

project : Land Adjacent 2 The Spinney, Eastern Road, RH16 3NG
project no : 9501
by : ICR
date : Apr 2023
no : 1

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**PROPOSED DEVELOPMENT AT
LAND ADJACENT 2 THE SPINNEY,
EASTERN ROAD,
HAYWARDS HEATH, RH16 3NG**

The following calculations and details are to be read in conjunction with all relevant drawings prepared by BPM ARCHITECTURAL SERVICES LTD.

All dimensions and assumptions made within the following calculations and details are to be checked on site, and any discrepancy reported to Ings Engineering Limited.

Design Documents

Loads to	BS 648 & BS 6399
Timber to	BS 5268
Steel to	BS 5950
Masonry to	BS 5628
Concrete to	BS 8110
Foundations to	BS 8004
Building Regulations to	Part A

Loading (kN/m²)	DL	+	LL	Total
Pitched Roof	0.85	+	0.75	1.60
Skelling	0.15	+	0.00	0.15
Attic Space	0.30	+	0.25	0.55
Flat Roof	0.75	+	0.75	1.50
Timber Floor	0.50	+	1.50	2.00
BB Floor	4.50	+	1.50	6.00
Internal Timber Stud	0.40			0.40
External Timber Stud	0.90			0.90
100 Block + Plaster	1.50			1.50
103 Brick+ Plaster	2.50			2.50
215 Brick	4.70			4.70
330 Brick	7.30			7.30

General Notes

These calculations are for obtaining Building Regulation Approval only.

Note also requirements of the Building Regulations outside the scope of these calculations i.e. Building Regulations Approved Documents B to M, which should be referred to as appropriate.

Works carried out on site prior to the relevant Building Regulation approval being obtained from the Local Authority, are undertaken entirely at the Contractor and Clients own risk.

Calculations are to be read in conjunction with all accompanying specification notes and architectural drawings.

Calculations have been produced from drawings only, unless noted.

No structural investigations unless noted have been undertaken and all assumed loadbearing walls and existing foundations must be physically exposed, checked and confirmed with the BUILDING CONTROL OFFICER on site prior to commencement of the works.

Work to site dimensions only. Do not use measurement shown in the calculations - these are for design analysis only

Any discrepancies are to be reported immediately to the designer for clarification.

The Contractor is responsible for all temporary works and for the stability of the works in progress including any necessary sacrificial jacks to take up beam deflections.

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The Construction (Design and Management) Regulations 2015 (CDM 2015)

1. Wherever possible all foreseeable risks to Health and Safety have been considered at Design Stage and suitable measures have been taken to combat such risks at source.

Where it has not been possible to reduce risk element to a minimum due to practical or physical considerations, the Contractor must employ competent people and establish safe methods of work to reduce the risk accordingly.

2. **Construction Stage Associated Risks**

The following hazards and/or risks have been identified and are to be addressed by the Contractor with regards to Health and Safety accordingly.

Contractor is to assess existing structural configurations and provide all necessary temporary support works

The contractor is to provide all necessary lifting equipment to install heavy building components

Contractor is to provide all necessary signage, fencing, hoarding etc to protect staff, visitors for the duration of the works

Normal Construction Hazards/Risks have been excluded from the above which are considered to be covered by the use of competent Contractors working in accordance with good working practice and compliance with current Health and Safety legislation.

3. **Maintenance, Cleaning or Demolition Associated Risks**

The following hazards and/or risks have been identified and are to be addressed by any person or Contractor undertaking maintenance, cleaning or demolition works to the above property.

All future works are to be carried out after consultation with a structural engineer or suitably qualified building professional

Normal Construction Hazards/Risks have been excluded from the above which are considered to be covered by the use of competent Contractors working in accordance with good working practice and compliance with current Health and Safety legislation.

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Beam: R1 - rafters						Span: 2.8 m.		
	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
U D	pitched roof	1.0*0.4	0		L	0.56	0.56	0.32
U L	pitched roof	0.75*0.4	0		L	0.42	0.42	0.24
Total load (unfactored):						0.98	0.98	0.56
Dead/Permanent (unfactored):					1.12 kN	0.56	0.56	0.32
Live/Variable (unfactored):					0.84 kN	0.42	0.42	0.24

Load types: U:UDL; Load positions: m. from R1; Load durations: D: Dead; L: Live

Maximum B.M. = 0.686 kNm (unfactored) at 1.40 m. from R1
Maximum S.F. = 0.98 kN (unfactored) at R1
Mid-span deflections: Dead: $0.32 \times 10^8 / EI$ (**E in N/mm², I in cm⁴**)
Live: $0.24 \times 10^8 / EI$
Total: $0.56 \times 10^8 / EI$

Timber beam calculation to BS5268 Part 2: 2002 using C16 timber

Use 47 x 147 C16 2.6 kg/m approx

$z = 169 \text{ cm}^3$ $I = 1,244 \text{ cm}^4$

Timber grade: C16 Load sharing system: $K_8 = 1.1$ [§2.10.11]

K_3 (loading duration factor) = 1.00 (long term)

K_7 (depth factor) = $(300/147)^{0.11} = 1.08$ [§2.10.6] K_8 (load sharing factor) = 1.1 [§2.9,2.10]

$E = 8,800 \text{ N/mm}^2$ (E_{mean})

Bending

Permissible bending stress, $\sigma_{m,adm} = \sigma_{m,g} \cdot K_3 \cdot K_7 \cdot K_8 = 5.3 \times 1.00 \times 1.08 \times 1.1 = 6.31 \text{ N/mm}^2$

Applied bending stress, $\sigma_{m,a} = 0.69 \times 1000/169 = 4.05 \text{ N/mm}^2$ OK

Shear

Permissible shear stress, $\tau_{adm, //} = \tau_{g, //} \cdot K_3 \cdot K_8 = 0.67 \times 1.00 \times 1.1 = 0.74 \text{ N/mm}^2$

Applied shear stress, $\tau_a = 0.98 \times 1000 \times 3 / (2 \times 47 \times 147) = 0.21 \text{ N/mm}^2$ OK

Deflection

Bending deflection = $0.560 \times 10^8 / (8,800 \times 1,244) = 5.1 \text{ mm}$

Mid-span shear deflection = $1.2M_0/GA$ ($G=E/16$) = $1.2 \times 0.69 \times 10^6 / ((8800/16) \times 47 \times 147) = 0.2 \text{ mm}$

Total deflection = $5.1 + 0.2 = 5.3 \text{ mm}$ (0.0019 L) $\leq 0.003 \text{ L}$ OK

provide new rafters @ 400 centres, min size as noted above. notched at supports to provide horizontal bearings (max 1/3rd depth notch)

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Beam: CJ1 - ceiling joists							Span: 3.2 m.	
	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
U D	attic space	0.3*0.4	0		L	0.19	0.19	0.16
U L	attic space	0.25*0.4	0		L	0.16	0.16	0.14
P L	man PL	0.9	L/2			0.45	0.45	0.61
Total load (unfactored): 1.60 kN						0.80	0.80	0.91
Dead/Permanent (unfactored):					0.38 kN	0.19	0.19	0.16
Live/Variable (unfactored):					1.22 kN	0.61	0.61	0.75

Load types: U:UDL; P:Point load; Load positions: m. from R1; Load durations: D: Dead; L: Live

Maximum B.M. = 1.00 kNm (unfactored) at 1.60 m. from R1
Maximum S.F. = 0.80 kN (unfactored) at R1
Mid-span deflections: Dead: $0.16 \times 10^8 / EI$ (**E in N/mm², I in cm⁴**)
Live: $0.75 \times 10^8 / EI$
Total: $0.91 \times 10^8 / EI$

Timber beam calculation to BS5268 Part 2: 2002 using C16 timber

Use 47 x 147 C16 2.6 kg/m approx

$z = 169 \text{ cm}^3$ $I = 1,244 \text{ cm}^4$

Timber grade: C16 Load sharing system: $K_8 = 1.1$ [§2.10.11]

K_3 (loading duration factor) = 1.00 (long term)

K_7 (depth factor) = $(300/147)^{0.11} = 1.08$ [§2.10.6] K_8 (load sharing factor) = 1.1 [§2.9,2.10]

$E = 8,800 \text{ N/mm}^2$ (E_{mean})

Bending

Permissible bending stress, $\sigma_{m,adm} = \sigma_{m,g} \cdot K_3 \cdot K_7 \cdot K_8 = 5.3 \times 1.00 \times 1.08 \times 1.1 = 6.31 \text{ N/mm}^2$

Applied bending stress, $\sigma_{m,a} = 1.00 \times 1000/169 = 5.92 \text{ N/mm}^2$ OK

Shear

Permissible shear stress, $\tau_{adm, //} = \tau_{g, //} \cdot K_3 \cdot K_8 = 0.67 \times 1.00 \times 1.1 = 0.74 \text{ N/mm}^2$

Applied shear stress, $\tau_a = 0.80 \times 1000 \times 3 / (2 \times 47 \times 147) = 0.17 \text{ N/mm}^2$ OK

Deflection

Bending deflection = $0.915 \times 10^8 / (8,800 \times 1,244) = 8.4 \text{ mm}$

Mid-span shear deflection = $1.2M_0 / GA$ ($G=E/16$) = $1.2 \times 1.00 \times 10^6 / ((8800/16) \times 47 \times 147) = 0.3 \text{ mm}$

Total deflection = $8.4 + 0.3 = 8.7 \text{ mm}$ (0.0027 L) $\leq 0.003 \text{ L}$ OK

provide new ceiling joists @ 400 centres, min size as noted above

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Beam: FJ1 - engineered floor joists by specialist supplier

Span: 5.0 m.

	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
U D	timber floor	0.5*0.4	0		L	0.50	0.50	1.63
U L	timber floor	1.5*0.4	0		L	1.50	1.50	4.88
P D	int stud	0.4*0.4*2.5	L/2			0.20	0.20	1.04
Total load (unfactored):						2.20	2.20	7.55
Dead/Permanent (unfactored):					1.40 kN	0.70	0.70	2.67
Live/Variable (unfactored):					3.00 kN	1.50	1.50	4.88
Total load (factored):					6.76 kN	3.38	3.38	

Load types: U:UDL; P:Point load; Load positions: m. from R1; Load durations: D: Dead; L: Live

Maximum B.M. = 4.57 kNm (factored) at 2.50 m. from R1

Maximum S.F. = 3.38 kN (factored) at R1

Mid-span deflections: Dead: $2.67 \times 10^8 / EI$ (**E in N/mm², I in cm⁴**)

Live: $4.88 \times 10^8 / EI$

Total: $7.55 \times 10^8 / EI$

User script generated output follows

Use 241mm deep TJI® joist

All values shown here are unfactored

TJI Series	Joist Wt kg/m	Moment cap kNm	Flexural Rigidity x 1E9	Max vert SF kN	Max React kN	Max int react kN	
105	2.8	2.79	344	4.64	3.44	7.50	38 x 35 flanges, 9.5mm web
110	3.1	3.29	401	4.83	3.56	7.80	45 x 35 flanges, 9.5mm web
360	3.7	6.47	710	5.19	4.39	9.88	58 x 35 flanges, 9.5mm web
560	5.3	9.94	1080	5.51	5.27	11.01	89 x 35 flanges, 11.1mm web

Data from TrusJoist Engineered Joists and Beam Technical Guide UK-1001 May 2006

Table above is based on uniform loads, load-sharing system, dry conditions

Use TJI Series 360 joist

Max BM = 3.00 kNm; Moment capacity = 6.47 kNm OK

Max SF = 3.38 kN; Shear capacity = 5.19 kN OK

Max end reaction = 4.39 kN; Actual: 2.20/2.20 kN OK

Bending deflection = $7.55 \times 10,000 / (710 \times 10) = 10.6$ mm

Shear deflection = $0.88 \times (5.00 \times 1000)^2 / (8 \times 710 \times 241 \times 9.5) = 1.7$ mm

Total deflection = $10.6 + 1.7 = 12.3$ mm (span/406) OK

N.B. NHBC recommend limiting unbraced floor joist deflection to 12mm

End of user script generated output

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Beam: T1							Span: 1.1 m.	
	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
U D	pitched roof	1.0*1.4	0		L	0.77	0.77	0.027
U L	pitched roof	0.75*1.4	0		L	0.58	0.58	0.020
Total load (unfactored):						1.35	1.35	0.047
Dead/Permanent (unfactored):					1.54 kN	0.77	0.77	0.027
Live/Variable (unfactored):					1.15 kN	0.58	0.58	0.020

Load types: U:UDL; Load positions: m. from R1; Load durations: D: Dead; L: Live

Maximum B.M. = 0.371 kNm (unfactored) at 0.55 m. from R1
Maximum S.F. = 1.35 kN (unfactored) at R1
Mid-span deflections: Dead: $0.027 \times 10^8 / EI$ (**E in N/mm², I in cm⁴**)
Live: $0.020 \times 10^8 / EI$
Total: $0.047 \times 10^8 / EI$

Timber beam calculation to BS5268 Part 2: 2002 using C16 timber

Use 47 x 147 C16 2.6 kg/m approx

$z = 169 \text{ cm}^3$ $I = 1,244 \text{ cm}^4$

Timber grade: C16 Single member: No load sharing

K_3 (loading duration factor) = 1.00 (long term)

K_7 (depth factor) = $(300/147)^{0.11} = 1.08$ [**§2.10.6**] K_8 (load sharing factor) = 1.0 [**§2.9, 2.10**]

$E = 5,800 \text{ N/mm}^2$ (E_{min})

Bending

Permissible bending stress, $\sigma_{m,adm} = \sigma_{m,g} \cdot K_3 \cdot K_7 \cdot K_8 = 5.3 \times 1.00 \times 1.08 \times 1.0 = 5.73 \text{ N/mm}^2$

Applied bending stress, $\sigma_{m,a} = 0.371 \times 1000 / 169 = 2.19 \text{ N/mm}^2$ OK

Shear

Permissible shear stress, $\tau_{adm, //} = \tau_{g, //} \cdot K_3 \cdot K_8 = 0.67 \times 1.00 \times 1.0 = 0.67 \text{ N/mm}^2$

Applied shear stress, $\tau_a = 1.35 \times 1000 \times 3 / (2 \times 47 \times 147) = 0.29 \text{ N/mm}^2$ OK

Deflection

Bending deflection = $0.047 \times 10^8 / (5,800 \times 1,244) = 0.6 \text{ mm}$

Mid-span shear deflection = $1.2M_0 / GA$ ($G = E/16$) = $1.2 \times 0.371 \times 10^6 / ((5800/16) \times 47 \times 147) = 0.2 \text{ mm}$

Total deflection = $0.6 + 0.2 = 0.8 \text{ mm}$ (0.0007 L) $\leq 0.003 \text{ L}$ OK

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Beam: T2		Span: 2.8 m.						
	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
U D	pitched roof	1.0*1.5/2	0		L	1.05	1.05	0.60
U L	pitched roof	0.75*1.5/2	0		L	0.79	0.79	0.45
Total load (unfactored):						1.84	1.84	1.05
Dead/Permanent (unfactored):					2.10 kN	1.05	1.05	0.60
Live/Variable (unfactored):					1.57 kN	0.79	0.79	0.45

Load types: U:UDL; Load positions: m. from R1; Load durations: D: Dead; L: Live

Maximum B.M. = 1.29 kNm (unfactored) at 1.40 m. from R1

Maximum S.F. = 1.84 kN (unfactored) at R1

Mid-span deflections: Dead: $0.60 \times 10^8 / EI$ (**E in N/mm², I in cm⁴**)

Live: $0.45 \times 10^8 / EI$

Total: $1.05 \times 10^8 / EI$

Timber beam calculation to BS5268 Part 2: 2002 using C16 timber

Use 2no 47 x 147 C16 5.1 kg/m approx

$z = 339 \text{ cm}^3$ $I = 2,488 \text{ cm}^4$

Timber grade: C16 2 members acting together: $K_8 = 1.1$ [§2.9]

K_3 (loading duration factor) = 1.00 (long term)

K_7 (depth factor) = $(300/147)^{0.11} = 1.08$ [§2.10.6] K_8 (load sharing factor) = 1.1 [§2.9, 2.10]

$E = 5,800 \times 1.14 = 6,612 \text{ N/mm}^2$ ($E_{min} \cdot K_9$)

Bending

Permissible bending stress, $\sigma_{m,adm} = \sigma_{m,g} \cdot K_3 \cdot K_7 \cdot K_8 = 5.3 \times 1.00 \times 1.08 \times 1.1 = 6.31 \text{ N/mm}^2$

Applied bending stress, $\sigma_{m,a} = 1.29 \times 1000/339 = 3.80 \text{ N/mm}^2$ OK

Shear

Permissible shear stress, $\tau_{adm, //} = \tau_{g, //} \cdot K_3 \cdot K_8 = 0.67 \times 1.00 \times 1.1 = 0.74 \text{ N/mm}^2$

Applied shear stress, $\tau_a = 1.84 \times 1000 \times 3 / (2 \times 94 \times 147) = 0.20 \text{ N/mm}^2$ OK

Deflection

Bending deflection = $1.05 \times 10^8 / (6,612 \times 2,488) = 6.4 \text{ mm}$

Mid-span shear deflection = $1.2M_0/GA$ ($G=E/16$) = $1.2 \times 1.29 \times 10^6 / ((6612/16) \times 94 \times 147) = 0.3 \text{ mm}$

Total deflection = $6.4 + 0.3 = 6.7 \text{ mm}$ ($0.0024 L$) $\leq 0.003L$ OK

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Beam: T3 - floor loading purposes only

Span: 1.1 m.

	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
U D	timber floor	0.5*5.0/2	0		L	0.69	0.69	0.02
U L	timber floor	1.5*5.0/2	0		L	2.06	2.06	0.07
U D	int stud	0.4*2.5	0		L	0.55	0.55	0.02
Total load (unfactored):						6.60 kN	3.30	3.30
								0.11
Dead/Permanent (unfactored):					2.47 kN	1.24	1.24	0.04
Live/Variable (unfactored):					4.13 kN	2.06	2.06	0.07
Total load (factored):					10.06 kN	5.03	5.03	

Load types: U:UDL; Load positions: m. from R1; Load durations: D: Dead; L: Live

Maximum B.M. = 1.38 kNm (factored) at 0.55 m. from R1

Maximum S.F. = 5.03 kN (factored) at R1

Mid-span deflections: Dead: $0.04 \times 10^8 / EI$ (**E in N/mm², I in cm⁴**)

Live: $0.07 \times 10^8 / EI$

Total: $0.11 \times 10^8 / EI$

provide engineered timber member to be designed and supplied by specialist engineered timber joist manufacturer

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Beam: T4 - floor loading purposes only

Span: 5.0 m.

	Load name	Loading w1	Start x1	Loading w2	End x2	R1 comp	R2 comp	Defl.
U D	timber floor	0.5*0.4	0		L	0.50	0.50	1.63
U L	timber floor	1.5*0.4	0		L	1.50	1.50	4.88
R D	int stud	0.4*2.5	0.3		1.8	1.19	0.31	2.24
P D	Beam: T3 - floor : R1	1.24 [B/F]	0.3			1.16	0.07	0.58
P L	Beam: T3 - floor : R1	2.06 [B/F]	0.3			1.94	0.12	0.96
Total load (unfactored):						8.80 kN	6.29	2.51
Dead/Permanent (unfactored):						3.74 kN	2.85	0.89
Live/Variable (unfactored):						5.06 kN	3.44	1.62
Total load (factored):						13.33 kN	9.49	3.84

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

Load durations: D: Dead; L: Live

Maximum B.M. = 5.95 kNm (factored) at 1.92 m. from R1

Maximum S.F. = 9.49 kN (factored) at R1

Mid-span deflections: Dead: $4.45 \times 10^8 / EI$ (**E in N/mm², I in cm⁴**)

Live: $5.84 \times 10^8 / EI$

Total: $10.29 \times 10^8 / EI$

provide engineered timber member to be designed and supplied by specialist engineered timber joist manufacturer

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Beam: T5 - floor loading purposes only

Span: 4.0 m.

	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
U D	timber floor	0.4*1.5/2	0		L	0.60	0.60	1.00
U L	timber floor	1.5*1.5/2	0		L	2.25	2.25	3.75
U D	int stud	0.4*2.5	0		L	2.00	2.00	3.33
Total load (unfactored):						4.85	4.85	8.08
Dead/Permanent (unfactored):					5.20 kN	2.60	2.60	4.33
Live/Variable (unfactored):					4.50 kN	2.25	2.25	3.75
Total load (factored):					14.48 kN	7.24	7.24	

Load types: U:UDL; Load positions: m. from R1; Load durations: D: Dead; L: Live

Maximum B.M. = 7.24 kNm (factored) at 2.00 m. from R1

Maximum S.F. = 7.24 kN (factored) at R1

Mid-span deflections: Dead: $4.33 \times 10^8 / EI$ (**E in N/mm², I in cm⁴**)

Live: $3.75 \times 10^8 / EI$

Total: $8.08 \times 10^8 / EI$

provide engineered timber member to be designed and supplied by specialist engineered timber joist manufacturer

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Beam: B1							Span: 6.1 m.
O	D	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp R2comp Defl.
		o.w.	0.5	0		L	1.52 1.52 9
U	D	roof+attic	1.15*3.2/2	0		L	5.61 5.61 33
U	L	attic space	1.0*3.2/2	0		L	4.88 4.88 29
U	D	pitched roof	1.0*2.8/2	0		L	4.27 4.27 25
U	L	pitched roof	0.75*2.8/2	0		L	3.20 3.20 19
Total load (unfactored):							39.0 kN 19.49 19.49 115
Dead/Permanent (unfactored):							22.8 kN 11.41 11.41 67
Live/Variable (unfactored):							16.2 kN 8.08 8.08 48
Total load (factored):							57.8 kN 28.90 28.90

Load types: O:Beam o.w.; U:UDL; Load positions: m. from R1; Load durations: D: Dead; L: Live

Maximum B.M. = 44.08 kNm (factored) at 3.05 m. from R1

Maximum S.F. = 28.9 kN (factored) at R1

Mid-span deflections: Dead: $67 \times 10^8 / EI$ (**E in N/mm², I in cm⁴**)

Live: $48 \times 10^8 / EI$

Total: $115 \times 10^8 / EI$

Beam calculation to BS5950-1:2000 using S355 steel

SECTION SIZE : 254 x 146 x 37 UB S355 (compact)

D=256.0 mm B=146.4 mm t=6.3 mm T=10.9 mm $I_x=5,540 \text{ cm}^4$ $r_x=3.48 \text{ cm}$ $S_x=483 \text{ cm}^3$ $x=24.4$

Section classification: T = 10.9mm $p_v = 355 \text{ N/mm}^2$ $\varepsilon = \sqrt{(275/355)} = 0.88$

(Table 11) Flange: $b/T = 73.2/10.9 = 6.72$ ($\leq 9\varepsilon$: Class 1 plastic)

Web: $d/t = 219.0/6.3 = 34.8$ ($\leq 80\varepsilon$: Class 1, plastic)

For design purposes section classification is Class 2, compact

Shear

Maximum S.F. = 29 kN

Shear capacity = $0.6 p_v t D = 0.6 \times 355 \times 6.3 \times 256.0/1000 = 344 \text{ kN OK}$

Bending

Maximum B.M. = 44.1 kNm

Moment capacity, $M_c = p_v S_x = 355 \times 483/1000 = 171 \text{ kNm OK}$

Lateral-torsional buckling

Beam is laterally restrained at supports only

Restraint condition at R1 and R2: Compression flange laterally unrestrained; both flanges free to rotate on plan. Partial torsional restraint by dead bearing of bottom flange to support (1.2L+2D)

Effective length = $1.2L+2D = 7.83 \text{ m}$. [Table 13]

Bending strength, $p_b = 88.4 \text{ N/mm}^2$

Maximum moment within segment, $M_x = 44.1 \text{ kNm}$

Equivalent uniform moment factor, $m_{LT} = 0.925$ ($M_2 = 33.1$, $M_3 = 44.1$, $M_4 = 33.1$)

Equivalent uniform moment = $0.925 \times 44.1 = 40.8 \text{ kNm}$

Buckling resistance moment, $M_b = p_b S_x = 88.4 \times 483/1000 = 42.7 \text{ kNm OK}$

Web buckling and crushing have not been checked

Deflection

LL deflection = $47.8 \times 10^8 / (205,000 \times 5,540) = 4.2 \text{ mm}$ (L/1450) OK

TL deflection = $115.2 \times 10^8 / (205,000 \times 5,540) = 10.1 \text{ mm}$ (L/601)

Bearings

254 x 146 x 37 UB stiff bearing length, $b_1 = t + 1.6r + 2T = 40.3 \text{ mm}$

R1: 225 x 100 mm Padstone

Factored reaction = $1.4 \times 11.41 + 1.6 \times 8.08 = 28.9 \text{ kN}$

Masonry: 100mm 3.6N/mm² solid block (SF>2.0), class (iii) mortar, normal const/normal mfr, Class 2 bearing

Local design strength (factored) = $3.5 \times 1.5/3.5 = 1.50 \text{ N/mm}^2$ (BS5628-1:2005 Table 2d/2e)

Factored stress under padstone = $28.9 \times 1000/225 \times 100 = 1.28 \text{ N/mm}^2 \text{ OK}$

provide min 100 mm bearing onto engineering brick padstone bedded on mortar off inner leaf of external cavity wall

R2: 147 x 100 mm Padstone

Factored reaction = $1.4 \times 11.41 + 1.6 \times 8.08 = 28.9 \text{ kN}$

Masonry: 100mm 7.3N/mm² solid block (SF>2.0), class (iii) mortar, normal const/normal mfr, Class 2 bearing

Local design strength (factored) = $6.4 \times 1.5/3.5 = 2.74 \text{ N/mm}^2$ (BS5628-1:2005 Table 2d/2e)

Factored stress under padstone = $28.9 \times 1000/147 \times 100 = 1.97 \text{ N/mm}^2 \text{ OK}$

provide min 100 mm bearing onto party wall masonry

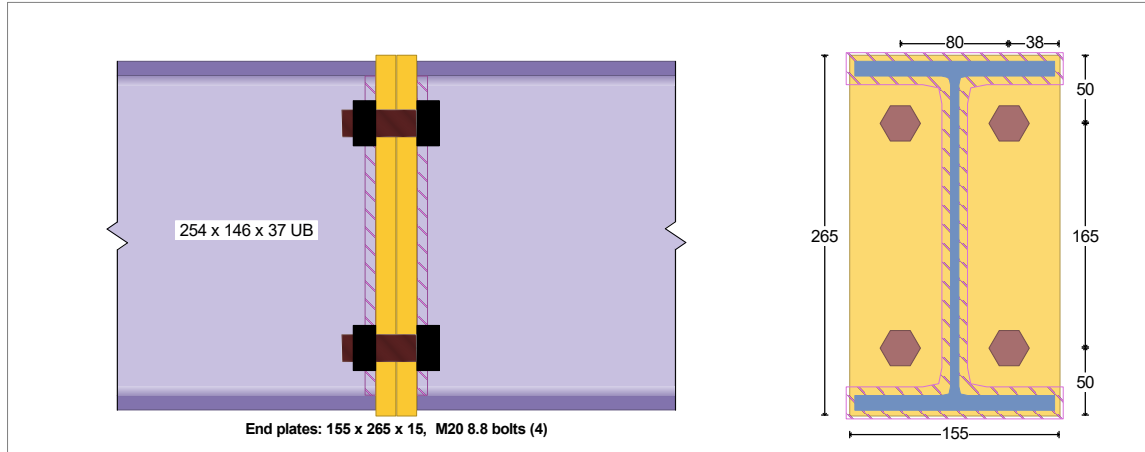
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End plate splice calculation

Beam: B1 6.1 m.

Section: **254 x 146 x 37 UB** S355



Splice 1: 2.1 m. from R1. B.M.: 39.8 kNm S.F.: 9.0 kN (factored)
Section dims: D = 256.0 B = 146.4 T = 10.9 t = 6.3 d = 219.0 r = 7.6

Use S275 end plates 265 x 155 x 15 mm, 2 pairs of M20 8.8 bolts

Bolts are at 80 mm cross centres; top of plate to top row of bolts: 50 mm; second row: 215 mm

8.8 bolt strength: Shear, $p_s = 375 \text{ N/mm}^2$, tension $p_t = 560 \text{ N/mm}^2$ [Tables 30/34]

Minimum fillet weld sizes: Top (compression) flange: nominal 6 mm (bearing fit assumed)

Bottom (tension) flange: $10.9/(2 \times 0.7) = 7.8 \text{ mm}$, say 8 mm (full strength)

Web: $6.3/(2 \times 0.7) = 4.5 \text{ mm}$, say 5 mm (full strength)

Basic detailing checks

Using M20 bolts, washer diameter 37 mm

Check plate height (265 mm) $\geq$ section depth (256 mm) OK

Check top bolt washer clears top flange weld: top of plate to top bolt (50 mm) $\geq$ 40 mm OK

Check bottom bolt washer clears bottom flange weld: top of plate to bottom bolt (215 mm) $\leq$ 223 mm OK

Check bolt washers clear web welds: bolt cross centres (80 mm) $\geq$ 54 mm OK

Check bolt cross centres $\geq$ 2.5D (50 mm) OK

Check bolt vertical centres (165 mm) $\geq$ 2.5D (50.0 mm) OK

Bolt edge distance (37.5 mm) $\geq$ 1.25D (25.0 mm) OK

Top bolt end distance (50 mm) $\geq$ 1.25D (25 mm) OK

Bottom bolt end distance (50 mm) $\geq$ 1.25D (25 mm) OK

All dimensional checks satisfied

Basis of design

This calculation is derived from the SCI publication P207: Design of Moment Connections. The compression zone is taken as acting at the centre of the top flange. The top pair of bolts are assumed to act in shear only. The bending moment at the splice is resisted by the other bolt row which may also be able to resist shear.

Bolt bearing capacity on plate = $20 \times 15 \times 460/1000 = 138 \text{ kN}$

Bolt shear capacity = $245 \times 375/1000 = 91.9 \text{ kN}$ ←

Enhanced bolt tension capacity, $P_t' = 245 \times 560/1000 = 137 \text{ kN}$

Max tensile length of web per bolt row, $L_t = (80/2) \times \tan 60 = 69.3 \text{ mm}$ above and below each row

N.B. the length below may not extend into the length above from the row below

Web tensile capacity per mm = $6.3 \times 355/1000 = 2.24 \text{ kN/mm}$

m_1 , bolt centre to 20% into web weld = 32.8 mm (5mm fillet welds assumed)

m_2 , bolt centre to 20% into bottom flange weld = 28.2 mm (8mm fillet welds assumed)

$\lambda_1 = m_1/(m_1 + e) = 0.47$ $\lambda_2 = m_2/(m_1 + e) = 0.40$ $\alpha = 6.214$ (P207 Appendix iii)

n , effective edge distance = $\min\{\text{edge distance}, 1.25m\} = 37.5 \text{ mm}$

$L_{eff}(i) = 2\pi m = 206.4 \text{ mm}$ $L_{eff}(ii) = 4m + 1.25e = 178.3 \text{ mm}$ $L_{eff}(iii) = \alpha m = 204.1 \text{ mm}$ (Table 2.4, $\alpha = 6.214$)

L_{eff} for bolts acting alone (Table 2.5)

Bottom row:

$L_{eff} = \min\{\max\{[ii+iii]/2, ii\}, i\} = 191.2 \text{ mm}$

Other rows:

$L_{eff} = \min\{ii, i\} = 178.3 \text{ mm}$

L_{eff} for bolts acting as a group (Table 2.6)

Bottom row:

$L_{eff} = \max\{ii/2, iii/2\} + p/2 = 137.1 \text{ mm}$

Non-bottom end row:

$L_{eff} = ii/2 + p/2 = 124.1 \text{ mm}$

$M_{p, \text{end plate T-stub plastic moment capacity}} = L_{eff} \times t^2 \times p_y/4$

Row capacity

Mode 1 (end plate yields at bolt positions and either side of web): $P_r = 4M_{p, \text{end plate}}$

Mode 2 (end plate yields either side of web; bolts fail in tension): $P_r = (2M_{p, \text{end plate}} + n(\Sigma P_t'))/(m + n)$

Mode 3 (bolt tensile failure): $P_r = \Sigma P_t'$

Bolt rows are numbered from the bottom (bottom pair = 1; top pair = 2)

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Row	L_{eff}	M_p	Mode 1	Mode 2	Mode 3	$L_{t,web}$	$P_{t,web}$	$P_{t,group}$ kN	P_i kN
1	191	2,958	360	230	274				230.3 ←

Check moment and shear capacity of splice

Compressive resistance of top flange = $1.4 \times 355 \times 146.4 \times 10.9/1000 = 793$ kN

Bolt line	Bolt LA mm	P_i kN	Row RM kNm	Bolt F_v/P_v	F_v kN
1	205	230.3	47.2	0.40	73.5
2	40	0.0	0.0	1.00	183.8
Totals		230.3	47.2		257.3

Section ultimate moment capacity = $p_y S_x = 171$ kNm; Splice moment capacity = **47.2** kNm (27.5%)

Splice 1: Design moment: 39.80 kNm Design SF: 9.00 kN

No further checking necessary

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Beam: B2						Span: 1.0 m.		
	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
U D	timber floor	0.5*4.5	0		L	1.13	1.13	0.03
U L	timber floor	1.5*4.5	0		L	3.38	3.38	0.09
U D	partition UDL	1.0*4.5	0		L	2.25	2.25	0.06
U D	masonry	1.5	0		L	0.75	0.75	0.02
Total load (unfactored):						15.00 kN	7.50	7.50
Dead/Permanent (unfactored):						8.25 kN	4.13	4.13
Live/Variable (unfactored):						6.75 kN	3.38	3.38
Total load (factored):						22.35 kN	11.18	11.18

Load types: U:UDL; Load positions: m. from R1; Load durations: D: Dead; L: Live

Maximum B.M. = 2.79 kNm (factored) at 0.50 m. from R1

Maximum S.F. = 11.18 kN (factored) at R1

Mid-span deflections: Dead: $0.11 \times 10^8 / EI$ (**E in N/mm², I in cm⁴**)

Live: $0.09 \times 10^8 / EI$

Total: $0.20 \times 10^8 / EI$

User script generated output follows

Use Stressline R15A 100x140 mm concrete lintel

Load capacity data and availability taken from Stressline Lintel Guide 2011 p.38

Check below may be invalid where member carries significant point loads

Required lintel length = 1200 mm. (span, 1.0m. is to centre of bearings), min 150mm bearings

Unfactored load carried by lintel = 15.00 kN

Average load = $15.00 / 1.00 = 15.00$ kN/m

Use **Stressline R15A 100x140 mm** concrete lintel

Lintel SWL = 28.2 kN/m (≥ 15.00 OK) Resistance moment = 3.7 kNm (≥ 1.88 OK)

End of user script generated output

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Beam: B3						Span: 3.2 m.		
	Load name	Loading w1	Start x1	Loading w2	End x2	R1 comp	R2 comp	Defl.
O D	o.w.	0.25	0		L	0.40	0.40	0.34
U D	timber floor	0.5*4.5	0		L	3.60	3.60	3.07
U L	timber floor	1.5*4.5	0		L	10.80	10.80	9.22
U D	partition UDL	1.0*4.5	0		L	7.20	7.20	6.14
Total load (unfactored):						44.0 kN	22.00	22.00
Dead/Permanent (unfactored):						22.4 kN	11.20	11.20
Live/Variable (unfactored):						21.6 kN	10.80	10.80
Total load (factored):						65.9 kN	32.96	32.96

Load types: O:Beam o.w.; U:UDL; Load positions: m. from R1; Load durations: D: Dead; L: Live

Maximum B.M. = 26.37 kNm (factored) at 1.60 m. from R1

Maximum S.F. = 33.0 kN (factored) at R1

Mid-span deflections: Dead: $9.56 \times 10^8 / EI$ (**E in N/mm², I in cm⁴**)

Live: $9.22 \times 10^8 / EI$

Total: $18.78 \times 10^8 / EI$

Beam calculation to BS5950-1:2000 using S355 steel

SECTION SIZE : 203 x 133 x 25 UB S355 (compact)

D=203.2 mm B=133.2 mm t=5.7 mm T=7.8 mm $I_x=2,340 \text{ cm}^4$ $r_y=3.10 \text{ cm}$ $S_x=258 \text{ cm}^3$ $x=25.6$

Section classification: T = 7.8mm $p_v = 355 \text{ N/mm}^2$ $\varepsilon = \sqrt{275/355} = 0.88$

(Table 11) Flange: $b/T = 66.6/7.8 = 8.54$ ($\leq 10\varepsilon$: Class 2 compact)

Web: $d/t = 172.4/5.7 = 30.2$ ($\leq 80\varepsilon$: Class 1, plastic)

For design purposes section classification is Class 2, compact

Shear

Maximum S.F. = 33 kN

Shear capacity = $0.6 p_v t D = 0.6 \times 355 \times 5.7 \times 203.2 / 1000 = 247 \text{ kN OK}$

Bending

Maximum B.M. = 26.4 kNm

Moment capacity, $M_c = p_v S_x = 355 \times 258 / 1000 = 91.6 \text{ kNm OK}$

Lateral-torsional buckling

Beam is laterally restrained at supports only

Restraint condition at R1 and R2: Compression flange laterally unrestrained; both flanges free to rotate on plan. Partial torsional restraint by dead bearing of bottom flange to support (1.2L+2D)

Effective length = $1.2L+2D = 4.25 \text{ m}$. [Table 13]

Bending strength, $p_b = 147 \text{ N/mm}^2$

Maximum moment within segment, $M_x = 26.4 \text{ kNm}$

Equivalent uniform moment factor, $m_{LT} = 0.925$ ($M_2 = 19.8, M_3 = 26.4, M_4 = 19.8$)

Equivalent uniform moment = $0.925 \times 26.4 = 24.39 \text{ kNm}$

Buckling resistance moment, $M_b = p_b S_x = 147 \times 258 / 1000 = 38.0 \text{ kNm OK}$

Web buckling and crushing have not been checked

Deflection

LL deflection = $9.22 \times 10^8 / (205,000 \times 2,340) = 1.9 \text{ mm}$ (L/1665) OK

TL deflection = $18.78 \times 10^8 / (205,000 \times 2,340) = 3.9 \text{ mm}$ (L/817)

Bearings

203 x 133 x 25 UB stiff bearing length, $b_1 = t + 1.6r + 2T = 33.5 \text{ mm}$

Masonry: 100mm 3.6N/mm² solid block (SF>2.0), class (iii) mortar, normal const/normal mfr, Class 2 bearing

Local design strength (factored) = $3.5 \times 1.5/3.5 = 1.50 \text{ N/mm}^2$ (BS5628-1:2005 Table 2d/2e)

R1: 225 x 100 mm Padstone

Factored reaction = $1.4 \times 11.20 + 1.6 \times 10.80 = 33.0 \text{ kN}$

Factored stress under padstone = $33.0 \times 1000 / 225 \times 100 = 1.46 \text{ N/mm}^2$ OK

provide min 150 mm bearing onto engineering brick padstone bedded on mortar off internal load bearing wall

R2: 225 x 100 mm Padstone

Factored reaction = $1.4 \times 11.20 + 1.6 \times 10.80 = 33.0 \text{ kN}$

Factored stress under padstone = $33.0 \times 1000 / 225 \times 100 = 1.46 \text{ N/mm}^2$ OK

provide min 100 mm bearing onto party wall masonry

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Beam: L1						Span: 1.1 m.		
	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
U D	pitched roof	1.0*1.4	0		L	0.77	0.77	0.03
U L	pitched roof	0.75*1.4	0		L	0.58	0.58	0.02
U D	cavity wall	3.3*1.2	0		L	2.18	2.18	0.08
U D	timber floor	0.5*2.5	0		L	0.69	0.69	0.02
U L	timber floor	1.5*2.5	0		L	2.06	2.06	0.07
U D	partition UDL	0.5*2.5	0		L	0.69	0.69	0.02
Total load (unfactored):						13.93 kN	6.96	6.96
Dead/Permanent (unfactored):						8.65 kN	4.32	4.32
Live/Variable (unfactored):						5.28 kN	2.64	2.64
Total load (factored):						20.55 kN	10.28	10.28

Load types: U:UDL; Load positions: m. from R1; Load durations: D: Dead; L: Live

Maximum B.M. = 2.83 kNm (factored) at 0.55 m. from R1

Maximum S.F. = 10.28 kN (factored) at R1

Mid-span deflections: Dead: $0.15 \times 10^8 / EI$ (**E in N/mm², I in cm⁴**)

Live: $0.09 \times 10^8 / EI$

Total: $0.24 \times 10^8 / EI$

**PROVIDE CG90/100 LINTEL FROM CATNIC OR SIMILAR APPROVED
FOR 90-105mm CAVITY
100-115mm WIDE INNER LEAF**

(SWL = 15 kN, L = 750-1500 mm)

(SWL = 18 kN, L = 1650-1800 mm)

(SWL = 20 kN, L = 1950-2100 mm)

(SWL = 22 kN, L = 2250-2400 mm)

(SWL = 26 kN, L = 2550-3600 mm)

- ENSURE MIN 150 mm BEARINGS ONTO SOUND MASONRY

- LINTEL TO BE INSTALLED IN ACCORDANCE WITH MANUFACTURERS RECOMMENDATIONS

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Beam: L2						Span: 2.7 m.		
	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
U D	pitched roof	1.0*1.4	0		L	1.89	1.89	0.97
U L	pitched roof	0.75*1.4	0		L	1.42	1.42	0.73
U D	cavity wall	3.3*1.2	0		L	5.35	5.35	2.74
U D	timber floor	0.5*2.5	0		L	1.69	1.69	0.86
U L	timber floor	1.5*2.5	0		L	5.06	5.06	2.59
U D	partition UDL	0.5*2.5	0		L	1.69	1.69	0.86
Total load (unfactored):						34.2 kN	17.09	17.09
								8.76
Dead/Permanent (unfactored):						21.2 kN	10.61	10.61
Live/Variable (unfactored):						13.0 kN	6.48	6.48
Total load (factored):						50.4 kN	25.22	25.22

Load types: U:UDL; Load positions: m. from R1; Load durations: D: Dead; L: Live

Maximum B.M. = 17.03 kNm (factored) at 1.35 m. from R1

Maximum S.F. = 25.22 kN (factored) at R1

Mid-span deflections: Dead: $5.44 \times 10^8 / EI$ (**E in N/mm², I in cm⁴**)

Live: $3.32 \times 10^8 / EI$

Total: $8.76 \times 10^8 / EI$

**PROVIDE CX90/100 LINTEL FROM CATNIC OR SIMILAR APPROVED
FOR 90-105mm CAVITY
100-115mm WIDE INNER LEAF**

(SWL = 60 kN, L = 900-2700 mm)

(SWL = 55 kN, L = 2850-3000 mm)

(SWL = 50 kN, L = 3300-3900 mm)

(SWL = 32 kN, L = 4200-4800 mm)

- ENSURE MIN 150 mm BEARINGS ONTO SOUND MASONRY

- LINTEL TO BE INSTALLED IN ACCORDANCE WITH MANUFACTURERS RECOMMENDATIONS

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Beam: L3						Span: 2.0 m.		
	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
U D	pitched roof	1.0*1.4	0		L	1.40	1.40	0.29
U L	pitched roof	0.75*1.4	0		L	1.05	1.05	0.22
U D	cavity wall	3.3*1.2	0		L	3.96	3.96	0.82
U D	timber floor	0.5*2.5	0		L	1.25	1.25	0.26
U L	timber floor	1.5*2.5	0		L	3.75	3.75	0.78
U D	partition UDL	0.5*2.5	0		L	1.25	1.25	0.26
Total load (unfactored):						25.32 kN	12.66	12.66
								2.64
Dead/Permanent (unfactored):						15.72 kN	7.86	7.86
Live/Variable (unfactored):						9.60 kN	4.80	4.80
Total load (factored):						37.37 kN	18.68	18.68

Load types: U:UDL; Load positions: m. from R1; Load durations: D: Dead; L: Live

Maximum B.M. = 9.34 kNm (factored) at 1.00 m. from R1

Maximum S.F. = 18.68 kN (factored) at R1

Mid-span deflections: Dead: $1.64 \times 10^8 / EI$ (**E in N/mm², I in cm⁴**)

Live: $1.00 \times 10^8 / EI$

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

**PROVIDE CH90/100 LINTEL FROM CATNIC OR SIMILAR APPROVED
FOR 90-105mm CAVITY
100-115mm WIDE INNER LEAF**

(SWL = 32 kN, L = 900-1800 mm)

(SWL = 48 kN, L = 1950-2100 mm)

(SWL = 45 kN, L = 2250-2400 mm)

- ENSURE MIN 150 mm BEARINGS ONTO SOUND MASONRY

- LINTEL TO BE INSTALLED IN ACCORDANCE WITH MANUFACTURERS RECOMMENDATIONS

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Pier: MP1

Loading

Pitched Roof	=	1.75	x	1.40	x	2.50	=	6.1	kN
Timber Floor	=	2.00	x	2.50	x	2.50	=	12.5	kN
Partition UDL	=	0.50	x	2.50	x	2.50	=	3.1	kN
Lintel Masonry	=	1.30	x	1.20	x	1.70	=	2.7	kN
Inner Leaf Over	=	1.30	x	0.80	x	1.00	=	1.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
Self Weight	=	1.30	x	0.80	x	1.00	=	1.0	kN
						Total	=	26.5	kN

Additional Foundations Loading

masonry	=	1.30	x	0.80	x	1.50	=	1.6	kN
	=	0.00	x	0.00	x	1.00	=	0.0	kN
	=	0.00	x	0.00	x	1.00	=	0.0	kN
	=	0.00	x	0.00	x	1.00	=	0.0	kN
						Total	=	1.6	kN

Pier length/Wall height	=	2.500	m		ym	=	3.5
Effective length/height factor	=	1.00			Gk/Qk	=	1.5
Effective thickness factor	=	1.35			(1.35 for standard 102 50 100 cavity wall)		

Pier Size

= 100 x 800mm

CSA RF = 0.82

L/b	=	2500	x	1	/	135	=	19
ex	=	13.5		ea	=	17.27	et =	25
em	=	25.4						
B=1.1(1-2xem/t)	=	0.69						

Basic Required	=	26.48	x	1.5	x	3.5	x	1000	=3.1N/mm ²
		100	x	800	x	0.69	x	0.82	

Therefore use a min of:

Allowable basic stress (exc. fk x 1.15)

3.6 N/mm² Blocks in 1:1:6 mortar

= 3.5 N/mm²

project : Land Adjacent 2 The Spinney, Eastern Road, RH16 3NG
project no : 9501
by : ICR
date : Apr 2023
no : 20

spithurst hub spithurst road
barcombe east sussex BN8 5EE
01273 470 066 info@ingsengineering.co.uk

Wall: W1

Loading

Roof+Rspace	=	2.15	x	6.10	x	0.50	=	6.6	kN
Inner Leaf Over	=	1.30	x	1.00	x	4.20	=	5.5	kN
Timber Floor	=	2.00	x	1.00	x	0.30	=	0.6	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
Self Weight	=	1.30	x	1.00	x	1.00	=	1.3	kN
						Total	=	13.9	kN

Additional Foundations Loading

masonry	=	1.30	x	1.00	x	1.50	=	2.0	kN
	=	0.00	x	0.00	x	1.00	=	0.0	kN
	=	0.00	x	0.00	x	1.00	=	0.0	kN
	=	0.00	x	0.00	x	1.00	=	0.0	kN
						Total	=	2.0	kN

Pier length/Wall height	=	2.500	m		ym	=	3.5
Effective length/height factor	=	1.00			Gk/Qk	=	1.5
Effective thickness factor	=	1.35			(1.35 for standard 102 50 100 cavity wall)		

Pier Size

CSA RF = 100 x 1000mm

CSA RF = 1.00

L/b	=	2500	x	1	/	135	=	19
ex	=	13.5		ea	=	17.27	et =	25
em	=	25.4						
B=1.1(1-2xem/t)	=	0.69						

Basic Required	=	13.92	x	1.5	x	3.5	x	1000	=1.1N/mm ²
		100	x	1000	x	0.69	x	1.00	

Therefore use a min of:

Allowable basic stress (exc. fk x 1.15)

3.6 N/mm² Blocks in 1:1:6 mortar

= 3.5 N/mm²

project : Land Adjacent 2 The Spinney, Eastern Road, RH16 3NG
project no : 9501
by : ICR
date : Apr 2023
no : 21

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01273 470 066 info@ingsengineering.co.uk

Wall: W2

Loading

Timber Floor	=	2.00	x	4.50	x	2.50	=	22.5	kN
Partiton UDL	=	0.50	x	4.50	x	2.50	=	5.6	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
Self Weight	=	1.50	x	1.00	x	1.00	=	1.5	kN
						Total	=	29.6	kN

Additional Foundations Loading

masonry	=	1.50	x	1.00	x	1.50	=	2.3	kN
	=	0.00	x	0.00	x	1.00	=	0.0	kN
	=	0.00	x	0.00	x	1.00	=	0.0	kN
	=	0.00	x	0.00	x	1.00	=	0.0	kN
						Total	=	2.3	kN

Pier length/Wall height	=	2.500	m	ym	=	3.5
Effective length/height factor	=	0.75		Gk/Qk	=	1.5
Effective thickness factor	=	1.00		(1.35 for standard 102 50 100 cavity wall)		

Pier Size

CSA RF	=	100 x 1000mm
	=	1.00
L/b	=	2500 x 0.75 / 100 = 19
ex	=	10 ea = 13.15 et = 19
em	=	19.1
B=1.1(1-2xem/t)	=	0.68

Basic Required	=	$\frac{29.63}{100} \times \frac{1.5}{1000} \times \frac{3.5}{0.68} \times \frac{1000}{1.00}$	=2.3N/mm ²
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Therefore use a min of:

Allowable basic stress (exc. fk x 1.15)	=	3.6 N/mm ² Blocks in 1:1:6 mortar
---	---	--

project : Land Adjacent 2 The Spinney, Eastern Road, RH16 3NG
project no : 9501
by : ICR
date : Apr 2023
no : 22

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01273 470 066 info@ingsengineering.co.uk

Foundation: F1

Loading

Pitched Roof	=	1.75	x	1.40	x	1.00	=	2.5	kN
Cavity Wall	=	3.30	x	3.50	x	1.00	=	11.6	kN
Timber Floor	=	2.50	x	2.50	x	1.00	=	6.3	kN
Beam & Block Floor	=	6.00	x	2.50	x	1.00	=	15.0	kN
Cavity Wall	=	4.00	x	1.00	x	0.45	=	1.8	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
						Total	=	37.1	kN

Provide Gen 1 mass conc. Foundation: 0.45m wide x 1.00m long x 0.60m thick
Ground Bearing Pressure = 38.9 / 0.45 x 1 = 87kN/m²

**Provide min 450mm wide GEN1 concrete foundations
to the satisfaction of the BCO.
Report any discrepancy for consideration.**

project : Land Adjacent 2 The Spinney, Eastern Road, RH16 3NG
project no : 9501
by : ICR
date : Apr 2023
no : 23

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01273 470 066 info@ingsengineering.co.uk

Foundation: F2

Loading

Pitched Roof	=	2.15	x	3.05	x	1.00	=	6.6	kN
Cavity Wall	=	3.30	x	6.60	x	1.00	=	21.8	kN
Timber Floor	=	2.00	x	0.30	x	1.00	=	0.6	kN
Beam & Block Floor	=	6.00	x	0.30	x	1.00	=	1.8	kN
Cavity Wall	=	4.00	x	1.00	x	0.45	=	1.8	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
						Total	=	32.5	kN

Provide Gen 1 mass conc. Foundation: 0.45m wide x 1.00m long x 0.60m thick
Ground Bearing Pressure = 34.4 / 0.45 x 1 = 77kN/m²

**Provide min 450mm wide GEN1 concrete foundations
to the satisfaction of the BCO.
Report any discrepancy for consideration.**

project : Land Adjacent 2 The Spinney, Eastern Road, RH16 3NG
project no : 9501
by : ICR
date : Apr 2023
no : 24

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Foundation: F3

Loading

Pitched Roof	=	2.15	x	5.80	x	1.00	=	12.5	kN
Cavity Wall	=	4.20	x	6.60	x	1.00	=	27.7	kN
Timber Floor	=	2.00	x	0.60	x	1.00	=	1.2	kN
Beam & Block Floor	=	6.00	x	0.60	x	1.00	=	3.6	kN
Cavity Wall	=	4.00	x	1.00	x	0.45	=	1.8	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
						Total	=	46.8	kN

Provide Gen 1 mass conc. Foundation: 0.45m wide x 1.00m long x 0.60m thick
Ground Bearing Pressure = 48.7 / 0.45 x 1 = 108kN/m²

**Provide min 450mm wide GEN1 concrete foundations
to the satisfaction of the BCO.
Report any discrepancy for consideration.**

project : Land Adjacent 2 The Spinney, Eastern Road, RH16 3NG
project no : 9501
by : ICR
date : Apr 2023
no : 25

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Foundation: F4

Loading

Timber Floor	=	2.50	x	4.50	x	1.00	=	11.3	kN
Masonry	=	1.50	x	2.50	x	1.00	=	3.8	kN
Beam & Block Floor	=	6.00	x	4.50	x	1.00	=	27.0	kN
Masonry	=	4.00	x	1.00	x	0.45	=	1.8	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
	=	0.00	x	1.00	x	1.00	=	0.0	kN
						Total	=	43.8	kN

Provide Gen 1 mass conc. Foundation: 0.45m wide x 1.00m long x 0.60m thick
Ground Bearing Pressure = 45.7 / 0.45 x 1 = 102kN/m²

**Provide min 450mm wide GEN1 concrete foundations
to the satisfaction of the BCO.
Report any discrepancy for consideration.**

Ings Engineering Ltd		Page 26
Spithurst Hub	Land Adjacent	
Spithurst Road	2 The Spinney	
Barcombe, East Sussex, BN8 5EE	Eastern Rd, Haywards Heath	
Date 09/04/2023 11:25	Designed by ICR	
File 9501_storage.SRCX	Checked by	
Innovyze		Source Control 2018.1

Rainfall Details

Rainfall Model	FSR	Winter Storms	Yes
Return Period (years)	100	Cv (Summer)	0.750
Region	England and Wales	Cv (Winter)	0.840
M5-60 (mm)	20.000	Shortest Storm (mins)	15
Ratio R	0.335	Longest Storm (mins)	10080
Summer Storms	Yes	Climate Change %	+40

Time Area Diagram

Total Area (ha) 0.038

Time (mins)	Area	Time (mins)	Area	Time (mins)	Area
From: To:	(ha)	From: To:	(ha)	From: To:	(ha)
0 4	0.013	4 8	0.013	8 12	0.013

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Ings Engineering Ltd		Page 27																				
Spithurst Hub Spithurst Road Barcombe, East Sussex, BN8 5EE	Land Adjacent 2 The Spinney Eastern Rd, Haywards Heath																					
Date 09/04/2023 11:25 File 9501_storage.SRCX	Designed by ICR Checked by																					
Innovyze	Source Control 2018.1																					
Model Details Storage is Online Cover Level (m) 10.000 Tank or Pond Structure Invert Level (m) 6.500 <table><thead><tr><th>Depth (m)</th><th>Area (m²)</th><th>Depth (m)</th><th>Area (m²)</th><th>Depth (m)</th><th>Area (m²)</th></tr></thead><tbody><tr><td>0.000</td><td>7.2</td><td>2.500</td><td>7.2</td><td>2.501</td><td>0.0</td></tr></tbody></table> Pump Outflow Control Invert Level (m) 6.500 <table><thead><tr><th>Depth (m)</th><th>Flow (l/s)</th><th>Depth (m)</th><th>Flow (l/s)</th></tr></thead><tbody><tr><td>1.000</td><td>1.0000</td><td>2.500</td><td>1.0000</td></tr></tbody></table>			Depth (m)	Area (m²)	Depth (m)	Area (m²)	Depth (m)	Area (m²)	0.000	7.2	2.500	7.2	2.501	0.0	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	1.000	1.0000	2.500	1.0000
Depth (m)	Area (m²)	Depth (m)	Area (m²)	Depth (m)	Area (m²)																	
0.000	7.2	2.500	7.2	2.501	0.0																	
Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)																			
1.000	1.0000	2.500	1.0000																			
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Spithurst Hub Spithurst Road Barcombe, East Sussex, BN8 5EE		Land Adjacent 2 The Spinney Eastern Rd, Haywards Heath			
Date 09/04/2023 11:26 File 9501_storage.SRCX		Designed by ICR Checked by			
Innovyze		Source Control 2018.1			
Summary of Results for 100 year Return Period (+40%)					
Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	7.673	1.173	1.0	8.4	O K
30 min Summer	8.049	1.549	1.0	11.2	O K
60 min Summer	8.399	1.899	1.0	13.7	O K
120 min Summer	8.624	2.124	1.0	15.3	O K
180 min Summer	8.668	2.168	1.0	15.6	O K
240 min Summer	8.663	2.163	1.0	15.6	O K
360 min Summer	8.610	2.110	1.0	15.2	O K
480 min Summer	8.522	2.022	1.0	14.6	O K
600 min Summer	8.420	1.920	1.0	13.8	O K
720 min Summer	8.314	1.814	1.0	13.1	O K
960 min Summer	8.109	1.609	1.0	11.6	O K
1440 min Summer	7.764	1.264	1.0	9.1	O K
2160 min Summer	7.463	0.963	1.0	6.9	O K
2880 min Summer	7.319	0.819	0.8	5.9	O K
4320 min Summer	7.139	0.639	0.6	4.6	O K
5760 min Summer	7.027	0.527	0.5	3.8	O K
7200 min Summer	6.954	0.454	0.5	3.3	O K
8640 min Summer	6.900	0.400	0.4	2.9	O K
10080 min Summer	6.859	0.359	0.4	2.6	O K
15 min Winter	7.822	1.322	1.0	9.5	O K
30 min Winter	8.252	1.752	1.0	12.6	O K
Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)	
15 min Summer	129.845	0.0	9.2	24	
30 min Summer	87.874	0.0	12.5	37	
60 min Summer	56.713	0.0	16.2	64	
120 min Summer	35.254	0.0	20.1	116	
180 min Summer	26.243	0.0	22.4	146	
240 min Summer	21.157	0.0	24.1	180	
360 min Summer	15.629	0.0	26.7	248	
480 min Summer	12.585	0.0	28.7	316	
600 min Summer	10.628	0.0	30.3	382	
720 min Summer	9.252	0.0	31.6	448	
960 min Summer	7.426	0.0	33.9	578	
1440 min Summer	5.437	0.0	37.2	816	
2160 min Summer	3.971	0.0	40.7	1156	
2880 min Summer	3.173	0.0	43.4	1524	
4320 min Summer	2.308	0.0	47.4	2248	
5760 min Summer	1.841	0.0	50.4	2952	
7200 min Summer	1.546	0.0	52.9	3680	
8640 min Summer	1.341	0.0	55.0	4416	
10080 min Summer	1.189	0.0	56.9	5144	
15 min Winter	129.845	0.0	10.4	24	
30 min Winter	87.874	0.0	14.0	38	
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Spithurst Hub		Land Adjacent			
Spithurst Road		2 The Spinney			
Barcombe, East Sussex, BN8 5EE		Eastern Rd, Haywards Heath			
Date 09/04/2023 11:26		Designed by ICR			
File 9501_storage.SRCX		Checked by			
Innovyze		Source Control 2018.1			
<u>Summary of Results for 100 year Return Period (+40%)</u>					
Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
60 min Winter	8.658	2.158	1.0	15.5	O K
120 min Winter	8.940	2.440	1.0	17.6	O K
180 min Winter	8.975	2.475	1.0	17.8	O K
240 min Winter	8.953	2.453	1.0	17.7	O K
360 min Winter	8.855	2.355	1.0	17.0	O K
480 min Winter	8.708	2.208	1.0	15.9	O K
600 min Winter	8.544	2.044	1.0	14.7	O K
720 min Winter	8.376	1.876	1.0	13.5	O K
960 min Winter	8.058	1.558	1.0	11.2	O K
1440 min Winter	7.577	1.077	1.0	7.8	O K
2160 min Winter	7.318	0.818	0.8	5.9	O K
2880 min Winter	7.173	0.673	0.7	4.8	O K
4320 min Winter	7.002	0.502	0.5	3.6	O K
5760 min Winter	6.905	0.405	0.4	2.9	O K
7200 min Winter	6.842	0.342	0.3	2.5	O K
8640 min Winter	6.798	0.298	0.3	2.1	O K
10080 min Winter	6.765	0.265	0.3	1.9	O K
Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)	
60 min Winter	56.713	0.0	18.1	64	
120 min Winter	35.254	0.0	22.5	120	
180 min Winter	26.243	0.0	25.1	168	
240 min Winter	21.157	0.0	27.0	192	
360 min Winter	15.629	0.0	29.9	270	
480 min Winter	12.585	0.0	32.1	344	
600 min Winter	10.628	0.0	33.9	414	
720 min Winter	9.252	0.0	35.4	484	
960 min Winter	7.426	0.0	37.9	612	
1440 min Winter	5.437	0.0	41.6	832	
2160 min Winter	3.971	0.0	45.6	1180	
2880 min Winter	3.173	0.0	48.6	1540	
4320 min Winter	2.308	0.0	53.0	2256	
5760 min Winter	1.841	0.0	56.4	3000	
7200 min Winter	1.546	0.0	59.2	3688	
8640 min Winter	1.341	0.0	61.6	4416	
10080 min Winter	1.189	0.0	63.7	5144	
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Spithurst Hub Spithurst Road Barcombe, East Sussex, BN8 5EE	Land Adjacent 2 The Spinney	
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