STRUCTURAL CALCULATIONS

FOR

SINGLE STOREY REAR EXTENSION

PROJECT

70 KENTLEA ROAD, LONDON SE28 0JZ

ON

13th JULY 2026

BY

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REFERENCE:

CALCULATIONS ARE PREPARED IN ACCORDANCE WITH THE FOLLOWING CODES OF PRACTICE:

BS 5268 [TIMBER SC3 GRADE]
CODE OF PRACTICE FOR USE OF TIMBER

BS 648 [WEIGHT OF BUILDING MATERIAL]
CODE OF PRACTICE FOR BUILDING MATERIAL

BS 5950 PART 1 – 2000
STRUCTURAL USE OF STEELWORK IN BUILDING

BS 5628 PART 1 – 1992
CODE OF PRACTICE FOR USE OF MASONRY

BS 6399 PART 1 – 1996
LOADING FOR BUILDINGS
[DEAD AND IMPOSED LOADS]

BS 6399 PART 3 - 1988 LOADING FOR BUILDINGS
[IMPOSED ROOF LOAD]

SPAN TABLES FOR SOLID TIMBER MEMBERS IN FLOORS, CEILINGS AND ROOFS FOR DWELLINGS --TRADA 2004

LOAD CALCULATIONS:

FLAT ROOF

	DEAD LOAD (kN/m^2)	LIVE LOAD (kN/m^2)
PLASTER BOARD	0.12	SNOW & REPAIR ACCESS = 0.75 kN/m^2
JOIST	0.18	-
FELT	0.15	-
CHIPPINGS	0.24	-
BOARDING	0.12	-
TOTAL	**0.81 kN/m^2**	**0.75 kN/m^2**

PITCHED ROOF

	DEAD LOAD (kN/m^2)	LIVE LOAD (kN/m^2)
RAFTERS	0.12	SNOW & REPAIR ACCESS = 0.75 kN /m2
BATTENS	0.04	-
FELT	0.05	-
TILES	0.68	-
	0.89 (On Slope)	-
	0.89 / Cos 45 Degrees	-
	= 1.26 (On Plan)	**0.75 kN/m^2**
TOTAL		

FLOOR

	DEAD LOAD (kN/m^2)	LIVE LOAD (kN/m^2)
CEILING	0.15	IMPOSED = 1.50 kN /m2
JOIST	0.15	-
BOARDING	0.20	-
TOTAL	**0.50 kN/m^2**	**1.50 kN/m^2**

CEILING

	DEAD LOAD (kN/m^2)	LIVE LOAD (kN/m^2)
CEILING	0.25	-
TOTAL	**0.25 kN/m^2**	-

BRICK WORK

	DEAD LOAD (kN/m^3)	LIVE LOAD (kN/m^3)
BRICKS	22.00	-
TOTAL	**22.00 kN**/m^3	-

BLOCK WORK

	DEAD LOAD (kN/m^3)	LIVE LOAD (kN/m^3)
BLOCKS	14.00	-
TOTAL	**14.00 kN**/m^3	-

INTERNAL STUDDING

	DEAD LOAD (kN/m^2)	LIVE LOAD (kN/m^2)
PLASTER BOARD	0.04	-
STUD	0.68	-
TOTAL	**0.72 kN/m^2**	-

EXTERNAL STUDDING

	DEAD LOAD (kN/m^2)	LIVE LOAD (kN/m^2)
PLASTER BOARD	0.12	-
TILING	0.68	-
FELT	0.05	-
BATTENS	0.04	-
STUD	0.12	-
TOTAL	**1.01 kN/m^2**	-

GENERAL AND SAFETY NOTES:

1. All dimensions, setting out and levels are to be verified on site with the architect prior to the commencement of any site work.

2. Building control approval must be obtained prior to the commencement of building works.

3.The contractor shall be responsible for and must take all necessary precautions to ensure the stability of the existing structure and earthworks on adjoining sites during the course of the contract.

4.Materials and constructions are to be in accordance with the relevant British Standards and Codes of Practice.

5.Any services or drainage which pass through the foundation are to be encased in a flexible sleeve.

6.The dimensions of all steel sections required should be measured on site by the client (or their representative contractor or steelwork fabricator). All Steel Beams to have minimum of 1/2hr Fire Resistance via ‘Nullifire’ Paint or 19mm Gyproc Plank tied with 1.6mm wire binding@100mm c/c and finished in Carlite Bonding 16mm thick.

7. It is the responsibility of the Client / Contractor to notify the designer of any discrepancies.

8. Note that all steel (Sizes & Quantity) shown on the drawings are should be checked prior to ordering any material.

9. Structural Drawings provided here are based on architectural drawings and site survey therefore are indicative only and are NOT TO SCALE.

10. All existing foundations and lintels to be exposed and to be checked for adequacy by Builder / Building Control Officer and or replaced if necessary.

11. Depth of all footings is to be approved by the local building inspector. Minimum of 1 m deep, advise engineer if footings are located within 30m of large trees or hedges. Also notify if water is struck, tree roots are found or clay soil appears very dry and brittle.

LOAD CALCULATIONS FOR STEEL BEAM 1:

CLEAR SPAN OF THE STEEL BEAM 1 = 7650(3900+3750)mm

LOADING:

DEAD LOAD ON STEEL BEAM 1:

PITCHED ROOF = 1.26 kN/m^2 x SPAN OF ROOF IN MTS.

= 1.26 kN/m^2 x 1.75

= 2.21 kN/m

TIMBER FLOOR = 0.50 kN/m^2 x SPAN OF FLOOR IN MTS

= 0.50 kN/m^2 x 2.5m

=1.25 kN/m

BRICK WORK = 22 kN/m^3 x THICKNES OF WALL x HEIGHT OF WALL IN MTS.
(225mm)

= 22 kN/m^3 x 0.225m x 2.60m

= 12.87 kN/m

PITCHED ROOF = 1.26 kN/m^2 x SPAN OF ROOF IN MTS.

= 1.26 kN/m^2 x 1.5

= 1.89 kN/m

THEREFORE TOTAL DEAD LOAD ON SB1=2.21+1.25+12.87+1.89= 18.22kN/m

LIVE LOAD ON STEEL BEAM 1:

PITCHED ROOF = 0.75 kN/m^2 x SPAN OF ROOF IN MTS.

= 0.75 kN/m^2 x 1.75m

= 1.31 kN/m

TIMBER FLOOR = 1.5 kN/m^2 x SPAN OF FLOOR IN MTS.

= 1.5 kN/m^2 x 2.50m

= 3.75 kN/m

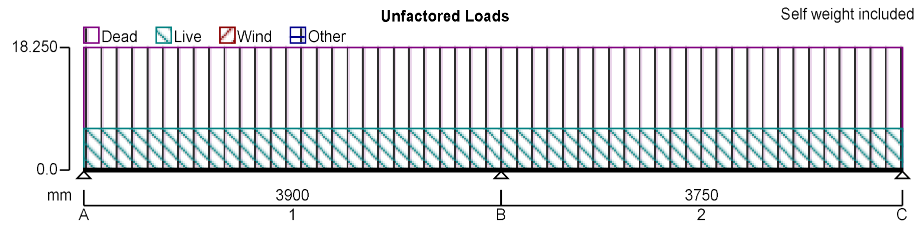
PITCHED ROOF = 0.75 kN/m^2 x SPAN OF ROOF IN MTS.

= 0.75 kN/m^2 x 1.5m

= 1.13 kN/m

THEREFORE TOTAL LIVE LOAD ON SB1 = 1.31 + 3.75 + 1.13 = 6.19 kN/m

STRUCTURAL REPORT FOR STEEL BEAM 1:



CONTINUOUS BEAM ANALYSIS - INPUT

BEAM DETAILS

Number of spans = **2**

Material Properties:

Modulus of elasticity = **205** kN/mm^2 Material density = **7860** kg/m^3

Support Conditions:

Support A Vertically **"Restrained"** Rotationally **"Free"**

Support B Vertically **"Restrained"** Rotationally **"Free"**

Support C Vertically **"Restrained"** Rotationally **"Free"**

Span Definitions:

Span 1 Length = **3900** mm Cross-sectional area = **5873** mm^2 Moment of inertia = **45.7×10^6** mm^4

Span 2 Length = **3750** mm Cross-sectional area = **5873** mm^2 Moment of inertia = **45.7×10^6** mm^4

LOADING DETAILS

Beam Loads:

Load 1 Self weight Dead load = **1.000**

Load 2 UDL Dead load **18.3** kN/m

Load 3 UDL Live load **6.2** kN/m

LOAD COMBINATIONS

Load combination 1

Span 1 1.4×Dead + 1.6×Live + 1×Wind + 1×Other

Span 2 1.4×Dead + 1.6×Live + 1×Wind + 1×Other

CONTINUOUS BEAM ANALYSIS - RESULTS

Support Reactions - Combination Summary

Support A Max react = **-53.5** kN Min react = **-53.5** kN Max mom = **0.0** kNm Min mom = **0.0** kNm

Support B Max react = **-172.7** kN Min react = **-172.7** kN Max mom = **0.0** kNm Min mom = **0.0** kNm

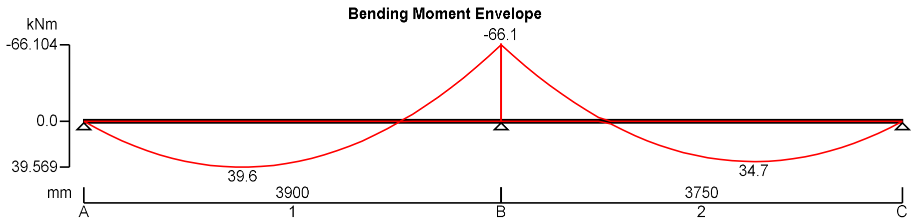
Support C Max react = **-50.1** kN Min react = **-50.1** kN Max mom = **0.0** kNm Min mom = **0.0** kNm

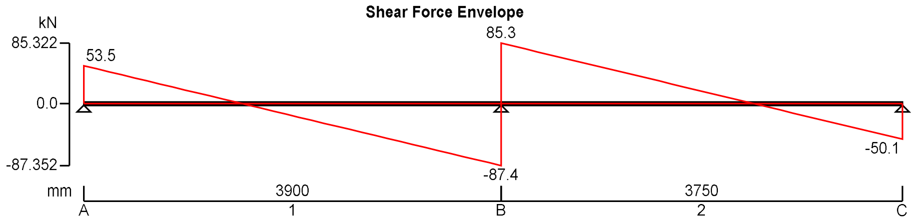
Beam Max/Min results - Combination Summary

Maximum shear = **85.3** kN Minimum shearF_{min} = **-87.4** kN

Maximum moment = **39.6** kNm Minimum moment = **-66.1** kNm

Maximum deflection = **5.1** mm Minimum deflection = **0.0** mm





DESIGN SPAN 1

Member design checks for a simply-supported steel beam to BS 5950 (with LTB)

Summary of results

Material	Grade = **"S355"**	p_y = **355** N/mm^2	
Section	**"UC 203x203x46"**	Classification	**"Semi-Compact"**

Check	*Load*	*Capacity*	*Notes*	*Result*
Deflection	δ_{y_max} = **0.9** mm	δ_{lim} = **10.8** mm	**Span / 360 or 11.0 mm**	**Pass**
Shear	F_{vy} = **87.4** kN	P_{vy} = **311.6** kN	**Low shear**	**Pass**
Moment	M_x = **66.1** kNm	M_{cx} = **159.6** kNm	**Low shear**	**Pass**
LTB	M_{LT} = **66.1** kNm	M_b / m_{LT} = **128.2** kNm	L_{E_LT} = **3.9** m m_{LT} = **1.00**	**Pass**

LOAD CALCULATIONS FOR STEEL BEAM 2:

CLEAR SPAN OF THE STEEL BEAM 2 = 7300(1750+3800+1750)mm

LOADING:

DEAD LOAD ON STEEL BEAM 2:

PITCHED ROOF = 1.26 kN/m^2 x SPAN OF ROOF IN MTS.

= 1.26 kN/m^2 x 1.5

= 1.89 kN/m

BRICK WORK = 22 kN/m^3 x THICKNES OF WALL x HEIGHT OF WALL IN MTS.
(100mm)

= 22 kN/m^3 x 0.100m x 0.4m (x2)

= 1.76 kN/m

THEREFORE TOTAL DEAD LOAD ON SB2 = 1.89 + 1.76 = 3.65kN/m

LIVE LOAD ON STEEL BEAM 2:

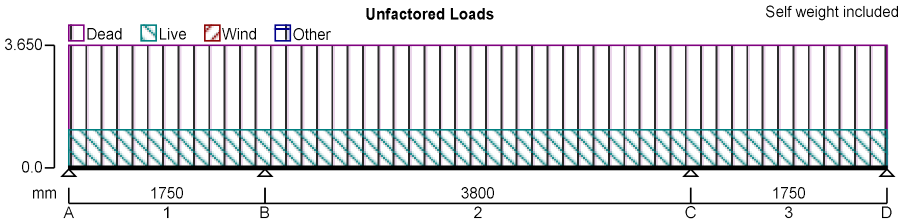
PITCHED ROOF = 0.75 kN/m^2 x SPAN OF ROOF IN MTS.

= 0.75 kN/m^2 x 1.5

= 1.13 kN/m

THEREFORE TOTAL LIVE LOAD ON SB2 = 1.13 kN/m

STRUCTURAL REPORT FOR STEEL BEAM 2:



CONTINUOUS BEAM ANALYSIS - INPUT

BEAM DETAILS

Number of spans = **3**

Material Properties:

Modulus of elasticity = **205** kN/mm^2 | Material density = **7860** kg/m^3

Support Conditions:

Support A	Vertically **"Restrained"**	Rotationally **"Free"**
Support B	Vertically **"Restrained"**	Rotationally **"Free"**
Support C	Vertically **"Restrained"**	Rotationally **"Free"**
Support D	Vertically **"Restrained"**	Rotationally **"Free"**

Span Definitions:

Span 1	Length = **1750** mm	Cross-sectional area = **2940** mm^2	Moment of inertia = **21.0×10^6** mm^4
Span 2	Length = **3800** mm	Cross-sectional area = **2940** mm^2	Moment of inertia = **21.0×10^6** mm^4
Span 3	Length = **1750** mm	Cross-sectional area = **2940** mm^2	Moment of inertia = **21.0×10^6** mm^4

LOADING DETAILS

Beam Loads:

Load 1	Self weight Dead load = **1.000**
Load 2	UDL Dead load **3.7** kN/m
Load 3	UDL Live load **1.1** kN/m

LOAD COMBINATIONS

Load combination 1

Span 1	1.4×Dead + 1.6×Live + 1×Wind + 1×Other
Span 2	1.4×Dead + 1.6×Live + 1×Wind + 1×Other
Span 3	1.4×Dead + 1.6×Live + 1×Wind + 1×Other

CONTINUOUS BEAM ANALYSIS - RESULTS

Support Reactions - Combination Summary

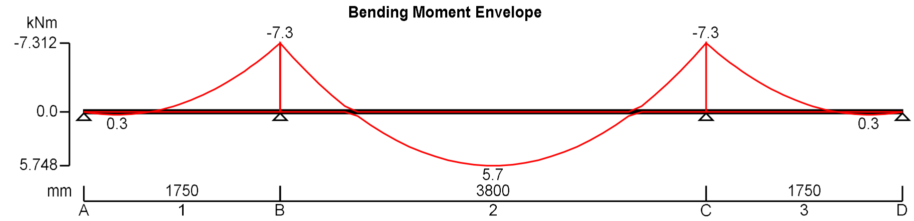
Support A	Max react = **-2.2** kN	Min react = **-2.2** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support B	Max react = **-24.3** kN	Min react = **-24.3** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support C	Max react = **-24.3** kN	Min react = **-24.3** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support D	Max react = **-2.2** kN	Min react = **-2.2** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm

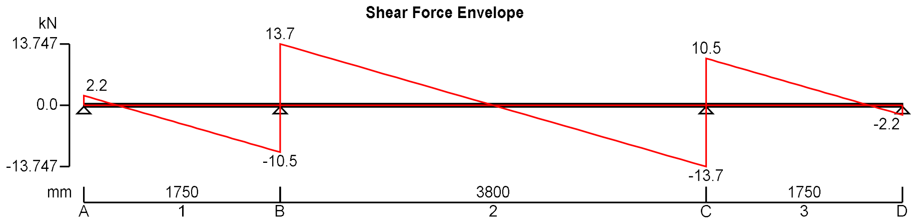
Beam Max/Min results - Combination Summary

Maximum shear = **13.7** kN | Minimum shearF_{min} = **-13.7** kN

Maximum moment = **5.7** kNm | Minimum moment = **-7.3** kNm

Maximum deflection = **1.5** mm | Minimum deflection = **-0.1** mm





DESIGN FOR WORST CASE

Member design checks for a simply-supported steel beam to BS 5950 (with LTB)

Summary of results

Material	Grade = **"S355"**	p_y = **355** N/mm^2	
Section	**"UB 203x102x23"**	Classification	**"Plastic"**

Check	*Load*	*Capacity*	*Notes*	*Result*
Deflection	δ_{y_max} = **0.2** mm	δ_{lim} = **10.6** mm	**Span / 360 or 11.0 mm**	**Pass**
Shear	F_{vy} = **13.7** kN	P_{vy} = **233.7** kN	**Low shear**	**Pass**
Moment	M_x = **7.3** kNm	M_{cx} = **83.1** kNm	**Low shear**	**Pass**
LTB	M_{LT} = **7.3** kNm	M_b / m_{LT} = **30.6** kNm	L_{E_LT} = **3.8** m m_{LT} = **1.00**	**Pass**

LOAD CALCULATIONS FOR STEEL COLUMN:

HEIGHT OF STEEL COLUMN = 2800mm

ASSUME A STEEL COLUMN = 203 x 203 x 46 UC

GRADE OF STEEL = ‘S355’

LOAD FROM STEEL BEAM 1 = 174.00 kN

CAP PLATE = 25mm THICK MILD STEEL PLATE

TOP BOLTS = 4NOS M20 GR.8.8 BOLTS

BASE PLATE = 400x400x25mm THICK MILD STEEL PLATE

RESIN BOLTS = 4NOS M20 M.L RESIN BOLTS 200mm LONG

LOAD CALCULATIONS FOR STEEL COLUMN:

STEEL MEMBER DESIGN TO BS5950: PART 1: 2000

Member design checks for a steel member to BS 5950:2000

Section properties

Try UC 203x203x46 section.

D = **203** mm B = **204** mm T = **11.0** mm t = **7.2** mm

d = **160.8** mm b = B / 2 = **101.8** mm u = **0.8465** x = **17.71**

A_g = A = **58.7** cm^2 I_x = **4568** cm^4 I_y = **1548** cm^4 r_x = **8.82** cm r_y = **5.13** cm

S_x = **497.4** cm^3 S_y = **230.9** cm^3 Z_x = **449.6** cm^3 Z_y = **152.1** cm^3

$A_{vy} = t \times D$ = **14.6** cm^2 $A_{vx} = 0.9 \times 2 \times T \times B$ = **40.3** cm^2 $d_x = B$ $d_y = d$

$S_{vx} = t \times D^2 / 4$ = **74.3** cm^3

Strut curve (b) for x-axis, curve (c) for y-axis. (Tables 23 & 24) $z_1 = 2.0$ $z_2 = 1.0$

Steel grade **"S355"** p_y = **355** N/mm^2 $p_{yw} = p_y$ $\varepsilon = \sqrt{(275\ N/mm^2 / p_y)}$ = **0.880** K_e = **1.1**

Cl. 3.1.1 & 3.4.3

Geometry

Member has no intermediate restraint.

L = **2800** mm

Buckling:	Restraint:	End 1:	End 2:
Normal to x-axis	Position	**X**	**X**
	Direction		
Normal to y-axis	Position	**X**	**X**
	Direction		

For strut buckling normal to x-axis K_x = **1.00** L_{Ex} = **2800** mm

For strut buckling normal to y-axis K_y = **1.00** L_{Ey} = **2800** mm

Cl. 4.3.5 & 4.7.3

Loading

Internal forces & moments on member under factored loading for uls design:

F_t = 0 kN F_c = **173.00** kN $n = F_c / (A \times p_y)$ = **0.083**

F_{vx} = 0 kN

F_{vy} = 0 kN

M_x = 0 kNm

M_y = 0 kNm

Section classification

b / T = **9.3** d / t = **22.3**

$r_1 = \min(1.0, \max(-1.0, F_c / (d \times t \times p_{yw})))$ = **0.421** $r_2 = F_c / (A_g \times p_{yw})$ = **0.083**

Section classification is Semi-Compact

Cl. 3.5.1 - 3.5.5

Compression resistance - strut buckling about x-axis

F_c = **173.0** kN L_{Ex} = **2.80** m $\lambda_x = L_{Ex} / r_x$ = **32**

Strut curve (b) applies $\lambda_0 = 0.2 \times (\pi^2 \times E_{S5950} / p_y)^{0.5}$ = **15** a_x = **3.5**

η_x = max(0, $a_x \times (\lambda_x - \lambda_0)$ / 1000) = **0.058** $\quad p_{ex} = \pi^2 \times E_{S5950} / \lambda_x^2$ = **2007** N/mm^2

$\phi_x = (p_y + (\eta_x + 1) \times p_{ex}) / 2 \quad p_{cx} = p_{ex} \times p_y / (\phi_x + (\phi_x^2 - p_{ex} \times p_y)^{0.5})$ = **332** N/mm^2

p_{cx} = **332** N/mm^2 $\quad P_{cx} = A \times p_{cx}$ = **1948.9** kN

Check $F_c \le P_{cx}$ Pass - Compression

Slenderness less than 180

Cl. 4.7.4 & Annex C

Compression resistance - strut buckling about y axis

F_c = **173.0** kN $\quad L_{Ey}$ = **2.80** m $\quad \lambda_y = L_{Ey} / r_y$ = **55**

Strut curve (c) applies $\quad \lambda_0 = 0.2 \times (\pi^2 \times E_{S5950} / p_y)^{0.5}$ = **15** $\quad a_y$ = **5.5**

η_y = max(0, $a_y \times (\lambda_y - \lambda_0)$ / 1000) = **0.217** $\quad p_{ey} = \pi^2 \times E_{S5950} / \lambda_y^2$ = **680** N/mm^2

$\phi_y = (p_y + (\eta_y + 1) \times p_{ey}) / 2 \quad p_{cy} = p_{ey} \times p_y / (\phi_y + (\phi_y^2 - p_{ey} \times p_y)^{0.5})$ = **262** N/mm^2

$P_{cy} = A \times p_{cy}$ = **1540.9** kN

Check $F_c \le P_{cy}$ Pass - Compression

Slenderness less than 180

Cl. 4.7.4 & Annex C

Results summary

Summary of results

Material	Grade = **"S355"**	p_y = **355** N/mm^2		
Section	**"UC 203x203x46"**	Classification	**"Semi-Compact"**	

Check	*Load*	*Capacity*	*Notes*	*Result*
Strut buckling (x-axis)	F_c = **173.0** kN	P_{cx} = **1948.9** kN	L_{Ex} = **2.8** m **Slenderness < 180**	**Pass**
Strut buckling (y-axis)	F_c = **173.0** kN	P_{cy} = **1540.9** kN	L_{Ey} = **2.8** m **Slenderness < 180**	**Pass**

LOAD CALCULATIONS FOR ENGINEERING BRICK PIER:

HEIGHT OF BRICK PIER= 2800mm

ASSUMING 330x330mm THICK NEW ENGINEERING BRICK PIER

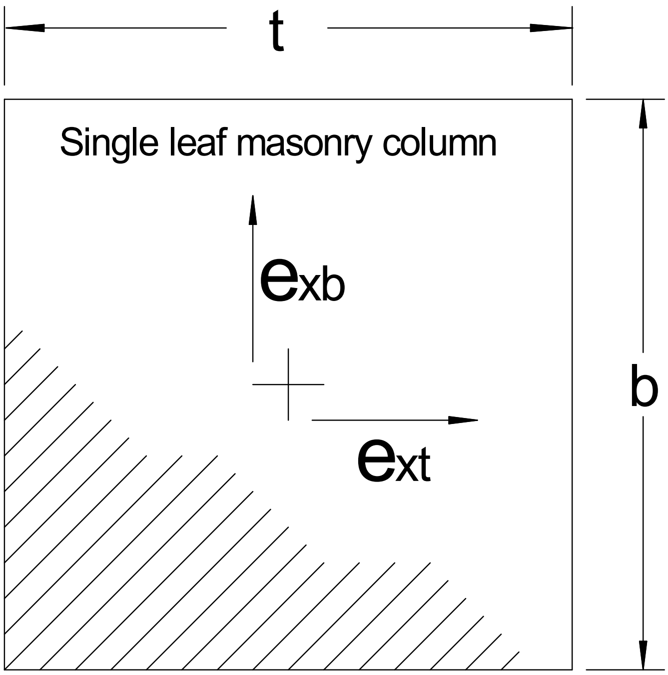
BRICK PIER TO BE MADE IN MORTAR III

LOADING (WORST CASE):

LOAD FROM THE STEEL BEAM 1 = 53.50 kN

LOAD CALCULATIONS FOR ENGINEERING BRICK PIER:

VERTICAL LOADING RECTANGULAR COLUMN (BS5628-1:2005)



Compressive strength from Table 2 BS5628:Part 1 - Clay or calcium silicate bricks

Mortar designation	Mortar = **"iii"**
Brick compressive strength	p_{unit} = **20.0** N/mm^2
Characteristic compressive strength	f_k = **5.00** N/mm^2
Column width	b = **330** mm
Column thickness	t = **330** mm
Column height	h = **2.80** m
Column slenderness - minor axis (Clause 24.1)	$\lambda_t = h/t =$ **8.48**
Column slenderness - major axis (Clause 24.1)	$\lambda_b = h/b =$ **8.48**
Maximum slenderness	$\lambda_{max} = \max(\lambda_t, \lambda_b) =$ **8.48**

Slenderness < 27 - OK

Partial safety factor for material (Table 4)	$\gamma_m =$ **3.5**

Load eccentricity

Eccentricity of applied load about minor axis	e_{xt} = **0.0** mm
Eccentricity of applied load about major axis	e_{xb} = **0.0** mm

Capacity reduction factor: minor axis

Eccentricity due to slenderness	$e_{at} = t \times (((h/t)^2/2400)-0.015) =$ **4.9** mm
Design eccentricity	$e_{tt} = 0.6\times\max(\text{abs}(e_{xt}), 0.05\times t)+ e_{at} =$ **14.8** mm
	$e_{mt} = \max(\text{abs}(e_{xt}),e_{tt}) =$ **14.8** mm
Capacity reduction factors	$\beta_{tcalc} = 1.1 \times (1 - (2 \times e_{mt} / t))=$ **1.00**
	$\beta_{max} = 1.0$
	$\beta_t = \min(\beta_{tcalc}, \beta_{max}) =$ **1.00**

Capacity reduction factor: major axis

Eccentricity due to slenderness	$e_{ab} = b \times (((h/b)^2/2400)-0.015) =$ **4.9** mm
Design eccentricity	$e_{tb} = 0.6\times \max(\text{abs}(e_{xb}), 0.05\times b)+ e_{ab} =$ **14.8** mm
	$e_{mb} = \max(\text{abs}(e_{xb}),e_{tb}) =$ **14.8** mm

Capacity reduction factors $\beta_{bcalc} = 1.1 \times (1 - (2 \times e_{mb} / b))=$ **1.00**

$\beta_{max} = 1.0$

$\beta_b = \min(\beta_{bcalc}, \beta_{max}) =$ **1.00**

Minimum capacity reduction factor $\beta_{min} = \min(\beta_t, \beta_b) =$ **1.00**

Design vertical load resistance

Compressive strength correction factor

Plan area of column $A = t \times b =$ **0.11** m^2

For small plan area (Clause 19.1.2) $c = \min(1.0, 0.7+(1.5\ m^{-2})\times A) =$ **0.86**

Design vertical load resistance $DVLR = \beta_{min} \times t \times b \times c \times f_k / \gamma_m =$ **134.313** kN

Applied factored vertical load on column $V =$ **53.500** kN

Column - OK

LOAD CALCULATIONS FOR 100mm STEEL POSTS 1 & 2:

HEIGHT OF SHS POST = 2300mm [Builder to confirm on site prior to ordering material]

ASSUME SHS POST = 100 x 100 x 5mm SHS

LOAD FROM STEEL BEAM 2 = 25.00 kN

TOP & BOTTOM PLATES = 15mm THICK

TOP BOLTS = 4NOS M20 GR.8.8 BOLTS

BOTTOM BOLTS = 4NOS M20 M.L RESIN BOLTS

LOAD CALCULATIONS FOR STEEL POST 1 & 2:

STEEL MEMBER DESIGN TO BS5950: PART 1: 2000

Member design checks for a steel member to BS 5950:2000

Section properties

Try SHS 100x100x5.0 section.

D = **100** mm B = **100** mm T = t = **5.0** mm A_g = A = **18.7** cm^2

d_x = D - 3 × t = **85.0** mm b_x = B - 3 × t = **85.0** mm $b_y = d_x$ $d_y = b_x$

A_{vy} = A × D / (D + B) = **9.4** cm^2 A_{vx} = A × B / (D + B) = **9.4** cm^2

I_x = **279** cm^4 I_y = **279** cm^4 r_x = **3.86** cm r_y = **3.86** cm

S_x = **66.4** cm^3 S_y = **66.4** cm^3 Z_x = **55.9** cm^3 Z_y = **55.9** cm^3

d_{vx} = A_{vx} / (2 × t) = **93.7** mm d_{vy} = A_{vy} / (2 × t) = **93.7** mm

S_{vx} = 2 × t × d_{vy}^2 / 4 = **21.9** cm^3 S_{vy} = 2 × t × d_{vx}^2 / 4 = **21.9** cm^3

Strut curve (a) for x-axis and y-axis. (Tables 23 & 24) $z_1 = 5/3$ $z_2 = 5/3$

Steel grade **"S355"** p_y = **355** N/mm^2 $p_{yw} = p_y$ $\varepsilon = \sqrt{(275 \text{ N/mm}^2 / p_y)}$ = **0.880** K_e = **1.1**

Cl. 3.1.1 & 3.4.3

Geometry

Member has no intermediate restraint.

L = **2300** mm

Buckling:	Restraint:	End 1:	End 2:
Normal to x-axis	Position	**X**	**X**
	Direction		
Normal to y-axis	Position	**X**	**X**
	Direction		

For strut buckling normal to x-axis K_x = **1.00** L_{Ex} = **2300** mm

For strut buckling normal to y-axis K_y = **1.00** L_{Ey} = **2300** mm

Cl. 4.3.5 & 4.7.3

Loading

Internal forces & moments on member under factored loading for uls design:

F_t = 0 kN F_c = **25.00** kN n = F_c / (A × p_y) = **0.038**

F_{vx} = 0 kN

F_{vy} = 0 kN

M_x = 0 kNm

M_y = 0 kNm

Section classification

b_x / t = **17.0** d_x / t = **17.0** b_y / t = **17.0** d_y / t = **17.0**

r_{1x} = min(1.0, max(-1.0, F_c / (2 × d_x × t × p_{yw}))) = **0.083**

r_{1y} = min(1.0, max(-1.0, F_c / (2 × d_y × t × p_{yw}))) = **0.083**

r_2 = F_c / (A_g × p_{yw}) = **0.038**

Section classification is Plastic

Cl. 3.5.1 - 3.5.5

Compression resistance - strut buckling about x-axis

F_c = **25.0** kN L_{Ex} = **2.30** m $\lambda_x = L_{Ex} / r_x$ = **60**

Strut curve (a) applies $\lambda_0 = 0.2 \times (\pi^2 \times E_{S5950} / p_y)^{0.5}$ = **15** a_x = **2.0**

$\eta_x = \max(0, a_x \times (\lambda_x - \lambda_0) / 1000)$ = **0.089** $p_{ex} = \pi^2 \times E_{S5950} / \lambda_x^2$ = **571** N/mm^2

$\phi_x = (p_y + (\eta_x + 1) \times p_{ex}) / 2$ $p_{cx} = p_{ex} \times p_y / (\phi_x + (\phi_x^2 - p_{ex} \times p_y)^{0.5})$ = **299** N/mm^2

p_{cx} = **299** N/mm^2 $P_{cx} = A \times p_{cx}$ = **560.3** kN

Check $F_c \le P_{cx}$ Pass - Compression

Slenderness less than 180

Cl. 4.7.4 & Annex C

Compression resistance - strut buckling about y axis

F_c = **25.0** kN L_{Ey} = **2.30** m $\lambda_y = L_{Ey} / r_y$ = **60**

Strut curve (a) applies $\lambda_0 = 0.2 \times (\pi^2 \times E_{S5950} / p_y)^{0.5}$ = **15** a_y = **2.0**

$\eta_y = \max(0, a_y \times (\lambda_y - \lambda_0) / 1000)$ = **0.089** $p_{ey} = \pi^2 \times E_{S5950} / \lambda_y^2$ = **571** N/mm^2

$\phi_y = (p_y + (\eta_y + 1) \times p_{ey}) / 2$ $p_{cy} = p_{ey} \times p_y / (\phi_y + (\phi_y^2 - p_{ey} \times p_y)^{0.5})$ = **299** N/mm^2

$P_{cy} = A \times p_{cy}$ = **560.3** kN

Check $F_c \le P_{cy}$ Pass - Compression

Slenderness less than 180

Cl. 4.7.4 & Annex C

Results summary

Summary of results

Material	Grade = **"S355"**	p_y = **355** N/mm^2	
Section	**"SHS 100x100x5.0"**	Classification	**"Plastic"**

Check	*Load*	*Capacity*	*Notes*	*Result*
Strut buckling (x-axis)	F_c = **25.0** kN	P_{cx} = **560.3** kN	L_{Ex} = **2.3** m **Slenderness < 180**	**Pass**
Strut buckling (y-axis)	F_c = **25.0** kN	P_{cy} = **560.3** kN	L_{Ey} = **2.3** m **Slenderness < 180**	**Pass**

LOAD CALCULATIONS FOR TIMBER RAFTERS:

SPAN OF RAFTERS = 3000mm

ASSUME RAFTERS = 50mm x 150mm

TIMBER GRADE = C24

JOISTS CENTERS = 400mm

LOADING:

DEAD LOAD ON TIMBER RAFTERS:

PITCHED ROOF = 1.26 kN/m^2 x SPAN OF ROOF IN MTS.

= 1.26 kN/m^2 x 0.40m

= 0.50 kN/m

THEREFORE TOTAL DEAD LOAD ON TIMBER RAFTERS = 0.50 kN/m

LIVE LOAD ON TIMBER RAFTERS:

PITCHED ROOF = 0.75 kN/m^2 x SPAN OF ROOF IN MTS.

= 0.75 kN/m^2 x 0.40m

= 0.30 kN/m

THEREFORE TOTAL LIVE LOAD ON TIMBER RAFTERS = 0.30 kN/m

LOAD CALCULATIONS FOR TIMBER RAFTERS:

Analysis for a simply-supported single-span timber beam to BS 5268

TEDDS calculation version 1.0.02

Span length & partial factors for loading

Span (mm)	Factors for moments & forces			Factors for deflection		
	γ_{fd}	γ_{fi}	γ_{fw}	γ_{dd}	γ_{di}	γ_{dw}
3000	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**

Loading data (unfactored)

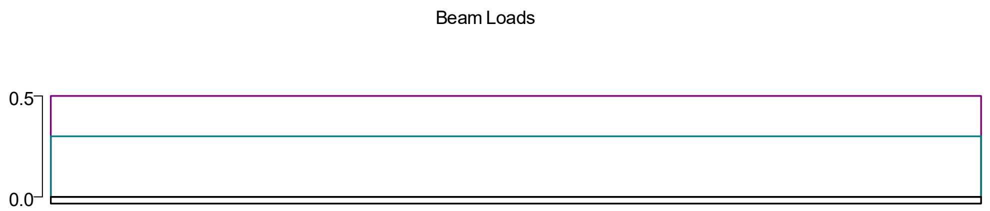
Ref.	Category	Type	Load kN/m	Position mm	Load kN/m	Position mm
1	**"Dead"**	UDL	**0.5**	**0**	-	**3000**
2	**"Imposed"**	UDL	**0.3**	**0**	-	**3000**

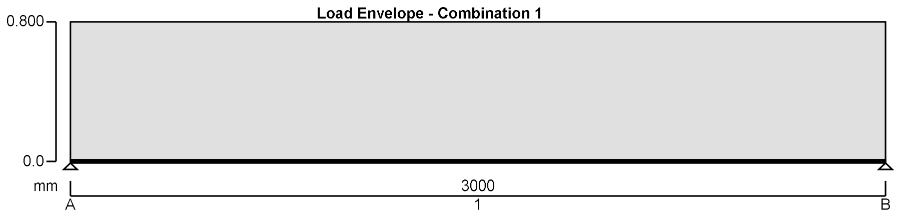
Analysis results - entire span

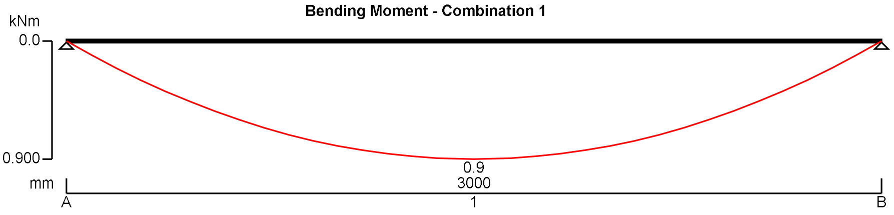
R_a	R_b	V	M		Deflection: δEI	
kN (fac)	kN (fac)	kN (fac)	kNm (fac)	Sense	kNm^3	Direction
1.2	**1.2**	**1.2**	**0.9**	**"Sagging"**	**0.84**	**"Down"**

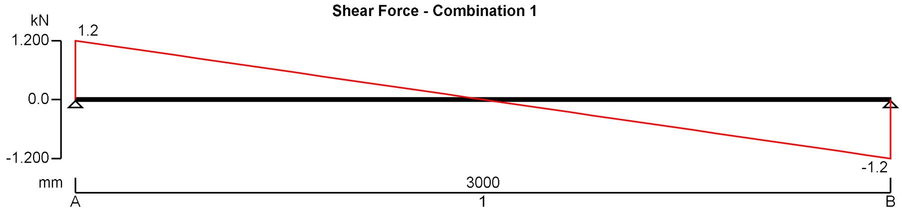
Unfactored support reactions

Support A	Dead load **-0.7** kN	Live load **-0.4** kN	Wind load **0.0** kN
Support B	Dead load **-0.7** kN	Live load **-0.4** kN	Wind load **0.0** kN









Member design checks for a simply-supported single-span timber beam to BS 5268

Timber member design BS 5268-2:2002

Summary of results				
Section size	D = **150** mm	B = **50** mm	A= **7500** mm^2	
Section properties (x-x)	I_{xx} = **14062500** mm^4	Z_{xx} = **187500** mm^3	r_{xx} = **43.3** mm	
(y-y)	I_{yy} = **1562500** mm^4	Z_{yy} = **62500** mm^3	r_{yy} = **14.4** mm	
Grade	**"C24"**	σ_c = **7.90** N/ mm^2		
Check	*Stress*	*Capacity*	*Notes*	*Result*
Bending stress	$\sigma_{m.a.para} = M / Z_{xx}$ = **4.80** N/mm^2	$\sigma_{m.adm.para}$ = **8.09** N/mm^2	Moment M = **0.9** kNm	**Pass**
Shear stress	τ_a = **0.24** N/mm^2	τ_{adm} = **0.71** N/mm^2	Shear V = **1.2** kN	**Pass**
Deflection	δ = **5.8** mm	14 mm		**Pass**
	δ / L_s = **0.00192**	0.003		**Pass**

LOAD CALCULATIONS FOR STRIP FOUNDATION:

ASSUMIND 600mm WIDE MASS CONCRETE FOUNDATION

ASSUMIND GOOD STRATA AT 1000mm DEEP MASS CONCRETE FOUNDATION

ASSUMED ECCENTRICITY = 0mm [TO BE CHECKED BY BUILDER ON SITE]

LOADING:

DEAD LOAD ON FOUNDATION:

BLOCK WORK = 14 kN/m^3 x THICKNES OF WALL x HEIGHT OF WALL IN MTS.
(100mm)
= 14 x 0.100 x 2.6m

= 3.64 kN/m

BRICK WORK = 22 kN/m^3 x THICKNES OF WALL x HEIGHT OF WALL IN MTS.
(100mm)
= 22 x 0.100 x 2.6m

= 5.72 kN/m

PITCHED ROOF = 1.26 kN/m^2 x SPAN OF ROOF IN MTS.

= 1.26 kN/m^2 x 1.5m

= 1.89 kN/m

THEREFORE TOTAL DEAD LOAD ON SF= 3.64 + 5.72 + 1.89 = 11.25 kN/m

LIVE LOAD ON STRIP FOUNDATION:

PITCHED ROOF = 0.75 kN/m^2 x SPAN OF ROOF IN MTS.

= 0.75 kN/m^2 x 1.5m

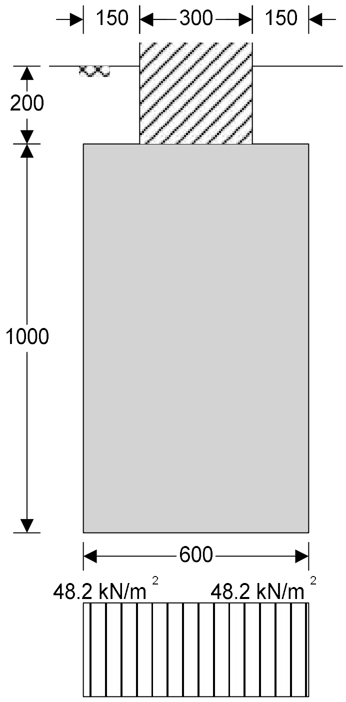
= 1.13 kN/m

THEREFORE TOTAL LIVE LOAD ON SF = 1.13 kN/m

STRUCTURAL REPORT FOR STRIP FOUNDATION:

STRIP FOOTING ANALYSIS AND DESIGN (BS8110-1:1997)

TEDDS calculation version 2.0.02.00



Strip footing details

Width of strip footing — B = **600** mm

Depth of strip footing — h = **1000** mm

Depth of soil over strip footing — h_{soil} = **200** mm

Density of concrete — ρ_{conc} = **23.6** kN/m^3

Load details

Load width — b = **300** mm

Load eccentricity — e_P = **0** mm

Soil details

Dense, moderately graded, sub-angular, gravel

Mobilisation factor — m = **1.5**

Density of soil — ρ_{soil} = **20.0** kN/m^3

Design shear strength — ϕ' = **25.0** deg

Design base friction — δ = **19.3** deg

Allowable bearing pressure — $P_{bearing}$ = **100** kN/m^2

Axial loading on strip footing

Dead axial load — P_G = **11.3** kN/m

Imposed axial load — P_Q = **1.1** kN/m

Wind axial load — P_W = **0.0** kN/m

Total axial load — P = **12.4** kN/m

Foundation loads

Dead surcharge load — F_{Gsur} = **0.000** kN/m^2

Imposed surcharge load — F_{Qsur} = **0.000** kN/m^2

Strip footing self weight — $F_{swt} = h \times \rho_{conc}$ = **23.600** kN/m^2

Soil self weight — $F_{soil} = h_{soil} \times \rho_{soil}$ = **4.000** kN/m^2

Total foundation load	$F = B \times (F_{Gsur} + F_{Qsur} + F_{swt} + F_{soil}) = \mathbf{16.6}$ kN/m

Calculate base reaction

Total base reaction	$T = F + P = \mathbf{28.9}$ kN/m

Eccentricity of base reaction in x	$e_T = (P \times e_P + M + H \times h) / T = \mathbf{0}$ mm

Base reaction acts within middle third of base

Calculate base pressures

$q_1 = (T / B) \times (1 - 6 \times e_T / B) = \mathbf{48.233}$ kN/m^2

$q_2 = (T / B) \times (1 + 6 \times e_T / B) = \mathbf{48.233}$ kN/m^2

Minimum base pressure	$q_{min} = \min(q_1, q_2) = \mathbf{48.233}$ kN/m^2

Maximum base pressure	$q_{max} = \max(q_1, q_2) = \mathbf{48.233}$ kN/m^2

PASS - Maximum base pressure is less than allowable bearing pressure

Material details

Characteristic strength of concrete	$f_{cu} = \mathbf{30}$ N/mm^2

Calculate base lengths

Left hand length	$B_L = B / 2 + e_P = \mathbf{300}$ mm

Right hand length	$B_R = B / 2 - e_P = \mathbf{300}$ mm

Calculate rate of change of base pressure

Length of base reaction	$B_x = B = \mathbf{600}$ mm

Rate of change of base pressure	$C_x = (q_1 - q_2) / B_x = \mathbf{0.000}$ kN/m^2/m

Calculate minimum depth of unreinforced strip footing

Average pressure to left of strip footing	$q_L = q_1 - C_x \times (B_L - b / 2) / 2 = \mathbf{48.233}$ kN/m^2

Minimum depth to left of strip footing	$h_{Lmin} = (B_L\text{-}b/2)\times\max(0.15\times[(q_L/1\text{ kN/m}^2)^2/(f_{cu}/1\text{ N/mm}^2)]^{1/4},1) = \mathbf{150}$ mm

Average pressure to right of strip footing	$q_R = q_2 + C_x \times (B_R - b / 2) / 2 = \mathbf{48.233}$ kN/m^2

Minimum depth to right of strip footing	$h_{Rmin} = (B_R\text{-}b/2)\times\max(0.15\times[(q_R/1\text{kN/m}^2)^2/(f_{cu}/1\text{N/mm}^2)]^{1/4},1) = \mathbf{150}$ mm

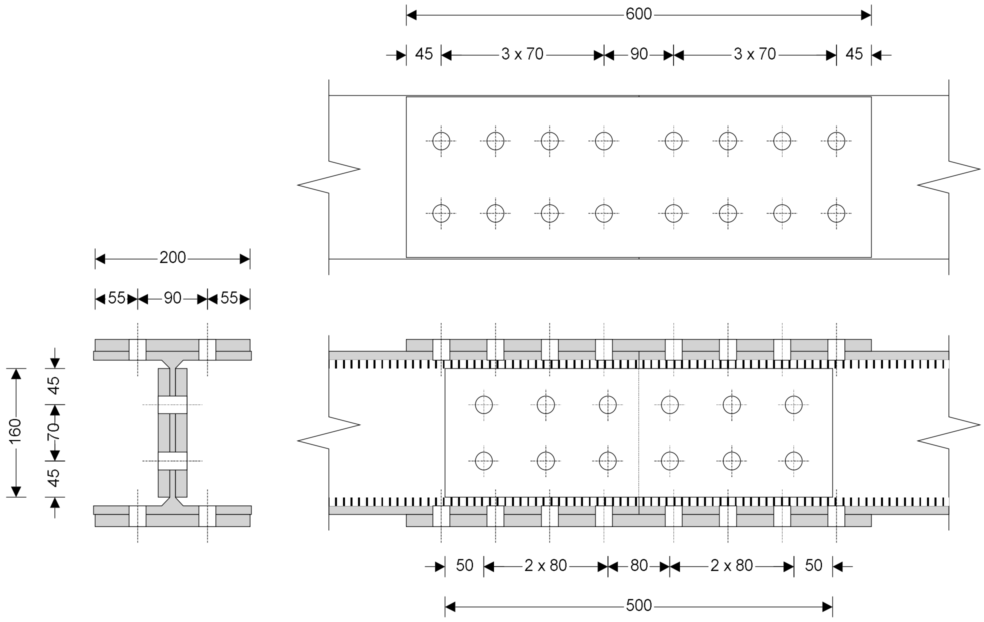
Minimum depth of unreinforced strip footing	$h_{min} = \max(h_{Lmin}, h_{Rmin}, 300\text{ mm}) = \mathbf{300}$ mm

PASS - Unreinforced strip footing depth is greater than minimum

SPLICE CONNECTION FOR STEEL BEAM 1 (203x203x46UC):

BOLTED COVER PLATE SPLICE CONNECTION TO BS5950-1:2000

TEDDS calculation version 1.0.02



Connection loads

Design moment	M = **66** kNm
Axial force in the member (compression +ve)	N = **0** kN
Shear force in the member	V = **173** kN

Steel beam details

Beam section classification	**UC 203x203x46**
Grade of steel section	**S355**
Section bearing strength	p_{bs_s} = **550** N/mm^2

General connection details

Grade of steel plate	**S355**
Plate bearing strength	p_{bs_p} = **550** N/mm^2
Bolt classification	**M20 (Torqued General Grade HSFG)**
Hole diameter	D_h = **22** mm
Bolt slip factor	μ = **0.50**
Hole type factor	K_s = 1.0

Flange plate details – plates bolted to one side of each flange

Thickness of flange plates	t_{fp} = **15** mm
Width of flange plates	b_{fp} = **200** mm
Length of flange plates	l_{fp} = **600** mm

Flange bolting details

Rows of flange bolts on each side of joint	n_{fb_r} = **4**
Bolts per row	n_{fb_p} = **2**
Total number of flange bolts each side of joint	$n_{fb} = n_{fb\_r} \times n_{fb\_p}$ = **8**
Spacing between rows of bolts	S_f = **70** mm

Spacing between rows of bolts across joint	S_{fc} = **90** mm
Spacing at end of flange plates	S_{fe} = **45** mm
Lateral spacing between central bolts	S_{flc} = **90** mm
Lateral spacing at edge of flange plates	S_{fle} = **55** mm

Web plate details - plates bolted to both sides of the web

Thickness of web plates	t_{wp} = **15** mm
Width of web plates	b_{wp} = **500** mm
Length of web plates	l_{wp} = **160** mm

Web bolting details

Rows of web bolts	n_{wb_r} = **2**
Bolts per row each side of joint	n_{wb_p} = **3**
Total number of web bolts each side of joint	$n_{wb} = n_{wb\_r} \times n_{wb\_p}$ = **6**
Spacing between rows of bolts	S_w = **70** mm
Spacing at end of web plates	S_{we} = **45** mm
Lateral spacing between bolts	S_{wl} = **80** mm
Lateral spacing between central bolts	S_{wlc} = **80** mm
Lateral spacing at edge of web plates	S_{wle} = **50** mm

Step 1 - Distribution of forces in member flanges

Forces in member tension flange	$T = [M / (D_b - T_b)] – N / 2$ = **343** kN
Forces in member compression flange	$C = [M / (D_b - T_b)] + N / 2$ = **343** kN
Force in the flange	$F_f = \max(T, C)$ = **343** kN

Step 2 - Calculate distribution of forces in member flanges

Check area of flange

Design strength of section	p_{ys} = **355** N/mm^2
Minimum required effective flange area	F_f / p_{ys} = **967** mm^2
Effective net area coefficient	K_e = **1.1**
Effective flange area	$A_{ef} = \min(K_e \times [B_b – (n_{fb\_p} \times D_h)] \times T_b, B_b \times T_b)$ = **1931** mm^2

PASS - Effective flange area is adequate

Check area of flange plates

Design strength of plates	p_{yp} = **355** N/mm^2
Minimum required effective flange plate area	F_f / p_{yp} = **967** mm^2
Effective flange plate area	$A_{ep} = \min(K_e \times [b_{fp} – (n_{fb\_p} \times D_h)] \times t_{fp}, b_{fp} \times t_{fp})$ = **2574** mm^2

PASS - Effective flange plate area is adequate

Step 3 - Design of flange bolts

Slip resistance of the bolt per interface	$S_{fb} = 1.1 \times K_s \times \mu \times P_p$ = **79.2** kN
Bearing capacity of the bolt in the flange	$P_{bg\_s} = 1.5 \times d \times T_b \times p_{bs\_s}$ = **181.5** kN
Bearing capacity of the bolt in the plate	$P_{bg\_p} = 1.5 \times d \times t_{fp} \times p_{bs\_p}$ = **247.5** kN
Average flange bolt end distance	$S_{fe\_ave} = S_{fe} + (n_{fb\_r} - 1) \times S_f / 2$ = **150** mm
Bearing capacity limit of the bolt in the plate	$P_{bg\_p\_lim} = 0.5 \times S_{fe\_ave} \times t_{fp} \times p_{bs\_p}$ = **618.8** kN
Bolt capacity	$P_s = \min(S_{fb}, P_{bg\_s}, P_{bg\_p}, P_{bg\_p\_lim})$ = **79.2** kN
Number of bolts required	$n_{fb\_req} = F_f / P_s$ = **4.3**
Number of bolts provided	n_{fb} = **8**

PASS - Flange plate bolting is adequate

Step 4 - Design of web plates and bolts

Check web plate in shear

Shear force in web plates	V = **173** kN
Gross shear area	$A_v = n_{wp} \times l_{wp} \times t_{wp}$ = **4800** mm^2

Net shear area (allowing for bolt holes)	$A_{v\_net} = n_{wp} \times (l_{wp} – n_{wb\_r} \times D_h) \times t_{wp} =$ **3480** mm^2
Net shear area limit	$0.85 \times A_v / K_e =$ **3709** mm^2
Net shear capacity of web plates	$p_{v\_net} = 0.7 \times K_e \times A_{v\_net} \times p_{yp} =$ **951** kN
Gross shear capacity of web plates	$p_{v\_gross} = 0.6 \times A_v \times p_{yp} =$ **1022** kN
Length of block shear face	$L_v = S_{we} + (n_{wb\_r} - 1) \times S_w =$ **115** mm
Length of block tension face	$L_t = S_{wle} + (n_{wb\_p} - 1) \times S_{wl} =$ **210** mm
Block shear coefficient	$k = if(n_{wb\_p} > 1, 2.5, 0.5) =$ **2.5**
Block shear capacity of web plates	$p_{v\_block} = 0.6 \times p_{yp} \times t_{wp} \times n_{wp} \times [L_v + K_e \times (L_t - k \times D_h)] =$ **1824** kN
Shear capacity of web plates	$p_v = \min(p_{v\_net}, p_{v\_gross}, p_{v\_block}) =$ **951** kN

PASS - Effective web plate area is adequate in shear

Check web plate in bending

Second moment of area of web plate	$I = (t_{wp} \times l_{wp}^3 / 12) – (n_{wb\_r} \times t_{wp} \times D_h^3 / 12) – (t_{wp} \times D_h \times K \times S_w^2)$
	$I =$ **4284880** mm^4
Distance from joint to centroid of bolt group	$a = [((n_{wb\_p} – 1) \times S_{wl}) + S_{wlc}] / 2 =$ **120** mm
Moment in web plate	$M_{wp} = V \times a =$ **20.8** kNm
Moment capacity of web plates	$M_{cap} = p_{yp} \times n_{wp} \times I / (l_{wp} / 2) =$ **38.0** kNm

PASS - Effective web plate area is adequate in bending

Check web plate bolts

Moment of inertia of bolt group	$I_{bg} =$ **32950** mm^2
Force on bolt due to direct shear	$F_v = V / n_{wb} =$ **28.8** kN
Vertical force on bolt due to moment	$F_{mv} = M_{wp} \times x / I_{bg} =$ **50.4** kN
Horizontal force on bolt due to moment	$F_{mh} = M_{wp} \times y / I_{bg} =$ **22.1** kN
Resultant bolt load	$F_r = \sqrt{((F_v + F_{mv})^2 + F_{mh}^2)} =$ **82.2** kN
Angle of the resultant bolt load	$\theta = atan(F_{mh} / (F_v + F_{mv})) =$ **15.6** deg
Minimum edge distance	$e_r = \min(S_{we} / \cos(\theta), S_{wle} / \cos(90 - \theta)) =$ **47** mm
Edge distance factor for web plate bearing	$K_{edge} = \min(e_r / (3 \times d), 1) =$ **0.8**
Slip resistance of the bolt per interface	$S_{fb} = 1.1 \times K_s \times \mu \times P_p =$ **79.2** kN
Bearing capacity of the bolt in the web	$P_{bg\_s} = 1.5 \times d \times t_b \times p_{bs\_s} =$ **118.8** kN
Bearing capacity of the bolt in the plate	$P_{bg\_p} = 1.5 \times K_{edge} \times d \times t_{wp} \times n_{wp} \times p_{bs\_p} =$ **385.4** kN
Bolt capacity	$P_s = \min(n_{wp} \times S_{fb}, P_{bg\_s}, P_{bg\_p}) =$ **118.8** kN

PASS - Web plate bolting is adequate

Connection summary

Beam classification	**UC 203x203x46**
Bolt classification	**M20 (Torqued General Grade HSFG)**
Flange plates	**600 mm x 200 mm x 15 mm to the outside of each flange**
Flange bolting	**16 No. total per flange - 4 No. rows of 2 No. bolts on each side of the joint**
Web plates	**160 mm x 500 mm x 15 mm on each side of the web**
Web bolting	**12 No. total - 2 No. rows of 3 No. bolts on each side of the joint**

Padstone Calculator

COMPLIES WITH LATEST EUROPEAN DESIGN CODES

structural calculations for padstones

Beam End Reaction = **13.70** kN (factored)
Factored Load at End of Beam

Variable Load Safety Factor = 1.5
Permanent Load Safety Factor = 1.35

Characteristic strength of masonry = **2.6** N/mm²

Width of beam end bearing = **100** mm
Length of beam end bearing = **200** mm

(Brickwork usually = 4.5 N/mm²)
(3.6N Blockwork usually = 2.6 N/mm²)
(A Engineering Brick = 13.2 N/mm²)
(B Engineering Brick = 10.5 N/mm²)
(Weak Brickwork = approx 2.8 N/mm²)
(7.3N Blockwork usually = 4.2 N/mm²)
(10.4N Blockwork usually = 5.4 N/mm²)

$\gamma m = 3.0$

Bearing Factor = **1.25**

Results

Maximum Bearing Stress =	1.08	N/mm²
Actual Bearing Stress =	0.69	N/mm²

Padstone Not Required

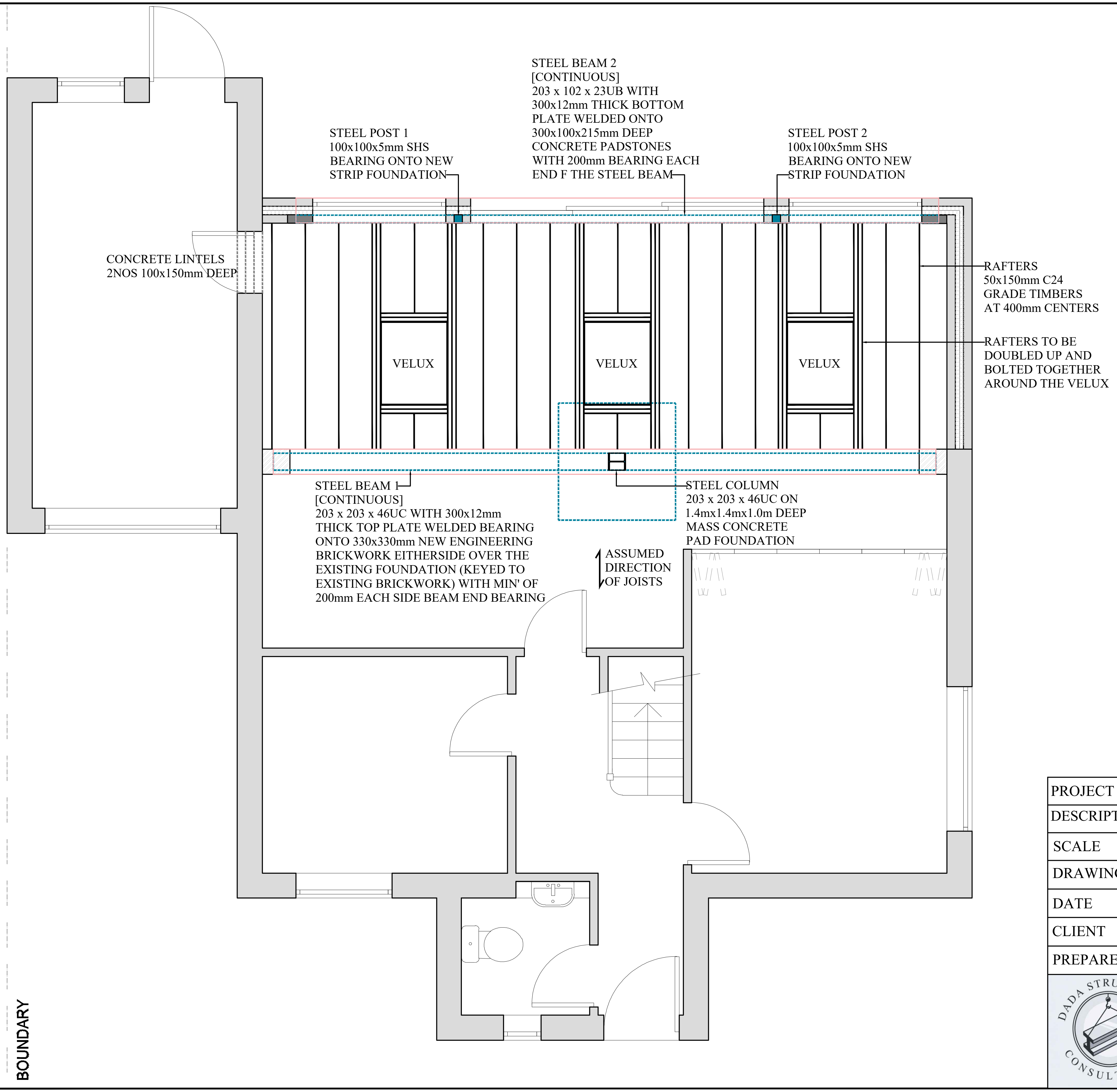
Padstone Results

Characteristic strength of Padstone = **30.0** N/mm²

Width of Padstone = **100** mm
Length of Padstone = **300** mm

(A Engineering Brick = 13.2 N/mm²)
(B Engineering Brick = 10.5 N/mm²)
(Concrete C15 = 15 N/mm²)
(Concrete C30 = 30 N/mm²)
(Concrete C40 = 40 N/mm²)
(Steel Plate = 275 N/mm²)

Allowable padstone stress =	12.50	N/mm²	
Stress under beam end bearing =	0.69	N/mm²	Therefore Padstone Stress OK
Allowable masonry stress =	1.08	N/mm²	
Stress under padstone =	0.46	N/mm²	Therefore Masonry Stress OK



GENERAL AND SAFETY NOTES:

1. ALL THE LOAD BEARING WALLS HAS TO BE CHECKED BY BUILDING OWNER / CONTRACTOR / BUILDING CONTROL OFFICER

2.EXISTING FOUNDATION ADEQUACY SHOULD BE CHECKED BY OWNER / CONTRACTOR / BUILDING CONTROL OFFICER PRIOR TO THE COMMENCEMENT OF WORK.

3. 30x5mm GALVANIZED IRON METAL STRAPS SHOULD BE PROVIDED WHERE EVER NECESSARY

4. IF NECESSARY CONTRACTOR MAY ALLOW FOR SPLICING OF STEEL WORK TO AID ERECTION.

5.FOR FIRE PROTECTION TO STEEL WORK REFER TO ARCHITECT DETAILS.

6.FOR ALL TEMPORARY WORKS INCLUDING DESIGN AND ERECTION CONTRACTOR IS RESPONSIBLE.

7.ALL THE TEMPORARY WORKS SHOULD BE CHECK BY SPECIALIST PRIOR TO ANY DEMO WORKS.

8. NOTE THAT ALL STEEL (SIZES & QUANTITY) SHOWN ON THE DRAWINGS ARE SHOULD BE CHECKED PRIOR TO START OF THE WORKS. IT IS THE RESPONSIBILITY OF THE CLIENT / CONTRACTOR TO NOTIFY THE DESIGNER OF ANY DISCREPANCIES.

PROJECT	70 KENTLEA ROAD, LONDON SE28 0JZ
DESCRIPTION	PROPOSED GROUND FLOOR STRUCTURAL PLAN
SCALE	NOT TO SCALE
DRAWINGS NO.	S01
DATE	13-07-2026
CLIENT	MR. SYED AFJAL ALI
PREPARED BY	MOHAMMED NAYEEM MSc. (Hons.)

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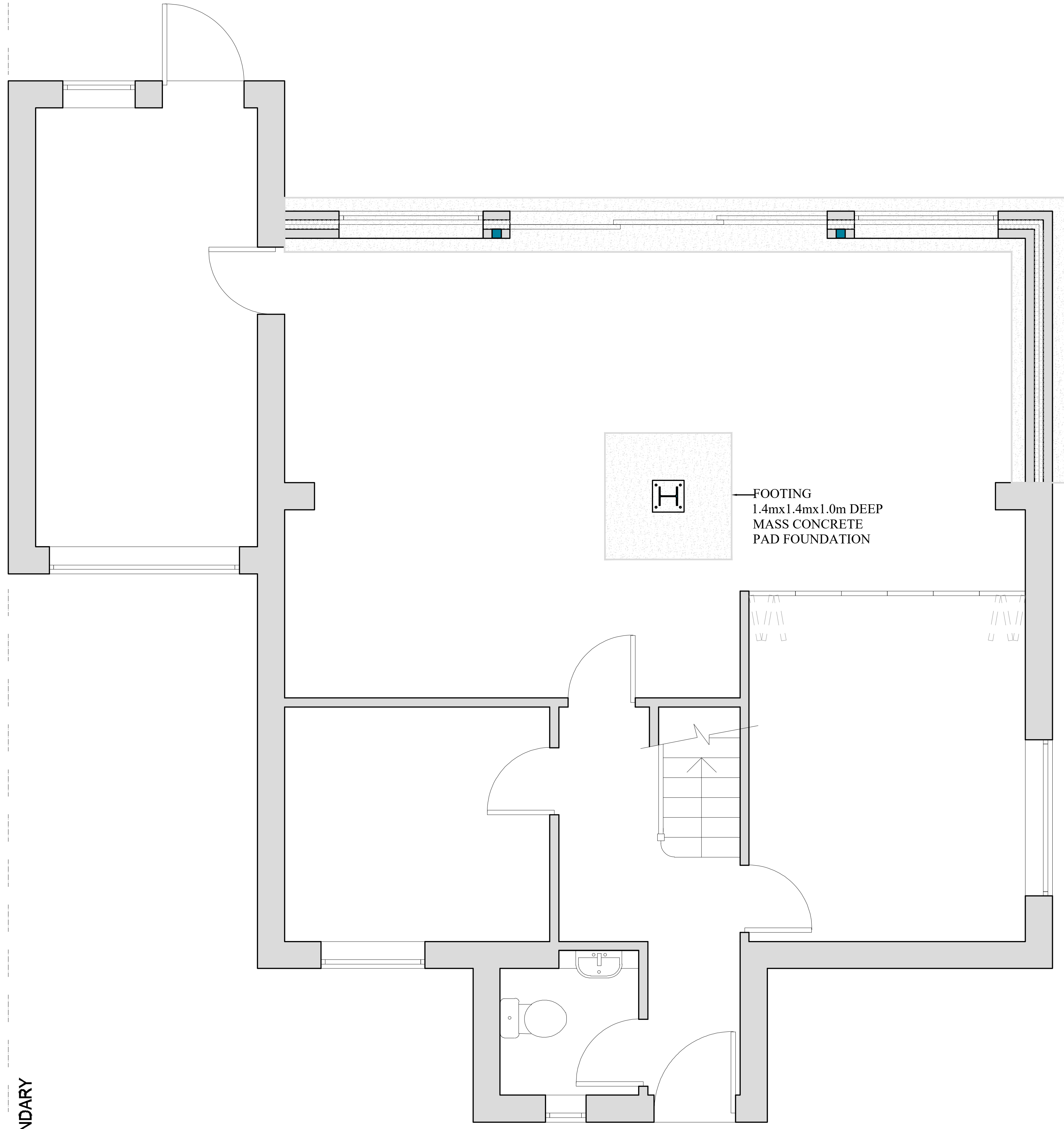

600mm WIDE MASS CONCRETE TAKEN DOWN TO LOAD BEARING SUB-SOIL A WITH A MINIMUM 1000mm DEPTH (TO BE CONFIRMED BY BUILDING CONTROL OFFICER)

FOUNDATION IS TO BE 600mm WIDE TRENCH CONCRETE FOOTING WITH 1:2:4 CONCRETE MIX.FOUNDATION TO BE 1M DEEP(TO BE DETERMINED BY BUILDING CONTROL).ANY DRAINS PASSING THROUGH FOUNDATION ARE TO BE PROTECTED USING UPVC SLEEVING AND 150X100mm RC BLOCK LINTEL.

FOUNDATION DEPTH ACCORDING TO NHBC FOUNDATION CALCULATOR.FINAL DESIGN AND DEPTHS TO BE DETERMINED BY BUILDING CONTROL

GENERAL AND SAFETY NOTES:

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2.EXISTING FOUNDATION ADEQUACY SHOULD BE CHECKED BY OWNER / CONTRACTOR / BUILDING CONTROL OFFICER PRIOR TO THE COMMENCEMENT OF WORK.

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PROJECT	70 KENTLEA ROAD, LONDON SE28 0JZ
DESCRIPTION	PROPOSED FOUNDATION STRUCTURAL PLAN
SCALE	NOT TO SCALE
DRAWINGS NO.	S02
DATE	13-07-2026
CLIENT	MR. SYED AFJAL ALI
PREPARED BY	MOHAMMED NAYEEM MSc. (Hons.)

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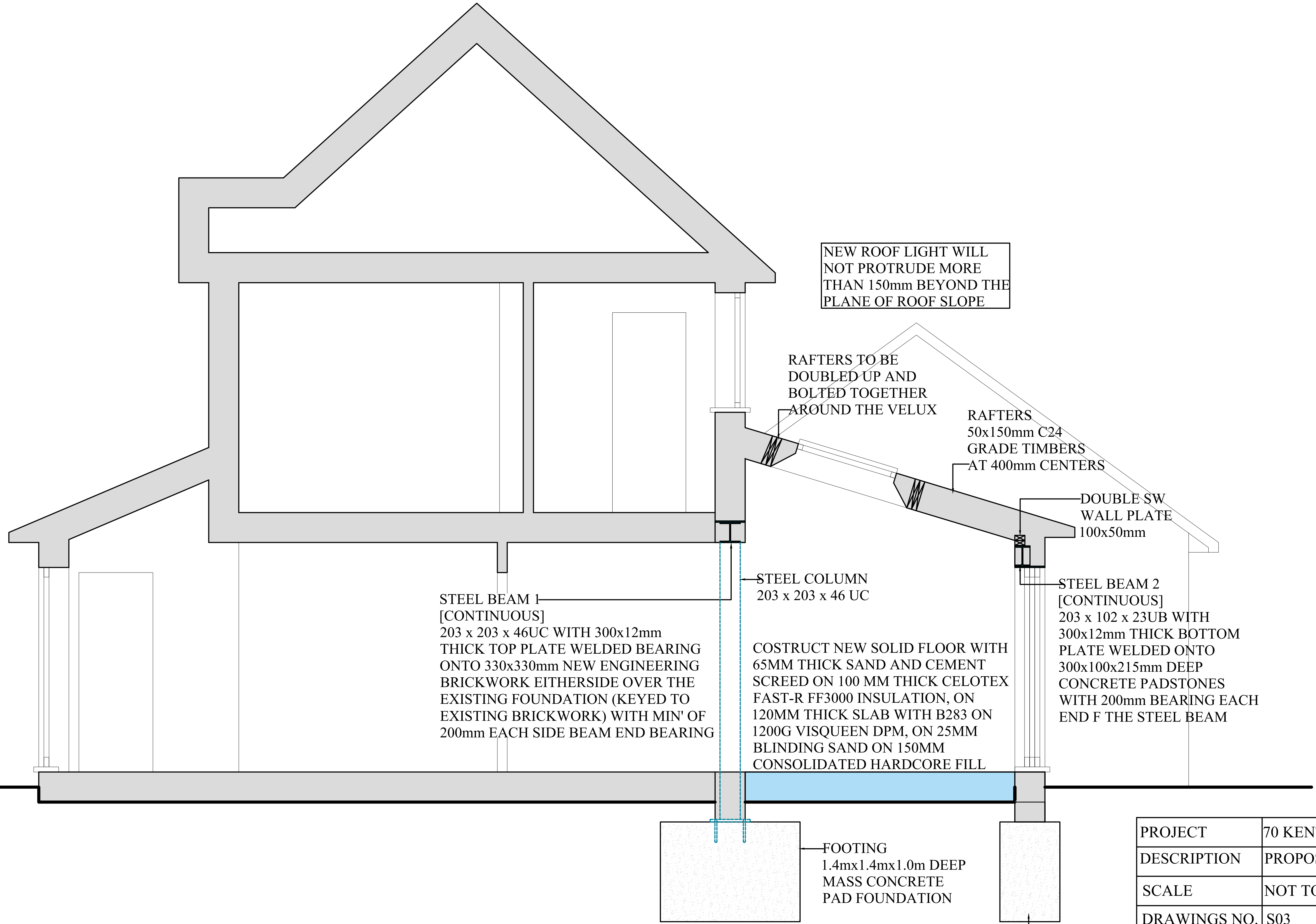

FOUNDATION DEPTH ACCORDING TO NHBC FOUNDATION CALCULATOR.FINAL DESIGN AND DEPTHS TO BE DETERMINED BY BUILDING CONTROL

600mm WIDE MASS CONCRETE TAKEN DOWN TO LOAD BEARING SUB-SOIL A WITH A MINIMUM 1000mm DEPTH (TO BE CONFIRMED BY BUILDING CONTROL OFFICER)

FOUNDATION IS TO BE 600mm WIDE TRENCH CONCRETE FOOTING WITH 1:2:4 CONCRETE MIX.FOUNDATION TO BE 1M DEEP(TO BE DETERMINED BY BUILDING CONTROL).ANY DRAINS PASSING THROUGH FOUNDATION ARE TO BE PROTECTED USING UPVC SLEEVING AND 150X100mm RC BLOCK LINTEL.

PROJECT	70 KENTLEA ROAD, LONDON SE28 0JZ
DESCRIPTION	PROPOSED SECTION
SCALE	NOT TO SCALE
DRAWINGS NO.	S03
DATE	13-07-2026
CLIENT	MR. SYED AFJAL ALI
PREPARED BY	MOHAMMED NAYEEM MSc. (Hons.)
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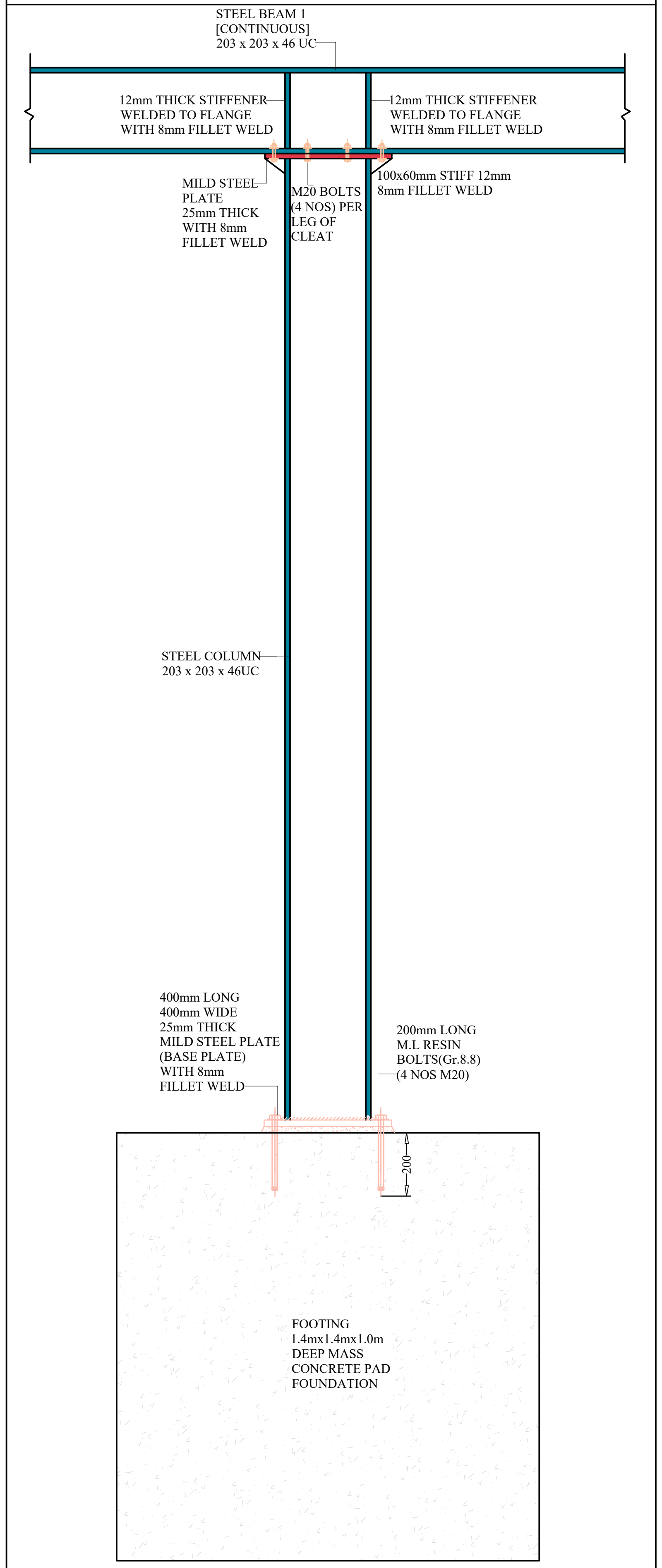
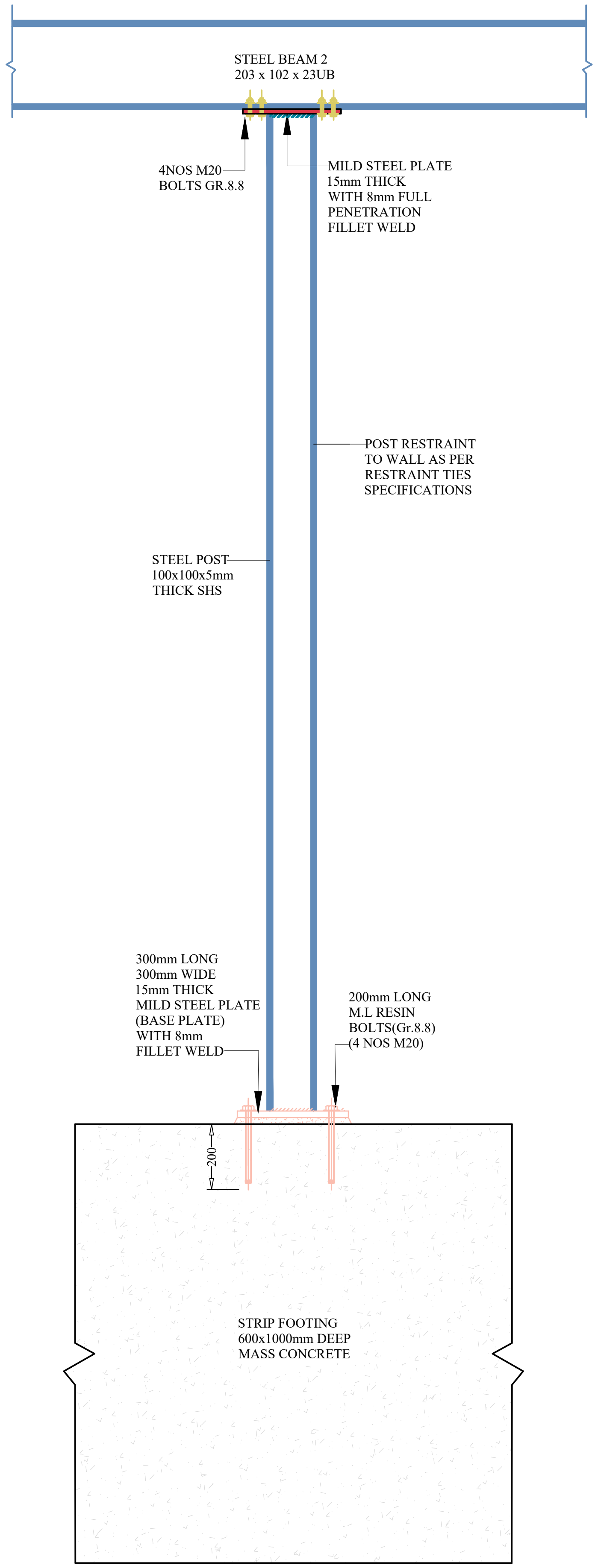
STEEL BEAM 1 TO STEEL COLUMN TO FOOTING CONNECTION DETAILS



STEEL BEAM 2 TO STEEL POST TO STRIP FOOTING CONNECTION DETAILS



PROJECT	70 KENTLEA ROAD, LONDON SE28 0JZ
DESCRIPTION	CONNECTIONS
SCALE	NOT TO SCALE
DRAWINGS NO.	S04
DATE	13-07-2026
CLIENT	MR. SYED AFJAL ALI
PREPARED BY	MOHAMMED NAYEEM MSc. (Hons.)
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