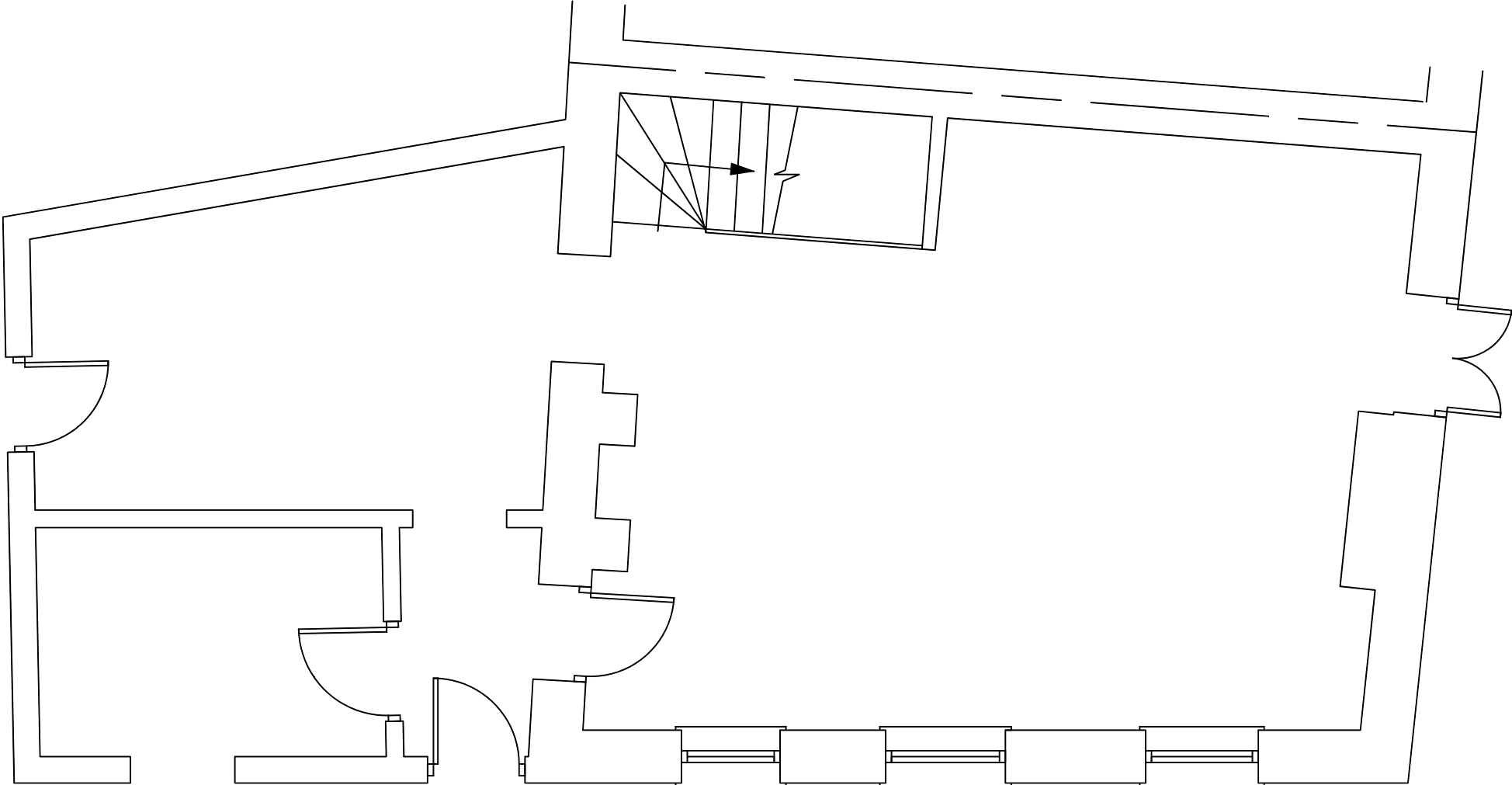
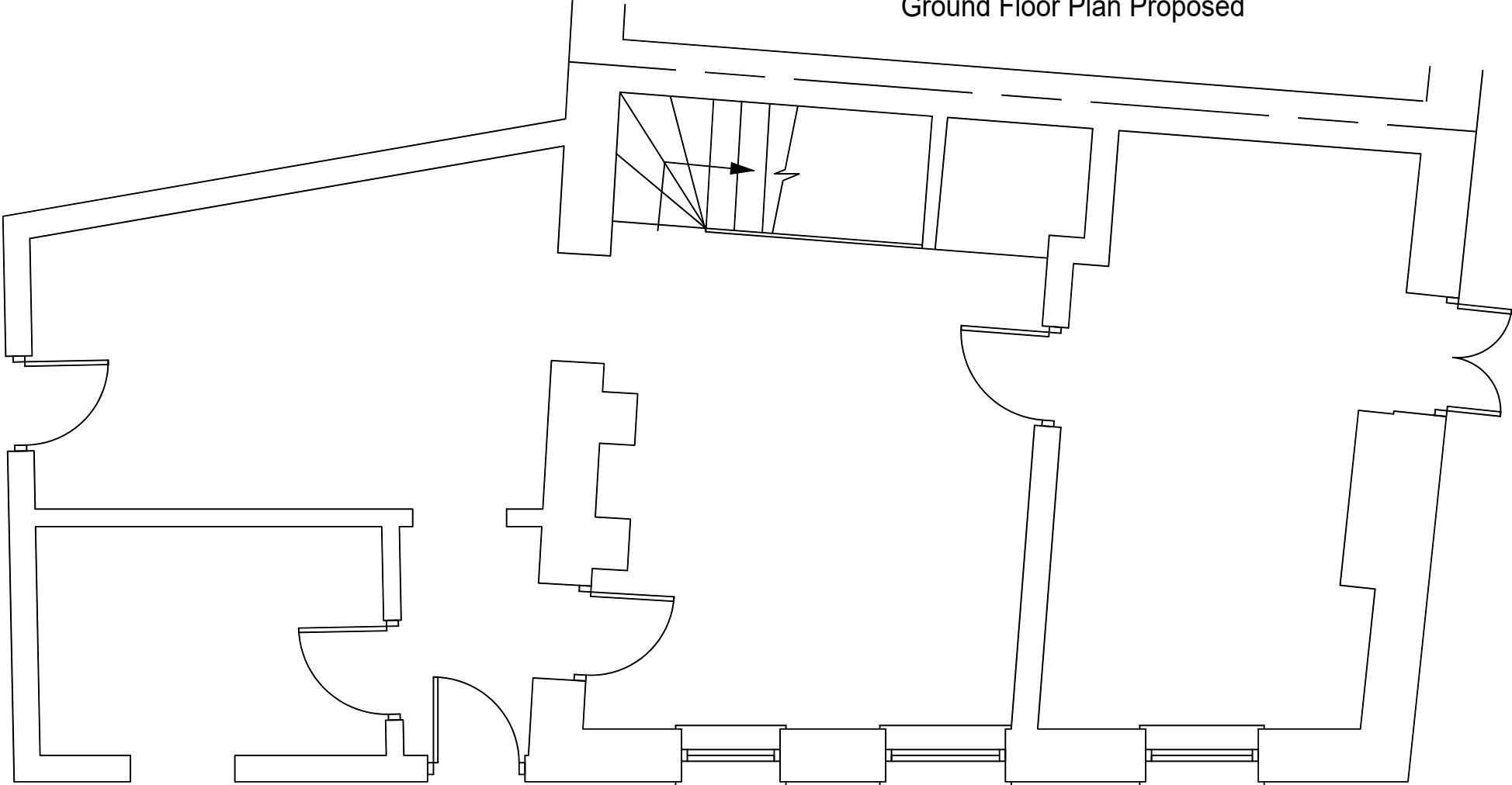
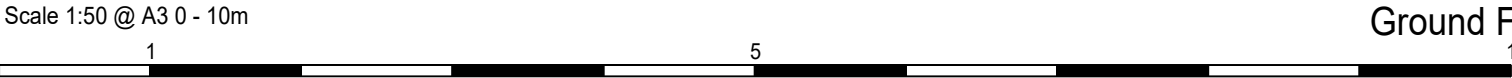




Ground Floor Plan Proposed



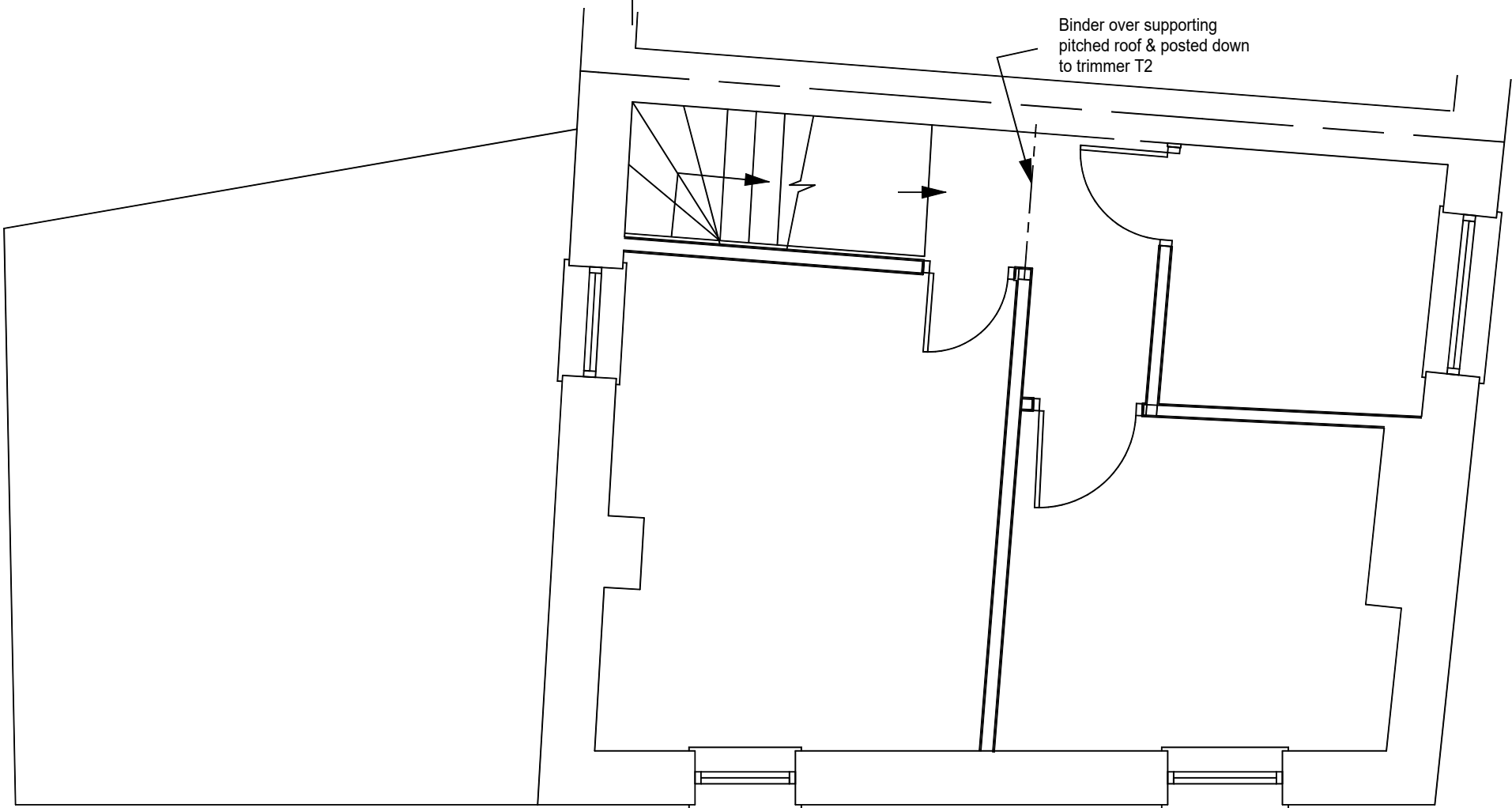
Ground Floor Plan Existing



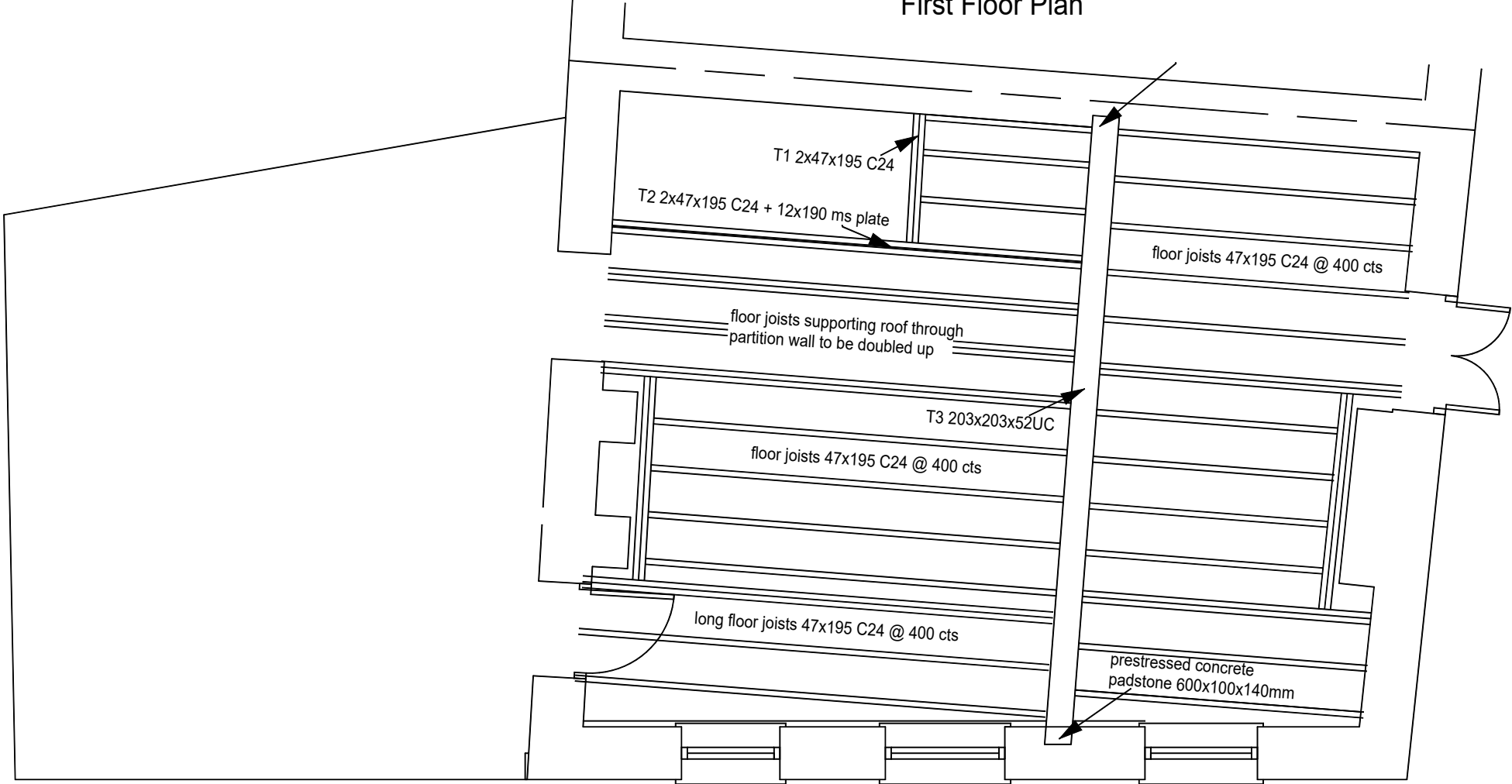
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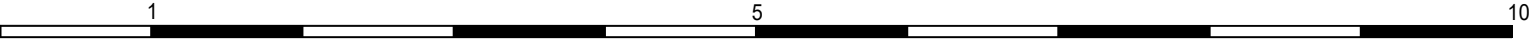


First Floor Plan



First Floor Structural Plan

Scale 1:50 @ A3 0 - 10m



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TIMBER FLOOR (to be min 1/2hrfr)

Reduc Micro 17 on 22mm t&g >15kg/m2 flooring screwed to floor joists on 47x195 C24 joists @ 400cts with noggins mid span & ends. hung on joists hangers fixed to plate bolted to walls. Insert 200mm wired Rockwool between Underside soundbreakers with 2x12.5mm plasterboard & skim coat. Provided ms anchors all round at 1200 cts

STEELWORK

Weldable structural steel in accordance with BS 4360. For Fire Resistance the steel work is to be encased with 15mm Glasroc S Frameless Encasement stapled with joints taped & sealed. Steel is to be free of rust & painted. Padstones @ all seatings, sizes as indicated on structural calculations. Min bearing 100mm on party walls & 150mm @ all other times. Ensure transfer of loads by dry packing as required. Double beams to be bolted @ 600 staggered cts with spacers between.

STUDWORK

Stud walls to be constructed in accordance with drawings, sole & head plates 47x97, uprights, 47x97 @ 400cts, 97x97 corner posts & noggins 1/3 & 2/3 Joists under walls parallel to walls to be min doubled up. 12.5mm plasterboard to either side to give 1/2hrfr, joints taped & sealed. Provide 100mm Rockwool between for sound insulation.

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First Floor Plans Existing & Proposed

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Beam: Roof Binder Over Stairs

Span: 1.1 m.

	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp
U G	o.w.	0.1	0		L	0.06	0.06
U G	Pitched Roof	1x5	0		L	2.75	2.75
U QH	Pitched Roof	0.75x5	0		L	2.06	2.06
Total load (unfactored): 9.74 kN						4.87	4.87
Dead/Permanent (unfactored): 5.61 kN						2.81	2.81
Live/Variable (unfactored): 4.13 kN						2.06	2.06
(6.10): 13.76 kN						6.88	6.88

Load types: U:UDL; Load positions are measured in m. from R1

Load durations: G: Dead; Qx: Imposed; QH: Roof

Maximum B.M. = 1.89 kNm (6.10) at 0.55 m. from R1

Maximum S.F. = 6.88 kN (6.10) at R1

Mid-span deflections: Dead: $0.10 \times 10^9/EI$ (E in N/mm^2 , I in cm^4)

Live: $0.07 \times 10^9/EI$

Total: $0.17 \times 10^9/EI$

Timber beam calculation to BS EN1995-1-1 using C24 timber

Use 2no 47 x 147 C24 5.8 kg/m approx

$W_{el,y} = 338.5 \text{ cm}^3$ $I_y = 2,488 \text{ cm}^4$

Timber grade: C24

Grade bending strength, $f_{m,k} = 24.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Grade shear strength, $f_{v,k} = 4.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

$E_{0.05} = 7,400 \text{ N/mm}^2$; $E_{0,mean} = 11,000 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Material partial factor, $g_M = 1.3$ [EC5 UK Table NA.3]

Loading modification factor, $k_{mod} = 0.6$ (Service class 1; Live load duration: Permanent) [EC5 Tables 2.2/3.1]

Load sharing factor, $k_{sys} = 1.0$

Deflection modification factor, $k_{def} = 0.60$ (Service class 1, solid timber/glulam/LVL) [EC5 Table 3.2]

Bending

Height factor, $k_h = \min((150/h)^{0.2}, 1.3) = (150/147)^{0.2} = 1.00$ [EC5 3.2(3)]

Design bending strength, $f_{m,y,d} = f_{m,k} \cdot k_{mod} \cdot k_h \cdot k_{sys} / g_M = 24.0 \times 0.60 \times 1.00 \times 1.0/1.30 = 11.12 \text{ N/mm}^2$

Design bending stress, $s_{m,y,d} = 1.89 \times 1000/339 = 5.59 \text{ N/mm}^2$ OK

Bending resistance = $11.12 \times 339/1000 = 3.77 \text{ kNm}$

Shear

Effective width for shear, $b_{ef} = k_{cr} \cdot b = 0.67 \times 94 = 63 \text{ mm}$. [A1:2008 (6.13a)]

Design shear strength, $f_{v,d} = f_{v,k} \cdot k_{mod} \cdot k_{sys} / g_M = 4.00 \times 0.60 \times 1.0/1.30 = 1.85 \text{ N/mm}^2$

Design shear stress, $s_{v,y,d} = 6.88 \times 1000 \times (3/2)/(63.0 \times 147) = 1.11 \text{ N/mm}^2$ OK

Shear resistance = $1.85 \times 63 \times 147 \times (2/3)/1000 = 11.39 \text{ kN}$

Deflection

Final deflection limit = $L/250 = 4.40 \text{ mm}$

Final deflection, $u_{fin} = S u_{inst} (1 + y_2 \cdot k_{def})$ [EC2 Eq.2.29/2.30]

Instantaneous mid-span shear deflection, $u_{inst,v} = 1.2 M_{unf} / (G_{mean} A)$ where ($G = E/16$)

$u_{inst,v} = 1.2 \times 1.34 \times 10^6 / ((11,000/16) \times 94 \times 147) = 0.17 \text{ mm}$

Final shear deflection is assumed to increase in proportion to total bending deflection

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Mid-span deflections:	x 1e ⁸ /EI	Inst. mm	y ₂	k _{def}	Fin. mm	
Dead:	0.10	0.36	1.00	0.60	0.57	
Live QH:	0.07	0.26	0.00	0.60	0.26	
Shear deflection:		0.17			0.23	
Total mm.		<u>0.79</u>			<u>1.06</u>	OK

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Beam: Floor Joists with point loads from roof & Wall

Span: 4.0 m.

	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp
U G	o.w.	0.1	0		L	0.20	0.20
U G	Floor	0.5x0.4	0		L	0.40	0.40
U QA	Floor	1.5x0.4	0		L	1.20	1.20
P G	Roof & Partition	2	3.2			0.40	1.60
Total load (unfactored): 5.60 kN						2.20	3.40
Dead/Permanent (unfactored): 3.20 kN						1.00	2.20
Live/Variable (unfactored): 2.40 kN						1.20	1.20
(6.10): 7.92 kN						3.15	4.77

Load types: U:UDL; P:Point load; Load positions are measured in m. from R1

Load durations: G: Dead; Qx: Imposed: QA: Residential

Maximum B.M. = 3.80 kNm (6.10) at 2.41 m. from R1

Maximum S.F. = -4.77 kN (6.10) at R2

Mid-span deflections: Dead: $2.51 \times 10^8/EI$ (E in N/mm^2 , I in cm^4)

Live: $2.00 \times 10^8/EI$

Total: $4.51 \times 10^8/EI$

Timber beam calculation to BS EN1995-1-1 using C24 timber

Use 2no 47 x 195 C24 7.7 kg/m approx

$W_{el,y} = 595.7 \text{ cm}^3$ $I_y = 5,808 \text{ cm}^4$

Timber grade: C24

Grade bending strength, $f_{m,k} = 24.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Grade shear strength, $f_{v,k} = 4.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

$E_{0.05} = 7,400 \text{ N/mm}^2$; $E_{0,mean} = 11,000 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Material partial factor, $g_M = 1.3$ [EC5 UK Table NA.3]

Loading modification factor, $k_{mod} = 0.6$ (Service class 1; Live load duration: Permanent) [EC5 Tables 2.2/3.1]

Load sharing factor, $k_{sys} = 1.0$

Deflection modification factor, $k_{def} = 0.60$ (Service class 1, solid timber/glulam/LVL) [EC5 Table 3.2]

Bending

Height factor, $k_h = 1.0$ [EC5 3.2(3)]

Design bending strength, $f_{m,y,d} = f_{m,k} \cdot k_{mod} \cdot k_h \cdot k_{sys} / g_M = 24.0 \times 0.60 \times 1.00 \times 1.0/1.30 = 11.08 \text{ N/mm}^2$

Design bending stress, $s_{m,y,d} = 3.80 \times 1000/596 = 6.38 \text{ N/mm}^2$ OK

Bending resistance = $11.08 \times 596/1000 = 6.60 \text{ kNm}$

Shear

Effective width for shear, $b_{ef} = k_{cr} \cdot b = 0.67 \times 94 = 63 \text{ mm}$. [A1:2008 (6.13a)]

Design shear strength, $f_{v,d} = f_{v,k} \cdot k_{mod} \cdot k_{sys} / g_M = 4.00 \times 0.60 \times 1.0/1.30 = 1.85 \text{ N/mm}^2$

Design shear stress, $s_{v,y,d} = 4.77 \times 1000 \times (3/2)/(63.0 \times 195) = 0.58 \text{ N/mm}^2$ OK

Shear resistance = $1.85 \times 63 \times 195 \times (2/3)/1000 = 15.12 \text{ kN}$

Deflection

Final deflection limit = $L/250 = 16.00 \text{ mm}$

Final deflection, $u_{fin} = S u_{inst} (1 + y_2 \cdot k_{def})$ [EC2 Eq.2.29/2.30]

Instantaneous mid-span shear deflection, $u_{inst,v} = 1.2 M_{unf} / (G_{mean} A)$ where ($G = E/16$)

$u_{inst,v} = 1.2 \times 2.60 \times 10^6 / ((11,000/16) \times 94 \times 195) = 0.25 \text{ mm}$

Final shear deflection is assumed to increase in proportion to total bending deflection

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Mid-span deflections:	$\times 10^8/EI$	Inst. mm	y_2	k_{def}	Fin. mm	
Dead:	2.51	3.94	1.00	0.60	6.30	
Live QA:	2.00	3.13	0.30	0.60	3.69	
Shear deflection:		0.25			0.35	
Total mm.		<u>7.31</u>			<u>10.34</u>	OK

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Beam: Trimmer T1

Span: 1.1 m.

	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp
U G	o.w.	0.15	0		L	0.08	0.08
U G	Floor	0.5x1.4/2	0		L	0.19	0.19
U QA	Floor	1.5x1.4/2	0		L	0.58	0.58
U G	Stairs	0.5x2.4/2	0		L	0.33	0.33
U QA	Stairs	1.5x2.4/2	0		L	0.99	0.99
Total load (unfactored): 4.35 kN						2.17	2.17
Dead/Permanent (unfactored): 1.21 kN						0.61	0.61
Live/Variable (unfactored): 3.13 kN						1.57	1.57
(6.10): 6.34 kN						3.17	3.17

Load types: U:UDL; Load positions are measured in m. from R1

Load durations: G: Dead; Qx: Imposed; QA: Residential

Maximum B.M. = 0.87 kNm (6.10) at 0.55 m. from R1

Maximum S.F. = 3.17 kN (6.10) at R1

Mid-span deflections: Dead: $0.02 \times 10^8/EI$ (E in N/mm^2 , I in cm^4)

Live: $0.05 \times 10^8/EI$

Total: $0.08 \times 10^8/EI$

Timber beam calculation to BS EN1995-1-1 using C24 timber

Use 2no 47 x 195 C24 7.7 kg/m approx

$W_{el,y} = 595.7 \text{ cm}^3$ $I_y = 5,808 \text{ cm}^4$

Timber grade: C24

Grade bending strength, $f_{m,k} = 24.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Grade shear strength, $f_{v,k} = 4.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

$E_{0.05} = 7,400 \text{ N/mm}^2$; $E_{0,mean} = 11,000 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Material partial factor, $g_M = 1.3$ [EC5 UK Table NA.3]

Loading modification factor, $k_{mod} = 0.6$ (Service class 1; Live load duration: Permanent) [EC5 Tables 2.2/3.1]

Load sharing factor, $k_{sys} = 1.0$

Deflection modification factor, $k_{def} = 0.60$ (Service class 1, solid timber/glulam/LVL) [EC5 Table 3.2]

Bending

Height factor, $k_h = 1.0$ [EC5 3.2(3)]

Design bending strength, $f_{m,y,d} = f_{m,k} \cdot k_{mod} \cdot k_h \cdot k_{sys} / g_M = 24.0 \times 0.60 \times 1.00 \times 1.0/1.30 = 11.08 \text{ N/mm}^2$

Design bending stress, $s_{m,y,d} = 0.87 \times 1000/596 = 1.46 \text{ N/mm}^2$ OK

Bending resistance = $11.08 \times 596/1000 = 6.60 \text{ kNm}$

Shear

Effective width for shear, $b_{ef} = k_{cr} \cdot b = 0.67 \times 94 = 63 \text{ mm}$. [A1:2008 (6.13a)]

Design shear strength, $f_{v,d} = f_{v,k} \cdot k_{mod} \cdot k_{sys} / g_M = 4.00 \times 0.60 \times 1.0/1.30 = 1.85 \text{ N/mm}^2$

Design shear stress, $s_{v,y,d} = 3.17 \times 1000 \times (3/2)/(63.0 \times 195) = 0.39 \text{ N/mm}^2$ OK

Shear resistance = $1.85 \times 63 \times 195 \times (2/3)/1000 = 15.12 \text{ kN}$

Deflection

Final deflection limit = $L/250 = 4.40 \text{ mm}$

Final deflection, $u_{fin} = S u_{inst} (1 + y_2 \cdot k_{def})$ [EC2 Eq.2.29/2.30]

Instantaneous mid-span shear deflection, $u_{inst,v} = 1.2 M_{unf} / (G_{mean} A)$ where ($G = E/16$)

$u_{inst,v} = 1.2 \times 0.597 \times 10^6 / ((11,000/16) \times 94 \times 195) = 0.06 \text{ mm}$

Final shear deflection is assumed to increase in proportion to total bending deflection

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Mid-span deflections:	$\times 10^8/EI$	Inst. mm	y_2	k_{def}	Fin. mm	
Dead:	0.02	0.03	1.00	0.60	0.05	
Live QA:	0.05	0.09	0.30	0.60	0.10	
Shear deflection:		0.06			0.07	
Total mm.		<u>0.17</u>			<u>0.23</u>	OK

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Beam: Trimmer T2

Span: 4.0 m.

	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp
U G	o.w.	0.3	0		L	0.60	0.60
U G	Floor	0.5x0.4	0		L	0.40	0.40
U QA	Floor	1.5x0.4	0		L	1.20	1.20
R G	Studwork Partition	0.5x2.4	0		3.2	2.30	1.54
P G	T1	2.2	2.5			0.83	1.38
P G	Roof Binder	5	3.2			1.00	4.00
Total load (unfactored): 15.44 kN						6.33	9.11
Dead/Permanent (unfactored): 13.04 kN						5.13	7.91
Live/Variable (unfactored): 2.40 kN						1.20	1.20
(6.10): 21.20 kN						8.72	12.48

Load types: U:UDL; R:Part UDL; P:Point load; Load positions are measured in m. from R1
Load durations: G: Dead; Qx: Imposed; QA: Residential

Maximum B.M. = 11.83 kNm (6.10) at 2.50 m. from R1

Maximum S.F. = -12.48 kN (6.10) at R2

Mid-span deflections: Dead: $11.76 \times 10^9/EI$ (E in N/mm^2 , I in cm^4)

Live: $2.00 \times 10^9/EI$

Total: $13.76 \times 10^9/EI$

Timber beam calculation to BS EN1995-1-1 using C24 timber

Use 2no 47 x 195 C24 + 12 x 190 flitch plate 25.6 kg/m approx

Timber: $W_{el,y} = 595.7 \text{ cm}^3$ $I_y = 5,808 \text{ cm}^4$

Flitch plate: $W_{el,y} = 72.2 \text{ cm}^3$ $I_y = 686 \text{ cm}^4$ $E_{steel} = 210,000 \text{ N/mm}^2$ $1,440 \times 10^9 \text{ Nmm}^2$

Timber grade: C24

Grade bending strength, $f_{m,k} = 24.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Grade shear strength, $f_{v,k} = 4.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

$E_{0.05} = 7,400 \text{ N/mm}^2$; $E_{0,mean} = 11,000 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Material partial factor, $\gamma_M = 1.3$ [EC5 UK Table NA.3]

Loading modification factor, $k_{mod} = 0.6$ (Service class 1; Live load duration: Permanent) [EC5 Tables 2.2/3.1]

Load sharing factor, $k_{sys} = 1.0$

Deflection modification factor, $k_{def} = 0.60$ (Service class 1, solid timber/glulam/LVL) [EC5 Table 3.2]

Loading will be carried by the timber members and flitch plate in proportion to their EI values. Checks are made using the mean and minimum E-values for timber to produce worst case stresses on timber and steel members respectively. See TRADA guidance document GD9, 2008, for more information.

Calculate timber bending and shear enhancement factor (k_{fi})

Design strengths can be multiplied by 1.04 if the flitch plate takes 0.2-0.8 of total load (using $E_{0,mean}$) GD9 Eq.(5)

$EI_{timber} = 11,000 \times 5,808 \times 10^4 = 639 \times 10^9 \text{ Nmm}^2$; $K_{timber} = EI_{timber}/EI_{total} = 639/(639 + 1,440) = 0.307$

$K_{steel} = 1 - 0.307 = 0.693$; $k_{fi} = 1.04$

Calculate effect of bolt holes

M16 bolts, staggered alternately above/below NA, centres offset 0 mm from NA: assume 17 mm holes

Bending stress factor for a single hole = $(h^2(h^2 - h.d' + 2d'.y)) / (h^4 - h^3d' + h.d'^3 + d'^4 - 12h.d'y^2)$ GD9 Eq.(10)

h = member height; d' = hole diameter; y = offset centre of hole from member centre

Resulting factors: Timber: 1.0 Steel: 1.0

Timber bending

Design bending moment, $M_{y,d} = 11.83 \text{ kNm}$

Design bending stress, $s_{m,y,d} = 0.307 \times 11.83 \times 1.000 \times 1000/595.7 = 6.10 \text{ N/mm}^2$

Height factor, $k_h = 1.0$ [EC5 3.2(3)]

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Design bending strength, $f_{m,y,d} = f_{m,k} \cdot k_{mod} \cdot k_{sys} \cdot k_{hy} \cdot k_{fl} / g_m$

$$f_{m,y,d} = 24.0 \times 0.60 \times 1.0 \times 1.00 \times 1.04/1.30 = 11.52 \text{ N/mm}^2 \text{ OK}$$

Timber shear

Design shear force, $F_{v,d} = 12.48 \text{ kN}$

Effective width for shear, $b_{ef} = k_{cr} \cdot b = 0.67 \times 94 = 63 \text{ mm}$. [A1:2008 (6.13a)]

Design shear stress, $t_a = 0.307 \times 12.48 \times 1000 \times (3/2)/(63 \times 195) = 0.47 \text{ N/mm}^2$

Design shear strength, $f_{v,d} = f_{v,k} \cdot k_{mod} \cdot k_{sys} \cdot k_{fl} / g_m = 4.00 \times 0.60 \times 1.0 \times 1.04/1.30 = 1.92 \text{ N/mm}^2 \text{ OK}$

Shear resistance = $1.92 \times 63.0 \times 195 \times (2/3)/(0.307 \times 1000) = 51.2 \text{ kN}$

Bearings

Grade compressive strength perpendicular to grain, $f_{c,90,d} = f_{c,90,k} \cdot k_{mod} \cdot k_{sys} \cdot k_{fl} / g_m$

$$f_{c,90,d} = 2.50 \times 0.60 \times 1.0 \times 1.04/1.3 = 1.20 \text{ N/mm}^2$$

Minimum bearing lengths:

$$R1: 8.72 \times 1000/(1.20 \times 94) = 78 \text{ mm}$$

(subject to adequate support under bearing)

$$R2: 12.5 \times 1000/(1.20 \times 94) = 111 \text{ mm}$$

Deflection

Calculate instantaneous and final deflections, $u_{inst}, u_{fin}; u_{fin} = S(1 + k_{def} \cdot y_2) u_{inst}$

Dur	u_{inst}	k_{def}	y_2	u_{fin}
G	11.76	0.60	1.00	18.82
QA	2.00	0.60	0.30	2.36
Tot	13.76			21.18

Instantaneous deflection

Timber carries $639/(639+1,440) = 0.307$ of total load (as above)

Bending deflection = $0.307 \times 13.76 \times 10^8/(639 \times 10^9/10^4) = 6.6 \text{ mm}$

Mid-span shear deflection = $0.307 \times 1.2 \times 8.06 \times 10^6/(E/16) \times 94 \times 195 = 0.2 \text{ mm}$ (0.036 of bending deflection)

Total deflection = $6.6 + 0.2 = 6.9 \text{ mm}$ (0.0017 L) OK

Final deflection

$$E_{fin} = E_{inst} \times u_{inst}/u_{fin}$$

$$E_{mean,fin} = 11,000 \times 13.76/21.18 = 7,148 \text{ N/mm}^2$$

Using $E_{mean,fin}$, Timber EI = $7,148 \times 5,808 \times 10^4 = 415 \times 10^9 \text{ Nmm}^2$

Timber carries $415/(415+1,440) = 0.224$ of total load

Final deflection = $0.224 \times 1.036 \times 21.18 \times 10^8/(415 \times 10^9/10^4) = 11.8 \text{ mm}$ (0.0030 L) OK

Flitch plate bending

$E_{min,fin} = 7,400 \times 13.76/21.18 = 4,808 \text{ N/mm}^2$; Timber EI = $4,808 \times 5,808 \times 10^4 = 279 \times 10^9 \text{ Nmm}^2$

Using $E_{min,fin}$ for timber, flitch plate carries $1,440/(279 + 1,440) = 0.838$ of total load

Per TRADA GD9, factor load by 1.1 to allow for slip and shear deflection in plate

Flitch plate $f_{bc} = 0.838 \times 11.83 \times 1.1 \times 1.000 \times 1000/72.2 = 150.9 \text{ N/mm}^2 \text{ OK}$

Moment capacity is limited by steel plate:

$$\text{Moment capacity} = 275 \times 72.2/(1000 \times 1.000 \times 1.1 \times 0.838) = 21.55 \text{ kNm (timber: 22.33 kNm)}$$

Bolting:

Use M16 4.6 bolts. Bolt numbers are calculated assuming worst case load on flitch plate

C24 characteristic density, $f_k = 350 \text{ kgm}^{-3}$; compressive strength 90° , $f_{c,90,k} = 2.5 \text{ N/mm}^2$

Embedment strength, load in direction of grain, $f_{h,0,k} = 0.082(1 - 0.01d)p_k = 0.082 \times (1 - 0.16) \times 350 = 24.11 \text{ N/mm}^2$ (8.32)

$k_{90} = 1.35 + 0.015d = 1.35 + 0.24 = 1.59$ EC5 (8.30) softwood

$f_{h,1,k}$ = embedment strength 90° to grain, $f_{h,90,k} = f_{h,0,k}/k_{90} = 24.11/1.59 = 15.16 \text{ N/mm}^2$ (8.31)

Bolt characteristic axial capacity, $F_{ax,Rk} = 3 \cdot f_{c,90,k} \cdot A_{washer} = 3 \times 2.5 \times 1,555/1000 = 11.66 \text{ kN}$ (compression under washer)

$F_{ax,Rk}$ has been calculated using a washer diameter of $3d$, $d+2$ hole

Characteristic bolt yield moment, $M_{y,Rk} = 0.3 \cdot f_{u,k} \cdot d^{2.6} = 0.3 \times 400 \times 16^{2.6} = 162141 \text{ Nmm}$ EC5 (8.30)

Bolt capacity, $F_{v,Rk}$ (per plane) EC5 Eq.(8.11)

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Eq.(8.11f) (timber bearing): $F_{v,Rk} = f_{h,1,k} \cdot t_1 \cdot d = 11.40 \text{ kN}$

Eq.(8.11g) (bolt bending): $F_{v,Rk} = f_{h,1,k} \cdot t_1 \cdot d \left(\sqrt{2 + 4M_{y,Rk} / (f_{h,1,k} \cdot d \cdot t_1^2)} - 1 \right) + F_{ax,Rk} / 4^* = 11.28 \text{ kN} <<<$

Eq.(8.11h) (bolt yielding): $F_{v,Rk} = 2.3 \sqrt{M_{y,Rk} \cdot f_{h,1,k} \cdot d} + F_{ax,Rk} / 4^* = 17.34 \text{ kN}$

* Rope effect: use (0.25 x Johansen load) [first term] if $< F_{ax,Rk} / 4$ EC5 8.2.2(2)

Bolt design load capacity (double shear), $F_{v,Rd} = k_{mod} \cdot F_{v,Rk} / \gamma_m = 2 \times 0.6 \times 11.28 / 1.3 = 10.42 \text{ kN}$

Bearings: R1 (8.72kN): Required number of bolts = $0.838 \times 8.72 / 10.42 = 0.70$ i.e. 1 bolt min.

R2 (12.5kN): Required number of bolts = $0.838 \times 12.5 / 10.42 = 1.00$ i.e. 2 bolts min.

For load transference a minimum of 2 bolts are also required across the span

Recommended bolting pattern across span: Bolts spaced at lesser of 2.5h or 600mm (i.e. 487 mm c/s) , alternately set max 49 mm (h/4) above and below centre line of beam with additional centred bolt(s) at each significant point load position - see GD9 for further guidance

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Beam: Trimmer T3

Span: 5.2 m.

	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp
U G	o.w.	0.55	0		L	1.43	1.43
U G	Studwork Partition	0.5x2.4	0		L	3.12	3.12
R G	Floor	0.5x4.2/2	0		1.1	1.03	0.12
R QA	Floor	1.5x4.2/2	0		1.1	3.10	0.37
R G	Floor	0.5x6.6/2	1.1		5.2	2.67	4.10
R QA	Floor	1.5x6.6/2	1.1		5.2	8.00	12.29
R G	Pitched Roof	1x5	1.1		2.5	4.58	2.42
R QH	Pitched Roof	0.75x5	1.1		2.5	3.43	1.82
P G	T2	10	1.1			7.88	2.12
Total load (unfactored): 63.03 kN						35.24	27.79
Dead/Permanent (unfactored): 34.02 kN						20.71	13.31
Live/Variable (unfactored): 29.01 kN						14.53	14.48
(6.10): 89.44 kN						49.76	39.68

Load types: U:UDL; R:Part UDL; P:Point load; Load positions are measured in m. from R1

Load durations: G: Dead; Qx: Imposed; QA: Residential; QH: Roof

Maximum B.M. = 64.4 kNm (6.10) at 2.20 m. from R1

Maximum S.F. = 49.8 kN (6.10) at R1

Mid-span deflections: Dead: $66.6 \times 10^8/EI$ (E in N/mm^2 , I in cm^4)

Live: $58.4 \times 10^8/EI$

Total: $125.0 \times 10^8/EI$

Beam calculation to BS EN1993.1.1 using S355 steel

SECTION SIZE : 203 x 203 x 52 UKC S355 (Class 1, plastic)

D=206.2 mm B=204.3 mm t=7.9 mm T=12.5 mm $I_y=5,260 \text{ cm}^4$ $i_z=5.18 \text{ cm}$ $W_{pl,y}=567 \text{ cm}^3$ $W_{el,y}=510 \text{ cm}^3$

Classification: Flange: $c/t = 88.0/12.5 = 7.04 \leq 9e(7.32)$: Class 1, plastic

EC3 Table 5.2 Web: $c/t = 160.8/7.9 = 20.4 \leq 72e(58.6)$: Class 1, plastic

Shear

Design shear force, $V_{Ed} = 49.76 \text{ kN}$

Shear area, $A_v = A - 2bt_f + (t_w + 2r)t_f = 66.3 \times 100 - 2 \times 204 \times 12.5 + (7.90 + 2 \times 10.2) \times 12.5 = 1,876 \text{ mm}^2$ [EC3 6.2.6 (3)]

Shear resistance, $V_{pl,Rd} = A_v \cdot (f_y / \sqrt{3}) / g_{M0} = 1,876 \times (355 / \sqrt{3}) / (1.0 \times 1000) = 385 \text{ kN} (> 49.76) \text{ OK}$ [EC3 6.2.6]

Shear buckling: $h_w/t_w = 181.2/7.9 = 22.94 \leq 72e(58.58)$: check not required [EC3 6.2.6(6)]

Bending

Moment resistance

Design moment, $M_{Ed} = 64.4 \text{ kNm}$

Moment resistance, $M_{c,y,Rd} = f_y \cdot W_{pl,y} = 355 \times 567/1000 = 201.3 \text{ kNm OK}$

Lateral-torsional buckling check

Beam is laterally restrained at supports only

Support conditions R1/R2: Compression flange laterally restrained. Nominal torsional restraint. Both flanges free to rotate on plan (1.0L)

Design buckling resistance moment, $M_{b,Rd} = c_{LT,mod} \cdot M_{c,Rd}$

$c_{LT,mod} = c_{LT}/f$ (but $\leq 1/\bar{I}_{LT}^2$ and ≤ 1.0) [Eq.6.58]

$f = 1 - 0.5(1 - k_c)[1 - 2(\bar{I}_{LT} - 0.8)^2]$ 6.3.2.3(2) $k_c = 1/\sqrt{C_1}$ [NA2.18]

Use buckling curve b: $a = 0.340$ [EC3 Tables 6.3/6.4 NA 2.17]

$c_{LT} = 1/[f_{LT} + \sqrt{(F_{LT}^2 - bI_{LT}^2)}]$ [EC3 (6.57)]

$F_{LT} = 0.5[1 + a_{LT}(\bar{I}_{LT} - \bar{I}_{LT,0}) + bI_{LT}^2]$

$\bar{I}_{LT,0} = 0.4$ $b = 0.75$ [EC3 UK NA 2.17]

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$$\bar{I}_{LT} = \sqrt{(f_y \cdot W_{pl,y} / M_{cr})}$$

$$M_{cr} = C_1(p^2 E I_z / L_e^2) \sqrt{(I_w / I_z + L_e^2 G I_T / p^2 E I_z)} \quad [\text{SN003: } k \text{ and } k_w \text{ taken as } 1.0, \text{ conservative}]$$

$$W_y = 567.0 \text{ cm}^3 \quad I_w = 0.167 \text{ dm}^6 \quad I_T = 31.8 \text{ cm}^4 \quad I_z = 1,780 \text{ cm}^4 \quad G = 81,000 \text{ N/mm}^2$$

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\bar{I}_z$	$\bar{I}_{LT}$	f_{LT}	C_{LT}	$C_{LT, mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-5.20	5.20	64.4	1.15*	0.93	263.2	1.314	0.874	0.867	0.775	0.802	201.3	161.3	OK

$$* C_1 = 4M_{max} / \sqrt{(M_{max}^2 + 4M_a^2 + 7M_b^2 + 4M_c^2)} \leq 2.5 \quad [M_{a,b,c} \text{ quarter pt moments}] \quad [\text{Wong \& Driver, AISC Eng. Journal, Q1 2010}]$$

Combined bending and shear

$$V_{Ed} \leq 0.5 V_{c,Rd} : \text{Check for bending/shear interaction not required} \quad [\text{EC3 6.2.8(2)}]$$

Web capacity at bearings

$$\text{Resistance of web to transverse forces, } F_{Rd} = f_{yw} \cdot L_{eff} \cdot t_w / \gamma_{M1}$$

$$f_{yw} = 355 \text{ N/mm}^2$$

$$C_F = 0.5 / \bar{I}_F \leq 1.0$$

$$F_{cr} = 0.9 k_f \cdot E \cdot (t_w^3 / h_w)$$

$$L_{eff} = C_F I_y$$

$$\bar{I}_F = \sqrt{(I_y \cdot t_w \cdot f_{yw} / F_{cr})}$$

$$k_F = 2 + 6((S_s + c) / h_w)^2 \leq 6$$

Type (c) load application assumed:

$$\text{Effective loaded length, } l_y = \min S_s + 2t_f(1 + \sqrt{(m_1 + m_2)}) \quad [\text{EC3-1-5 Eq.6.10}],$$

$$l_e + t_f \sqrt{(m_1/2 + (l_e/t_f)^2 + m_2)} \quad [\text{Eq.6.11}]$$

$$\text{or } l_e + t_f \sqrt{(m_1 + m_2)} \quad [\text{Eq.6.12}]$$

$$l_e = k_F \cdot E \cdot t_w^2 / (2 \cdot f_{yw} \cdot h_w) \leq S_s + c \quad [\text{Eq.6.13}]$$

$$m_1 = f_{yf} \cdot b_f / (f_{yw} \cdot t_w) \quad m_2 = 0.02(h_w/t_f)^2 \text{ if } \bar{I}_F > 0.5 \text{ else } 0.0$$

Reaction R1: 49.76 kN

Required minimum stiff bearing length, $S_s = 0 \text{ mm}$

c (end of beam to stiff bearing) taken as 0.0

$$m_1 = 25.9 \quad m_2 = 0.00 \quad F_{cr} = 1,029 \text{ kN} \quad k_F = 2.00 \quad l_e = 0.00 \quad l_y = 44.95 \quad \bar{I}_F = 0.350 \quad C_F = 1.00 \quad L_{eff} = 44.95$$

$$\text{Resistance of web to transverse forces, } F_{Rd} = 355 \times 44.95 \times 7.9 / (1000 \times 1.0) = 126 \text{ kN OK}$$

Reaction R2: 39.68 kN

Required minimum stiff bearing length, $S_s = 0 \text{ mm}$

c (end of beam to stiff bearing) taken as 0.0

$$m_1 = 25.9 \quad m_2 = 0.00 \quad F_{cr} = 1,029 \text{ kN} \quad k_F = 2.00 \quad l_e = 0.00 \quad l_y = 44.95 \quad \bar{I}_F = 0.350 \quad C_F = 1.00 \quad L_{eff} = 44.95$$

$$\text{Resistance of web to transverse forces, } F_{Rd} = 355 \times 44.95 \times 7.9 / (1000 \times 1.0) = 126 \text{ kN OK}$$

Deflection

$$\text{LL deflection} = 58.4 \times 1e8 / (210,000 \times 5,260) = 5.3 \text{ mm} \quad (L/984) \text{ OK}$$

$$\text{TL deflection} = 125.0 \times 1e8 / (210,000 \times 5,260) = 11.3 \text{ mm} \quad (L/460)$$

Bearings

$$203 \times 203 \times 52 \text{ UKC stiff bearing length, } b_1 = t + 1.6r + 2T = 49.2 \text{ mm}$$

Masonry: 7N/mm² brick, shape factor, $d = 0.850$ [Table A1 BS EN 772-1]

$$\text{Normalised compressive strength, } f_b = 7.00 \times 0.850 = 5.95 \text{ N/mm}^2$$

$$\text{Mortar: M2: } f_m = 2.0 \text{ N/mm}^2$$

$$K = 0.5, a = 0.7, B = 0.3 \quad [\text{EC6-1-1 UK NA 2012 Table NA.4}]$$

$$\text{Characteristic compressive strength, } f_k = K \cdot f_b^a \cdot f_m^B = 0.50 \times 5.95^{0.7} \times 2.00^{0.3} = 2.15 \text{ N/mm}^2 \quad [\text{EC6-1 Eq(3.1)}]$$

$$\text{Category II mfr, class 2 execution: } g_m = 3.0 \quad [\text{EC6-3 UK NA Table NA.1}]$$

$$\text{Use concentrated load enhancement factor of } 1.2 \quad \text{EC6-3 4.3}$$

where a_1 = distance from end of wall to edge of loaded area and h_c = height to level of load

Enhancement factor requirements (not checked by EuroBeam): [EC6-3 4.3]

1. Bearing area must not exceed 1/4 cross sectional area of wall or $2t^2$ where t is wall thickness
2. Eccentricity must not exceed $t/4$

3. Adequacy of wall strength at mid-height should be verified assuming load spread of 60°

$$\text{Design compressive strength, } f_d = f_k / g_m = 2.15 / 3.0 = 0.715 \text{ N/mm}^2 \times 1.200 = 0.858 \text{ N/mm}^2$$

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R1 (49.76 kN) : 600 x 100 mm padstone

Stress under padstone = $49.76 \times 1000 / 600 \times 100 = 0.83 \text{ N/mm}^2$ OK

R2 (39.68 kN) : 500 x 100 mm padstone

Stress under padstone = $39.68 \times 1000 / 500 \times 100 = 0.79 \text{ N/mm}^2$ OK

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Beam: Floor Joists with point loads from roof & Wall

Span: 4.0 m.

	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp
U G	o.w.	0.1	0		L	0.20	0.20
U G	Floor	0.5x0.3	0		L	0.30	0.30
U QA	Floor	1.5x0.3	0		L	0.90	0.90
P G	Partition	0.4	3.2			0.08	0.32
Total load (unfactored): 3.20 kN						1.48	1.72
Dead/Permanent (unfactored): 1.40 kN						0.58	0.82
Live/Variable (unfactored): 1.80 kN						0.90	0.90
(6.10): 4.59 kN						2.13	2.46

Load types: U:UDL; P:Point load; Load positions are measured in m. from R1

Load durations: G: Dead; Qx: Imposed; QA: Residential

Maximum B.M. = 2.25 kNm (6.10) at 2.11 m. from R1

Maximum S.F. = -2.46 kN (6.10) at R2

Mid-span deflections: Dead: $1.14 \times 10^8/EI$ (E in N/mm^2 , I in cm^4)

Live: $1.50 \times 10^8/EI$

Total: $2.64 \times 10^8/EI$

Timber beam calculation to BS EN1995-1-1 using C24 timber

Use 47 x 195 C24 3.8 kg/m approx

$W_{el,y} = 297.9 \text{ cm}^3$ $I_y = 2,904 \text{ cm}^4$

Timber grade: C24

Grade bending strength, $f_{m,k} = 24.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Grade shear strength, $f_{v,k} = 4.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

$E_{0.05} = 7,400 \text{ N/mm}^2$; $E_{0,mean} = 11,000 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Material partial factor, $g_M = 1.3$ [EC5 UK Table NA.3]

Loading modification factor, $k_{mod} = 0.6$ (Service class 1; Live load duration: Permanent) [EC5 Tables 2.2/3.1]

Load sharing factor, $k_{sys} = 1.1$ [EC5 6.6(2)]

Deflection modification factor, $k_{def} = 0.60$ (Service class 1, solid timber/glulam/LVL) [EC5 Table 3.2]

Bending

Height factor, $k_h = 1.0$ [EC5 3.2(3)]

Design bending strength, $f_{m,y,d} = f_{m,k} \cdot k_{mod} \cdot k_h \cdot k_{sys} / g_M = 24.0 \times 0.60 \times 1.00 \times 1.1/1.30 = 12.18 \text{ N/mm}^2$

Design bending stress, $s_{m,y,d} = 2.25 \times 1000/298 = 7.54 \text{ N/mm}^2$ OK

Bending resistance = $12.18 \times 298/1000 = 3.63 \text{ kNm}$

Shear

Effective width for shear, $b_{ef} = k_{cr} \cdot b = 0.67 \times 47 = 31.5 \text{ mm}$. [A1:2008 (6.13a)]

Design shear strength, $f_{v,d} = f_{v,k} \cdot k_{mod} \cdot k_{sys} / g_M = 4.00 \times 0.60 \times 1.1/1.30 = 2.03 \text{ N/mm}^2$

Design shear stress, $s_{v,y,d} = 2.46 \times 1000 \times (3/2)/(31.5 \times 195) = 0.60 \text{ N/mm}^2$ OK

Shear resistance = $2.03 \times 31.5 \times 195 \times (2/3)/1000 = 8.31 \text{ kN}$

Deflection

Final deflection limit = $L/250 = 16.00 \text{ mm}$

Final deflection, $u_{fin} = S u_{inst} (1 + y_2 \cdot k_{def})$ [EC2 Eq.2.29/2.30]

Instantaneous mid-span shear deflection, $u_{inst,v} = 1.2 M_{unf} / (G_{mean} A)$ where ($G = E/16$)

$u_{inst,v} = 1.2 \times 1.56 \times 10^6 / ((11,000/16) \times 47 \times 195) = 0.30 \text{ mm}$

Final shear deflection is assumed to increase in proportion to total bending deflection

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Mid-span deflections:	$\times 10^8/EI$	Inst. mm	y_2	k_{def}	Fin. mm	
Dead:	1.14	3.56	1.00	0.60	5.69	
Live QA:	1.50	4.70	0.30	0.60	5.54	
Shear deflection:		0.30			0.40	
Total mm.		<u>8.55</u>			<u>11.64</u>	OK