \times 84417.478\ \text{cm}^4)$
$= 14.463\ \text{N/mm}^2$

Maximum stress in the steel section

σ_c $= \sigma_{G1,a} + \sigma_{G2,a} + \sigma_{Q1,a}$
$= 104.718\ \text{N/mm}^2 + 2.284\ \text{N/mm}^2 + 14.463\ \text{N/mm}^2$
$= 121.466\ \text{N/mm}^2$

The extreme fiber stress slightly exceeds the steel section's yield strength (275 N/mm²). However, this has minimal impact on deflection, keeping it below the limit. Thus, upgrading the steel grade or size to reduce stress is unnecessary.

Stresses in the concrete flange due to permanent actions on the composite beam

$\sigma_{G,2,c}$ $= g_{2,sls} \times L^2 \times (h_a + h_s - z_{el,nL}) / (8 \times I_{y,nL} \times n_L)$
$= 1.125\ \text{kN/m} \times (10\ \text{m})^2 \times (453.4\ \text{mm} + 140\ \text{mm} - 218.657\ \text{mm})/(8 \times 67302.058\ \text{cm}^4 \times 29.129)$
$= 0.269\ \text{N/mm}^2$

$\sigma_{Q,1,c}$ $= q_{1,sls} \times L^2 \times (h_a + h_s - z_{el,n0}) / (8 \times I_{y,n0} \times n_0)$
$= 7.03125\ \text{kN/m} \times (10\ \text{m})^2 \times (453.4\ \text{mm} + 140\ \text{mm} - 333.083\ \text{mm}) / (8 \times 100646.199\ \text{cm}^4 \times 6.774)$
$= 3.356\ \text{N/mm}^2$

Maximum stress in the concrete

$\sigma_{c,concrete}$ $= \sigma_{G,2,c} + \sigma_{Q,1,c}$
$= 0.2689\ \text{N/mm}^2 + 3.356\ \text{N/mm}^2$
$= 3.625\ \text{N/mm}^2$

2.19.5. Natural frequency

To calculate the natural frequency, SLS values include the full permanent load and 10% of variable actions in service.

w $= g_{1,sls} + g_{2,sls} + (q_{1,sls} \times 0.1)$
$= 10.864$ kN/m + 1.125 kN/m + (7.031 kN/m × 0.1)
$= 12.693$ kN/m

The deflection under this load is:

$\delta_{G1,freq}$ $= 5 \times w \times L^4 / (384 \times E \times I_{y,nd})$
$= 5 \times 12.693 \text{ kN/m} \times (10 \text{ m})^4 / (384 \times 210 \text{ kN/mm}^2 \times 104418.031 \text{ cm}^4)$
$= 7.537$ mm

The natural frequency of the beam is therefore:

f $= 18 / \sqrt{(\delta_{G1,freq})} = 18 / \sqrt{(7.537 \text{ mm})} = 6.557$

Thus f > 4 Hz. Therefore, the beam is OK for initial calculation purposes.

3. Column Design

Unless stated otherwise, the design is in accordance with BS EN1993−1−1:2005 and UK National Annex.

Resistance of cross-sections,	γ_{M0}	= 1
Resistance of members to instability,	γ_{M1}	= 1

Column details

Column section;	UC 305x305x97	
Steel grade,	S355	
Yield strength,	f_y	= 355 N/mm^2
Ultimate strength,	f_u	= 470 N/mm^2
Modulus of elasticity,	E	= 210 kN/mm^2
Poisson's ratio,	ν	= 0.3
Shear modulus,	G	$= E / [2 \times (1 + \nu)] = 80.8$ kN/mm^2

Section properties

For a 457 × 191 × 67 UKB in S355 steel		
Depth,	h	= 307.9 mm
Width,	b	= 305.3 mm
Web thickness,	t_w	= 9.9 mm
Flange thickness,	t_f	= 15.4 mm
Root radius,	r	= 15.2 mm
Depth between fillets,	d	= 246.7 mm
Second moment of area y axis,	I_y	= 22200 cm^4
Second moment of area z axis,	I_z	= 7310 cm^4
Elastic modulus y axis,	$W_{el,y}$	= 1450 cm^3
Plastic modulus y axis,	$W_{pl,y}$	= 1590 cm^3
Area,	A_a	= 123 cm^2
Modulus of elasticity,	E	= 210 kN/mm^2
Mass per metre,		= 96.9 kg/m

Column geometry

The column is not part of a sway frame in both axes.

Length for buckling − Major axis, L_y = 3500 mm

Length for buckling − Minor axis, L_z = 3500 mm

Column loading

Axial load N_{Ed} = 2760 kN (Compression)

For pinned connections the column only carries axial loads and no moments. Subsequently no shear in the columns.

Buckling lengths

As the column in pin ended with no intermediate restraints, the buckling length may be taken as the actual length of the column.

End restraint factor y-y, K_y = 1.000

Buckling length, $L_{cr,y}$ $= L_y \times K_y$
$= 3500 \text{ mm} \times 1.000$
$= 3500 \text{ mm}$

End restraint factor z-z, K_z = 1.000

Buckling length, $L_{cr,z}$ $= L_z \times K_z$
$= 3500 \text{ mm} \times 1.000$
$= 3500 \text{ mm}$

Web section classification (Table 5.2)

Coefficient depending on f_y, ε $= \sqrt{(235 \text{ N/mm}^2 / f_y)}$
$= \sqrt{(235 \text{ N/mm}^2 / 355 \text{ N/mm}^2)}$
$= 0.814$

Depth between fillets,

c_w $= h - 2 \times (t_f + r)$
$= 307.9 \text{ mm} - 2 \times (15.4 \text{ mm} + 15.2 \text{ mm})$
$= 237.1 \text{ mm}$

Ratio of c/t, $ratio_w$ $= 237.1 \text{ mm} / 9.9 \text{ mm}$
$= 23.95$

Length of web taken by axial load,

l_w $= \min(N_{Ed} / (f_y \times t_w), c_w)$
$= \min(2760 \text{ kN} / (355 \text{ N/mm}^2 \times 9.9 \text{ mm}), 237.1 \text{ mm})$
$= 237.1 \text{ mm}$

For class 1 & 2 proportion in compression,

α $= (c_w/2 + l_w/2) / c_w$
$= (237.1 \text{ mm} /2 + 237.1 \text{ mm} /2) / 237.1 \text{ mm}$
$= 1.000$

Limit for class 1 web, $Limit_{1w}$ $= (396 \times \varepsilon) / (13 \times \alpha - 1)$
$= (396 \times 0.814) / (13 \times 1.000 - 1)$
$= 26.85$
The web is class 1

Flange section classification (Table 5.2)

Outstand length,

c_f $= (b - t_w) / 2 - r$
$= (305.3\ \text{mm} - 9.9\ \text{mm}) / 2 - 15.2\ \text{mm}$
$= 127.7\ \text{mm}$

Ratio of c/t, $ratio_f$ $= c_f / t_f$
$= 127.7\ \text{mm} / 15.4\ \text{mm}$
$= 8.29$

Limit for class 1 flange, $Limit_{1f}$ $= 9 \times \varepsilon = 9 \times 0.814 = 7.32$
Limit for class 2 flange, $Limit_{2f}$ $= 10 \times \varepsilon = 10 \times 0.814 = 8.14$
Limit for class 3 flange, $Limit_{3f}$ $= 14 \times \varepsilon = 14 \times 0.814 = 11.39$

The flange is class 3

Overall section classification, The section is class 3

Resistance of cross section (cl. 6.2.4)

Design force, N_{Ed} $= 2760\ \text{kN}$

Design resistance,

$N_{c,Rd}$ $= N_{pl,Rd}$
$= A \times f_y / \gamma_{M0}$
$= 123\ \text{cm}^2 \times 355\ \text{N/mm}^2 / 1$
$= 4434\ \text{kN}$

$= N_{Ed} / N_{c,Rd}$
$= 2760\ \text{kN} / 4434\ \text{kN}$
$= 0.622$ $\therefore$ OK

Buckling resistance (cl. 6.3)

Yield strength for buckling resistance, f_y $= 355\ \text{N/mm}^2$

Flexural buckling − Major axis

Elastic critical buckling force,

$N_{cr,y}$ $= \pi^2 \times E \times I_y / L_{cr,y}^2$
$= \pi^2 \times 210\ \text{kN/mm}^2 \times 22200\ \text{cm}^4 / (3500\ \text{mm})^2$
$= 38076\ \text{kN}$

Non-dimensional slenderness,

λ_y $= \sqrt{(A \times f_y / N_{cr,y})}$
$= \sqrt{(123\ \text{cm}^2 \times 355\ \text{N/mm}^2 / 38076\ \text{kN})}$
$= 0.341$

Buckling curve *(Table 6.2)*, b

Imperfection factor *(Table 6.1)*, α_y $= 0.34$

Parameter Φ,

Φ_y $= 0.5 \times [1 + \alpha_y \times (\lambda_y - 0.2) + \lambda_y^2]$
$= 0.5 \times [1 + 0.34 \times (0.341 - 0.2) + 0.341^2]$
$= 0.582$

Reduction factor,

χ_y $= \min(1.0, 1 / [\Phi_y + \sqrt{(\Phi_y^2 - \lambda_y^2)}])$
$= \min(1.0, 1 / [0.582 + \sqrt{(0.582^2 - 0.341^2)}])$
$= 0.949$

Design buckling resistance,

$N_{b,y,Rd}$ $= \chi_y \times A \times f_y / \gamma_{M1}$

$= 0.949 \times 123\ cm^2 \times 355\ N/mm^2 / 1$
$= 4206.8\ kN$

$= N_{Ed} / N_{b,y,Rd}$
$= 2760\ kN / 4206.8\ kN$
$= 0.656$ ∴ OK

Flexural buckling − Minor axis

Elastic critical buckling force,

$N_{cr,z}$ $= \pi^2 \times E \times I_z / L_{cr,z}^2$
$= \pi^2 \times 210\ kN/mm^2 \times 7310\ cm^4 / (3500\ mm)^2$
$= 12367\ kN$

Non−dimensional slenderness,

λ_z $= \sqrt{(A \times f_y / N_{cr,z})}$
$= \sqrt{(123\ cm^2 \times 355\ N/mm^2 / 12367\ kN)}$
$= 0.599$

Buckling curve (Table 6.2), c

Imperfection factor (Table 6.1), α_z $= 0.49$

Parameter Φ,

Φ_z $= 0.5 \times [1 + \alpha_z \times (\lambda_z - 0.2) + \lambda_z^2]$
$= 0.5 \times [1 + 0.49 \times (0.599 - 0.2) + 0.599^2]$
$= 0.777$

Reduction factor,

χ_z $= \min(1.0, 1 / [\Phi_z + \sqrt{(\Phi_z^2 - \lambda_z^2)}])$
$= \min(1.0, 1 / [0.777 + \sqrt{(0.777^2 - 0.599^2)}])$
$= 0.786$

Design buckling resistance,

$N_{b,z,Rd}$ $= \chi_z \times A \times f_y / \gamma_{M1}$
$= 0.786 \times 123\ cm^2 \times 355\ N/mm^2 / 1$
$= 3485.6\ kN$

$= 2760\ kN / 3485.6\ kN$
$= 0.792$ ∴ OK

Torsional and torsional−flexural buckling (cl. 6.3.1.4)

Torsional buckling length factor, K_T $= 1.00$

Effective buckling length, $L_{cr,T}$ $= K_T \times \max(L_y, L_z)$
$= 3500\ mm$

Distance from shear centre to centroid along major axis,

y_0 $= 0.0\ mm$
z_0 $= 0.0\ mm$

Distance from shear centre to centroid along minor axis,

z_0 $= 0.0\ mm$
i_0 $= \sqrt{(i_y^2 + i_z^2 + y_0^2 + z_0^2)}$
$= 154.5\ mm$
β_T $= 1 - (y_0 / i_0)^2$
$= 1.000$

Elastic critical torsional buckling force,

$N_{cr,T}$ $= 1 / i_0^2 \times (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2)$
$= 14411$ kN
Elastic critical torsional−flexural buckling force,
$N_{cr,TF}$ $= N_{cr,y}/(2\times\beta_T)\times[1+N_{cr,T}/N_{cr,y}-\sqrt{[(1-N_{cr,T}/N_{cr,y})^2 + 4\times(y_0/i_0)^2\times N_{cr,T}/N_{cr,y}]}]$
$= 14411$ kN
Non−dimensional slenderness,
λ_T $= \sqrt{(A \times f_y / \min(N_{cr,T}, N_{cr,TF}))}$
$= \sqrt{(123 \text{ cm}^2 \times 355 \text{ N/mm}^2 / \min(14411 \text{ kN}, 14411 \text{ kN}))}$
$= 0.555$
Buckling curve (Table 6.2), c

Imperfection factor (Table 6.1), α_T $= 0.49$
Parameter Φ,
Φ_T $= 0.5 \times [1 + \alpha_T \times (\lambda_T - 0.2) + \lambda_T^2]$
$= 0.5 \times [1 + 0.49 \times (0.555 - 0.2) + 0.555^2]$
$= 0.741$
Reduction factor,
χ_T $= \min(1.0, 1 / [\Phi_T + \sqrt{(\Phi_T^2 - \lambda_T^2)}])$
$= \min(1.0, 1 / [0.741 + \sqrt{(0.741^2 - 0.555^2)}])$
$= 0.812$
Design buckling resistance,
$N_{b,T,Rd}$ $= \chi_T \times A \times f_y / \gamma_{M1}$
$= 0.812 \times 123 \text{ cm}^2 \times 355 \text{ N/mm}^2 / 1$
$= 3599.9$ kN

$= N_{Ed} / N_{b,T,Rd}$
$= 2760 \text{ kN} / 3599.9 \text{ kN}$
$= 0.767$ ∴ OK

Minimum buckling resistance

Minimum buckling resistance,
$N_{b,Rd}$ $= \min(N_{b,y,Rd}, N_{b,z,Rd}, N_{b,T,Rd})$
$= \min(4206.8 \text{ kN}, 3485.6 \text{ kN}, 3599.9 \text{ kN})$
$= 3485.6$ kN

$= N_{Ed} / N_{b,Rd}$
$= 2760 \text{ kN} / 3485.6 \text{ kN}$
$= 0.792$ ∴ OK

4. Connection Design

Unless stated otherwise, the design is in accordance with BS EN1993−1−1:2005 and BS EN1993−1−8:2005.

Tie force

T_i $= 0.8 (g_k + \psi_1 q_k)sL$ where $\psi = 0.5$ for domestic/residetial
$= 0.8 (3.14 \text{ kN/m}^2 + 0.5 \times 1.875 \text{ kN/m}^2) \times 7.5 \text{ m} \times 10 \text{ m}$
$= 244.65$ kN

Connection details

Connection type, Partial depth end plate
Number of supported beams, 1 supported beam

Partial factors

Resistance of cross–section,	γ_{M0}	= 1.00
Resistance of members to instability,	γ_{M1}	= 1.00
Resistance of bolts,	$\gamma_{M2,b}$	= 1.25
Structural integrity,	$\gamma_{M,u}$	= 1.10

Supporting column details

Section name,	UC 305x305x97	
Steel grade,	S355	
Yield strength,	f_y	= 355 N/mm^2
Ultimate strength,	f_u	= 470 N/mm^2

Design forces

Design shear,	V_{Ed1}	= 160.5 kN
Design tying force,	F_{Ed1}	= 244.65 kN

Supported beam details

Section name,	UB 457x191x67	
Steel grade,	S355	
Yield strength,	$f_{y,b}$	= 355 N/mm^2
Ultimate strength,	$f_{u,b}$	= 470 N/mm^2
Correlation factor,	$\beta_{w,b}$	= 0.9

End plate details

Plate height,	h_p	= 290 mm
Plate width,	b_p	= 150 mm
Plate thickness,	t_p	= 10 mm
Plate grade,	S275	
Yield strength,	$f_{y,p}$	= 275 N/mm^2
Ultimate strength,	$f_{u,p}$	= 410 N/mm^2
Correlation factor,	$\beta_{w,p}$	= 0.85

Bolt details

Number of bolt rows,	$n_{1,1}$	= 4
Total number of bolts,	n_b	= 8
End distance,	e_1	= 40 mm
Edge distance,	e_2	= 30 mm
Pitch,	p_1	= 70 mm
Gauge,	p_3	= 90 mm
Bolt hole,	d_0	= 22 mm
Bolt size,	M20	
Bolt grade,	8.8	
Yield strength,	$f_{y,bolt}$	= 640 N/mm^2
Ultimate strength,	$f_{u,bolt}$	= 800 N/mm^2

Check 1: Recommended detailing practice

Minimum plate height,	$0.6 \times h_b$	= 0.6 × 453.4 mm = 272 mm
Actual plate height,	h_p	= 290 mm
Maximum depth to plate,	50 mm	
Actual depth to plate,	d_p	= 50 mm
Maximum plate thickness,	10 mm	
Actual plate thickness,	t_p	= 10 mm

Minimum bolt gauge, 90 mm
Actual bolt gauge, p_3 = 90 mm

∴ OK – Recommended detailing practices are met

Check 2: Supported beam − Welds

Weld leg size, s_w $= 8.0$ mm
Minimum weld throat thickness, $0.48 \times t_{w,b}$ $= 0.48 \times 8.5$ mm
$= 4.1$ mm
Effective weld throat thickness, a_w $= 0.7 \times s_w$
$= 0.7 \times 8.0$ mm
$= 5.6$ mm
Correlation factor, β_w $= \text{Min}(\beta_{w,b}, \beta_{w,p})$
$= \text{Min}(0.9, 0.85)$
$= 0.85$
Design shear strength,
$f_{vw,d}$ $= \text{Min}(f_{u,b},f_{u,p}) / \sqrt{(3)} / (\beta_w \times \gamma_{M2,c})$
$= \text{Min}(800\ \text{N/mm}^2, 410\ \text{N/mm}^2) / \sqrt{(3)} / (0.85 \times 1.10)$
$= 253.17\ \text{N/mm}^2$
Design resistance,
$F_{w,Rd}$ $= f_{vw,d} \times a_w \times h_p$
$= 253.17\ \text{N/mm}^2 \times 5.6\ \text{mm} \times 290\ \text{mm}$
$= 411.15$ kN
Design weld force,
$F_{w,Ed}$ $= \sqrt{((160.5\ \text{kN})^2 + (244.65\ \text{kN})^2)} / 2$
$= \sqrt{(V_{Ed1}^2 + F_{Ed1}^2)} / 2$
$= 146.3$ kN
Utilisation, $= F_{w,Ed} / F_{w,Rd}$
$= 146.3\ \text{kN} / 411.15\ \text{kN}$
$= 0.356$ ∴ OK

Check 4: Supported beam − Web in shear

Shear area, A_v $= 0.9 \times h_p \times t_{w,b}$
$= 0.9 \times 290\ \text{mm} \times 8.5\ \text{mm}$
$= 2219\ \text{mm}^2$
Plastic shear resistance of beam web,
$V_{pl,Rd}$ $= A_v \times (f_{y,b} / \sqrt{(3)}) / \gamma_{M0}$
$= 2219\ \text{mm}^2 \times (355\ \text{N/mm}^2 / \sqrt{(3)}) / 1.00$
$= 454.7$ kN
Design shear resistance, $V_{c,Rd}$ $= V_{pl,Rd}$
$= 454.7$ kN
Utilisation, $= V_{Ed1} / V_{c,Rd}$
$= 160.5\ \text{kN} / 454.7\ \text{kN}$
$= 0.353$ ∴ OK

Check 8: Connection − Bolt group

Bolt tensile stress area, A_s $= 245\ \text{mm}^2$
Bolt shear stress factor, α_v $= 0.6$
Bolt shear resistance,
$F_{v,Rd}$ $= \alpha_v \times f_{u,bolt} \times A_s / \gamma_{M2,b}$
$= 0.6 \times 800\ \text{N/mm}^2 \times 245\ \text{mm}^2 / 1.25$

$= 94.08\ \text{kN}$

For the end plate,

$\alpha_{b,p}$ $= \text{Min}(e_1 / (3 \times d_0), p_1 / (3 \times d_0) - 1/4, f_{u,bolt} / f_{u,p}, 1)$

$= \text{Min}(40\ \text{mm} / (3 \times 22\ \text{mm}), 70\ \text{mm} / (3 \times 22\ \text{mm}) - 1/4, 800\ \text{N/mm}^2 / 410\ \text{N/mm}^2, 1)$

$= 0.61$

$k_{1,p}$ $= \text{Min}(2.8 \times e_2 / d_0 - 1.7, 1.4 \times p_3 / d_0 - 1.7, 2.5)$

$= \text{Min}(2.8 \times 30\ \text{mm} / 22\ \text{mm} - 1.7, 1.4 \times 90\ \text{mm} / 22\ \text{mm} - 1.7, 2.5)$

$= 2.12$

For the supporting member,

$\alpha_{b,2}$ $= \text{Min}(p_1 / (3 \times d_0) - 1/4, f_{u,bolt} / f_u, 1)$

$= \text{Min}(70\ \text{mm} / (3 \times 22\ \text{mm}) - 1/4, 800\ \text{N/mm}^2 / 470\ \text{N/mm}^2, 1)$

$= 0.81$

$k_{1,2}$ $= \text{Min}(1.4 \times p_3 / d_0 - 1.7, 2.5)$

$= \text{Min}(1.4 \times 90\ \text{mm} / 22\ \text{mm} - 1.7, 2.5)$

$= 2.5$

Bearing on the end plate,

$F_{b,Rd,p}$ $= k_{1,p} \times 0.61 \times 410\ \text{N/mm}^2 \times d_b \times t_p / \gamma_{M2,b}$

$= 2.12 \times \alpha_{b,p} \times 470\ \text{N/mm}^2 \times d_b \times 10\ \text{mm} / 1.25$

$= 84.21\ \text{kN}$

Bearing on the supporting member,

$F_{b,Rd,2}$ $= k_{1,2} \times \alpha_{b,2} \times f_u \times d_b \times t_w / \gamma_{M2,b}$

$= 2.5 \times 0.81 \times 470\ \text{N/mm}^2 \times 22\ \text{mm} \times 8.5\ \text{mm} / 1.25$

$= 150.87\ \text{kN}$

Minimum bearing resistance, $F_{b,Rd1}$ $= \text{Min}(F_{b,Rd,p}, F_{b,Rd,2})$

$= \text{Min}(84.21\ \text{kN}, 150.87\ \text{kN})$

$= 84.21\ \text{kN}$

Resistance of the bolt group, F_{Rd} $= 0.8 \times n_b \times F_{v,Rd}$

$= 0.8 \times 8 \times 94.08\ \text{kN}$

$= 602.11\ \text{kN}$

Utilisation, $= V_{Ed1} / F_{Rd}$

$= 160.5\ \text{kN} / 602.11\ \text{kN}$

$= 0.267 \therefore$ OK

Check 9: Connection − End plate in shear

Net shear area, $A_{v,net}$ $= t_p \times (h_p - n_{1,1} \times d_0)$

$= 2020\ \text{mm}^2$

Edge shear area, A_{nt} $= t_p \times (e_2 - d_0 / 2)$

$= 190\ \text{mm}^2$

Shear area from end bolt, A_{nv} $= t_p \times (h_p - e_1 - (n_{1,1} - 0.5) \times d_0)$

$= 1730\ \text{mm}^2$

Gross section shear resistance,

$V_{Rd,g}$ $= (2 \times h_p \times t_p) / 1.27 \times f_{y,p} / (\sqrt{(3)} \times \gamma_{M0})$

$= 725.1\ \text{kN}$

Net section shear resistance,

$V_{Rd,n}$ $= 2 \times A_{v,net,A\_c1} \times f_{u,plate,A\_c1} / (\sqrt{(3)} \times \gamma_{M2,c})$

$= 869.38\ \text{kN}$

Block tearing resistance,

$V_{Rd,b}$ $= 2 \times (f_{u,p} \times A_{nt} / \gamma_{M2,c} + f_{y,p} \times A_{nv} / (\sqrt{(3)} \times \gamma_{M0}))$

$= 690.99\ \text{kN}$

End plate in−plane bending resistance, $h_p < 1.36p_3$ − No additional requirements

End plate shear resistance, $V_{Rd,pl,min} = \mathrm{Min}(V_{Rd,g}, V_{Rd,n}, V_{Rd,b})$

$= 690.99$ kN

Utilisation, $= 0.232$ ∴ OK

Check 10: Supporting column − Shear

Minimum top distance, $e_t = 5 \times d_b$

$= 100$ mm

Minimum bottom distance, $e_b = \mathrm{Min}(p_3 / 2, 5 \times d_b)$

$= 45$ mm

Shear area of supporting member, $A_v = t_w \times (e_t + (n_{1,1} - 1) \times p_1 + e_b)$

$= 3515$ mm^2

Net shear area of supporting member, $A_{v,net} = A_v - n_{1,1} \times d_0 \times t_w$

$= 2643$ mm^2

Local shear resistance,

$V_{Rd,min} = \mathrm{Min}(A_v \times f_y / (\sqrt{3} \times \gamma_{M0}), A_{v,net} \times f_u / (\sqrt{3} \times \gamma_{M2,c}))$

$= 652.07$ kN

Utilisation,

$= V_{Ed1} / 2 / V_{Rd,min}$

$= 0.123$ ∴ OK

Check 11: Tying resistance − Plate and bolts

Effective end distance,

$e_{1A} = \mathrm{Min}(e_1, 0.5 \times (p_3 - t_{w,b} - 2 \times a_w \times \sqrt{2}) + d_0/2)$

$= 40$ mm

Effective bolt pitch,

$p_{1A} = \mathrm{Min}(p_1, p_3 - t_{w,b} - 2 \times a_w \times \sqrt{2} + d_0)$

$= 70$ mm

Minimum end distance, $e_{min} = e_2 = 30$ mm

Bolt factor, $k_2 = 0.9$

Distance from weld throat to bolt,

$m_w = (p_3 - t_{w,b} - 2 \times 0.8 \times a_w \times \sqrt{2}) / 2$

$= 34.4$ mm

$n_w = \mathrm{Min}(e_{min}, 1.25 \times m_w) = 30$ mm

Width across bolt head points, $d_w = 33$ mm

$e_w = d_w / 4 = 8.3$ mm

Effective length of equivalent T−stub, $\Sigma l_{eff} = 2 \times e_{1A} + (n_{1,1} - 1) \times p_{1A}$

$= 290.0$ mm

Moment resistance of plate, $M_{pl,1,Rd,u} = (0.25 \times \Sigma l_{eff} \times t_p^2 \times f_{u,p}) / \gamma_{M,u}$

$= 2.7$ kNm

$M_{pl,2,Rd,u} = M_{pl,1,Rd,u}$

$= 2.7$ kNm

Mode 1 plate failure,

$F_{Rd,u,1} = (8 \times n_w - 2 \times e_w) \times M_{pl,1,Rd,u} / (2 \times m_w \times n_w - e_w \times (m_w + n_w))$

$= 393.86$ kN

Individual bolt resistance, $F_{t,Rd,u} = k_2 \times f_{u,bolt} \times A_s / \gamma_{M,u}$

$= 160.36$ kN

Group bolt resistance, $\Sigma F_{t,Rd,u} = n_b \times F_{t,Rd,u}$

$= 1282.91$ kN

Mode 2 bolt and plate failure,

$F_{Rd,u,2} = (2 \times M_{pl,2,Rd,u} + n_w \times \Sigma F_{t,Rd,u}) / (m_w + n_w)$

= 681.4 kN

Mode 3 bolt failure,

$F_{Rd,u,3}$ = $\Sigma F_{t,Rd,u}$

= 1282.91 kN

Minimum resistance,

$F_{Rd,u,min}$ = Min($F_{Rd,u,1}$, $F_{Rd,u,2}$, $F_{Rd,u,3}$)

= 393.86 kN

Utilisation, F_{Ed1} / $F_{Rd,u,min}$ = 0.621∴ OK

Check 12: Tying resistance − Supported beam web

Web resistance, $F_{Rd,u}$ = ($t_{w,b}$ × h_p × $f_{u,b}$) / $\gamma_{M,u}$

= 1053.23 kN

Utilisation, F_{Ed1} / $F_{Rd,u}$ = 0.232 ∴ OK

4.1.1.1. Check 14: Tying resistance − Supporting column web

Geometric properties,

η_1 = (($n_{1,1}$ − 1) × p_1 − $n_{1,1}$ / 2 × d_0) / d_f

= 0.7

β_1 = p_3 / d_f = 0.38

γ_1 = d_0 / d_f = 0.09

Web moment resistance, $M_{pl,Rd,u}$ = f_u × t_w^2 / (4 × $\gamma_{M,u}$)

= 10.47 kNm/m

Web resistance,

$F_{Rd,u}$ = 8 × $M_{pl,Rd,u}$ / (1 − β_1) × (η_1 + 1.5 × √(1 − β_1) × √(1 − γ_1))

= 246.43 kN

Utilisation, F_{Ed1} / $F_{Rd,u}$ = 0.993 ∴ OK

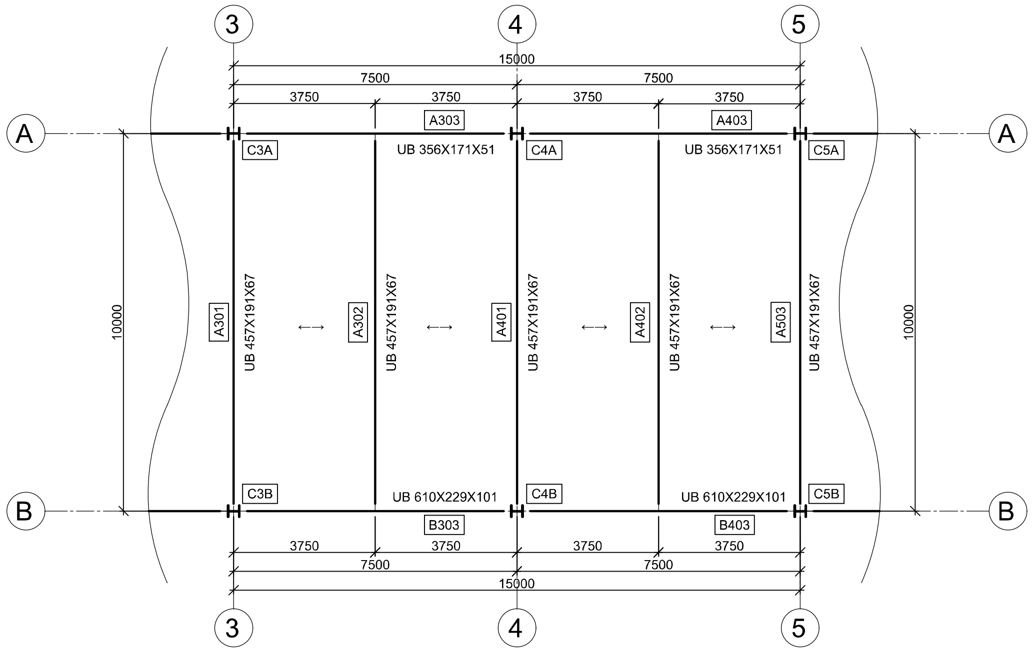
5. Design Summary

The composite beam design utilizes a 457x191x67 UKB in S355 steel, acting compositely with a Tata Steel ComFlor® 60 profiled steel decking slab. This combination optimizes structural efficiency by leveraging composite action, which reduces material usage while maintaining load-bearing capacity. Shear connectors are spaced at 300 mm centres to ensure effective interaction between the steel beam and concrete slab, achieving a degree of shear connection (η = 0.49) that exceeds the minimum codified requirement of 0.4.

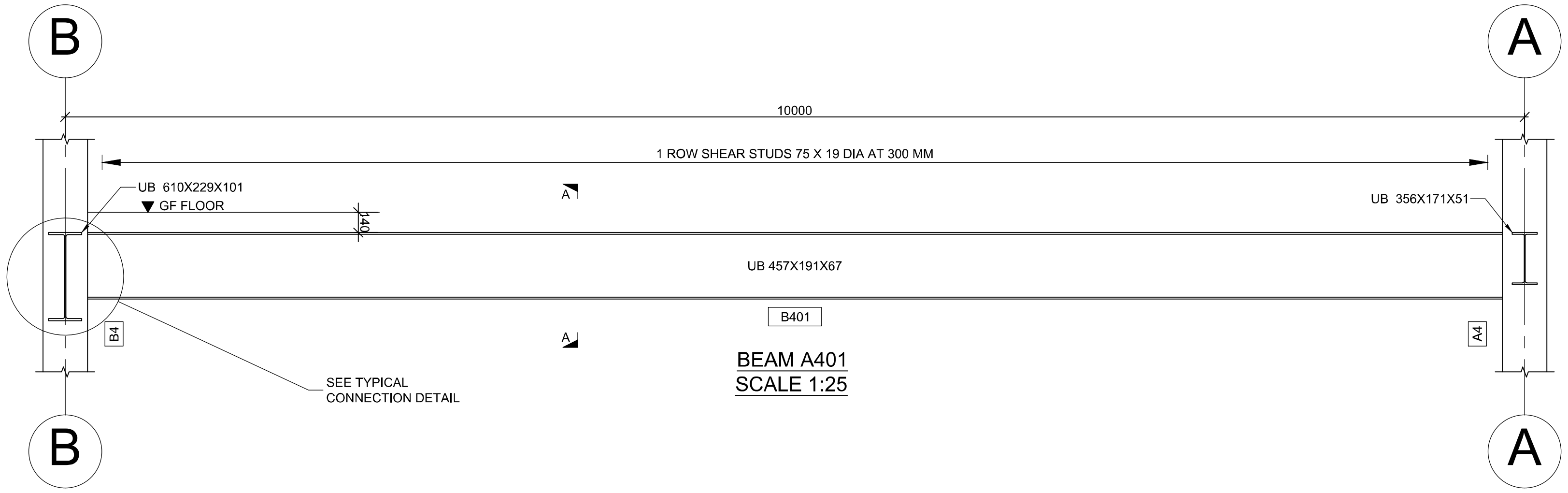
The beam design accounts for both construction-stage and composite-stage loading conditions, with deflection limits verified to meet serviceability requirements (L / 360). The column design employs a UC 305x305x97 section in S355 steel to resist significant axial loads (N_{Ed} = 2760 kN) while maintaining stability against buckling. The column's cross-section is classified as Class 3 due to its flange properties, which influence its elastic behaviour under loading. Flexural buckling resistance is analysed along both major and minor axes using reduction factors (χ_y = 0.949, χ_z = 0.786), ensuring stability under axial compression. Additional checks confirm that torsional and torsional-flexural buckling are not critical failure modes for the column.

The connection design features a partial-depth end plate with M20 bolts (grade 8.8), facilitating robust load transfer under combined shear and tying forces. The end plate is fabricated from S275 steel with a thickness of 10 mm, balancing strength and weldability. Bolts are arranged in four rows with appropriate edge distances to prevent failure modes such as bearing or shear tear-out. Welds are designed with an effective throat thickness of 5.6 mm to ensure adequate shear transfer without over-reliance on bolt capacity

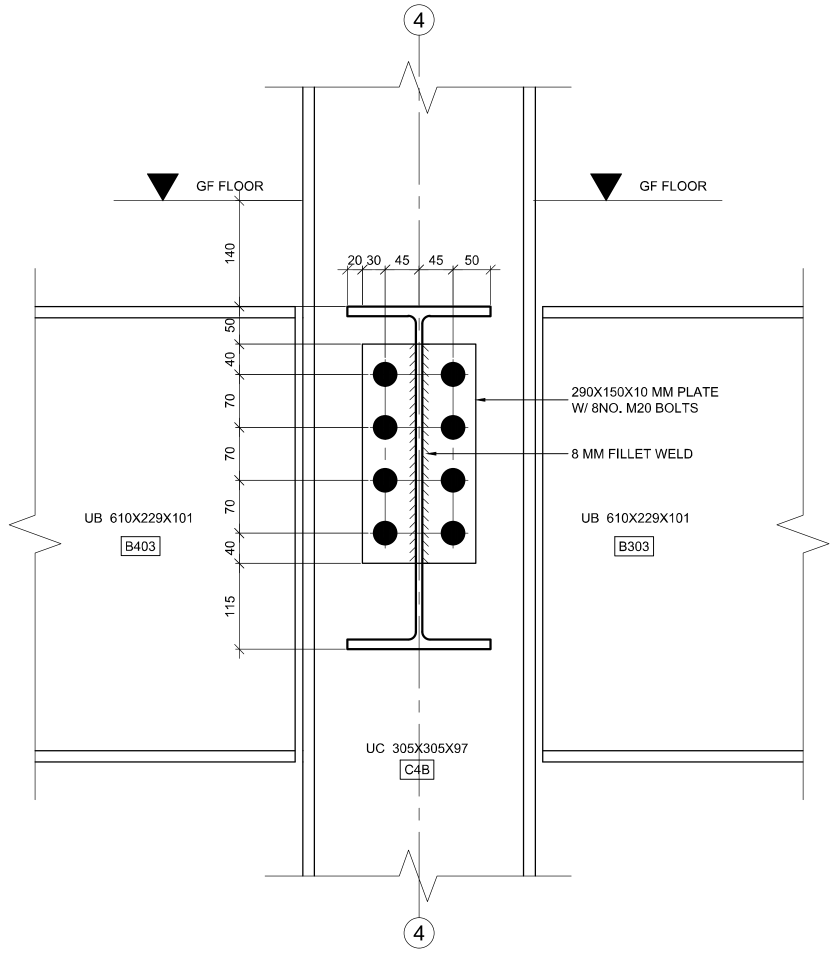
6. Structural Drawings



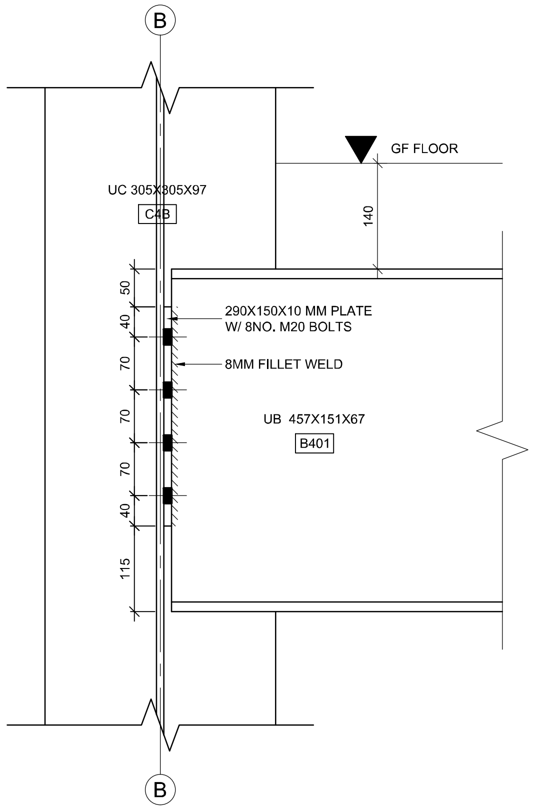
SUB-STRUCTURE LAYOUT PLAN
SCALE 1:100



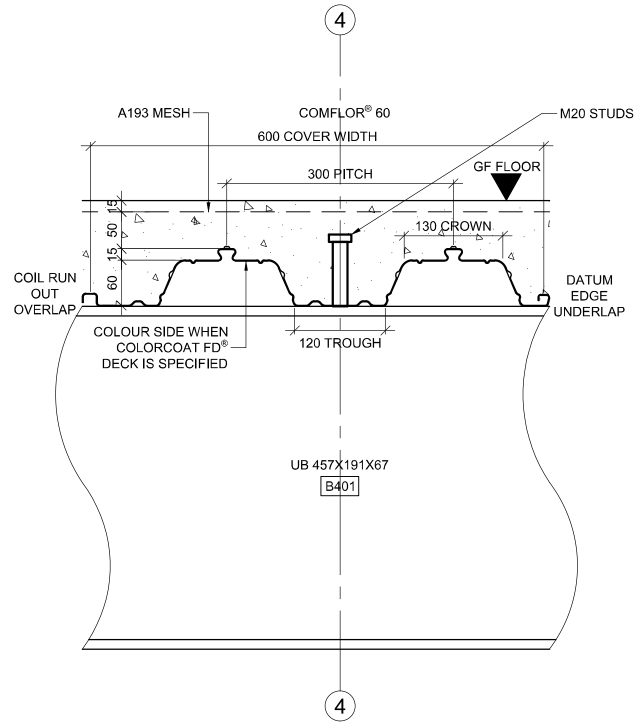
BEAM A401
SCALE 1:25



TYPICAL CONNECTION DETAIL
SCALE 1:5



TYPICAL CONNECTION
DETAIL ELEVATION
SCALE 1:5



SECTION A-A
SCALE 1:5

NOTES

1. All dimensions are in millimeters, unless otherwise stated.
2. This drawing must not be scaled ,only figured dimensions should be used.
3. This drawing must be read in conjunction with relevant Architectural drawings.
4. All steel to BS EN 10025 grade S355.
5. All beam marks to be at north or east end. All column marks to be on flange facing north or east.
6. All beams at 140 below floor level except where shown in brackets e.g. (-580).
7. ←→ indicates the direction of metal decking or cladding.

CLIENT

NAME:

SIGN:............................ DATE:.............................

PROJECT

ENGM042 STEELWORK DRAWINGS

DRAWING TITLE

SUB-STRUCTURE DETAILS

DRAWN BY:

ALLAN NDUBI MAUGO

CHECKED BY:

NAME: ....

DATE: .... SIGN: ....

SHOP DRAWINGS

UNIVERSITY OF SURREY

ISSUE DATE	JAN, 2025
DRAWING NO:	001 - SHT 001

7. Research

7.1. Literature Review

Robustness refers to the ability of a structure to withstand localized failures without leading to disproportionate or catastrophic collapse. Progressive collapse is often triggered by the sudden loss of a critical structural element, such as a column, which initiates a chain reaction of failures. Finite element (FE) modelling is as a powerful tool for analysing this phenomenon. Li et al. (2018) validated FE models against experimental data to study the dynamic responses and collapse modes of steel frames under column removal scenarios. Their research introduced a bearing capacity-based robustness index that accounts for dynamic effects and plastic force redistribution, offering a quantitative measure to evaluate robustness. The study revealed that factors such as damping, strain rate sensitivity, and the relative sizes of beams and columns influence robustness. Frames with strong beams and weak columns demonstrated higher robustness compared to their counterparts with weak beams and strong columns (Li et al., 2018)

Adam et al. (2018) highlighted the importance of beam-column connections in resisting progressive collapse. Tests on rigid and semi-rigid connections showed that rigid connections generally exhibit superior load and deformation capacities. Experimental setups often simulate column loss through pushdown tests or dynamic removal methods, revealing critical load redistribution mechanisms such as catenary action and arching behaviour. These mechanisms serve as secondary lines of defence, allowing structures to absorb additional loads after initial failures (Kiakojouri et al., 2020).

Design strategies for improving robustness often focus on enhancing redundancy and ductility. Redundancy involves providing multiple load paths to redistribute forces in case of local failures, while ductility ensures that structural elements can undergo significant deformations without losing load-carrying capacity. Steeves and Oudah (2024) proposed a holistic structural robustness index that incorporates redundancy and system ductility. Their framework demonstrated that upgrading critical elements, such as transitioning failure modes from buckling to strain hardening, improves both robustness and redundancy (Steeves and Oudah, 2024). Similarly, retrofitting techniques like adding braces or using fibre-reinforced polymers (FRPs) have shown promise in increasing alternative load paths and initial resistance against collapse (Kiakojouri et al., 2020).

High-strength steels and advanced connection designs have been investigated for their potential to enhance robustness. However, Kiakojouri et al. (2020) cautioned that certain design choices might inadvertently promote progressive collapse under specific conditions. For example, overly strong connections can propagate damage from the affected area to undamaged parts of the structure in a "zipper-like" fashion. This highlights the need for balanced design approaches that consider both local and global failure mechanisms.

Energy-based methods have also gained traction for assessing robustness. El-Hajj Diab et al. (2022) introduced an energy-based characterization approach that evaluates the energy required for initial failure and subsequent collapse propagation. Their multi-scenario analysis identified critical failure scenarios and provided a framework for comparing different design configurations. This method allows engineers to prioritize retrofitting measures based on their effectiveness in mitigating progressive collapse (El Hajj Diab et al., 2022).

7.1.1. Quantification of Structural Robustness

1. Deterministic Measures: These include indices based on load redistribution capacity or residual strength after damage. For example, bearing capacity-based indices compare

the load-carrying capacity of intact and damaged structures (Li et al., 2018; Steeves and Oudah, 2024).

2. Probabilistic Measures: These incorporate uncertainties in material properties, loading conditions, and connection behaviour using probabilistic risk assessments. Fragility curves derived from Monte Carlo simulations are often employed to estimate collapse probabilities under various scenarios (Adam et al., 2018; Kiakojouri et al., 2020).
3. Energy-Based Indicators: Recent studies propose energy-based robustness indices that evaluate the energy required for initial failure and subsequent collapse propagation. These measures provide insights into the structure's ability to dissipate energy during extreme events (El Hajj Diab et al., 2022).
4. Scenario-Based Analysis: Multi-scenario analyses investigate potential column loss scenarios to identify critical failure paths and assess robustness under diverse conditions. Pareto front optimization is used to balance competing objectives such as minimizing collapse extent and maximizing resistance (El Hajj Diab et al., 2022).

Despite these advancements, there is no universally accepted framework for robustness quantification. Other challenges include:

- Lack of a unified theory for robustness assessment that integrates elastic-plastic behaviour, dynamic effects, and local damage redistribution comprehensively (Li et al., 2018; Steeves and Oudah, 2024).
- Limited experimental data on full-scale steel frames under multi-hazard scenarios (e.g., combined seismic and blast loads) (Kiakojouri et al., 2020).
- The need for cost-effective retrofitting strategies that balance robustness improvements with economic constraints and environmental sustainability (Steeves and Oudah, 2024).
- Development of innovative resisting mechanisms beyond traditional approaches like catenary action or arching behaviour.

7.2. Methodology

An isolated representative model separately modelled in RFEM as shown in Figure 8. Part of the roof where the transfer structure exists, and the sixth floor were isolated for this analysis. The column subject to removal connects nodes 18-38. The beam of interest runs from node 16 to node 20. The following assumptions were made;

- The rest of the beam spans were ignored, only focusing on the sections up to the column on either side of the notional column.
- The transfer structure at the roof is infinitely rigid.
- The act of removal itself did induce any forces, static or dynamic. Equally, no benefit was taken for enhanced material strength due to fast strain rate effects. (Institution of Structural Engineers, 2010).

At first an alternative load path analysis is carried out and the robustness of the structure measured. The structural robustness metric used to quantify the extent of failure propagation in the structure following an initial local failure (column loss) is the Failure Propagation Index (FPI). It is calculated as the maximum degree of failure propagation (DFP_i) across all N considered failure scenarios. The degree of failure propagation for each scenario i is defined as the ratio of the metric $M(CP_i)$, representing the extent of the collapsed part after propagation, to $M(IDP_i)$, which quantifies the initially damaged part. Mathematically, it is expressed as:

$$DFP_i = M(CP_i) / M(IDP_i) \text{ if } M(IDP_i) > 0, \text{ otherwise } DFP_i = 0$$

The FPI is then determined as:

$$FPI = \max (DFP_i)$$

7.3. Results

After the column loss, the system was able to redistribute the forces and find alternative load paths. The load path defined first by the composite slabs loading the secondary beams (1-16, 2-17, 3-18, 4-19, 5-20). The secondary beams point loads the main beams at their respective end nodes. For those connected to a column they load directly onto the column. Therefore, beam 16-20 is made up of two simple spans supported by columns 16-36, 18-38 and 20-40. The transfer structure is represented by infinitely rigid members 36-40 and 31-35. Before the column removal, deflections were as low as 0.6 mm. Moments were low at 58.51 kN for each span.

After column removal, deflections increased to 11 mm. Moments increased to 116.87 kNm. The main beam rests on the secondary beams, which are supporting the main beam through cantilever action. This is evident by the shear force and bending moment diagram of the beam shown in Figure 9 and Figure 10 respectively. This means that an alternative load path has been found by the system. As the system did not collapse the failure propagation index is equal to 0.

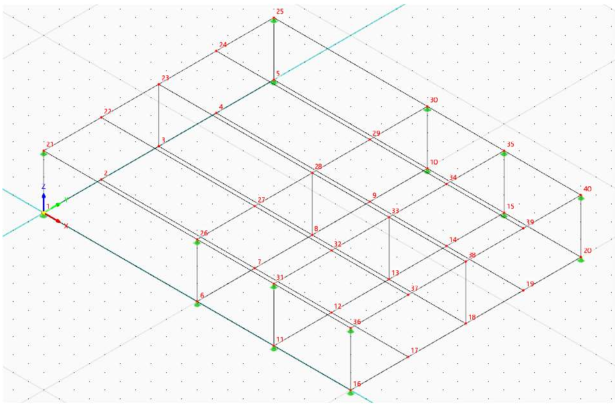


Figure 8: Simplified Finite Element Model

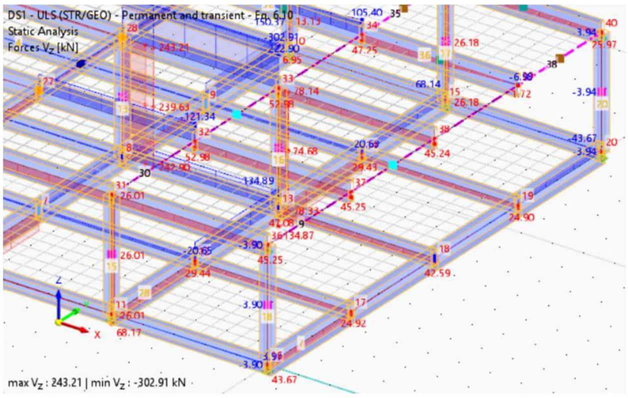


Figure 9: Shear Forces after column loss

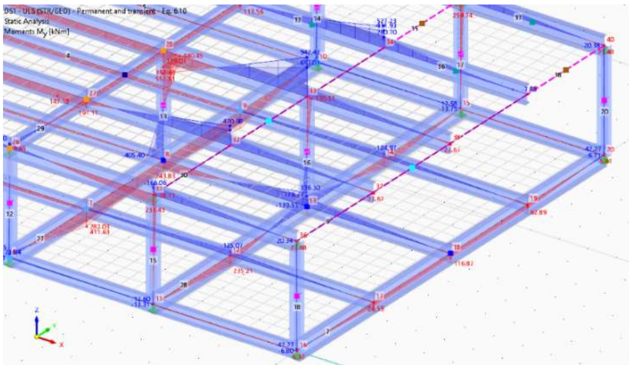


Figure 10: Bending Moments after column loss

7.4. Discussion

The calculated Failure Propagation Index (FPI) of 0 indicates that no further collapse occurred beyond the initially damaged region. This reflects an ideal scenario where failure propagation is fully arrested, demonstrating a high level of robustness. The FPI metric effectively quantifies robustness by focusing on the extent of failure propagation, making it a valuable tool for assessing structural performance under localized damage scenarios (El Hajj Diab et al., 2022). Compared to other robustness measures, such as energy-based indices or residual capacity metrics, FPI offers simplicity and calculability while maintaining expressiveness regarding collapse propagation (Adam et al., 2018; El Hajj Diab et al., 2022).

While FPI captures failure propagation, it does not account for other aspects of robustness, such as energy dissipation capacity or ductility. For instance, energy-based measures like those proposed by El-Hajj Diab et al. (2022) assess both propagation extent and the energy required for initial failure, offering a more comprehensive view of robustness under dynamic conditions. Similarly, probabilistic measures incorporate uncertainties in loading and material properties but are computationally intensive and less practical for routine applications (Li et al., 2018).

A point of concern is on node 18 where a connection exists where there is maximum moment along the beam as common initiation points for progressive collapse in steel frames (Li et al., 2018). The structure demonstrates some degree of robustness by developing an alternative load path. Enhancing connection detailing can improve post-failure resistance and delay collapse propagation however overly strong connections can propagate damage from the affected area to undamaged parts of the structure in a "zipper-like" fashion.

Codified approaches to structural robustness, such as those in EN 1991-1-7 or UFC 4-023-03, typically emphasize prescriptive design methods like alternate load path (ALP) analysis or key element design. These methods aim to ensure that structures can withstand localized failures without disproportionate collapse (Kiakojouri et al., 2020; Steeves and Oudah, 2024). The ALP method was utilised to demonstrate that the structure can redistribute loads effectively after column removal.

However, codified measures often lack quantitative metrics like FPI, for assessing robustness explicitly. Instead, they rely on qualitative assessments or simplified deterministic checks (e.g., demand-capacity ratios). While these approaches are practical for design purposes, they may not fully capture the nuanced behaviour of structures under extreme events. For example, deterministic measures such as residual capacity ratios fail to account for ductility or dynamic effects comprehensively (Li et al., 2018; Steeves and Oudah, 2024). In contrast, indices like FPI provide a more transparent and quantifiable measure of robustness but may require additional computational effort.

Integrating non-structural elements like slabs or partitions has been shown to contribute significantly to alternative load paths and could be explored further for this system (Adam et al., 2018; Kiakojouri et al., 2020).

Also, the infinitely rigid transfer members (36-40 and 31-35) likely contributed to limiting collapse propagation by confining load redistribution within a localized region. This behavior aligns with segmentation strategies proposed in robustness literature, which aim to isolate damage and prevent global collapse .(Adam et al., 2018; El Hajj Diab et al., 2022)

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