



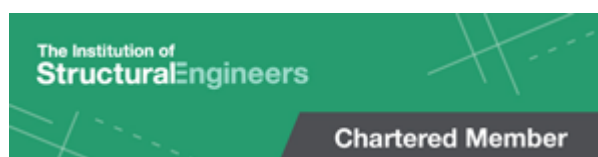
Chartered Structural Engineer

STRUCTURAL CALCULATIONS
FOR
THE OLD SCHOOL HOUSE, HAGUE STREET,
GLOSSOP

REVISION A

Ref: 2186 – CAL A

Any dimensions quoted or used within the calculations are for use within the calculations only and should not be taken as any actual dimensions of any existing or proposed elements.



THE OLD SCHOOL HOUSE, HAGUE STREET, GLOSSOP

STRUCTURAL CALCULATIONS

REVISION A

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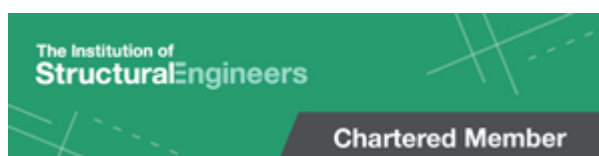
A2 Lower Ground Floor Plan - SK01

A2 Upper Ground Floor Plan Showing Beams Over - SK02

A2 Lower Ground Floor Plan Showing Floor Structure Over - SK03

Appendix

Architect's Layouts	A01-21
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01

Project No :

2186

Designer :

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Checked :

Project :

The Old School House, Hague Street, Glossop

Date :

25/08/2026

I

LOADING:-

- DESIGN CARRIED OUT IN LINE WITH EUROCODES

(SEE ARCHITECTS LAYOUTS IN APPENDIX)

- PITCHED ROOF:-

PERMANENT:

- NATURAL STONE FLAGS	=	1.00
- INSULATION AND BATTENS	=	0.10
- RAFTERS	=	0.15
- CEILING & SERVICES	=	0.25
		<u>1.50</u>

kN/m² ON FLAT

$$\text{- PLAN LOADING} = 1.50 / \cos 40 = \underline{\underline{1.96}}$$

kN/m² ON PLAN

VARIABLE:

$$\text{- SNOW / ACCESS} = \underline{\underline{0.60}}$$

kN/m²

- TIMBER FLOOR:-

PERMANENT:

- BOARDS	=	0.15
- JOISTS	=	0.20
- CEILING & SERVICES	=	0.25
- PARTITIONS	=	0.50
		<u>1.10</u>

kN/m²

VARIABLE:

$$\text{- DOMESTIC} = \underline{\underline{1.50}}$$

kN/m²

- INTERNAL WALL:-

PERMANENT:

- PLASTER	=	0.36
- 102 BRICK	=	2.10
- PLASTER	=	0.36
		<u>2.82</u>

kN/m²



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2

BEAM OVER INTERNAL WALL REMOVAL:-

- BEAM CARRIES WALL, FLOOR & ROOF OVER

- SPAN = 2.20 m

- UDL LOAD:

- ROOF:

PERM. = 1.96 x 3.00

VAR. = 0.60 x 3.00

DL	LL
5.88	
	1.80

- 1ST FLOOR:

PERM. = 1.10 x 3.00

VAR. = 1.50 x 3.00

DL	LL
3.30	
	4.50

- WALL:

PERM. = 2.82 x 3.00

DL	LL
8.46	
17.64	6.30

kN/m

(SEE CALCULATIONS THAT FOLLOW)

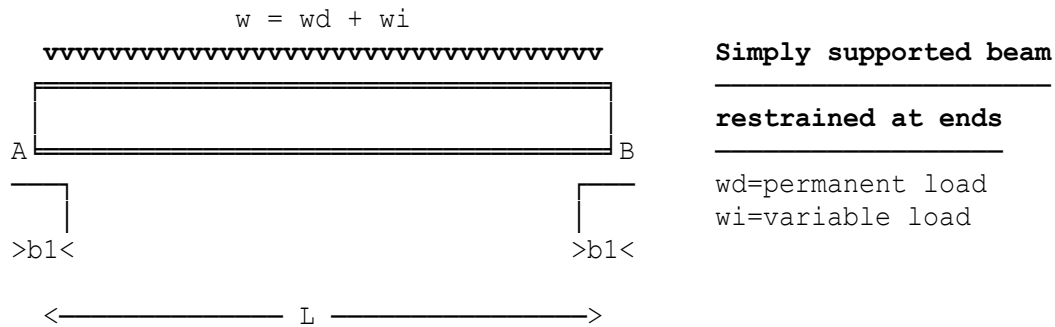
PROVIDE

- 152 x 89 UB 16 STEEL BEAM
OVER INTERNAL WALL REMOVAL
SUPPORTING WALL, FLOOR & ROOF OVER

Location: BEAM OVER INTERNAL WALL REMOVAL

Calculations are in accordance with BS EN 1993-1-1:2005 Eurocode 3: Design of steel structures and the non-contradictory, complimentary information sheets provided by Access Steel.

The beam under consideration is assumed to be a hot rolled section.



Beam span $L=2.2$ m

Loading (unfactored)

Permanent UDL (including S.W) $w_d=18$ kN/m
 Variable UDL $w_i=6.3$ kN/m

Unfactored loads

Distributed load $w'=w_d+w_i=18+6.3=24.3$ kN/m

Unfactored shear forces

At end B $V_b'=w'*L/2=24.3*2.2/2=26.73$ kN
 At end A $V_a'=w'*L-V_b'=24.3*2.2-26.73=26.73$ kN

Factored loads

Distributed load $w'=w_d*\gamma_mG+w_i*\gamma_mQ=18*1.35+6.3*1.5=33.75$ kN/m

Factored shear forces

At end B $V_b=w'*L/2=33.75*2.2/2=37.125$ kN
 At end A $V_a=w'*L-V_b=33.75*2.2-37.125=37.125$ kN
 Max shear is same at both ends $V_e=V_a=37.125$ kN

Factored moment

Maximum bending moment at centre $M=w \cdot L^2/8=33.75 \cdot 2.2^2/8$
=20.419 kNm
Corresponding shear force $V=0$ kN
Maximum moment $MyEd=M=20.419$ kNm
Maximum shear force $VzEd=Vb=37.125$ kN

Steel section properties

152 x 89 x 16 UKB
Dimensions (mm): $h=152.4$ $b=88.7$ $tw=4.5$ $tf=7.7$ $r=7.6$
Properties (cm): $Iy=834$ $Iz=89.8$ $Wply=123$ $Wplz=31.2$ $It=3.56$
 $A=20.3$ $iy=6.4097$ $iz=2.1032$

Strength of steel - Table 3.1 (amended by N.A.)

The material thickness is 7.7 mm and steel grade is S 275.
Yield strength $fy=275$ N/mm²
Ultimate strength $fu=410$ N/mm²
Young's modulus $E=210000$ N/mm²

Section classification - BS EN 1993-1-1:2005

Classify outstand element of compression flange:
Parameter (Table 5.2) $e=(235/fy)^{0.5}=(235/275)^{0.5}$
=0.92442
Outstand $c=(b-tw-2 \cdot r)/2=(88.7-4.5-2 \cdot 7.6)/2$
=34.5 mm
Ratio $c't=c/tf=34.5/7.7=4.4805$
As $c/tf \leq 9e$ ($4.4805 \leq 8.3197$), outstand element of compression flange is classified as Class 1 plastic.
Classify web element of section:
Depth between fillets $C=h-2 \cdot (tf+r)=152.4-2 \cdot (7.7+7.6)$
=121.8 mm
Ratio $C't=C/tw=121.8/4.5=27.067$
As $C/tw \leq 72e$ ($27.067 \leq 66.558$), web element in bending is classified as Class 1 plastic.

Moment resistance - Clause 6.2.5

Moment resistance $McRd=Wply \cdot fy/10^3=123 \cdot 275/10^3$
=33.825 kNm
Unity factor $unity=MyEd/McRd=20.419/33.825$
=0.60366
Section chosen is considered suitable.

Plastic shear resistance - Clause 6.2.6 & 6.2.8

In the absence of torsion the shear plastic resistance depends on:
Shear area $Avz=A \cdot 100-2 \cdot b \cdot tf+(tw+2 \cdot r) \cdot tf$
=20.3*100-2*88.7*7.7+(4.5+2*7.6)
*7.7
=815.71 mm²
Shear plastic resistance $VplRd=Avz \cdot fy/SQR(3)/1000$
=815.71*275/SQR(3)/1000
=129.51 kN

Unity factor $Unity = V_{zEd} / V_{plRd} = 37.125 / 129.51 = 0.28665$

Section chosen is considered suitable.

Note: No M-V interaction has to be considered as since the maximum moment occurs near the mid-span and the maximum shear force is obtained at the supports.

Shear buckling resistance- EN 1993-1-1, Clause 6.2.6(6)

Depth between the flanges $hw = h - 2 * tf = 152.4 - 2 * 7.7 = 137 \text{ mm}$
 Buckling ratio $hw/t = hw/tw = 137 / 4.5 = 30.444$
 As $hw/tw \leq 72e$ (66.558), no check for shear buckling is required.

Lateral torsional buckling

Shear modulus $G = 80770 \text{ N/mm}^2$
 Warping constant $I_w = I_z / 4 * ((h - tf) / 10)^2 = 89.8 / 4 * ((152.4 - 7.7) / 10)^2 = 4700.6 \text{ cm}^6$

Terms for critical moment M_{cr} :
 Euler term $E_t = \pi^2 * E * I_z / (L^2 * 100) = 3.1416^2 * 210000 * 89.8 / (2.2^2 * 100) = 384548 \text{ N}$

Load to shear centre distance $z_g = h / 2 = 152.4 / 2 = 76.2 \text{ mm}$
 Shear modulus term $G_t = L^2 * G * I_t / (\pi^2 * E * I_z) * 10^6 = 2.2^2 * 80770 * 3.56 / (3.1416^2 * 210000 * 89.8) * 10^6 = 7477.4 \text{ mm}^2$

Square root term $S_r = (I_w * 100 / I_z + G_t)^{0.5} = (4700.6 * 100 / 89.8 + 7477.4)^{0.5} = 112.75 \text{ mm}$

Coefficient C_1 $C_1 = 1.132$
 Critical moment $M_{cr} = C_1 * E_t * S_r / 10^6 = 1.132 * 384548 * 112.75 / 10^6 = 49.08 \text{ kNm}$

Non-dimensional slenderness $\lambda_{LT} = (W_{ply} * f_y / (M_{cr} * 10^3))^{0.5} = (123 * 275 / (49.08 * 10^3))^{0.5} = 0.83017$

Limiting slenderness value $\lambda_{LT0} = 0.4$
 Ratio $h'b = h / b = 152.4 / 88.7 = 1.7182$
 As $h/b \leq 2$, imperfection value $a_{LT} = 0.34$
 Factor $\phi_{LT} = 0.5 * (1 + a_{LT} * (\lambda_{LT} - \lambda_{LT0}) + 0.75 * \lambda_{LT}^2) = 0.5 * (1 + 0.34 * (0.83017 - 0.4) + 0.75 * 0.83017^2) = 0.83157$

Modification factor $\chi_{LT} = 1 / (\phi_{LT} + (\phi_{LT}^2 - 0.75 * \lambda_{LT}^2)^{0.5}) = 1 / (0.83157 + (0.83157^2 - 0.75 * 0.83017^2)^{0.5}) = 0.80035$

Factor $k_c = 1 / (C_1)^{0.5} = 1 / (1.132)^{0.5} = 0.93989$

Moment distribution factor $f = 1 - 0.5 * (1 - k_c) * (1 - 2 * (\lambda_{LT} - 0.8)^2) = 1 - 0.5 * (1 - 0.93989) * (1 - 2 * (0.83017 - 0.8)^2)$

Modified χ_{LT} factor $=0.97$
 $\chi_{LT} = \chi_{LT} / f = 0.80035 / 0.97$
 $= 0.8251$
Design buckling resistance moment $M_{bRd} = \chi_{LT} \cdot W_{ply} \cdot f_y / 10^3$
 $= 0.8251 \cdot 123 \cdot 275 / 10^3$
 $= 27.909 \text{ kNm}$
Unity factor $unitb = M_{yEd} / M_{bRd} = 20.419 / 27.909$
 $= 0.73162$
Section chosen is considered suitable.

Deflection

UDL for deflection calculation $w_u = w_d + w_i = 18 + 6.3 = 24.3 \text{ kN/m}$
Central UDL deflection $DEL = 5 \cdot w_u \cdot L^4 / (384 \cdot E \cdot I_y) \cdot 10^8$
 $= 5 \cdot 24.3 \cdot 2.2^4 / (384 \cdot 210000 \cdot 834) \cdot 10^8$
 $= 4.232 \text{ mm}$
Deflection to span ratio 1: 520

SECTION	152 x 89 x 16 UKB Grade S 275
DESIGN	Maximum moment 20.419 kNm
SUMMARY	Moment resistance 33.825 kNm
	Unity factor bending $0.60366 \leq 1$
	Maximum shear force 37.125 kN
	Shear plastic resistance 129.51 kN
	Unity factor shear $0.28665 \leq 1$
	Deflection to span ratio 1: 520
	Buckling resistance BM 27.909 kNm
	Unity factor buckling $0.73162 \leq 1$



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3

STRENGTHENING TO EXISTING BEAM OVER LOWER GROUND FLOOR:-

- BEAM CARRIES FLOOR OVER
- SPAN = 3.70 m
- UDL LOAD: (WITHOUT ADDITIONAL BEAM TO FRONT HALF OF DINING ROOM)

- UPPER GROUND FLOOR:

PERM. = 0.60 x 2.20

VAR. = 1.50 x 2.20

	DL	LL
=	1.32	
=		3.30
	1.32	3.30

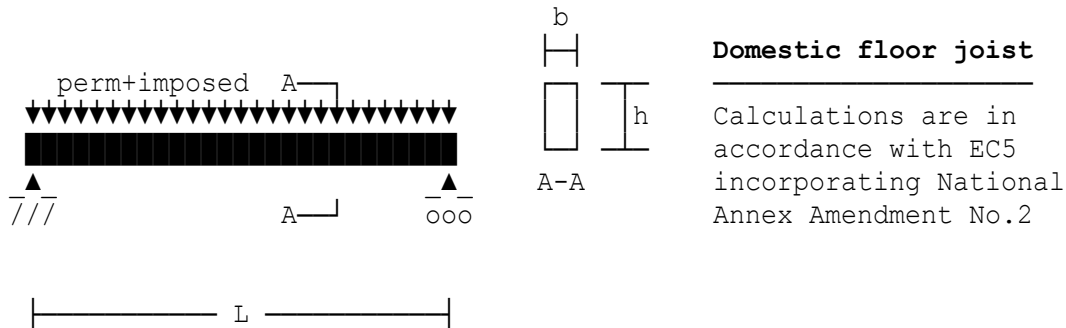
kN/m

(SEE CALCULATIONS THAT FOLLOW)

PROVIDE

- ADDITIONAL 100 x 50 PFC HOT ROLLED STEEL CHANNEL BOLTED TO EXISTING TIMBER BEAM OVER LOWER GROUND FLOOR SUPPORTING FLOOR OVER

Location: STRENGTHENING TO EXISTING BEAM OVER LOWEER GND FLR -
 EXTG FLR



The following assumptions are made in these calculations:

- that the timber has a moisture content of service class 1 or 2
- the floor can adequately distribute any concentrated point load to at least two joists
- the centres of joists do not exceed 600 mm
- that load sharing of the joists can occur.

Effective span of joist $L=4.4$ m
 Centres of joists $crs=450$ mm
 Permanent load including s.weight $G_k=0.15$ kN/m²
 Imposed udl load (on floor) $Q_k=0.375$ kN/m²
 Imposed point load (on one joist) $PL=0$ kN
 Depth of section $h=125$ mm
 Width of section $b=75$ mm
 Joist laterally restrained with support restraint.
 Bearing length $lb=50$ mm
 Timber strength class C16.
 Timber service class 2: Internal use, heated environment or similar with a max moisture content of 15%.

Characteristic strength properties - to BS EN 338:2016(E)

Bending parallel to grain $f_{mk}=16$ N/mm²
 Shear parallel to grain $f_{vk}=3.2$ N/mm²
 Compression perpend.to grain $f_{c90k}=2.2$ N/mm²
 Mean modulus of elasticity $E_{0mean}=8000$ N/mm²
 5th percentile mod of elasticity $E_{005}=5400$ N/mm²

Partial safety factors

Permanent actions factor $\gamma_G=1.35$
 Variable actions factor $\gamma_Q=1.5$
 Material partial factor $\gamma_M=2.0$ (package marked)

Design actions

Permanent duration load $F_{dperm}=\gamma_G \cdot G_k \cdot crs / 1000$
 $=1.35 \cdot 0.15 \cdot 450 / 1000$
 $=0.091125$ kN/m
 Medium-term duration load $F_{dmed}=(\gamma_G \cdot G_k + \gamma_Q \cdot Q_k) \cdot crs / 1000$
 $=(1.35 \cdot 0.15 + 1.5 \cdot 0.375) \cdot 450 / 1000$
 $=0.34425$ kN/m

The medium-term duration design load will produce the most critical case and will therefore be considered in the following calculations.

Modification factors

Creep deformation factor $k_{def}=0.8$
Medium-term duration factor $k_{mod}=0.8$
Depth factor (Clause 3.2) $k_h=(150/h)^{0.2}=(150/125)^{0.2}=1.0371$
Load sharing factor $k_{sys}=1.1$
Bearing modification factor $k_{c90}=1.5$
Notch factor for shear $k_v=1.0$
The shear strength parallel to the grain will be reduced by 33% to allow for possible drying splits and glue-line failure (refer to EC5 Part 1-1 Clause 6.1.7(2) for further clarification).
Shear reduction factor $k_{sh}=0.67$

Lateral stability factor - EC5 Part 1-1, Clause 6.3.3

As lateral displacement of the compressive edge is prevented throughout the length of the joists by the decking and torsional rotation is also prevented at the supports by adequate blocking then $k_{crit} = 1.0$
Lateral stability factor $k_{crit}=1.0$

Design strengths

Bending parallel to grain $f_{md}=k_h*k_{sys}*k_{mod}*k_{crit}*f_{mk}/\gamma_{M1}$
 $=1.0371*1.1*0.8*1*16/2$
 $=7.3014 \text{ N/mm}^2$
Bearing strength perp. to grain $f_{c90d}=k_{c90}*k_{sys}*k_{mod}*f_{c90k}/\gamma_{M1}$
 $=1.5*1.1*0.8*2.2/2$
 $=1.452 \text{ N/mm}^2$
Shear strength parallel to grain $f_{vd}=k_{sh}*k_v*k_{sys}*k_{mod}*f_{vk}/\gamma_{M1}$
 $=0.67*1*1.1*0.8*3.2/2$
 $=0.94336 \text{ N/mm}^2$

Design bending Strength

UDL loading case:
Design bending moment $M_1=F_{dmed}*L^2*10^6/8$
 $=0.34425*4.4^2*10^6/8$
 $=833085 \text{ Nmm}$

Point load loading case:
Bending moment from perm.load $M_2=F_{dperm}*L^2*10^6/8$
 $=0.091125*4.4^2*10^6/8$
 $=220523 \text{ Nmm}$
Bending moment from point load $M_3=PL*\gamma_{M1}*L*10^6/4=0*1.5*4.4*10^6/4$
 $=0 \text{ Nmm}$
Total design bending moment $M_4=M_2+M_3=220523+0=220523 \text{ Nmm}$
Worst case design moment $M_d=M_1=833085 \text{ Nmm}$
Section modulus about y-y $W_y=b*h^2/6=75*125^2/6=195313 \text{ mm}^3$
Design bending stress $s_{myd}=M_d/W_y=833085/195313$
 $=4.2654 \text{ N/mm}^2$
Since $s_{myd} < f_{md}$ ($4.2654 \text{ N/mm}^2 < 7.3014 \text{ N/mm}^2$) design bending stress is less than the design bending strength, therefore OK.

Deflections

For serviceability limit state the load factors for both permanent and variable loads is equal to 1.0.

Bending moment (permanent loads) $MG = 1.0 \cdot G_k \cdot c_{rs} / 1000 \cdot L^2 / 8$
 $= 1.0 \cdot 0.15 \cdot 450 / 1000 \cdot 4.4^2 / 8$
 $= 0.16335 \text{ kNm}$

Bending moment (variable loads) $MQ = 1.0 \cdot Q_k \cdot c_{rs} / 1000 \cdot L^2 / 8$
 $= 1.0 \cdot 0.375 \cdot 450 / 1000 \cdot 4.4^2 / 8$
 $= 0.40838 \text{ kNm}$

Second moment of area (y-y axis) $I_y = b \cdot h^3 / 12 = 75 \cdot 125^3 / 12 = 12.207E6 \text{ mm}^4$

(a) Instantaneous deflection: For bending and shear due to permanent action, G_k only;

$$u_{ip} = 5 \cdot G_k \cdot c_{rs} / 1000 \cdot (L \cdot 10^3)^4 / (384 \cdot E_{0mean} \cdot I_y) + 19.2 \cdot MG \cdot 10^6 / (b \cdot h \cdot E_{0mean})$$
$$= 5 \cdot 0.15 \cdot 450 / 1000 \cdot (4.4 \cdot 10^3)^4 / (384 \cdot 8000 \cdot 12.207E6) + 19.2 \cdot 0.16335 \cdot 10^6 / (75 \cdot 125 \cdot 8000)$$
$$= 3.4151 \text{ mm}$$

Instantaneous deflection: For bending and shear due to UDL variable action only;

$$u_{iv1} = 5 \cdot Q_k \cdot c_{rs} / 1000 \cdot (L \cdot 10^3)^4 / (384 \cdot E_{0mean} \cdot I_y) + 19.2 \cdot MQ \cdot 10^6 / (b \cdot h \cdot E_{0mean})$$
$$= 5 \cdot 0.375 \cdot 450 / 1000 \cdot (4.4 \cdot 10^3)^4 / (384 \cdot 8000 \cdot 12.207E6) + 19.2 \cdot 0.40838 \cdot 10^6 / (75 \cdot 125 \cdot 8000)$$
$$= 8.5378 \text{ mm}$$

For bending and shear due to the Point Load variable action only;

$$u_{iv2} = PL \cdot 1000 \cdot ((L \cdot 1000)^3 / (48 \cdot E_{0mean} \cdot I_y) + (1.5 \cdot L \cdot 1000) / (5 \cdot E_{0mean} / 16 \cdot b \cdot h))$$
$$= 0 \cdot 1000 \cdot ((4.4 \cdot 1000)^3 / (48 \cdot 8000 \cdot 12.207E6) + (1.5 \cdot 4.4 \cdot 1000) / (5 \cdot 8000 / 16 \cdot 75 \cdot 125))$$
$$= 0 \text{ mm}$$

Variable action due to UDL governs and will be considered below.

Instantaneous deflection $u_{iv} = u_{iv1} = 8.5378 \text{ mm}$

(b) Final deflection: The following deflection takes into account bending, shear and creep effects due to permanent and variable actions.

Final deflection $u_{cf} = u_{ip} \cdot (1 + k_{def}) + u_{iv} \cdot (1 + \psi_2 \cdot k_{def})$
 $= 3.4151 \cdot (1 + 0.8) + 8.5378 \cdot (1 + 0.3 \cdot 0.8)$
 $= 16.734 \text{ mm}$

Allowable combined final deflection due to all actions is $L/250 = 17.6 \text{ mm}$

Since $u_{cf} \leq u_{acf}$ ($16.734 \text{ mm} \leq 17.6 \text{ mm}$) the combined final deflection due to all variable actions is less than the allowable.

Floor vibrations

Floor vibrations are beyond the scope of this proforma. EC5 Clause 7.3 requires floor vibrations to be controlled to an acceptable level and it provides a design method for dwellings.

Design Shear and bearing strengths

UDL case:

Design shear due to UDL load $V_{d1} = F_{dmed} \cdot L / 2 = 0.34425 \cdot 4.4 / 2$
 $= 0.75735 \text{ kN}$

Point load case:

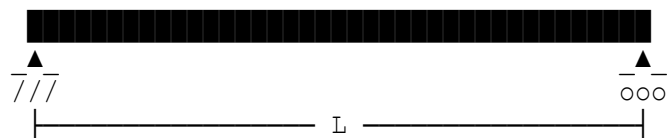
Design shear due to point load $V_{d2} = F_{dperm} \cdot L / 2 + PL \cdot \gamma_{mQ}$
 $= 0.091125 \cdot 4.4 / 2 + 0 \cdot 1.5$
 $= 0.20048 \text{ kN}$

Governing design shear is due to UDL $V_d = V_{d1} = 0.75735 \text{ kN}$
Design shear stress $s_{vd} = V_d * 1000 / (0.67 * b * h)$
 $= 0.75735 * 1000 / (0.67 * 75 * 125)$
 $= 0.12057 \text{ N/mm}^2$
Since $s_{vd} \leq f_{vd}$ ($0.12057 \text{ N/mm}^2 \leq 0.94336 \text{ N/mm}^2$) the design shear stress does not exceed the design shear strength, therefore OK.
Design bearing stress on support $s_{c90d} = V_d * 1000 / (l_b * b)$
 $= 0.75735 * 1000 / (50 * 75)$
 $= 0.20196 \text{ N/mm}^2$
Since $s_{c90d} \leq f_{c90d}$ ($0.20196 \text{ N/mm}^2 \leq 1.452 \text{ N/mm}^2$) design bearing stress does not exceed the design bearing strength, therefore OK.

	Joists: 125 mm x 75 mm @ 450 mm crs
	Timber strength class C16.
	Bending stress 4.2654 N/mm²
	Design bending strength 7.3014 N/mm²
DESIGN	Combined total deflection 16.734 mm
SUMMARY	Allowable deflection 17.6 mm
	Shear stress 0.12057 N/mm²
	Design shear strength 0.94336 N/mm²
	Bearing stress 0.20196 N/mm²
	Design bearing strength 1.452 N/mm²

Location: STRENGTHENING OF EXISTING BEAM OVER LOWER GND FLR - EXTG BEAM

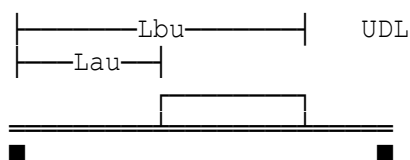
Timber beam to EC5 - incorporating national amendment No.2



Simply supported beam subject to vertical loads.

Beam span

$L=3.7$ m



Distances are measured from left hand support.

Uniformly distributed load 1 of 1

Dist. from left support to start $L_{au}(1)=0$ m

Distance from left support to end $L_{bu}(1)=3.7$ m

Permanent load (unfactored) $G_{ku}(1)=0.075$ kN/m

Variable load 1 (unfactored) $Q_{ua}(1)=0.188$ kN/m

Characteristic values of actions

Permanent:

Reaction at left hand end 0.13875 kN

Reaction at right hand end 0.13875 kN

Maximum span moment $M_{kp}=0.12834$ kNm

Maximum shear force $F_{kp}=0.13875$ kN

Variable 1:

Reaction at left hand end 0.3478 kN

Reaction at right hand end 0.3478 kN

Maximum span moment $M_{kv1}=0.32172$ kNm

Maximum shear force $F_{kv1}=0.3478$ kN

Partial safety factors

Permanent actions factor $\gamma_G=1.35$

Variable actions factor $\gamma_Q=1.5$

Material partial factor γ_M $\gamma_M=2.0$ (package marked)

Design values of combined actions for ULS

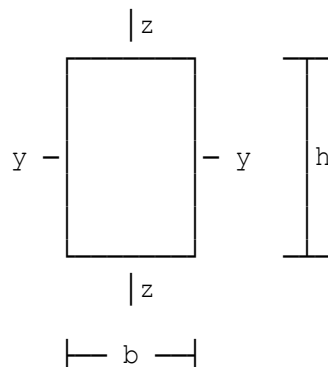
Design bending moment $M_d=\gamma_G M_{kp}+\gamma_Q M_{kv1}$
 $=1.35 \times 0.12834+1.5 \times 0.32172$
 $=0.65584$ kNm

Design shear force $V_d=\gamma_G F_{kp}+\gamma_Q F_{kv1}$
 $=1.35 \times 0.13875+1.5 \times 0.3478$
 $=0.70901$ kN

Design axial load (+ve compress) Nd=0 kN

Note: The combination load factor " ψ_0 " will not be used as there is only one variable action present.

Section design parameters



Depth of section h=100 mm
 Width of section b=100 mm
 Eff.length for bending about yy ley=3700 mm
 Eff.length for bending about zz lez=450 mm
 Length of bearing lb=50 mm
 Timber strength class C16.

Section properties

Inertia about yy axis $I_y = b \cdot h^3 / 12 = 100 \cdot 100^3 / 12 = 8.3333E6 \text{ mm}^4$
 Inertia about zz axis $I_z = h \cdot b^3 / 12 = 100 \cdot 100^3 / 12 = 8.3333E6 \text{ mm}^4$
 Area of cross section $A = b \cdot h = 100 \cdot 100 = 10000 \text{ mm}^2$
 Radius of gyration about yy axis $i_y = \sqrt{I_y / A} = \sqrt{8.3333E6 / 10000} = 28.868 \text{ mm}$
 Radius of gyration about zz axis $i_z = \sqrt{I_z / A} = \sqrt{8.3333E6 / 10000} = 28.868 \text{ mm}$

Slenderness

Slenderness ratio about yy axis $\lambda_{bdy} = l_{ey} / i_y = 3700 / 28.868 = 128.17$
 Slenderness ratio about zz axis $\lambda_{bdz} = l_{ez} / i_z = 450 / 28.868 = 15.588$
 Controlling slenderness $\lambda = \lambda_{bdy} = 128.17$

Characteristic strength properties - to BS EN 338:2016(E)

Compression parallel to grain $f_{c0k} = 17 \text{ N/mm}^2$
 Bending parallel to grain $f_{mk} = 16 \text{ N/mm}^2$
 Shear parallel to grain $f_{vk} = 3.2 \text{ N/mm}^2$
 Compression perp.to grain $f_{c90k} = 2.2 \text{ N/mm}^2$
 Mean modulus of elasticity $E_{0mean} = 8000 \text{ N/mm}^2$
 5th percentile MOE $E_{005} = 5400 \text{ N/mm}^2$

Modification factors

Timber service class 2: Internal use, heated environment or similar with a max moisture content of 15%.

Creep deformation factor $k_{def}=0.8$
Medium-term duration factor $k_{mod}=0.8$
Depth factor (Clause 3.2) $k_h=(150/h)^{0.2}=(150/100)^{0.2}=1.0845$

Member is not in a load-sharing system as defined by Clause 6.6.

Load-sharing modification factor $k_{sys}=1.0$

No notches exist at the support $k_v=1.0$

Bearing modification factor $k_{c90}=1.5$

The shear strength parallel to the grain will be reduced by 33% to allow for possible drying splits and glue-line failure (refer to EC5 Part 1-1 Clause 6.1.7(2) for further clarification).

Shear reduction factor $k_{sh}=0.67$

Lateral stability factor - EC5 Part 1-1 Clause 6.3.3

As lateral displacement of the compressive edge is prevented throughout the length of the member and torsional rotation is also prevented at the supports then $k_{crit} = 1.0$.

Lateral stability factor $k_{crit}=1.0$

Design strengths

Design bending strength $f_{md}=k_{mod}*k_h*k_{sys}*k_{crit}*f_{mk}/\gamma_{mM}$
 $=0.8*1.0845*1*1*16/2$
 $=6.9406 \text{ N/mm}^2$

Design shear strength // grain $f_{vd}=k_{sh}*k_{mod}*k_v*k_{sys}*f_{vk}/\gamma_{mM}$
 $=0.67*0.8*1*1*3.2/2$
 $=0.8576 \text{ N/mm}^2$

Design compr strength $\perp$ grain $f_{c90d}=k_{mod}*k_{c90}*k_{sys}*f_{c90k}/\gamma_{mM}$
 $=0.8*1.5*1*2.2/2$
 $=1.32 \text{ N/mm}^2$

Design bending strength

Design bending stress $s_{md}=M_d*1E6*(h/2)/I_y$
 $=0.65584*1E6*(100/2)/8.3333E6$
 $=3.935 \text{ N/mm}^2$

Since $s_{md} \leq f_{md}$ ($3.935 \text{ N/mm}^2 \leq 6.9406 \text{ N/mm}^2$) design bending stress within the design bending strength.

Check deflection - with shear and creep effects considered

Mean value of modulus of elasticity $E_{0mean}=8000 \text{ N/mm}^2$

Deflection based on $E_{0mean}=8000 \text{ N/mm}^2$

$d_{ld}=def_p * E' * I' * 10^{12} / (E_{0mean} * I_y)$
 $=0.31236E-3 * 9E6 * 65.104E-6 * 10^{12} / (8000 * 8.3333E6)$
 $=2.7454$

$i_{ld}=def_{v1} * E' * I' * 10^{12} / (E_{0mean} * I_y)$
 $=0.78298E-3 * 9E6 * 65.104E-6 * 10^{12} / (8000 * 8.3333E6)$
 $=6.8817$

Permanent load deflec (no shear) $dld=dld=2.7454$ mm
Variable load deflec (no shear) $ild=ild=6.8817$ mm
Total perm and variab deflection 9.627 mm
Modulus of rigidity $G=E0mean/16=8000/16=500$ N/mm²
Shape factor for rect section $KF=1.2$
Shear area for beam $Ay=h*b/KF=100*100/1.2=8333.3$ mm²
Total permanent and variable $WT'=WT'p+WT'v1=0.2775+0.6956$
 $=0.9731$ kN

If the total permanent & variable loads are assumed to be applied as a UDL, then the additional shear deflection is as follows:
Deflection due to shear $dsu=WT'*L*10^6/(8*Ay*G)$
 $=0.9731*3.7*10^6/(8*8333.3*500)$
 $=0.10801$ mm

Shear deflection $dels=dsu*(Mkp+Mkv1)/(WT'*L/8)$
 $=0.10801*(0.12834+0.32172)$
 $/(0.9731*3.7/8)$
 $=0.10801$ mm

Limiting deflection $DELlim=L/250*10^3=3.7/250*10^3$
 $=14.8$ mm

Final deflection (inclusive of shear and creep effects)
 $DEL=dld*(1+kdef)+ild*(1+psi2*kdef)+dels*(1+kdef)$
 $=2.7454*(1+0.8)+6.8817*(1+0.3*0.8)+0.10801*(1+0.8)$
 $=13.669$ mm

Since $DEL \leq DELlim$ (13.669 mm $\leq$ 14.8 mm), OK for deflection.

Shear and bearing strengths

Shear stress (no notches) $svd=Vd*1000/(0.67*b*h)$
 $=0.70901*1000/(0.67*100*100)$
 $=0.10582$ N/mm²

Since $svd \leq fvd$ (0.10582 N/mm² $\leq$ 0.8576 N/mm²) shear stress does not exceed the deign shear strength, therefore OK.

Bearing stress on wall plate $sc90d=Vd*1000/(lb*b)$
 $=0.70901*1000/(50*100)$
 $=0.1418$ N/mm²

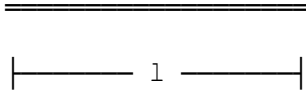
As $sc90d \leq fc90d$ (0.1418 N/mm² $\leq$ 1.32 N/mm²), bearing stress does not exceed the design bearing strength, therefore OK.

DESIGN SUMMARY	Member: 100 mm x 100 mm	
	Timber strength class C16.	
	Moisture service class	2
	Design bending stress	3.935 N/mm²
	Design bending strength	6.9406 N/mm²
	Deflection	13.669 mm
	Limiting deflection	14.8 mm
	Design shear stress	0.10582 N/mm²
	Design shear strength	0.8576 N/mm²
	Design bearing stress	0.1418 N/mm²
	Design bearing strength	1.32 N/mm²

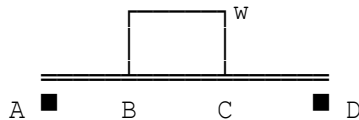
Location: STRENGTHENING TO EXISTING BEAM OVER LOWER GND FLR - EXTG
 ANGLE

Shear, moment and deflection formulas for beams generally taken from
 'Formulas for Stress and Strain' by Roark, published by McGraw-Hill
 with additions from 'Steel Designer's Manual'.

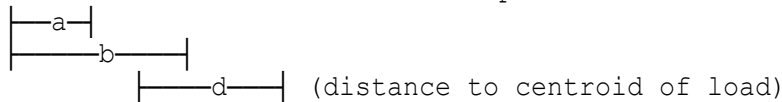
Inertia of beam $I=1.04\text{E-}6 \text{ m}^4$
 Elastic modulus for beam $E=205\text{E}6 \text{ kN/m}^2$



Length of beam $l=3.70 \text{ m}$

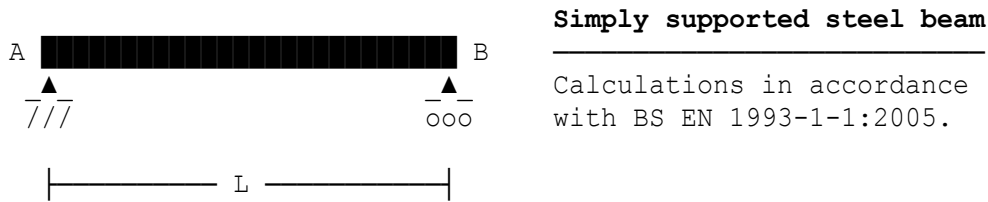


M (moment) is positive when clockwise;
 V (shear) is positive when upward;
 y (deflection) is positive when upward;
 w is positive when downward.



Distance from A to B $a=0 \text{ m}$
 Distance from A to C $b=3.7 \text{ m}$
 Magnitude of UDL $w=1.15 \text{ kN/m}$
 Length over which load applied $c=b-a=3.7-0=3.7 \text{ m}$
 Distance from D to cent of load $d=l-b/2-a/2=3.7-3.7/2-0/2=1.85 \text{ m}$
 Total load $W=w*c=1.15*3.7=4.255 \text{ kN}$
 Reaction at A $R1=W*d/l=4.255*1.85/3.7=2.1275 \text{ kN}$
 Reaction at D $R2=W*(a+c/2)/l=4.255*(0+3.7/2)/3.7=2.1275 \text{ kN}$
 Maximum bending moment $M=W*(d/l)*(a+c*d/(2*l))=4.255*(1.85/3.7)*(0+3.7*1.85/(2*3.7))=1.9679 \text{ kNm}$
 Distance to centre $x=l/2=3.7/2=1.85 \text{ m}$
 Deflection constant $k1=1/(48*E*I)=1/(48*205\text{E}6*1.04\text{E-}6)=97.717\text{E-}6$
 Deflection constant $k2=8*R1*(x^3-l^2*x)=8*2.1275*(1.85^3-3.7^2*1.85)=-323.29$
 Deflection constant $k3=(8*d^3-2*b*c^2+c^3)/l=(8*1.85^3-2*3.7*3.7^2+3.7^3)/3.7=11.522\text{E-}15 \text{ m}^2$
 Central deflection $y=k1*(k2+W*x*(k3+2*c^2)-2*W*(x-a)^4/c)=97.717\text{E-}6*(-323.29+4.255*1.85*(11.522\text{E-}15+2*3.7^2)-2*4.255*(1.85-0)^4/3.7)=-0.013163 \text{ m}$

Location: STRENGTHENING TO EXISTING BEAM OVER LOWER GND FLR - NEW



Beam span $L=3.70$ m

Steel section properties

100 x 50 UKPFC
 Dimensions (mm): $h=100$ $b=50$ $t_w=5$ $t_f=8.5$ $r=9$
 Properties (cm): $I_y=208$ $I_z=32.3$ $W_{ply}=48.9$ $W_{plz}=17.5$ $I_t=2.53$
 $A=13$ $C_y=1.7341$ $I_w=0.491E-3$ $i_y=4$ $i_z=1.5763$

Strength of steel - Table 3.1 (amended by N.A.)

The material thickness is 8.5 mm and steel grade is S 275.
 Yield strength $f_y=275$ N/mm²
 Ultimate strength $f_u=410$ N/mm²
 Young's modulus $E=210000$ N/mm²
 Shear modulus $G=81000$ N/mm²

Loading (unfactored)

Permanent UDL (including S.W) $w_d=0.76$ kN/m
 Variable UDL $w_i=1.65$ kN/m

Unfactored loads

Distributed load $w'=w_d+w_i=0.76+1.65=2.41$ kN/m

Unfactored shear forces

At end B $V_b'=w'*L/2=2.41*3.7/2=4.4585$ kN
 At end A $V_a'=w'*L-V_b'=2.41*3.7-4.4585=4.4585$ kN

Factored loads

Distributed load $w'=w_d*\gamma_G+w_i*\gamma_Q=0.76*1.35+1.65*1.5=3.501$ kN/m

Factored shear forces

At end B $V_b=w'*L/2=3.501*3.7/2=6.4769$ kN
 At end A $V_a=w'*L-V_b=3.501*3.7-6.4769=6.4769$ kN
 Max shear is same at both ends $V_e=V_a=6.4769$ kN

Factored moment

Maximum bending moment at centre $M=w'L^2/8=3.501*3.7^2/8$
=5.9911 kNm
Corresponding shear force $V=0$ kN
Maximum moment $MyEd=M=5.9911$ kNm
Maximum shear force $VzEd=Vb=6.4769$ kN

Deflection

UDL for deflection calculation $wu=wd+wi=0.76+1.65=2.41$ kN/m
Central UDL deflection $DEL=5*wu*L^4/(384*E*Iy)*10^8$
=5*2.41*3.7^4/(384*210000*208)*10^8
=13.464 mm
Deflection to span ratio 1: 275
Limiting deflection (brittle) $DELlim=L*1000/360=3.7*1000/360$
=10.278 mm

Section classification - BS EN 1993-1-1:2005

Classify outstand element of compression flange:
Parameter (Table 5.2) $e=(235/fy)^{0.5}=(235/275)^{0.5}$
=0.92442
Outstand $c=b-tw-r=50-5-9=36$ mm
Ratio $c't=c/tf=36/8.5=4.2353$
As $c/tf \leq 9e$ ($4.2353 \leq 8.3197$), outstand element of compression flange is classified as Class 1 plastic.
Part subject to bending (web):
Depth between fillets $C=h-2*(tf+r)=100-2*(8.5+9)$
=65 mm
Ratio $C't=C/tw=65/5=13$
As $C/tw \leq 72e$ ($13 \leq 66.558$), web element in compression is classified as Class 1 plastic.
Part subject to compression (flange):
Depth between fillets $C=b-(tw+r)=50-(5+9)=36$ mm
Ratio $C't=C/tf=36/8.5=4.2353$
As $C/tf \leq 33e$ ($4.2353 \leq 30.506$), web element in compression is classified as Class 1 plastic.
Hence, the overall section classification is Class 1.

Moment resistance - Clause 6.2.5

Moment resistance $McRd=Wply*fy/10^3=48.9*275/10^3$
=13.448 kNm
Unity factor $unity=MyEd/McRd=5.9911/13.448$
=0.44552
Section chosen is considered suitable.

Plastic shear resistance - Clause 6.2.6 & 6.2.8

In the absence of torsion the shear plastic resistance depends on:
Shear area $Avz=A*100-2*b*tf+(tw+r)*tf$
=13*100-2*50*8.5+(5+9)*8.5
=569 mm²
Shear plastic resistance $VplRd=Avz*fy/SQR(3)/1000$
=569*275/SQR(3)/1000
=90.341 kN

Unity factor $Unity = V_z E_d / V_{pl} R_d = 6.4769 / 90.341 = 0.071693$

Section chosen is considered suitable.

Note: No M-V interaction has to be considered as since the maximum moment occurs near the mid-span and the maximum shear force is obtained at the supports.

Shear buckling resistance- EN 1993-1-1, Clause 6.2.6(6)

Depth between the flanges $hw = h - 2 * tf = 100 - 2 * 8.5 = 83 \text{ mm}$
 Buckling ratio $hw' t = hw / tw = 83 / 5 = 16.6$
 As $hw / tw \leq 72e$ (66.558), no check for shear buckling is required.

Coefficient C1

Eff.length between restraints $LT = 4.64 \text{ m}$
 Coefficient C1 $C1 = 1.13$

Lateral torsional buckling

Warping constant $I_w = 491 \text{ cm}^6$
 Terms for Critical moment M_{cr}:
 Euler term $Et = \pi^2 * E * I_z / (LT^2 * 100) = 3.1416^2 * 210000 * 32.3 / (4.64^2 * 100) = 31095 \text{ N}$
 Load to shear centre distance $zg = h / 2 = 100 / 2 = 50 \text{ mm}$
 Shear modulus term $Gt = LT^2 * G * It / (\pi^2 * E * I_z) * 10^6 = 4.64^2 * 81000 * 2.53 / (3.1416^2 * 210000 * 32.3) * 10^6 = 65905 \text{ mm}^2$
 Square root term $Sr = (I_w * 100 / I_z + Gt)^{0.5} = (491 * 100 / 32.3 + 65905)^{0.5} = 259.66 \text{ mm}$
 Critical moment $M_{cr} = C1 * Et * Sr / 10^6 = 1.13 * 31095 * 259.66 / 10^6 = 9.1238 \text{ kNm}$
 Non-dimensional slenderness $\lambda_{mLT} = (W_{ply} * f_y / (M_{cr} * 10^3))^{0.5} = (48.9 * 275 / (9.1238 * 10^3))^{0.5} = 1.214$
 Limiting slenderness value $\lambda_{mLT0} = 0.4$
 Imperfection factor $a_{LT} = 0.76$ (curve d)
 compute $comp1 = a_{LT} * (\lambda_{mLT} - \lambda_{mLT0}) = 0.76 * (1.214 - 0.4) = 0.61867$
 compute $comp2 = 0.75 * \lambda_{mLT}^2 = 0.75 * 1.214^2 = 1.1054$
 Factor $\phi_{LT} = 0.5 * (1 + comp1 + comp2) = 0.5 * (1 + 0.61867 + 1.1054) = 1.362$
 Modification factor $\chi_{LT} = 1 / (\phi_{LT} + (\phi_{LT}^2 - 0.75 * \lambda_{mLT}^2)^{0.5}) = 1 / (1.362 + (1.362^2 - 0.75 * 1.214^2)^{0.5}) = 0.44885$

Moment distribution factor $f=1.0$
Modified χ_{LT} factor $\chi_{LT} = \chi_{LT}/f = 0.44885/1 = 0.44885$
Design buckling resistance moment $M_{bRd} = \chi_{LT} \cdot W_{ply} \cdot f_y / 10^3$
 $= 0.44885 \cdot 48.9 \cdot 275 / 10^3$
 $= 6.0359 \text{ kNm}$
Unity factor $unitb = M_{yEd} / M_{bRd} = 5.9911 / 6.0359$
 $= 0.99258$
Section chosen is suitable.

**PARALLEL FLANGE CHANNEL
DESIGN SUMMARY**

100 x 50 Grade S 275
Maximum shear force 6.4769 kN
Shear plastic resist. 90.341 kN
Design moment 5.9911 kNm
Moment resistance 13.448 kNm
Buckling resistance 6.0359 kNm
Central deflection 13.464 mm
Limiting deflection 10.278 mm

Factored end shears	[	Factored shear at A	6.4769 kN
		Factored shear at B	6.4769 kN



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Date :

25/08/2026

4

NEW BEAM OVER LOWER GROUND FLOOR:-

- BEAM CARRIES FLOOR OVER
- SPAN = 3.70 m
- UDL LOAD: (WITHOUT STRENGTHENED BEAM TOWARDS REAR OF DINING ROOM)

- UPPER GROUND FLOOR:

PERM. = 0.60 x 2.20

VAR. = 1.50 x 2.20

	DL	LL
=	1.32	
=		3.30
	1.32	3.30

kN/m

(SEE CALCULATIONS THAT FOLLOW)

PROVIDE

- NEW 100 SHS 10.0 HOT ROLLED STEEL BEAM
OVER FRONT OF LOWER GROUND FLOOR
SUPPORTING FLOOR OVER

Factored moment

Maximum bending moment at centre $M=w'*L^2/8=7.137*3.7^2/8$
=12.213 kNm
Corresponding shear force $V=0$ kN
Maximum moment $MyEd=M=12.213$ kNm
Maximum shear force $VzEd=Vb=13.203$ kN

Steel section properties

100 x 100 x 10 SHS - Hot finished.
Properties (cm): $A=34.9$ $iy=3.64$ $Wely=92.4$ $Wply=116$ $Iy=462$ $It=761$
 $Wt=133$

Strength of steel - Table 3.1 (amended by N.A.)

The material thickness is 10 mm and steel grade is S 275.
Yield strength $fy=275$ N/mm²
Ultimate strength $fu=410$ N/mm²
Young's modulus $E=210000$ N/mm²

Section classification - BS EN 1993-1-1:2005

Parameter (Table 5.2) $e=(235/fy)^{0.5}=(235/275)^{0.5}$
=0.92442
Depth between fillets $C=b-3*t=100-3*10=70$ mm
Ratio $C't=C/t=70/10=7$
Internal part subject to compression (flange):
As $C/t \leq 33e$ ($7 \leq 30.506$), internal element in compression
is classified as Class 1 plastic.
Internal part subject to bending (web):
Depth between fillets $C=h-3*t=100-3*10=70$ mm
Ratio $C't=C/t=70/10=7$
As $C/t \leq 72e$ ($7 \leq 66.558$), web element in bending
is classified as Class 1 plastic.
Section classification is therefore Class 1.

Moment resistance

Moment resistance $McRd=Wply*fy/10^3=116*275/10^3$
=31.9 kNm
Unity factor $unity=MyEd/McRd=12.213/31.9$
=0.38286
Section chosen is suitable.

Shear plastic resistance

In the absence of torsion the shear plastic resistance depends on:
Shear Area $Avz=A*100*h/(b+h)$
=34.9*100*100/(100+100)
=1745 mm²
Shear plastic resistance $VplRd=Avz*fy/SQR(3)/1000$
=1745*275/SQR(3)/1000
=277.06 kN

Unity factor $Unity = V_{zEd} / V_{plRd} = 13.203 / 277.06 = 0.047656$

Section chosen is suitable.

Note: No M-V interaction has to be considered as since the maximum moment occurs near the mid-span and the maximum shear force is obtained at the supports.

Coefficient C1

The beam is simply supported and is not restrained at point loads.

Effective length parameter $K = 1.0$
Length for LTB $LT = 3.7 \text{ m}$
Coefficient $C1 = 1.13$ (based on UDL)

Lateral torsional buckling

Shear modulus $G = 80770 \text{ N/mm}^2$
Terms for Critical moment M_{cr} :
Euler term $E_t = \frac{\pi^2 E I_z}{(K \cdot LT)^2 \cdot 100} = \frac{3.1416^2 \cdot 210000 \cdot 462}{(1 \cdot 3.7)^2 \cdot 100} = 699451 \text{ N}$
Shear modulus term $G_t = \frac{(K \cdot LT)^2 \cdot G \cdot I_t}{(\pi^2 E I_z) \cdot 10^6} = \frac{(1 \cdot 3.7)^2 \cdot 80770 \cdot 761}{(3.1416^2 \cdot 210000 \cdot 462) \cdot 10^6} = 878774 \text{ mm}^2$
Critical moment $M_{cr} = C1 \cdot E_t \cdot G_t^{0.5} / 10^6 = 1.13 \cdot 699451 \cdot 878774^{0.5} / 10^6 = 740.93 \text{ kNm}$
Non-dimensional slenderness $\lambda_{LT} = (W_{ply} \cdot f_y / (M_{cr} \cdot 10^3))^{0.5} = (116 \cdot 275 / (740.93 \cdot 10^3))^{0.5} = 0.2075$
Limiting slenderness value $\lambda_{LT0} = 0.4$
As $\lambda_{LT} \leq \lambda_{LT0}$,
The segment of the beam is not subject to lateral torsional buckling.
Design buckling resistance moment $M_{bRd} = W_{ply} \cdot f_y / 10^3 = 116 \cdot 275 / 10^3 = 31.9 \text{ kNm}$
Unity factor $unitb = M_{yEd} / M_{bRd} = 12.213 / 31.9 = 0.38286$

Section chosen is suitable.

Deflection

UDL for deflection calculation $w_u = w_d + w_i = 1.62 + 3.3 = 4.92 \text{ kN/m}$
Central UDL deflection $DEL = \frac{5 \cdot w_u \cdot L^4}{(384 \cdot E \cdot I_y) \cdot 10^8} = \frac{5 \cdot 4.92 \cdot 3.7^4}{(384 \cdot 210000 \cdot 462) \cdot 10^8} = 12.375 \text{ mm}$
Deflection to span ratio 1: 299

HOT FINISHED
SQUARE HOLLOW SECTION
DESIGN
SUMMARY

In accordance with EN 10210
100 x 100 x 10 SHS Grade S 275
Maximum moment 12.213 kNm
Moment resistance 31.9 kNm
Unity factor bending $0.38286 \leq 1$
Maximum shear force 13.203 kN
Shear plastic resistance 277.06 kN
Unity factor shear $0.047656 \leq 1$
Deflection/span ratio 299
Buckling resistance 31.9 kNm

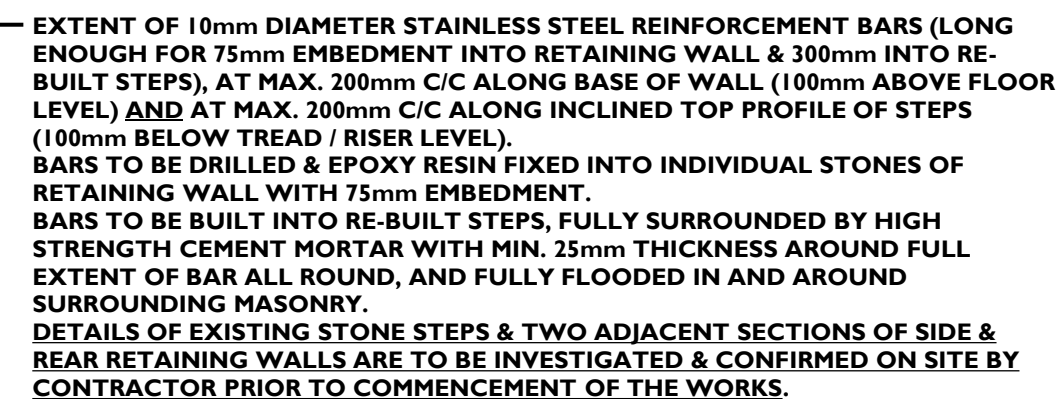


NOTES:

1. P1 DENOTES C20 CONCRETE PADSTONE; 215 dp x 440 LONG x THICKNESS OF WALL LEAF.
2. P2 DENOTES C20 CONCRETE PADSTONE; 215 dp x 300 LONG x THICKNESS OF WALL LEAF.
3. P3 DENOTES C20 CONCRETE PADSTONE; 215 dp x 215 LONG x THICKNESS OF WALL LEAF.
4. ALL SETTING OUT TO BE DETERMINED BY THE ARCHITECT AND CONFIRMED ON SITE.
5. BEAMS, AND LINTELS, TO HAVE MINIMUM 150mm LONG BEARING LENGTH ONTO PADSTONES / MASONRY AT ENDS, OR FULL THICKNESS OF WALL LEAF / SUPPORTING BEAM, WHICHEVER IN THE LESSER, UNLESS NOTED OR SHOWN OTHERWISE.
6. BEAMS IN PAIRS SIDE BY SIDE ARE TO BE BOLTED TOGETHER THROUGH THEIR WEBS WITH M16 BOLTS IN STEEL SPACER TUBES AT MAXIMUM 450mm CENTRES AND AT EACH END, UNLESS NOTED OTHERWISE.
7. ALL STEEL TO STEEL INTERSECTIONS / JUNCTIONS ARE TO BE JOINED TOGETHER WITH BOLTED CONNECTIONS WITH A MINIMUM OF 4No. M16 BOLTS PER CONNECTION WITH MIN 8mm THICK FULL DEPTH END PLATES WELDED TO SECTIONS WITH MIN. 4mm FULL PROFILE CONTINUOUS FILLET WELDS, DESIGNED AND DETAILED BY STEELWORK FABRICATOR.
8. ALL REQUIREMENTS FOR STEELWORK PROTECTIVE COATING, WATERPROOFING, DAMP PROOFING INCLUDING DMP & DPCs, ALL FIRE SAFETY & PROTECTION AND GAS PROTECTION REQUIREMENTS ARE TO BE IN LINE WITH DETAILS BY OTHERS.

CONTRACTOR TO DESIGN, DETAIL, INSTALL & MANAGE ALL TEMPORARY WORKS & PROPPING TO SECTIONS OF SIDE & REAR RETAINING WALLS BEHIND EXISTING STONE STEPS, PRIOR TO, DURING & FOLLOWING DISMANTLING AND RE-BUILDING OF STEPS. DETAILS OF EXISTING STONE STEPS & TWO ADJACENT SECTIONS OF SIDE & REAR RETAINING WALLS ARE TO BE INVESTIGATED & CONFIRMED ON SITE BY CONTRACTOR PRIOR TO COMMENCEMENT OF THE WORKS.

NO ASSESEMENT HAS BEEN MADE ON THE SUITABILITY OF ANY EXISTING OR PROPOSED FOUNDATIONS.



ALL TEMPORARY WORKS TO BE DESIGNED & DETAILED BY OTHERS, WHO HAVE THE NECESSARY SKILL, KNOWLEDGE, EXPERIENCE & TRAINING TO CARRY THIS OUT, IN ORDER TO MAINTAIN VERTICAL AND LATERAL SUPPORT & STABILITY AT ALL STAGES OF THE THE WORKS.

NOTES: (CONT. FROM ABOVE)

9. TIMBERS IN PAIRS, THREES OR MORE SIDE BY SIDE ARE TO BE SCREWED TOGETHER WITH 5mm DIAMETER SCREWS, FULLY THROUGH EACH TIMBER, AT MAXIMUM 200mm CENTRES, AND AT EACH END.

10. ANY AND ALL SLIDING & FOLDING DOORS TO HAVE THEIR WEIGHT SUPPORTED AT THEIR BASE AND NOT HUNG FROM THE BEAM / LINTEL AT DOOR HEAD.

11. ALL WORK TO BE CARRIED OUT IN LINE WITH LATEST VERSION OF BUILDING REGULATIONS APPROVED DOCUMENTS, UNLESS NOTED OTHERWISE.

STEEL & TIMBER BEAMS, SUPPORTING EXISTING STRUCTURE, HAVE BEEN DESIGNED FOR DEFLECTIONS TO BE WITHIN THE LIMITS SET OUT IN THE RELEVANT STANDARD DESIGN CODES. HOWEVER, THESE DEFLECTIONS, UPON TRANSFER OF LOADS FROM TEMPORARY PROPPING SUPPORTS, MAY RESULT IN INITIAL CRACKING WITHIN CEILINGS & WALLS, AND MAY ALSO CAUSE DOORS TO STICK OR PREVENT THEM FROM CLOSING. THIS PHENOMENON IS NOT IN ITSELF A SIGN OF STRUCTURAL FAILURE OR INSUFFICIENCY. THESE SETTLING CRACKS SHOULD NOT CONTINUE TO

**ALL REQUIREMENTS & DETAILS FOR MASONRY
MOVEMENT JOINTS ARE TO BE ASSESSED AND
DETERMINED BY ARCHITECT OR OTHERS.**

	Sht No. / Rev SK01
	Project No : 2186
Project : The Old School House, Hague Street, Glossop	Date : 27/08/2026

Drawing Title :

LOWER GROUND FLOOR PLAN

NOTES: (CONT. FROM ABOVE)

9. TIMBERS IN PAIRS, THREES OR MORE SIDE BY SIDE ARE TO BE SCREWED TOGETHER WITH 5mm DIAMETER SCREWS, FULLY THROUGH EACH TIMBER, AT MAXIMUM 200mm CENTRES, AND AT EACH END.

10. ANY AND ALL SLIDING & FOLDING DOORS TO HAVE THEIR WEIGHT SUPPORTED AT THEIR BASE AND NOT HUNG FROM THE BEAM / LINTEL AT DOOR HEAD.

11. ALL WORK TO BE CARRIED OUT IN LINE WITH LATEST VERSION OF BUILDING REGULATIONS APPROVED DOCUMENTS, UNLESS NOTED OTHERWISE.

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**ALL REQUIREMENTS & DETAILS FOR MASONRY
MOVEMENT JOINTS ARE TO BE ASSESSED AND
DETERMINED BY ARCHITECT OR OTHERS.**

**ALL NEW BLOCKWORK TO BE
CONSTRUCTED FROM SOLID
CONCRETE BLOCKS WITH MINIMUM
UNIT STRENGTH OF 7.3N/mm², UNLESS
NOTED OTHERWISE.**

ALL TEMPORARY WORKS TO BE DESIGNED & DETAILED BY OTHERS, WHO HAVE THE NECESSARY SKILL, KNOWLEDGE, EXPERIENCE & TRAINING TO CARRY THIS OUT, IN ORDER TO MAINTAIN VERTICAL AND LATERAL SUPPORT & STABILITY AT ALL STAGES OF THE THE WORKS.

(DO NOT SCALE FROM THIS SKETCH)



NOTES:

1. P1 DENOTES C20 CONCRETE PADSTONE; 215 dp x 440 LONG x THICKNESS OF WALL LEAF.
2. P2 DENOTES C20 CONCRETE PADSTONE; 215 dp x 300 LONG x THICKNESS OF WALL LEAF.
3. P3 DENOTES C20 CONCRETE PADSTONE; 215 dp x 215 LONG x THICKNESS OF WALL LEAF.
4. ALL SETTING OUT TO BE DETERMINED BY THE ARCHITECT AND CONFIRMED ON SITE.
5. BEAMS, AND LINTELS, TO HAVE MINIMUM 150mm LONG BEARING LENGTH ONTO PADSTONES / MASONRY AT ENDS, OR FULL THICKNESS OF WALL LEAF / SUPPORTING BEAM, WHICHEVER IN THE LESSER, UNLESS NOTED OR SHOWN OTHERWISE.
6. BEAMS IN PAIRS SIDE BY SIDE ARE TO BE BOLTED TOGETHER THROUGH THEIR WEBS WITH M16 BOLTS IN STEEL SPACER TUBES AT MAXIMUM 450mm CENTRES AND AT EACH END, UNLESS NOTED OTHERWISE.
7. ALL STEEL TO STEEL INTERSECTIONS / JUNCTIONS ARE TO BE JOINED TOGETHER WITH BOLTED CONNECTIONS WITH A MINIMUM OF 4No. M16 BOLTS PER CONNECTION WITH MIN 8mm THICK FULL DEPTH END PLATES WELDED TO SECTIONS WITH MIN. 4mm FULL PROFILE CONTINUOUS FILLET WELDS, DESIGNED AND DETAILED BY STEELWORK FABRICATOR.
8. ALL REQUIREMENTS FOR STEELWORK PROTECTIVE COATING, WATERPROOFING, DAMP PROOFING INCLUDING DMP & DPCs, ALL FIRE SAFETY & PROTECTION AND GAS PROTECTION REQUIREMENTS ARE TO BE IN LINE WITH DETAILS BY OTHERS.

ALL TEMPORARY WORKS TO BE DESIGNED & DETAILED BY OTHERS, WHO HAVE THE NECESSARY SKILL, KNOWLEDGE, EXPERIENCE & TRAINING TO CARRY THIS OUT, IN ORDER TO MAINTAIN VERTICAL AND LATERAL SUPPORT & STABILITY AT ALL STAGES OF THE THE WORKS.

JJ

JOHN JORDAN

CONSULTING

JOHN JORDAN

Chartered Structural Engineer

NOTES:
1. P1 DENOTES C20 CONCRETE PADSTONE; 215 dp x 440 LONG x THICKNESS OF WALL LEAF.
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Drawn :
JJ

Checked :

Project :
The Old School House, Hague Street, Glossop

Sht No. / Rev
SK03

Project No :
2186

Date :
29/09/2026

Drawing Title :

LOWER GROUND FLOOR PLAN - SHOWING BEAMS OVER

EXISTING PERIMETER & INTERNAL WALLS ARE ASSUMED TO BE CONSTRUCTED FROM SOLID DENSE CONCRETE BLOCKWORK OR BRICK THROUGHOUT. THIS IS TO BE INVERSTIGATED AND CONFIRMED ON SITE, BY OTHERS, PRIOR TO THE COMMENCEMENT OF THE WORKS. IF FOUND TO BE CONSTRUCTED FROM HOLLOW BLOCKS OR 'THERMALITE' TYPE, LOW STRENGTH BLOCKS, THEN FURTHER ADVICE IS TO BE SOUGHT FROM THE STRUCTURAL ENGINEER PRIOR TO COMMENCEMENT OF THE WORKS.

NOTES: (CONT. FROM ABOVE)
9. TIMBERS IN PAIRS, THREES OR MORE SIDE BY SIDE ARE TO BE SCREWED TOGETHER WITH 5mm DIAMETER SCREWS, FULLY THROUGH EACH TIMBER, AT MAXIMUM 200mm CENTRES, AND AT EACH END.
10. ANY AND ALL SLIDING & FOLDING DOORS TO HAVE THEIR WEIGHT SUPPORTED AT THEIR BASE AND NOT HUNG FROM THE BEAM / LINTEL AT DOOR HEAD.
11. ALL WORK TO BE CARRIED OUT IN LINE WITH LATEST VERSION OF BUILDING REGULATIONS APPROVED DOCUMENTS, UNLESS NOTED OTHERWISE.

STEEL & TIMBER BEAMS, SUPPORTING EXISTING STRUCTURE, HAVE BEEN DESIGNED FOR DEFLECTIONS TO BE WITHIN THE LIMITS SET OUT IN THE RELEVANT STANDARD DESIGN CODES. HOWEVER, THESE DEFLECTIONS, UPON TRANSFER OF LOADS FROM TEMPORARY PROPPING SUPPORTS, MAY RESULT IN INITIAL CRACKING WITHIN CEILINGS & WALLS, AND MAY ALSO CAUSE DOORS TO STICK OR PREVENT THEM FROM CLOSING. THIS PHENOMENON IS NOT IN ITSELF A SIGN OF STRUCTURAL FAILURE OR INSUFFICIENCY. THESE SETTLING CRACKS SHOULD NOT CONTINUE TO

ALL REQUIREMENTS & DETAILS FOR MASONRY MOVEMENT JOINTS ARE TO BE ASSESSED AND DETERMINED BY ARCHITECT OR OTHERS.

WALLS SUPPORTING BEAM ENDS, AND / OR ANY ADDITIONAL LOADING, ARE TO BE CHECKED ON SITE, BY OTHERS, TO VERIFY THAT THEY ARE OF LOAD BEARING MASONRY, OF EITHER SOLID DENSE CONCRETE BLOCK, SOLID CLAY BRICK OR GOOD QUALITY, HIGH STRENGTH BONDED NATURAL STONE MASONRY CONSTRUCTION, WHICH CONTINUES DOWN TO A SUPPORTING FOUNDATION.

NO ASSESEMENT HAS BEEN MADE ON THE SUITABILITY OF ANY EXISTING OR PROPOSED FOUNDATIONS.

(DO NOT SCALE FROM THIS SKETCH)

ALL NEW BLOCKWORK TO BE CONSTRUCTED FROM SOLID CONCRETE BLOCKS WITH MINIMUM UNIT STRENGTH OF 7.3N/mm², UNLESS

ALL TEMPORARY WORKS TO BE DESIGNED & DETAILED BY OTHERS, WHO HAVE THE NECESSARY SKILL, KNOWLEDGE, EXPERIENCE & TRAINING TO CARRY THIS OUT, IN ORDER TO MAINTAIN VERTICAL AND LATERAL SUPPORT & STABILITY AT ALL STAGES OF THE THE WORKS.

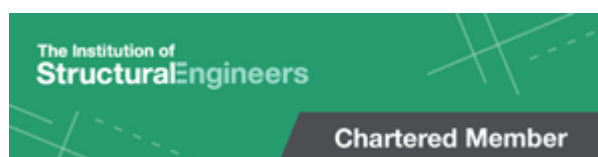
102 Chantry Road, Disley, Stockport, Cheshire SK12 2DN - 07804 679 386 john@jordan-consulting.co.uk www.jordan-consulting.co.uk

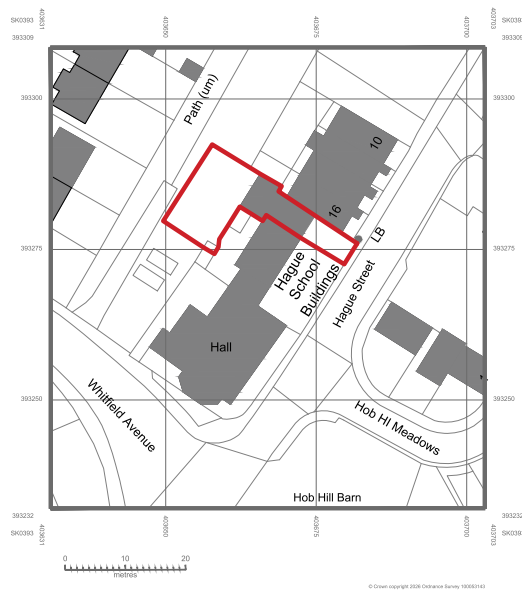
THE OLD SCHOOL HOUSE, HAGUE STREET,
GLOSSOP

STRUCTURAL CALCULATIONS

Appendix

Architect's Layouts





LOCATION PLAN



SWOFO
ARCHITECTS

07884183625
studio@swofa.co.uk
www.swofa.co.uk

REV	DATE		A_OSH_L_100	A
A	02.03.2026	A - Initial Issue	LOCATION PLAN	
			Old School House	1:1250
			Hague Street, Gloosop, Derbyshire, SK13 8NR	@ A4
			Nathy Connolly	

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REVISION	DATE	NOTES
A	02.03.2026	A - Planning Issue
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EXISTING



A_OSH_S_200			
SITE PLAN			
Old School House			
Hague Street Glossop SK13 8NR			
Nathy Connolly			
A	1:200 @ A3		

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REVISION	DATE	NOTES
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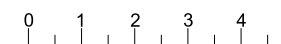
CAUTION NOTES

CAUTION

See notes on drawings and designers risk assessment. This as been developed to reflect the scope of this project only.

- 1 - Asbestos. No survey carried out. Survey to be completed by others.
- 2 - Stability of structure to be reviewed by structural engineer. Given the condition there are no concerns beyond a normal habitable house.
- 3 - A histroic house may conceal hidden hazards such as lead paint, lime plaster with animal hair.
- 4 - Existing remedial work has not been reviewed, but assumed to be signed of by qualified engineer or building control officer.

RISK



A_OSH_E_150

RISK PLAN PLAN

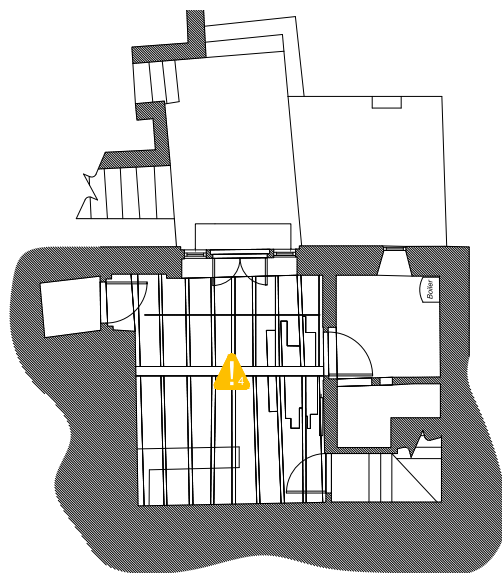
Old School House

Hague Street Glossop SK13 8NR

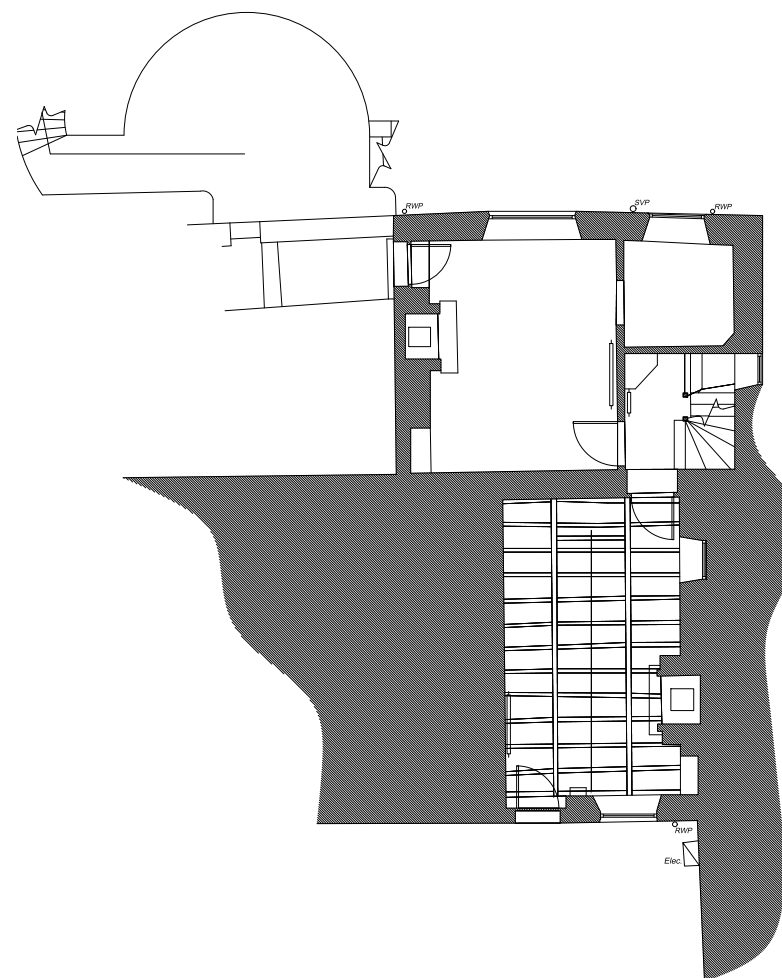
Nathy Connolly

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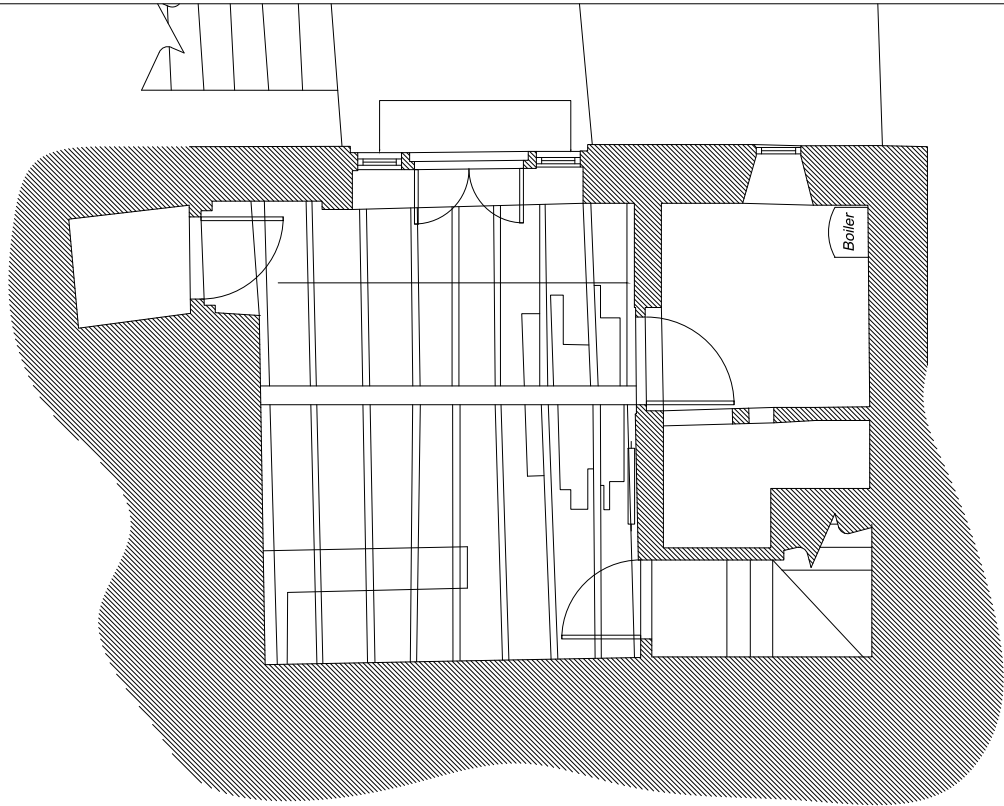
1:100
@ A3



LOWER GROUND FLOOR



UPPER GROUND FLOOR



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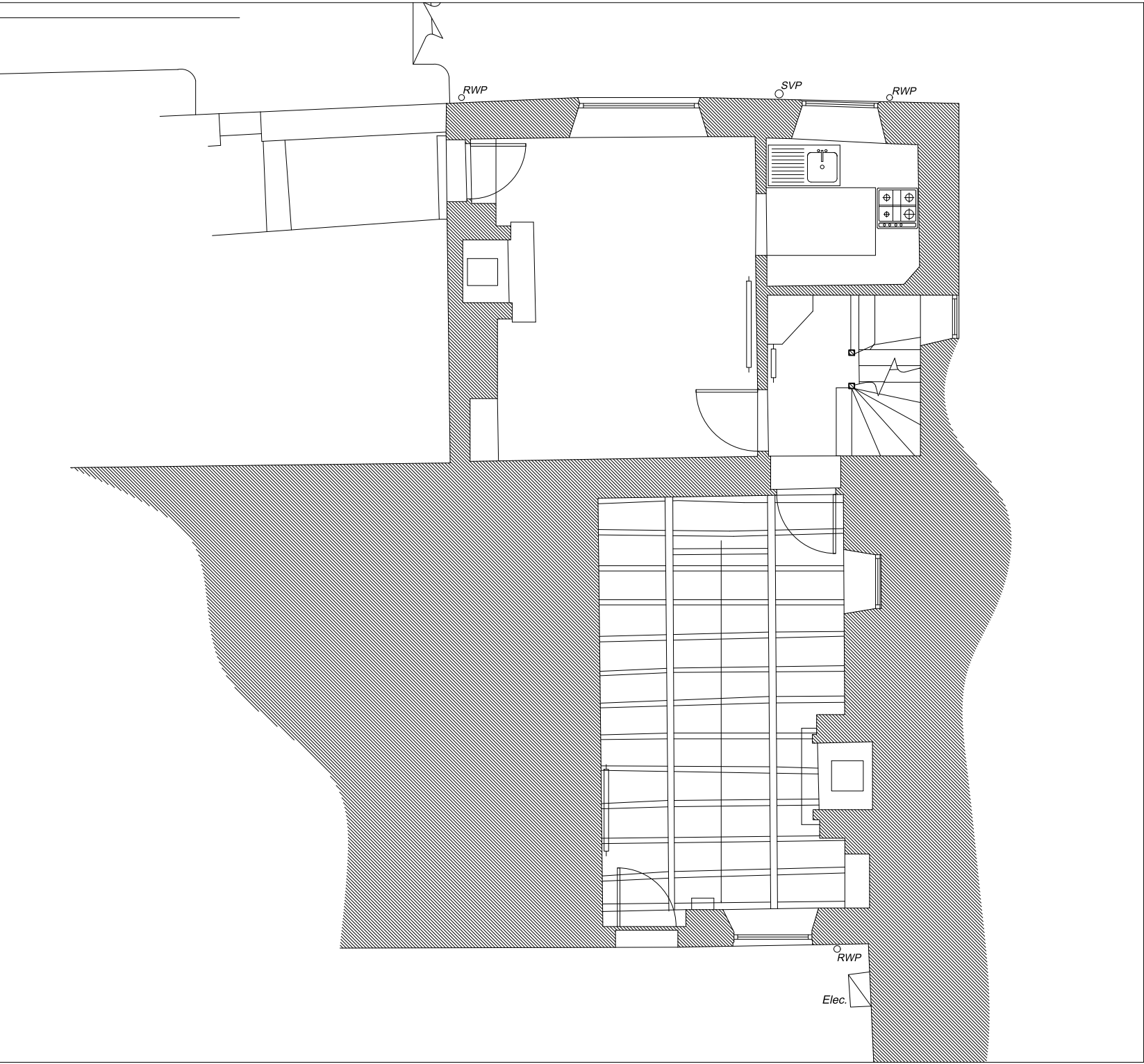
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EXISTING



A_OSH_E_300			
LOWER GROUND FLOOR PLAN			
Old School House			
Hague Street Glossop SK13 8NR			
Nathy Connolly			
A	1:50 @ A3		



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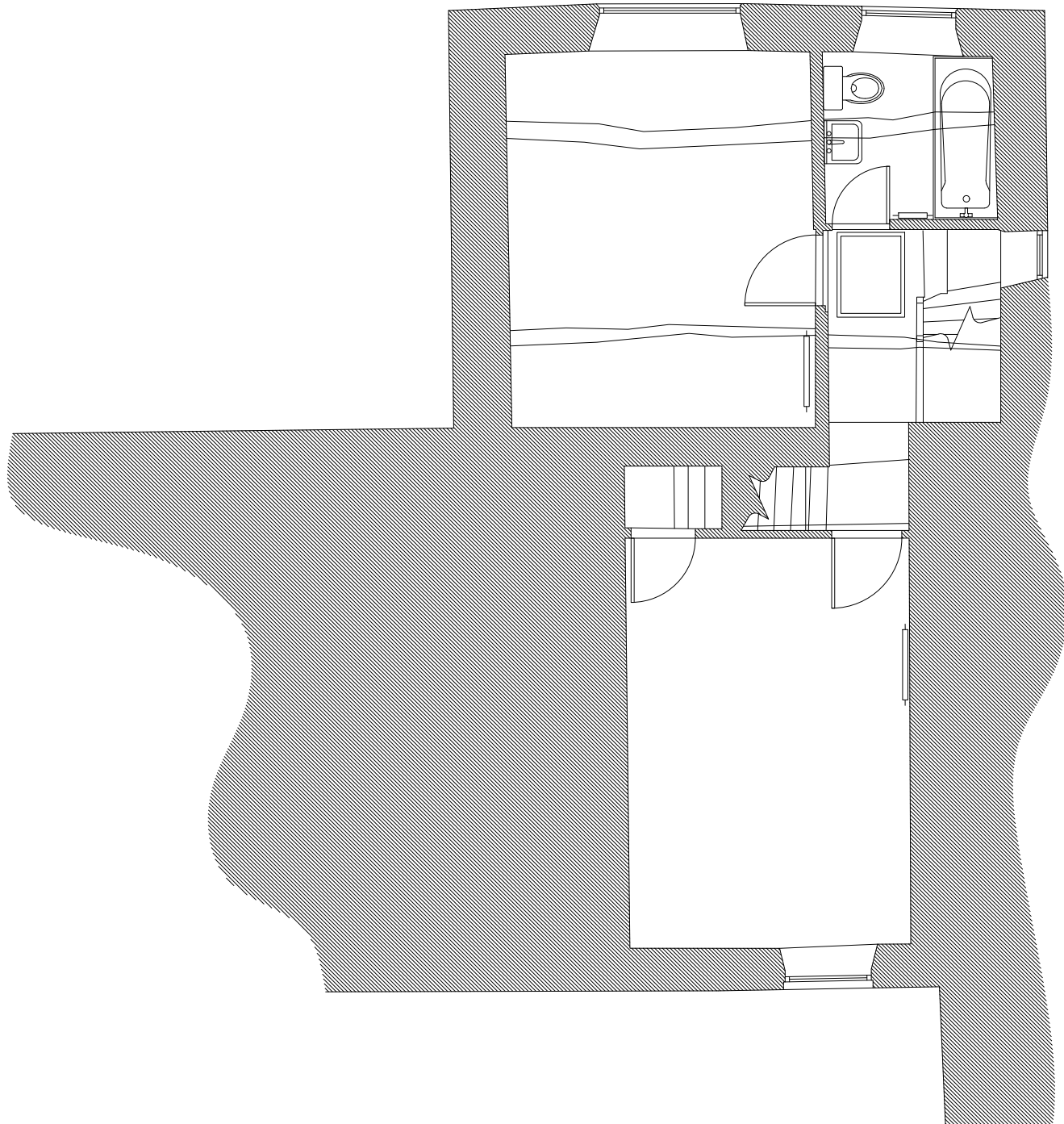
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REVISION	DATE	NOTES
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EXISTING



A_OSH_E_301			
UPPER GROUND FLOOR PLAN			
Old School House			
Hague Street Glossop SK13 8NR			
Nathy Connolly			
A	1:50 @ A3		



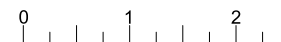
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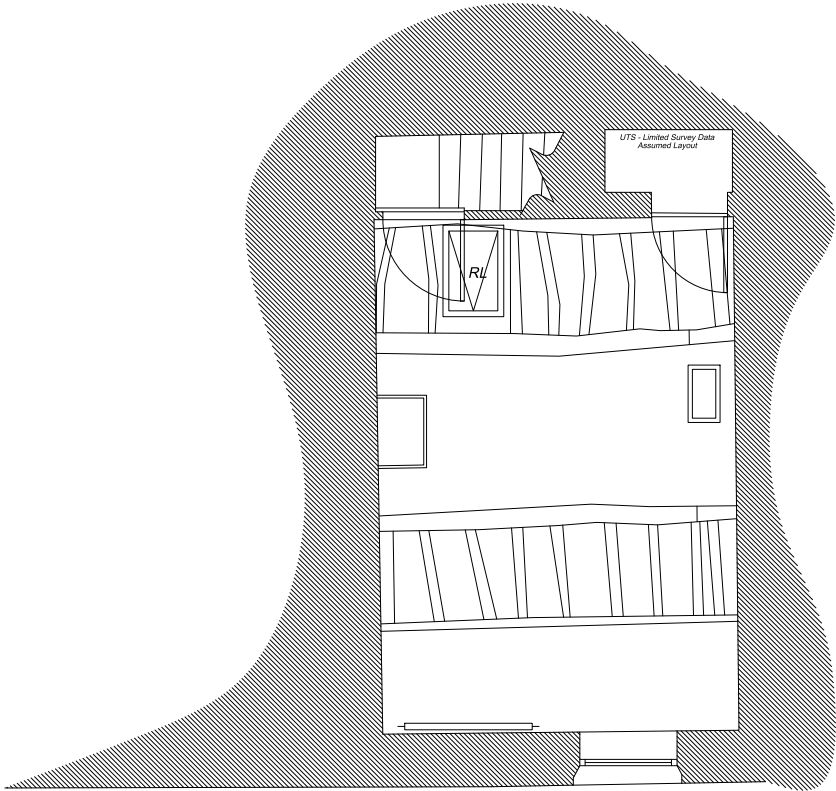
EXISTING



A_OSH_E_302			
FIRST FLOOR PLAN			
Old School House			
Hague Street Glossop SK13 8NR			
Nathy Connolly			
A	1:50 @ A3		

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REVISION	DATE	NOTES
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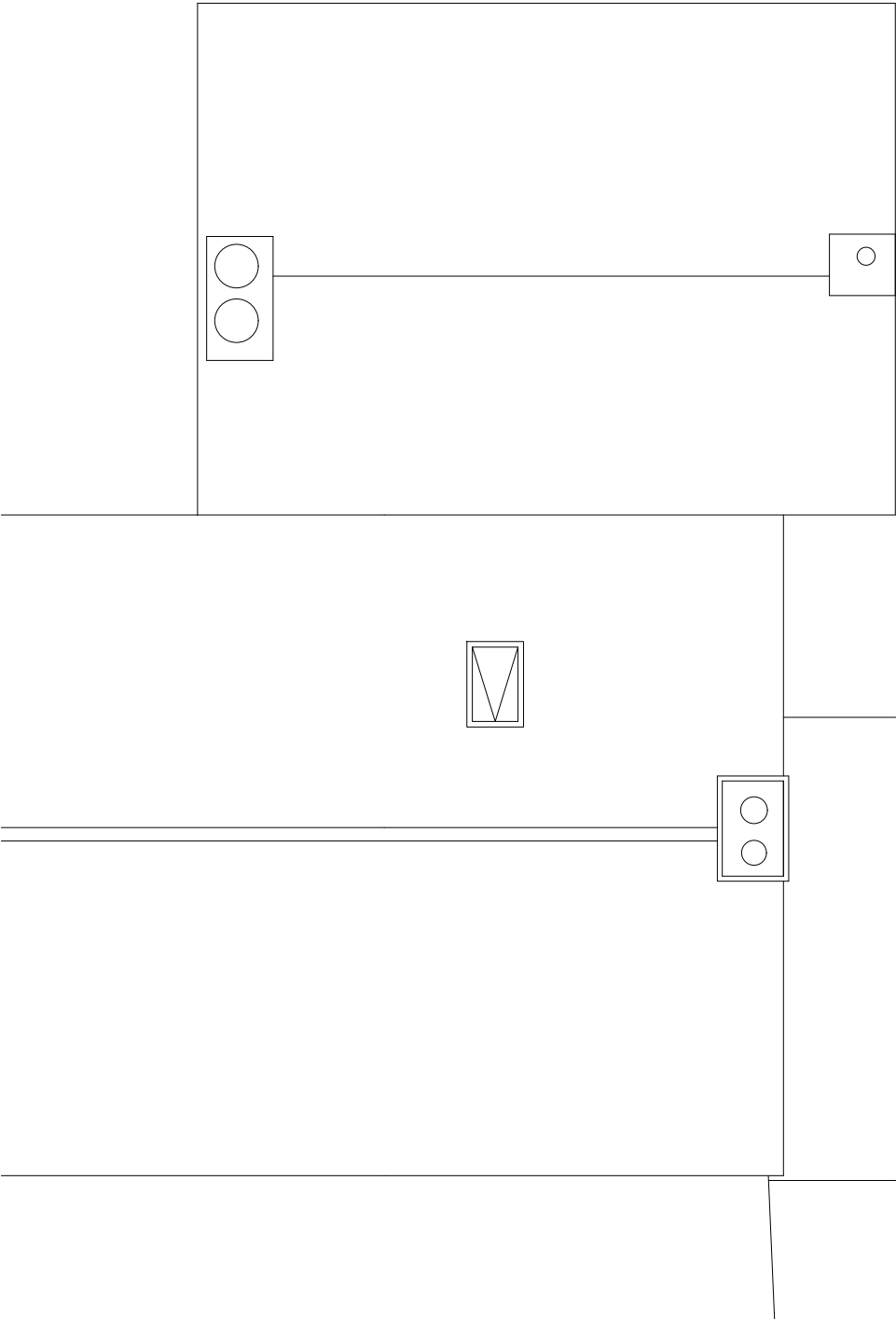
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SECOND FLOOR PLAN			
Old School House			
Hague Street Glossop SK13 8NR			
Nathy Connolly			
A	1:50 @ A3		

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REVISION	DATE	NOTES
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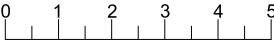
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ROOF PLAN			
Old School House			
Hague Street Glossop SK13 8NR			
Nathy Connolly			
A	1:50 @ A3		

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REVISION	DATE	NOTES
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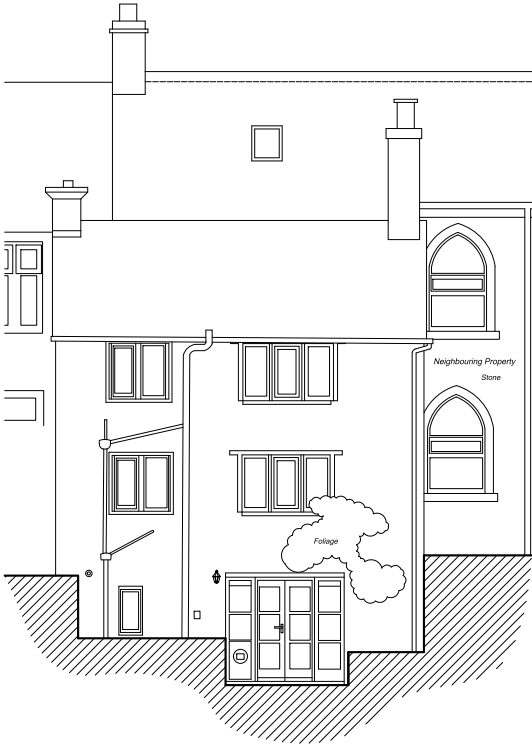
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FRONT ELEVATION				
Old School House				
Hague Street Glossop SK13 8NR				
Nathy Connolly				
A	1:50 @ A3			

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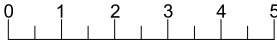
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REVISION	DATE	NOTES
A	02.03.2026	A - Draft Issue
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EXISTING



A_OSH_E_401				
REAR ELEVATION				
Old School House				
Hague Street Glossop SK13 8NR				
Nathy Connolly				
A	1:50 @ A3			

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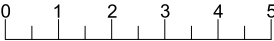
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REVISION	DATE	NOTES
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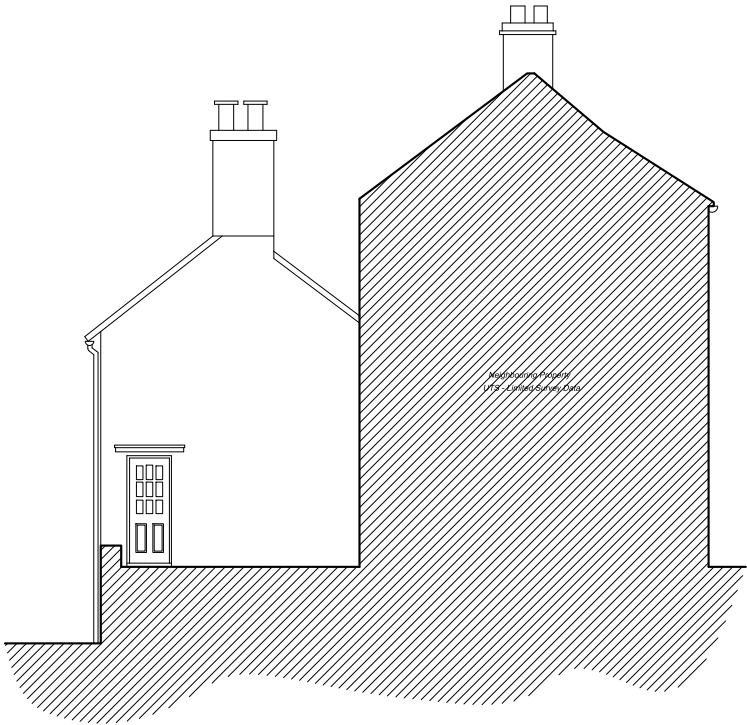
EXISTING



A_OSH_E_402				
SIDE ELEVATION				
Old School House				
Hague Street Glossop SK13 8NR				
Nathy Connolly				
A	1:50 @ A3			

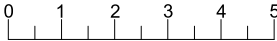
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REVISION	DATE	NOTES
A	02.03.2026	A - Draft Issue
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Datum = 200.00m

EXISTING



A_ OSH_E_403				
SIDE ELEVATION				
Old School House				
Hague Street Glossop SK13 8NR				
Nathy Connolly				
A	1:10 @ A3			

NEW DOORS
TBC?

DOOR

Existing frame to be reused, with membrane passing behind frame. Door resued, modern door leaf.

RADON SUMP

Under new floor radon sump ducting to external wall as indicated, under floor.

EXISTING FLOOR

Remove existing floor - mixture of compressed stone and concrete.

NEW FLOOR

Compacted aggregate

Concrete slab 150mm TBC.

Cavity drainage membrane (protection against moisture)

150mm of insulation

65mm of screed with under floor heating pipes.

Floor finish TBC

CEILING

New ceiling. Existing ceiling is joists (modern) with underside of floor boards. New ceiling with insulation between ceiling and floor. Ceiling creates fire and acoustic separation and conceals services,

NEW WALL BUILD UP

Remove existing paint and modern plaster.

New cavity drainage membrane fixed to wall to prevent water ingress into cellar.

Insulated lining to walls.

THIS COULD BE A NUMBER OF SOLUTIONS TO BE DISCUSSED
WITH CLIENT:

1 Studding, insulation with plaster board.

2 - Wood fibre board with lime plaster for more natural finish

3 - Insulating plaster.

NEW WINDOW
TBC?

TOILET AND SINK

Use existing drainage route and water supply in cellar.

BOILER

Maintain existing boiler location, with vent through external wall as existing. To be located in cupboards.

SHOWER

EXISTING DOOR AND FRAME

Adjust existing door to accommodate cavity drainage membrane. With removal of existing plaster and finish this is likely to result in minimal change.

WALLING MEMBRANE

Cavity Drainage membrane to be return around intermediate walls to provide continuity of protection. Internal walls can have reduced insulation.

EXISTING DOOR AND FRAME

Adjust existing door to accommodate cavity drainage membrane. With removal of existing plaster and finish this is likely to result in minimal change.

STEPS

Existing stone steps to be removed, cut back and repalced to suit new opening width. Existing stones used. If new stone required these will be of similar stone and cut to replicate existing stone steps.

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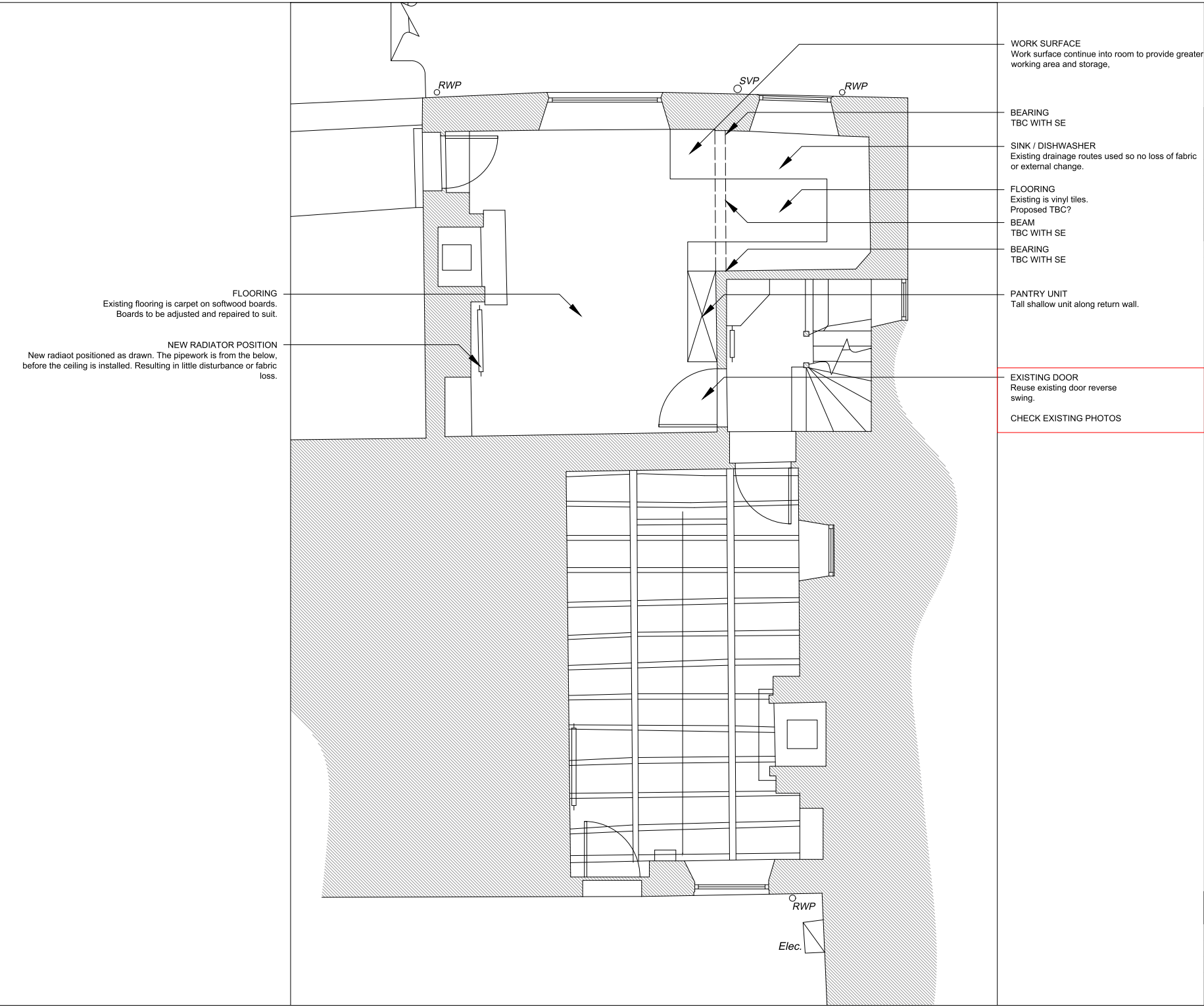
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REVISION	DATE	NOTES
A	05.03.2026	A - Draft Issue
B	29.06.2026	B - Draft Update
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PROPOSED



A_OSH_P_300			
LOWER GROUND FLOOR PLAN			
Old School House			
Hague Street Glossop SK13 8NR			
Nathy Connolly			
C	1:50 @ A3		



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www.swofo.co.uk

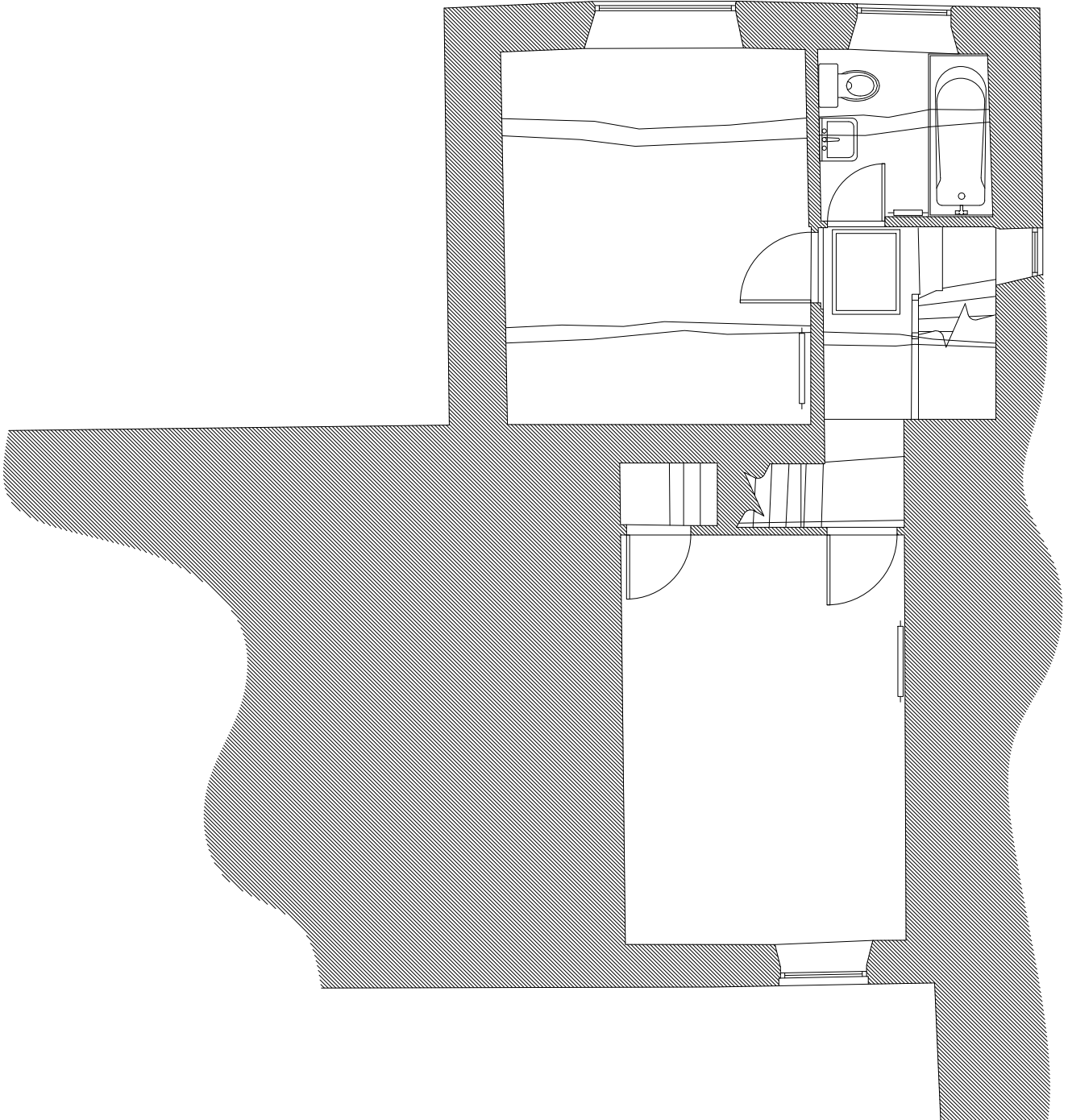
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PROPOSED

0 1 2

A_OSH_P_301			
UPPER GROUND FLOOR PLAN			
Old School House			
Hague Street Glossop SK13 8NR			
Nathy Connolly			
C	1:50 @ A3		



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PROPOSED



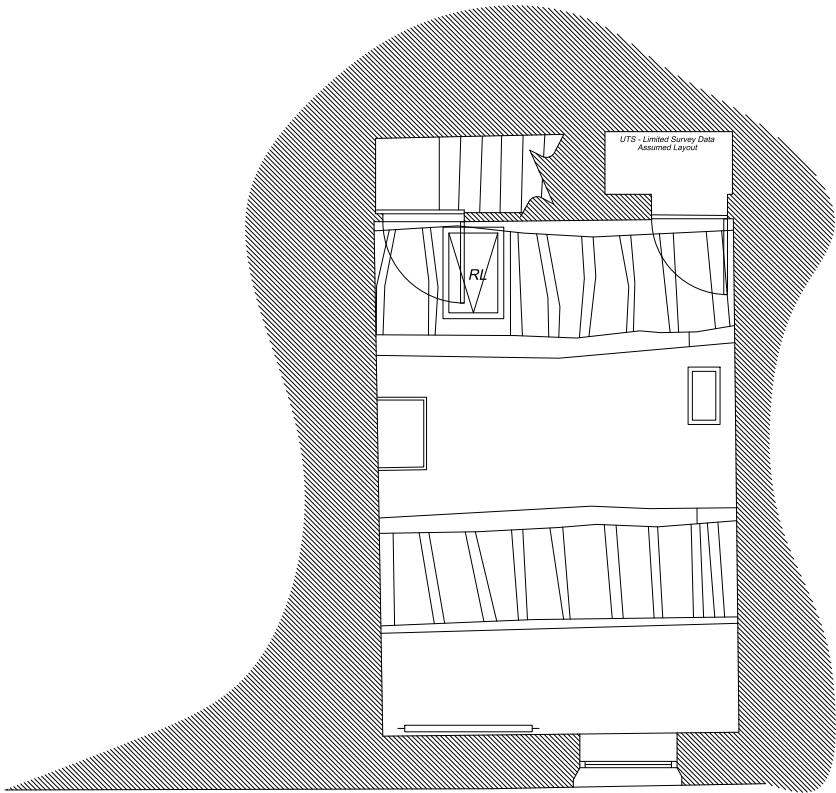
A_OSH_P_302			
FIRST FLOOR PLAN			
Old School House			
Hague Street Glossop SK13 8NR			
Nathy Connolly			
C	1:50 @ A3		

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PROPOSED



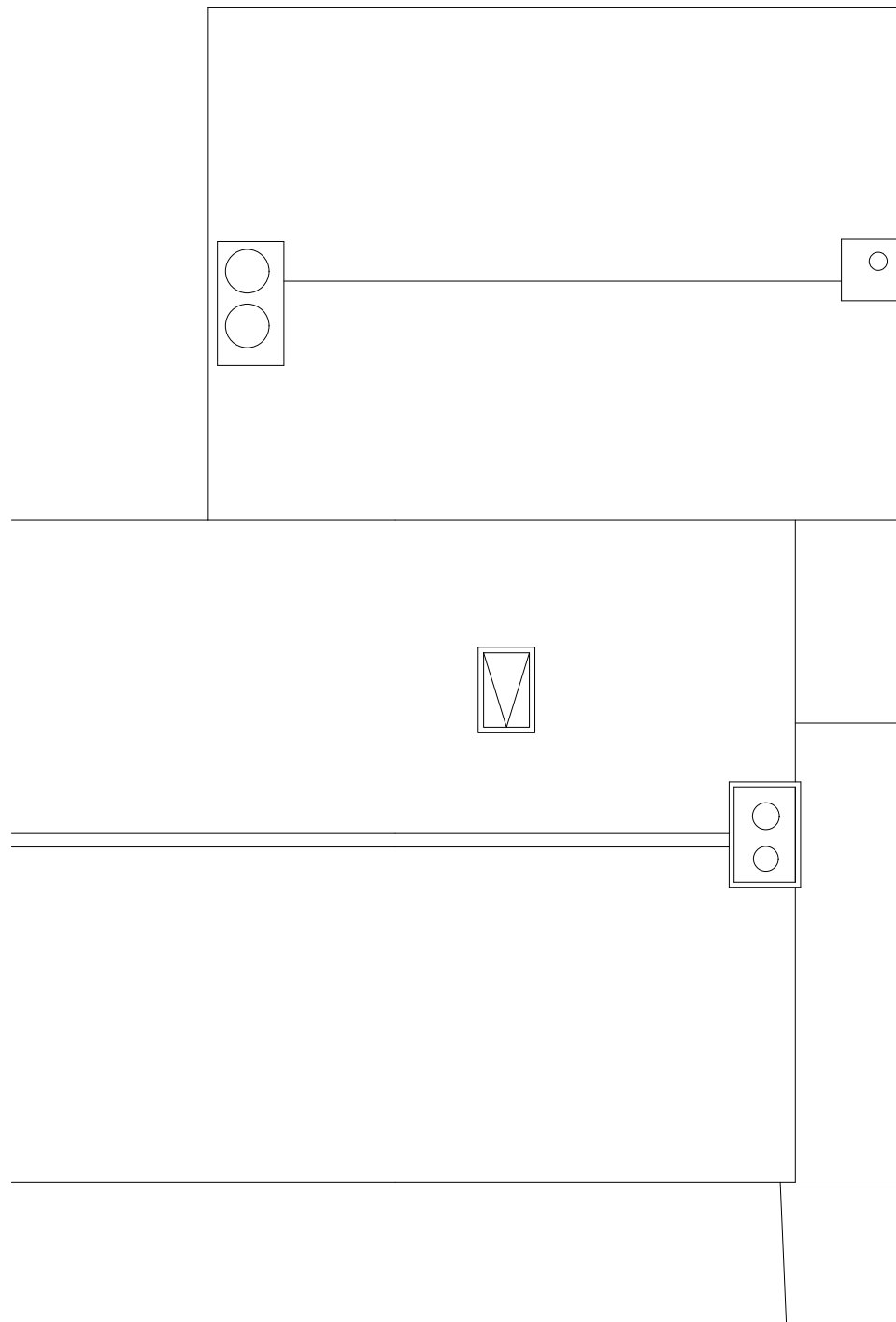
A_OSH_P_303			
SECOND FLOOR PLAN			
Old School House			
Hague Street Glossop SK13 8NR			
Nathy Connolly			
C	1:50 @ A3		

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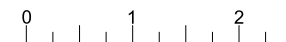
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PROPOSED



A_OSH_P_304			
ROOF PLAN			
Old School House			
Hague Street Glossop SK13 8NR			
Nathy Connolly			
C	1:50 @ A3		

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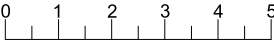
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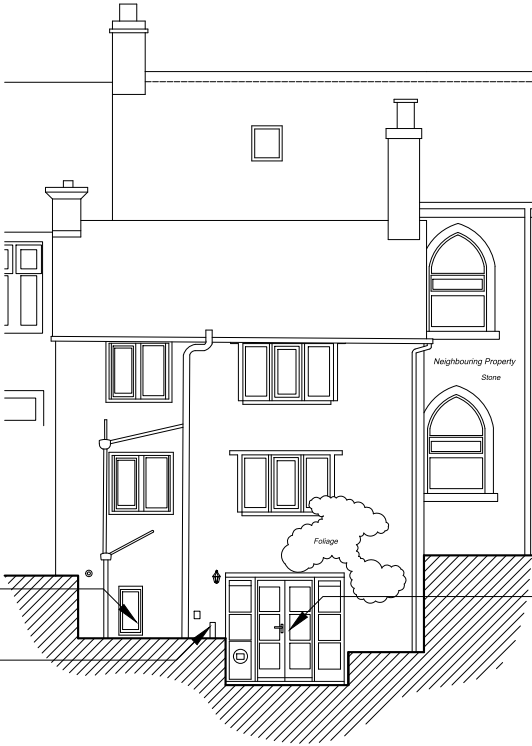
PROPOSED



A_OSH_P_400			
FRONT ELEVATION			
Old School House			
Hague Street Glossop SK13 8NR			
Nathy Connolly			
C	1:100 @ A3		

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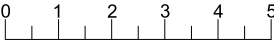


NEW WINDOW
TBC?

NEW DOORS
TBC?

RADON SUMP
Under new floor radon sump ducting to external wall as indicated, under floor.

PROPOSED



A_OSH_P_401				
REAR ELEVATION				
Old School House				
Hague Street Glossop SK13 8NR				
Nathy Connolly				
C	1:100 @ A3			

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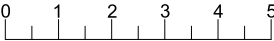
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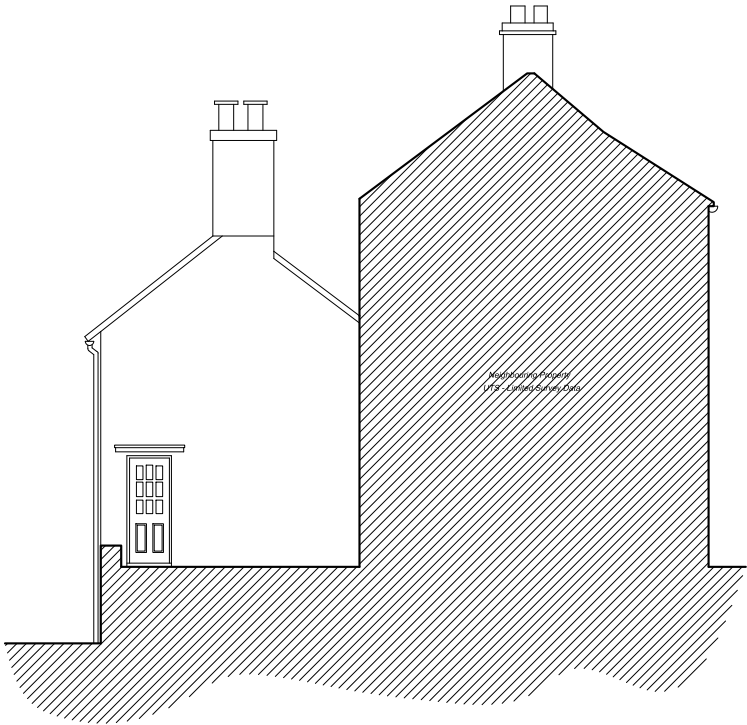
PROPOSED



A_OSH_P_402				
SIDE ELEVATION				
Old School House				
Hague Street Glossop SK13 8NR				
Nathy Connolly				
C	1:100 @ A3			

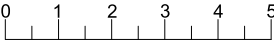
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-	-	-
-	-	-



Datum = 200.00m

PROPOSED



A_OSH_P_403				
SIDE ELEVATION				
Old School House				
Hague Street Glossop SK13 8NR				
Nathy Connolly				
C	1:100 @ A3			