Design Team:

Example Structural Calculations

designteam

Calc By	Project	Job No.	
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Introduction:

The following calculations are to size structural members for the new rear extension at

Calculations are prepared in accordance with the following Standards:

Weight of building materials	BS 648
Loading	BS 6399, BS-EN 1991
Structural concrete	BS 8110
Structural steel	BS 5950, BS-EN 1993
Structural timber	BS 5268, BS-EN 1995

Loading Data:

Timber Pitched Roof:

Dead Load

- Tiles 0.75 kNm^{-2}
- Battens 0.05 kNm^{-2}
- Felt 0.05 kNm^{-2}
- Rafters 0.20 kNm^{-2}
- Ceiling/Services 0.15 kNm^{-2}

Total 1.20 kNm^{-2} (on slope)

1.20 kNm^{-2} / cos (20) = 1.28 kNm^{-2} (on plane)

Imposed Load 0.60 kNm^{-2}

Timber Floor:

Dead Load

- Boarding 0.15 kNm^{-2}
- Joists 0.20 kNm^{-2}
- Ceiling/Services 0.15 kNm^{-2}

Total 0.50 kNm^{-2}

Imposed Load 1.50 kNm^{-2}

Brickwork:

Dead Load γ_{brick} = 18 kNm^{-3}

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Timber Stud Wall:

Dead Load

- Stud wall 1.00 kNm^{-2}

Wind load on walls:

Imposed Load 0.70 kNm^{-2}

Fire resistance:

Fire resistance period R = 30 min

Exposure to fire Exposed on more than one side

Soil bearing capacity: P = 91kPa

Note:

Calculations to be checked by local Authority before work commences. Client to ensure all of contractors' works on site to comply with and meet Approval of the relevant British Standards and the Local Authority including Building Control and Planning Departments. Maximum liability for these works is equal to the fee being paid.
Dimensions: Note that all dimensions shown on the drawings are indicative and should be checked prior to start of the works on site. It is the responsibility of the client to notify the Designer of any discrepancies. The same applies to the alignment of walls and general layouts. All existing foundations and lintels to be exposed to verify suitability and to be checked for adequacy and/or replaced or surrounded in 150mm concrete cover if necessary. Prior to commencement a trial hole and /or soil report/investigation and an inspection of any trees in the areas may be required.
Structural Calculations Loadings: We do not have access to any plans of building as they were originally constructed. Consequently structural calculations are based upon assumptions. The client should be aware that the calculations have limitations based upon the information forthcoming. We are not privy to the original construction details of the building from the time of its construction.
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. The dimensions of all steel sections required should be measured on site by the client (or their representative contractor or steelwork fabricator). Where the wall above is wider than the steel (supporting it) below there is the need to weld a 12mm thick plate to the top flange/s of the steels to ensure that the wall/steel interface are the identical width. All below ground steelwork must be concrete encased with minimum 100mm thickness.

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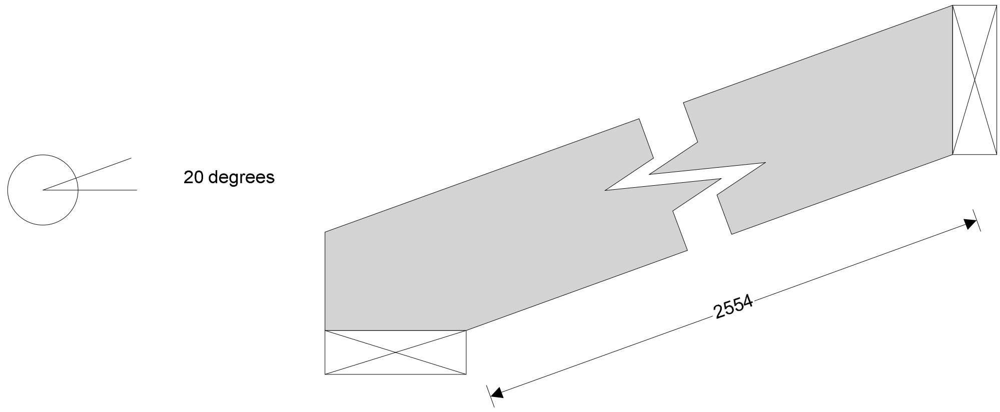
TIMBER RAFTER 'T-1'

TIMBER RAFTER DESIGN (BS5268-2:2002)

TEDDS calculation version 1.0.03



Rafter details

Breadth of timber sections	b = **47** mm	Depth of timber sections	h = **150** mm
Rafter spacing	s = **400** mm	Rafter span	**Single span**
Clear length of span on slope	L_{cl} = **2554** mm	Rafter slope	α = **20.0** deg
Timber strength class	**C24**		

Section properties

Cross sectional area of rafter	A = **7050** mm^2	Section modulus	Z = **176250** mm^3
Radius of gyration	r = **43** mm	Second moment of area	I = **13218750** mm^4

Loading details

Rafter self weight	F_j = **0.02** kN/m	Dead load on slope	F_d = **1.20** kN/m^2
Imposed snow load on plan	F_u = **0.75** kN/m^2	Imposed point load	F_p = **0.90** kN

Modification factors

Section depth factor	K_7 = **1.08**	Load sharing factor	K_8 = **1.10**

Consider long term load condition

Load duration factor	K_3 = **1.00**	Total UDL perp. to rafter	F = **0.474** kN/m
Notional bearing length	L_b = **5** mm	Effective span	L_{eff} = **2559** mm

Check bending stress

Permissible bending stress	σ_{m_adm} = **8.904** N/mm^2	Applied bending stress	σ_{m_max} = **2.200** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Permissible comp. stress	σ_{c_adm} = **5.848** N/mm^2	Applied compressive stress	σ_{c_max} = **0.330** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Combined loading check **0.308 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Permissible shear stress	τ_{adm} = **0.781** N/mm^2	Applied shear stress	τ_{max} = **0.129** N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection δ_{adm} = **7.677** mm | Total deflection δ_{max} = **1.951** mm

PASS - Total deflection within permissible limits

<u>Consider medium term load condition</u>

Load duration factor K_3 = **1.25** | Total UDL perp. to rafter F = **0.739** kN/m

Notional bearing length L_b = **8** mm | Effective span L_{eff} = **2562** mm

Check bending stress

Permissible bending stress σ_{m_adm} = **11.130** N/mm^2 | Applied bending stress σ_{m_max} = **3.438** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Permissible comp. stress σ_{c_adm} = **6.978** N/mm^2 | Applied compressive stress σ_{c_max} = **0.515** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Combined loading check **0.390 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Permissible shear stress τ_{adm} = **0.976** N/mm^2 | Applied shear stress τ_{max} = **0.201** N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection δ_{adm} = **7.685** mm | Total deflection δ_{max} = **3.054** mm

PASS - Total deflection within permissible limits

<u>Consider short term load condition</u>

Load duration factor K_3 = **1.50** | Total UDL perp. to rafter F = **0.474** kN/m

Notional bearing length L_b = **8** mm | Effective span L_{eff} = **2562** mm

Check bending stress

Permissible bending stress σ_{m_adm} = **13.355** N/mm^2 | Applied bending stress σ_{m_max} = **5.280** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Permissible comp. stress σ_{c_adm} = **7.965** N/mm^2 | Applied compressive stress σ_{c_max} = **0.374** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Combined loading check **0.449 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Permissible shear stress τ_{adm} = **1.172** N/mm^2 | Applied shear stress τ_{max} = **0.309** N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection δ_{adm} = **7.687** mm | Total deflection δ_{max} = **4.174** mm

PASS - Total deflection within permissible limits

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STEEL BEAM 'SB-1'

Loading

Dead load:

Pitched roof: $g_1 = 1.28\ kN/m^2 \times 2.60\ m =$ **3.33** kN/m

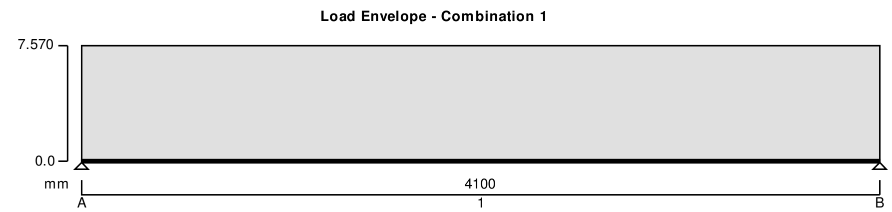
Imposed load:

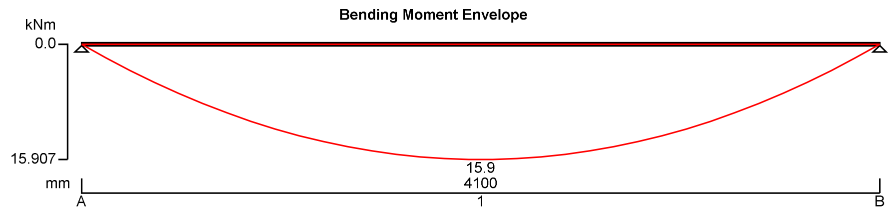
Pitched roof: $q_1 = 0.60\ kN/m^2 \times 2.60\ m =$ **1.56** kN/m

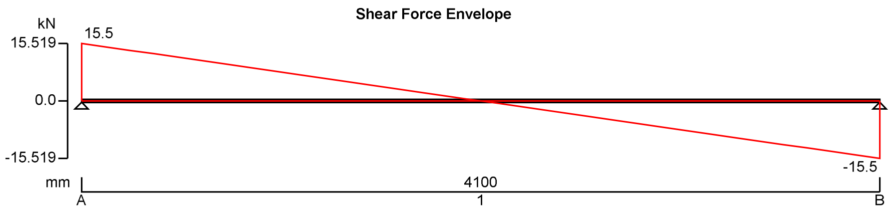
STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.05







Support conditions

Support A — Vertically restrained; Rotationally free

Support B — Vertically restrained; Rotationally free

Applied loading

Beam loads — Dead self weight of beam × 1; Dead full UDL 3.33 kN/m; Imposed full UDL 1.56 kN/m

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Load combinations

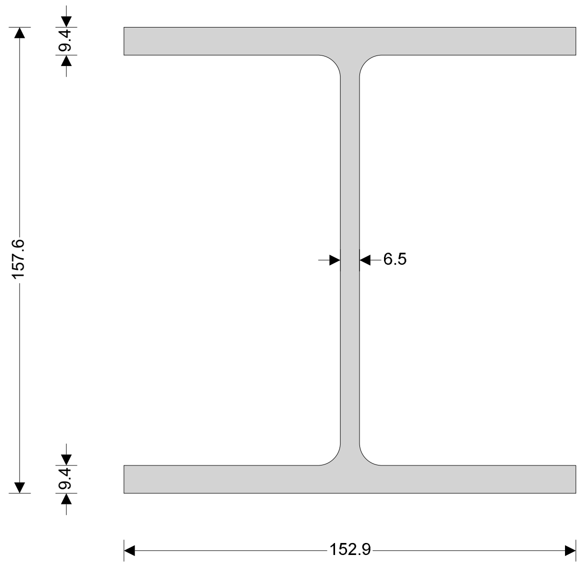
Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
	Span 1	Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **15.9** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **15.5** kN	V_{min} = **-15.5** kN
Deflection	δ_{max} = **5.3** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **15.5** kN	R_{A_min} = **15.5** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **7.4** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **3.2** kN	
Maximum reaction at support B	R_{B_max} = **15.5** kN	R_{B_min} = **15.5** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **7.4** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **3.2** kN	

Section details

Section type **UC 152x152x30 (BS4-1)** Steel grade **S275**



Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **1.00** Section classification **Plastic**

Shear capacity - Section 4.2.3

Design shear force F_v = **15.5** kN Design shear resistance P_v = **169** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M = **15.9** kNm Moment capacity low shear M_c = **68.1** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **41.4** kNm M_b / m_{LT} = **41.4** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection δ_{lim} = **11.389** mm

Maximum deflection δ = **5.323** mm

PASS - Maximum deflection does not exceed deflection limit

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STEEL BEAM 'SB-2'

Loading

Dead load:

Reaction from SB-1: g_1 = 7.40 kN / 1.80 m = **4.11** kN/m

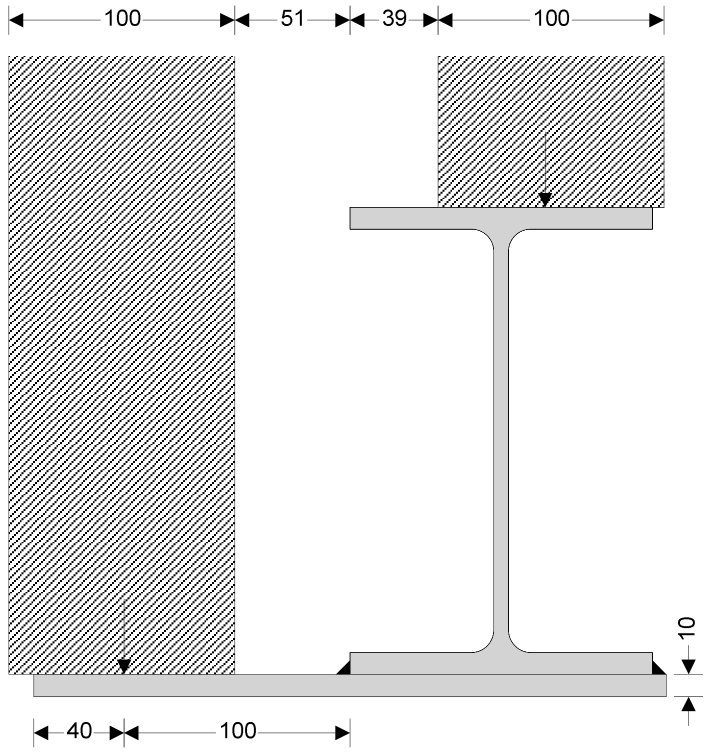
Imposed load:

Reaction from SB-1: q_1 = 3.20 kN / 1.80 m = **1.78** kN/m

STEEL MASONRY SUPPORT

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

Tedds calculation version 1.0.04



Steel member details

Torsion beam	UB 203x133x30
Masonry support plate	User
Steel grade of support plate	S275
Design strength of support plate	p_{ysb} = **275** N/mm^2
Modulus of elasticity	E = 205000 N/mm^2
Constant	$\varepsilon = \sqrt{(275N/mm^2 / p_{ysb})}$ = **1.000**
Length of plate beyond beam	l_h = **140** mm
Total length of plate	l_{plate} = **280** mm
Thickness of plate	t_{sb} = **10** mm
Width of main beam	B_{mb} = **134** mm
Area of plate	$A_{sbu} = t_{sb} \times l_{plate}$ = **2800.0** mm^2
Distance from weld position to CoG	$c_{yysb} = l_h / 2 - (l_{plate} - l_h) / 2$ = **0** mm

Supported materials detail

Density of masonry on main beam	$\rho_{m,mb}$ = **18.0** kN/m^3
Width of masonry on main beam	b_{mmb} = **100** mm
Height of masonry on main beam	h_{mmb} = **900** mm
Eccentricity of main beam material	e_{mb} = **39** mm
Add dead force main beam (not from masonry)	P_{Gaddmb} = **4.1** kN/m
Add live force main beam (not from masonry)	P_{Qaddmb} = **1.8** kN/m
Density of masonry on support beam	$\rho_{m,sb}$ = **18.0** kN/m^3
Width of masonry on support beam	b_{msb} = **100** mm
Height of masonry on support beam	h_{msb} = **900** mm
Add dead force support beam (not from masonry)	P_{Gaddsb} = **0.0** kN/m
Add live force support beam (not from masonry)	P_{Qaddsb} = **0.0** kN/m

Geometry

Cavity width	c = **90** mm
Supported width of masonry	$d_m = l_h + e_{mb} - c$ = **89** mm

Biaxial stress effects in the plate (SCI-P-110)

Maximum overall bending moment	M_x = **29.1** kNm
Dist to NA combined section (CoG torsion beam)	$y_{e,all} = (D_{mb} + t_{sb}) \times A_{sbu} / (2 \times (A_{mb} + A_{sbu}))$ = **46** mm
Second moment of area of combined section	$I_{xx,all} = (I_{xxmb} + A_{mb} \times y_{e,all}^2) + A_{sbu} \times (D_{mb} / 2 + t_{sb} / 2 - y_{e,all})^2$ = **4794** cm^4
Elastic section modulus of combined section	$Z_{xx,all} = I_{xx,all} / (D_{mb} / 2 + t_{sb} - y_{e,all})$ = **709.66** cm^3
Section modulus of plate	$Z_{xx,plate} = 1m \times t_{sb}^2 / (6 \times 1m)$ = **16.67** cm^3/m
Eccentricity of support beam masonry	e_1 = **100** mm
Force of masonry on support plate	$P_1 = (b_{msb} \times h_{msb} \times \rho_{m,sb} + P_{Gaddsb}) \times \gamma_{fG} + P_{Qaddsb} \times \gamma_{fQ}$ = **2.3** kN/m
Bending at heel	$M_{x,plate} = P_1 \times e_1$ = **0.2** kNm/m
Moment capacity of plate	$M_c = 1.2 \times Z_{x,plate} \times p_{ysb}$ = **5.5** kNm/m

PASS - Design strength exceeds stress at heel

Longitudinal stress due to overall bending	$\sigma_1 = M_x / Z_{xx,all}$ = **41.1** N/mm^2
Constant relating to Von Mises curve	$c_{fp} = (4 \times p_{ysb}^2 - 3 \times \sigma_1^2)^{0.5}$ = **545.4** N/mm^2
Transverse bending stress ratio limit	$\alpha_{ts} = (c_{fp}^2 - \sigma_1^2) / (2 \times c_{fp} \times p_{ysb})$ = **0.986**
Transverse bending stress ratio	$\alpha_{ls} = M_{x,plate} / M_c$ = **0.041**

PASS - Transverse bending stress ratio less than allowable limit

Deflection at toe

Unfactored force on support angle	$P_{1SLS} = b_{msb} \times h_{msb} \times \rho_{m,sb} + P_{Gaddsb} + P_{Qaddsb}$ = **1.6** kN/m
Distance from weld to load position	$a_m = e_1$ = **100** mm
Length of load resultant to edge of plate	$b_m = l_h - e_1$ = **40** mm
Dist from weld to load position as ratio of length	$a_l = a_m / (a_m + b_m)$ = **0.714**
Effective second moment of inertia	$I_{eff\_def} = t_{sb}^3 / 12$ = **83333** mm^4/m
Deflection at toe	$\delta = (a_l^2 \times (3 - a_l) / 6) \times (P_{1SLS} \times (a_m + b_m)^3) / (E_{S5950} \times I_{eff\_def})$ = **0.05** mm
Deflection limit	δ_{lim} = **1.99** mm

PASS - Deflection is within specified criteria

Weld details - assume a full length weld and that the plate acts as a propped cantilever with the prop at the weld position and the fixed end at the centre of the torsion beam

Leg length of weld	s_{weld} = **6** mm
Throat size of weld	$a_{weld} = 1/\sqrt{(2)} \times s_{weld}$ = **4.2** mm
Shear force at weld position	$R_A = P_1 \times maX((1 + (3 \times e_1) / (2 \times B_{mb} / 2)), 1.4)$ = **7.3** kN/m

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Maximum possible force in plate	$R_p = (l_h + B_{mb}) \times t_{sb} \times p_{ysb}$ = **753.2** kN
Longitudinal shear between beam and plate	$R_l = 2 \times R_p / L$ = **367.4** kN/m
Horizontal shear between beam and plate	$R_h = P_l \times e_l / (s_{weld} / 2 + t_{sb} / 2)$ = **28.4** kN/m
Resultant weld force	$R_{weld} = (R_A^2 + R_l^2 + R_h^2)^{0.5}$ = **0.369** kN/mm
Strength of weld (Table 37)	p_{weld} = **220.0** N/mm^2
Capacity of full length weld	$p_{c,weld} = a_{weld} \times p_{weld}$ = **0.933** kN/mm

PASS - Capacity of weld exceeds resultant force on weld

Torsional loading ULS

Loading of support beam masonry	$w_{1ULS} = (h_{msb} \times b_{msb} \times \rho_{m,sb} + P_{Gaddsb}) \times \gamma_{fG} + P_{Qaddsb} \times \gamma_{fQ}$ = **2.27** kN/m
Loading of main beam masonry	$w_{2ULS} = (h_{mmb} \times b_{mmb} \times \rho_{m,mb} + P_{Gaddmb}) \times \gamma_{fG} + P_{Qaddmb} \times \gamma_{fQ}$ = **10.87** kN/m
Self weight of support beam	$w_{3ULS} = A_{sbu} \times \rho_{sb} \times \gamma_{fG}$ = **0.31** kN/m

Torsional loading SLS

Loading of support beam masonry	$w_{1SLS} = h_{msb} \times b_{msb} \times \rho_{m,sb} + P_{Gaddsb} + P_{Qaddsb}$ = **1.62** kN/m
Loading of main beam masonry	$w_{2SLS} = h_{mmb} \times b_{mmb} \times \rho_{m,mb} + P_{Gaddmb} + P_{Qaddmb}$ = **7.51** kN/m
Self weight of support beam	$w_{3SLS} = A_{sbu} \times \rho_{sb}$ = **0.22** kN/m

Eccentricities

Distance to shear centre of main beam	e_{0mb} = **0** mm
Eccentricity of support beam masonry	$e_{1mb} = (B_{mb} + b_{msb}) / 2 + c - e_{mb}$ = **168** mm
Eccentricity of main beam masonry	$e_{2mb} = (B_{mb} - b_{mmb}) / 2 - e_{mb}$ = **-22** mm
Eccentricity of support beam	$e_{3mb} = B_{mb} / 2 + c_{yysb}$ = **67** mm

Torsional effects

Applied torque (ULS)	$T_{qULS} = abs(w_{1ULS} \times e_{1mb} + w_{2ULS} \times e_{2mb} + w_{3ULS} \times e_{3mb})$ = **0.16** kNm/m
Total torque (ULS)	$T_q = T_{qULS} \times L$ = **0.66** kNm
Applied torque (SLS)	$T_{qSLS} = abs(w_{1SLS} \times e_{1mb} + w_{2SLS} \times e_{2mb} + w_{3SLS} \times e_{3mb})$ = **0.12** kNm/m
Total torque (SLS)	$T_{qu} = T_{qSLS} \times L$ = **0.50** kNm

<u>STEEL BEAM TORSION DESIGN</u>

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

Tedds calculation version 2.0.02

Section details

Section type	UB 203x133x30
Steel grade	S275
Design stength	$p_{yw} = p_y$ = **275** N/mm^2
Constant	$\varepsilon = \sqrt{(275 \text{ N/mm}^2 / p_y)}$ = **1.000**

Geometry - Beam unrestrained against lateral-torsional buckling between supports.

Effective span	L = **4100** mm
Length of segment for LT buckling	L_{LT} = **4100** mm

Compression flanges laterally unrestrained

Partial torsional restraint against rotation about longitudinal axis provided by connection of bottom flange to supports

Effective length for LT buckling	$L_{E\_LT} = L_{LT} \times 1.0 + 2 \times D$ = **4514** mm

Loading - Torsional loading comprises only full-length uniformly distributed load(s)

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

Applied shear force	F_{vy} = **28.4** kN

Maximum bending moment | $M_{LT} = M_x =$ **29.14** kNm
Applied torque | $T_q =$ **0.66** kNm
Minor axis bending moment | $M_y = 0$ kNm
Compression force | $F_c = 0$ kN

Equivalent uniform moment factors

EUM factor (Cl. 4.3.6.6 and T18) | $m_{LT} =$ **1.000**

Torsional deflection parameters

Beam is torsion fixed and warping free at each end. (as defined in SCI-P-057 section 2.1.6) - Appendix B case 4

Dist along the beam for first derivative of twist | $z_1 = 0$ mm
Dist along the beam for second derivative of twist | $z_2 = L / 2 =$ **2050** mm
First derivative of angle of twist | $\phi'_1 = T_q / (G \times J) \times a / L \times [L^2 / (2 \times a) \times (1 / L - 2 \times z_1 / L^2) + \sinh(z_1 / a) - \tanh(L / (2 \times a)) \times \cosh(z_1 / a)] \times 1$ rads = **2.21×10^{-2}** rads/m
Third derivative of angle of twist | $\phi'''_1 = T_q / (G \times J \times a^2) \times a/L \times [\sinh(z_1 / a) - \tanh(L / (2 \times a)) \times \cosh(z_1 / a)] \times 1$ rads = **-1.99×10^{-2}** rads/m^3
Angle of twist | $\phi_2 = T_q \times a / (G \times J) \times a / L \times [L^2 / (2 \times a^2) \times (z_2 / L - z_2^2 / L^2) + \cosh(z_2 / a) - \tanh(L / (2 \times a)) \times \sinh(z_2 / a) - 1] \times 1$ rads = **0.028** rads
Second derivative of angle of twist | $\phi''_2 = T_q / (G \times J \times a) \times a / L \times [\cosh(z_2 / a) - \tanh(L / (2 \times a)) \times \sinh(z_2 / a) - 1] \times 1$ rads = **-1.52×10^{-2}** rads/m^2

Design parameters

Total angle of twist | $\phi = abs(\phi_2) =$ **0.028** rads
First derivative of ϕ | $\phi' = abs(\phi'_1) =$ **2.21×10^{-2}** rads/m
Second derivative of ϕ | $\phi'' = abs(\phi''_2) =$ **1.52×10^{-2}** rads/m^2
Third derivative of ϕ | $\phi''' = abs(\phi'''_1) =$ **1.99×10^{-2}** rads/m^3

Section classification

$b / T =$ **7.0**
$d / t =$ **26.9**
$r_{1s} = \min(1.0, \max(-1.0, F_c / (d \times t \times p_{yw}))) =$ **0.000**
$r_{2s} = F_c / (A_g \times p_{yw}) =$ **0.000**

Section classification is plastic

Shear capacity (parallel to y-axis)

Design shear force | $F_{vy} =$ **28.4** kN
Design shear resistance (Cl. 4.2.3) | $P_{vy} = 0.6 \times p_y \times A_{vy} =$ **218.4** kN

Pass - Shear

Moment capacity (x-axis)

Design bending moment | $M_x =$ **29.1** kNm
Moment capacity | $M_{cxu} = p_y \times S_x =$ **86.5** kNm
Moment capacity low shear (Cl. 4.2.5.1) | $M_{cx} = \min(p_y \times S_x, 1.2 \times p_y \times Z_x) =$ **86.5** kNm

Pass - Moment capacity exceeds design bending moment

Lateral torsional buckling

Effective length for lateral torsional buckling | $L_{E\_LT} =$ **4514** mm
Slenderness ratio | $\lambda = L_{E\_LT} / r_y =$ **142**
Buckling parameter | $u =$ **0.881**
Flange ratio | $\eta = 0.5$
Torsional index | $x =$ **21.5**
Slenderness factor | $v = 1 / (1 + 0.05 \times (\lambda / x)^2)^{0.25} =$ **0.75**

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Ratio - cl 4.3.6.9	$\beta_w = 1.0 =$ **1.000**
Equvalent slenderness – cl 4.3.6.7	$\lambda_{LT} = u \times v \times \lambda \times \sqrt{(\beta_w)} =$ **94**
Limiting slendernes – Annex B2.2	$\lambda_{L0} = 0.4 \times \sqrt{(\pi^2 \times E_{S5950} / p_y)} =$ **34**
Euler stress	$p_E = \pi^2 \times E_{S5950} / \lambda_{LT}^2 =$ **230** N/mm^2
Perry factor	$\eta_{LT} = \max(7.0 \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) =$ **0.42**
	$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 =$ **300287454.68**
Bending strength	$p_b = p_E \times p_y / (\phi_{LT} + \sqrt{(\phi_{LT}^2 - p_E \times p_y)}) =$ **136** N/mm^2
Buckling resistance moment	$M_b = p_b \times S_x =$ **42.8** kNm
Max moment governing buckling resistance	$M_{LT} =$ **29.1** kNm
Equiv uniform moment factor for LTB	$m_{LT} =$ **1.00**
	$M_b / m_{LT} =$ **42.8** kNm

Pass - lat. tors. buckling

Buckling under combined bending & torsion -SCI-P-057 section 2.3

For simplicity, a conservative check is applied using the maximum stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Span factor	$L / a =$ **4.22**
Angle of twist	$\phi =$ **0.028** rads
Second derivative of ϕ	$\phi'' =$ **15.2×10^{-3}** rads/m^2
Induced minor axis moment	$M_{yt} = M_x \times \phi / 1$ rad $=$ **0.80** kNm
Normal stress at flange tip due to M_{yt}	$\sigma_{byt} = M_{yt} / Z_y =$ **14** N/mm^2
Normal stress at flange tip due to warping	$\sigma_w = E_{S5950} \times W_{n0} \times \phi'' / 1$ rad $=$ **21** N/mm^2
Interaction index	$i_b = M_x \times m_{LT} / M_b + (\sigma_{byt} + \sigma_w) / p_y \times (1 + 0.5 \times M_x \times m_{LT} / M_b) =$ **0.85**

Pass - Combined bending and torsion check satisfied

Local capacity under combined bending & torsion

For simplicity, a conservative check is applied using the maximum stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Max. direct stress due to M_x	$\sigma_{bx} = M_x / Z_x =$ **104** N/mm^2
Combined stress - eqn 2.22	$\sigma_{bx} + \sigma_{byt} + \sigma_w =$ **139** N/mm^2
Design strength	$p_y =$ **275** N/mm^2

Pass - Local capacity

Combined shear stresses - SCI-P-057 section 2.3

For simplicity, a conservative check is applied using the maximum shear stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Max shear stresses due to bending in web	$\tau_{bw} = F_{vy} \times Q_w / (I_x \times t) =$ **24** N/mm^2
Max shear stresses due to bending in flange	$\tau_{bf} = F_{vy} \times Q_f / (I_x \times T) =$ **6** N/mm^2
Max shear stresses due to torsion in web	$\tau_{tw} = \mathrm{abs}(G \times t \times \phi' / 1\mathrm{rad}) =$ **11** N/mm^2
Max shear stresses due to torsion in flange	$\tau_{tf} = \mathrm{abs}(G \times T \times \phi' / 1\ \mathrm{rad}) =$ **17** N/mm^2
Max shear stresses due to warping in flange	$\tau_{wf} = \mathrm{abs}(-E_{S5950} \times S_{w1} \times \phi''' / 1\ \mathrm{rad} / T) =$ **1** N/mm^2
Amp shear stress torsion & warping in web	$\tau_{vtw} = \tau_{tw} \times (1 + 0.5 \times M_x \times m_{LT} / M_b) =$ **15** N/mm^2
Amp shear stress torsion & warping in flange	$\tau_{vtf} = (\tau_{tf} + \tau_{wf}) \times (1 + 0.5 \times M_x \times m_{LT} / M_b) =$ **24** N/mm^2

Combined shear stresses due to bending, torsion & warping:

Combined shear stresses in web	$\tau_w = \tau_{bw} + \tau_{vtw} =$ **39** N/mm^2
Combined shear stresses in flange	$\tau_f = \tau_{bf} + \tau_{vtf} =$ **30** N/mm^2
Shear strength	$p_v = 0.6 \times p_y =$ **165** N/mm^2

Pass - Combined shear stresses

Deflection

Maximum y-axis deflection δ_{y_max} = **6.0** mm

Deflection limit - cl. 2.5.2 δ_{lim} = min(L/ k_δ, δ_{lim_abs}) = **10.0** mm

Pass - Deflection within specified limit

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GROUND FLOOR STEEL FRAME

Loading

- **SB-3**

Dead load:

Pitched roof: $g_1 = 1.28\ kN/m^2 \times 2.60\ m = \mathbf{3.33}\ kN/m$

Timber floor: $g_2 = 0.50\ kN/m^2 \times 1.40\ m \times 2 = \mathbf{1.40}\ kN/m$

Solid masonry wall: $g_3 = 18\ kN/m^3 \times 0.25\ m \times 2.90\ m = \mathbf{13.05}\ kN/m$

Tiled stud wall *(possible loft)*: $g_4 = 1.00\ kN/m^2 \times 2.60\ m = \mathbf{2.60}\ kN/m$

Total_Dead = $g_1 + g_2 + g_3 + g_4 = \mathbf{20.38}\ kN/m$

Imposed load:

Pitched roof: $q_1 = 0.60\ kN/m^2 \times 2.60\ m = \mathbf{1.56}\ kN/m$

Timber floor: $q_2 = 1.50\ kN/m^2 \times 1.40\ m \times 2 = \mathbf{4.20}\ kN/m$

Total_Imposed = $q_1 + q_2 = \mathbf{5.76}\ kN/m$

- **SB-4**

Dead load:

Solid masonry wall: $g_1 = 18\ kN/m^3 \times 0.25\ m \times 2.90\ m = \mathbf{13.05}\ kN/m$

Tiled stud wall *(possible loft)*: $g_2 = 1.00\ kN/m^2 \times 2.60\ m = \mathbf{2.60}\ kN/m$

Reaction from SB-1: $g_3 = 7.40\ kN / 1.80\ m = \mathbf{4.11}\ kN/m$

Total_Dead = $g_1 + g_2 + g_3 = \mathbf{19.76}\ kN/m$

Imposed load:

Reaction from SB-1: $q_1 = 3.20\ kN / 1.80\ m = \mathbf{1.78}\ kN/m$

Wind load:

Wind: $w_1 = 0.70\ kN/m^2 \times 9.80\ m^2 = \mathbf{6.86}\ kN$

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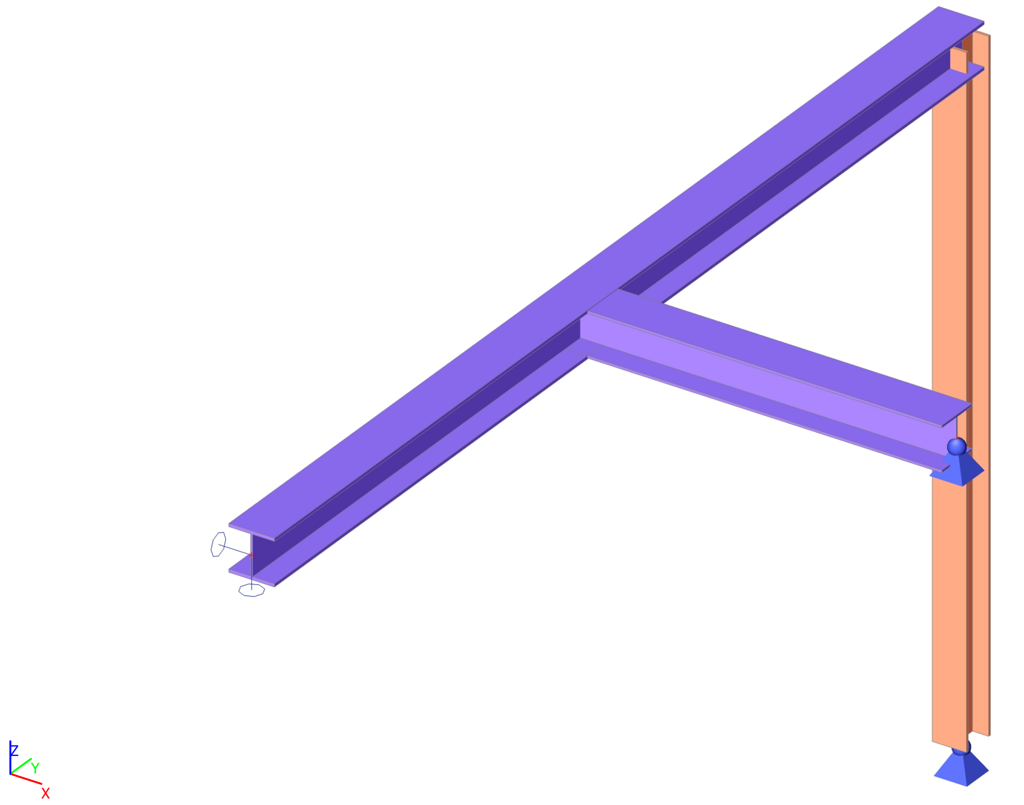
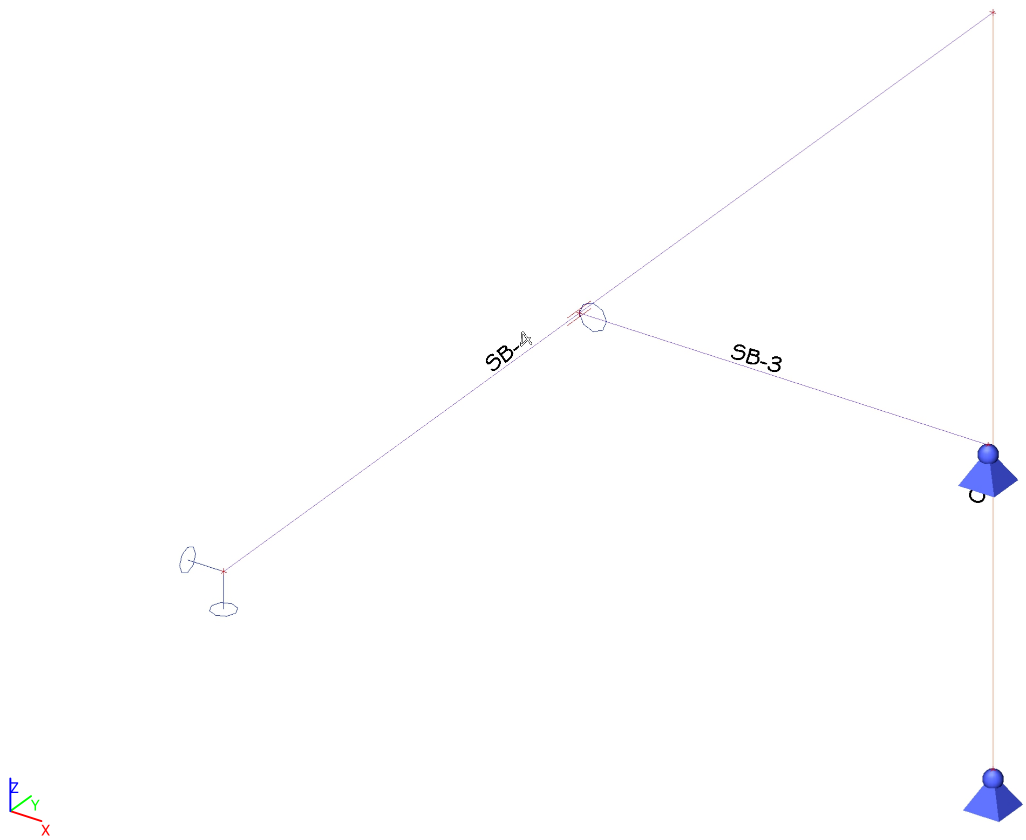
Analysis model





Members 1D

Name	Cross-section	Material	Length [m]	Beg. node	End node
SB-4	SB-4 - UC203/203/60	S 275	4.80	N1	N2
SB-3	SB-3 - UC203/203/46	S 275	1.70	N3	N4
C-1	C-1 - UC152/152/37	S 275	3.00	N5	N2

Note: All lengths of steel members are for analysis purposes only, accurate lengths are to be measured on site prior to ordering steel members.

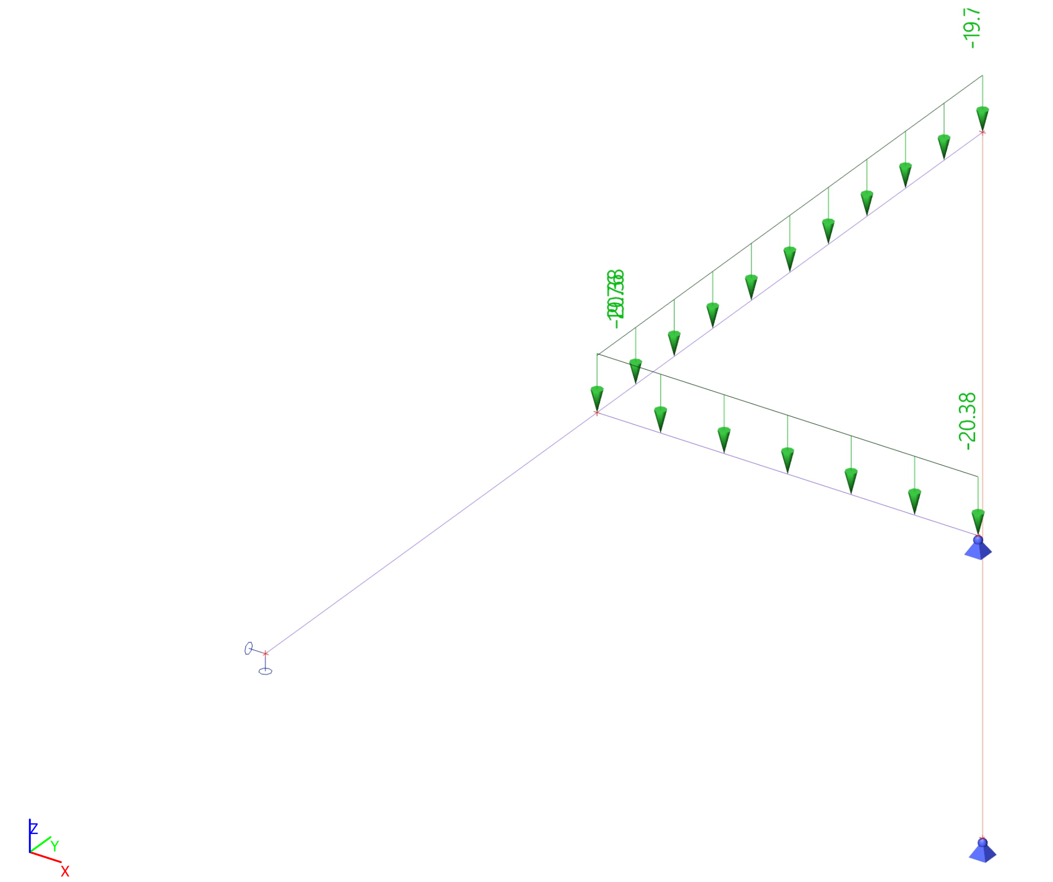
Nodal supports

Name	Node	System	Type	X	Y	Z	Rx	Ry	Rz
Sn1	N4	GCS	Standard	Rigid	Rigid	Rigid	Free	Free	Free
Sn2	N1	GCS	Standard	Rigid	Free	Rigid	Free	Free	Free
Sn3	N5	GCS	Standard	Rigid	Rigid	Rigid	Free	Free	Free

Load cases

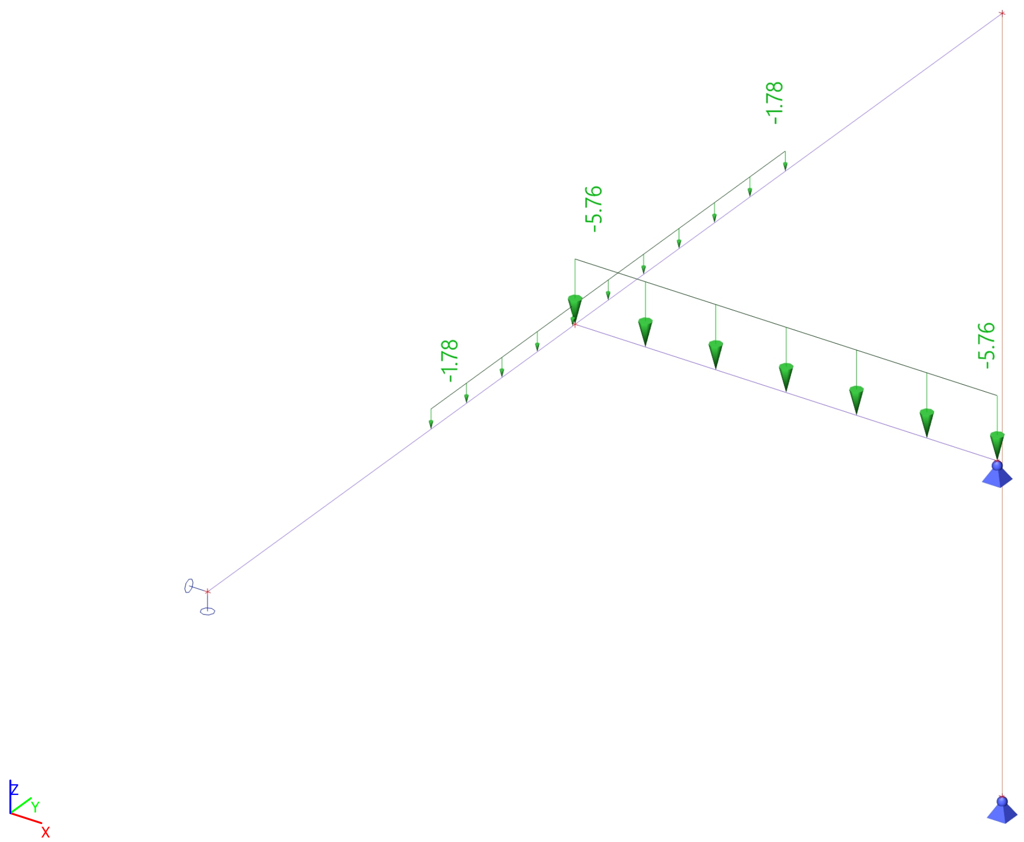
Name	Description Spec	Action type Load type	Load group	Direction
LC1	Self Weight	Permanent Self weight	LG1	-Z

Name	Description Spec	Action type Load type	Load group
LC2	Dead Load	Permanent Standard	LG1

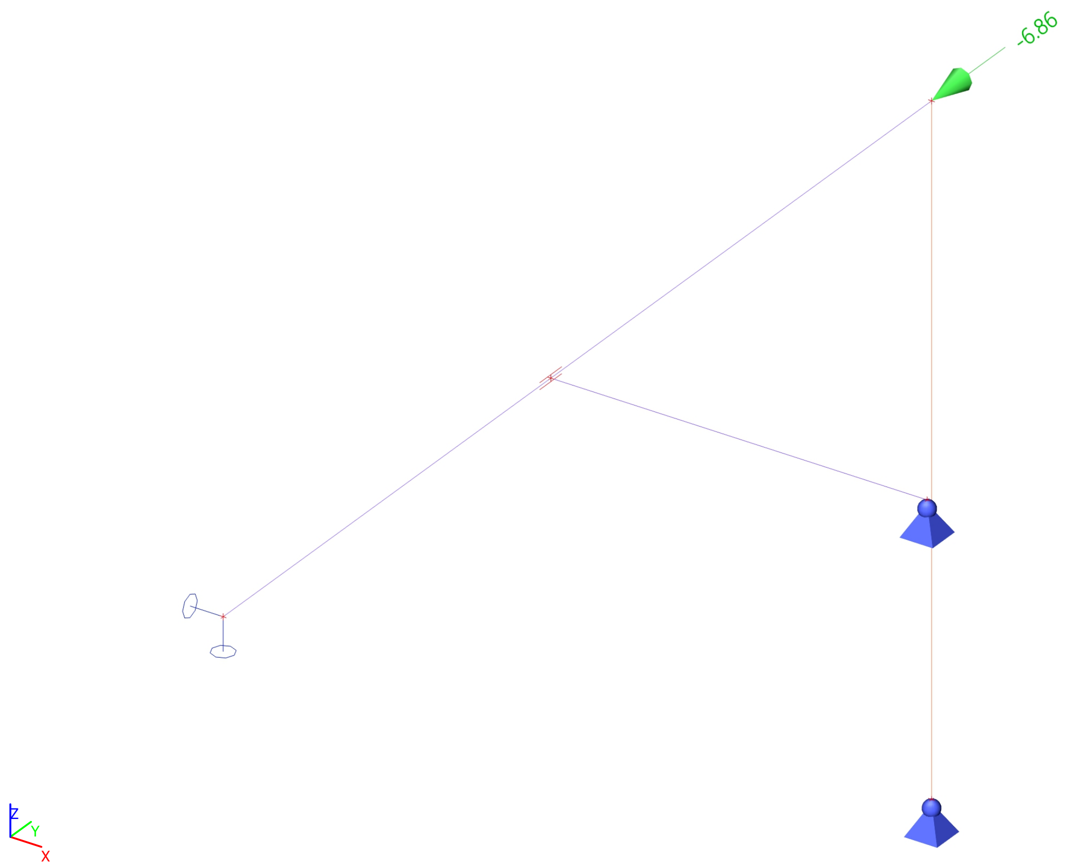


Name	Description Spec	Action type Load type	Load group	Duration	Master load case
LC3	Imposed Load	Variable	LG2	Medium	None

Name	Description Spec	Action type Load type	Load group	Duration	Master load case
	Standard	Static			



Name	Description Spec	Action type Load type	Load group	Duration	Master load case
LC4	Wind	Variable	LG3	Medium	None
	Standard	Static			



Member loads

Line force

Name	Member Load case	Type System	Dir Distribution	Value - P_1 [kN/m] Value - P_2 [kN/m]	Pos x_1 Pos x_2	Coor Loc	Orig	Ecc ey [m] Ecc ez [m]
LF1	SB-3	Force	Z	-20.38	0.000	Rela	From start	0.00
	LC2 - Dead Load	GCS	Uniform		1.000	Length		0.00
LF2	SB-4	Force	Z	-19.76	0.000	Abso	From end	0.00
	LC2 - Dead Load	GCS	Uniform		2.580	Length		0.00
LF3	SB-3	Force	Z	-5.76	0.000	Rela	From start	0.00
	LC3 - Imposed Load	GCS	Uniform		1.000	Length		0.00
LF4	SB-4	Force	Z	-1.78	1.310	Abso	From end	0.00
	LC3 - Imposed Load	GCS	Uniform		3.450	Length		0.00

Point force in node

Name	Node	Load case	System	Dir	Type	Value - F [kN]
F1	N2	LC4 - Wind	GCS	Y	Force	-6.86

Combinations

Name	Description	Type	Load cases	Coeff. [-]
CO1	Strength	Envelope - ultimate	LC1 - Self Weight	1.40
			LC2 - Dead Load	1.40
CO2	Strength	Envelope - ultimate	LC1 - Self Weight	1.20
			LC2 - Dead Load	1.20
			LC3 - Imposed Load	1.20
			LC4 - Wind	1.20
CO3	Strength	Envelope - ultimate	LC1 - Self Weight	1.40
			LC2 - Dead Load	1.40

Name	Description	Type	Load cases	Coeff. [-]
			LC3 - Imposed Load	1.60
CO4	Strength	Envelope - ultimate	LC1 - Self Weight	1.00
			LC2 - Dead Load	1.00
			LC4 - Wind	1.40
CO5	Service	Envelope - ultimate	LC1 - Self Weight	1.00
			LC2 - Dead Load	1.00
CO6	Service	Envelope - ultimate	LC1 - Self Weight	1.00
			LC2 - Dead Load	1.00
			LC3 - Imposed Load	1.00
			LC4 - Wind	1.00
CO7	Service	Envelope - ultimate	LC1 - Self Weight	1.00
			LC2 - Dead Load	1.00
			LC4 - Wind	1.00
CO8	Service	Envelope - ultimate	LC1 - Self Weight	1.00
			LC2 - Dead Load	1.00
			LC3 - Imposed Load	1.00

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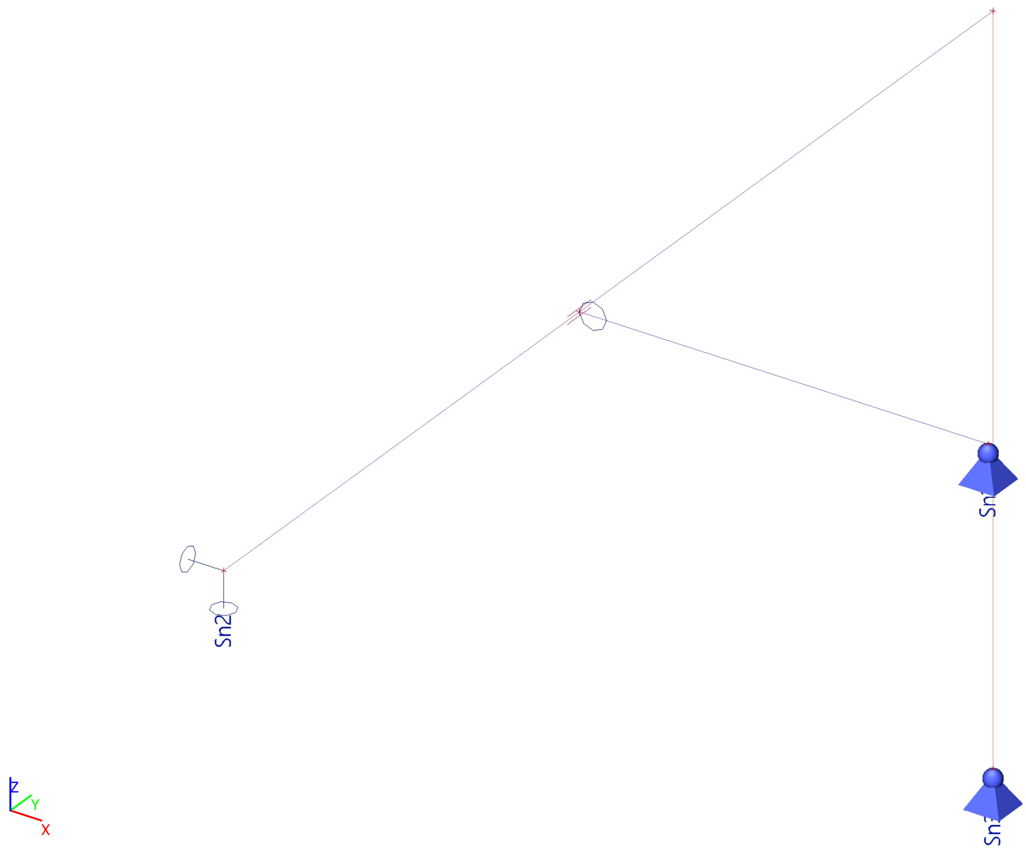
RESULTS - REACTIONS



Reactions - Self weight + Dead load

Linear calculation
Combination: CO5
System: Global
Extreme: Member
Selection: All
Nodal reactions

Name	Case	R_x [kN]	R_y [kN]	R_z [kN]	M_x [kNm]	M_y [kNm]	M_z [kNm]	e_x [mm]	e_y [mm]
Sn1/N4	CO5/1	**3.62**	**4.73**	**17.71**	**0.00**	**0.00**	**0.00**	0.0	0.0
Sn2/N1	CO5/1	**-3.62**	**0.00**	**21.67**	**0.00**	**0.00**	**0.00**	0.0	0.0
Sn3/N5	CO5/1	**0.00**	**-4.73**	**50.93**	**0.00**	**0.00**	**0.00**	0.0	0.0

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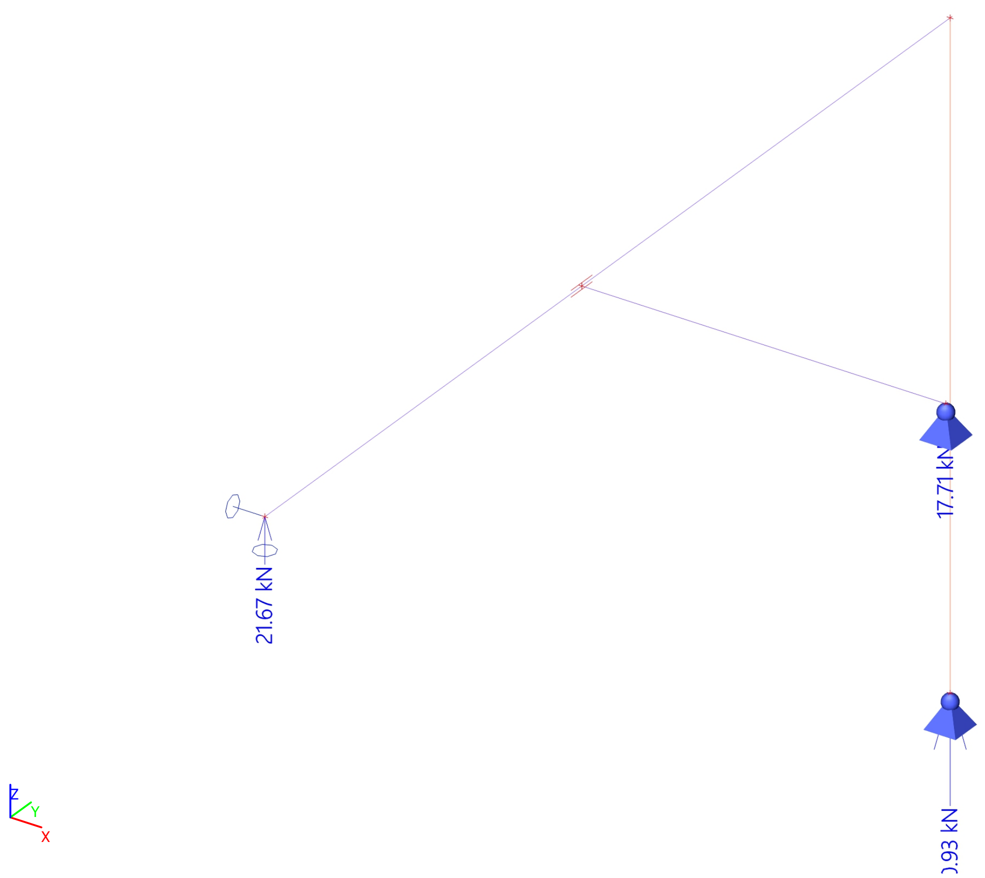
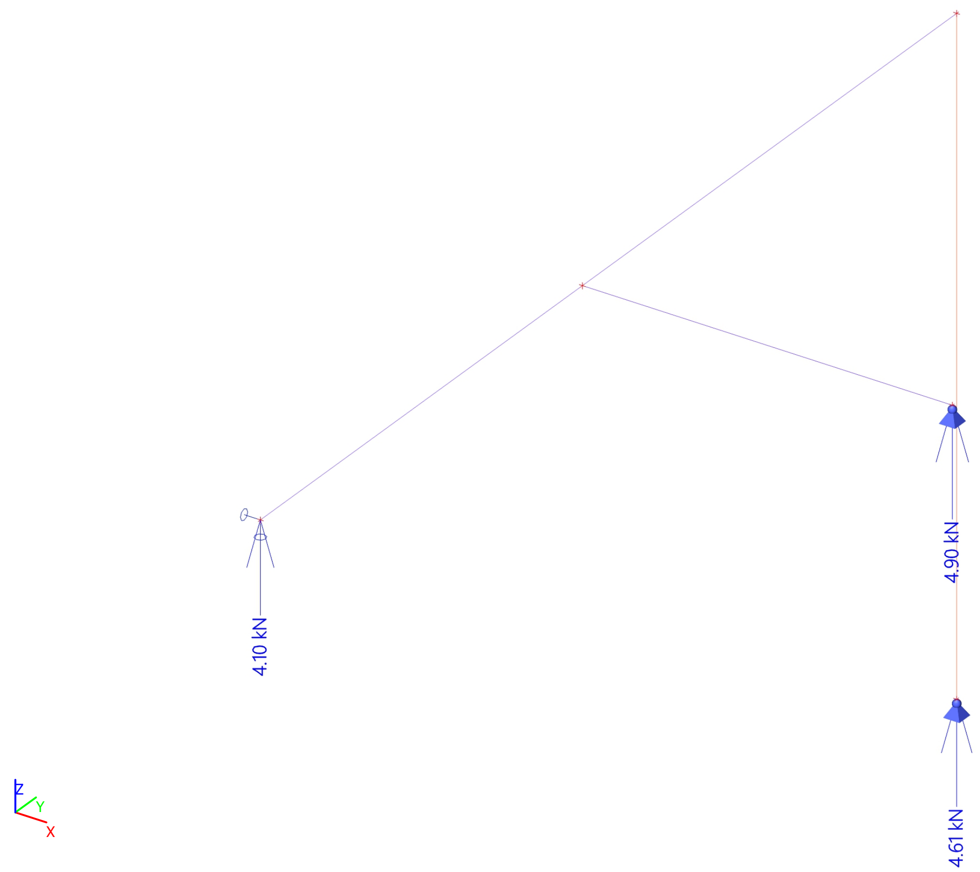


Reactions - Imposed load

Linear calculation
Load case: LC3
System: Global
Extreme: Member
Selection: All

Nodal reactions

Name	Case	R_x [kN]	R_y [kN]	R_z [kN]	M_x [kNm]	M_y [kNm]	M_z [kNm]	e_x [mm]	e_y [mm]
Sn1/N4	LC3	**0.52**	**0.68**	**4.90**	**0.00**	**0.00**	**0.00**	0.0	0.0
Sn2/N1	LC3	**-0.52**	**0.00**	**4.10**	**0.00**	**0.00**	**0.00**	0.0	0.0
Sn3/N5	LC3	**0.00**	**-0.68**	**4.61**	**0.00**	**0.00**	**0.00**	0.0	0.0



Reactions - Wind load

Linear calculation
Load case: LC4
System: Global
Extreme: Member
Selection: All

Nodal reactions

Name	Case	R_x [kN]	R_y [kN]	R_z [kN]	M_x [kNm]	M_y [kNm]	M_z [kNm]	e_x [mm]	e_y [mm]
Sn1/N4	LC4	**3.96**	**5.18**	**0.00**	**0.00**	**0.00**	**0.00**	-	-
Sn2/N1	LC4	**-3.96**	**0.00**	**1.05**	**0.00**	**0.00**	**0.00**	0.0	0.0
Sn3/N5	LC4	**0.00**	**1.68**	**-1.05**	**0.00**	**0.00**	**0.00**	0.0	0.0

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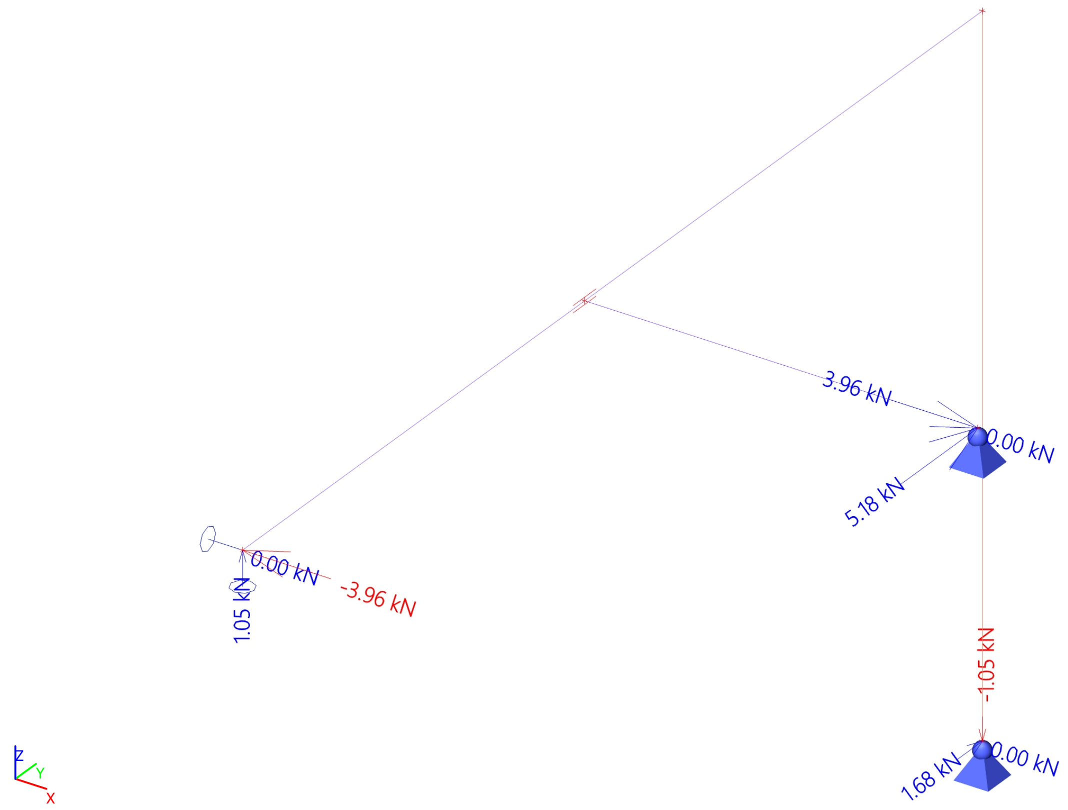
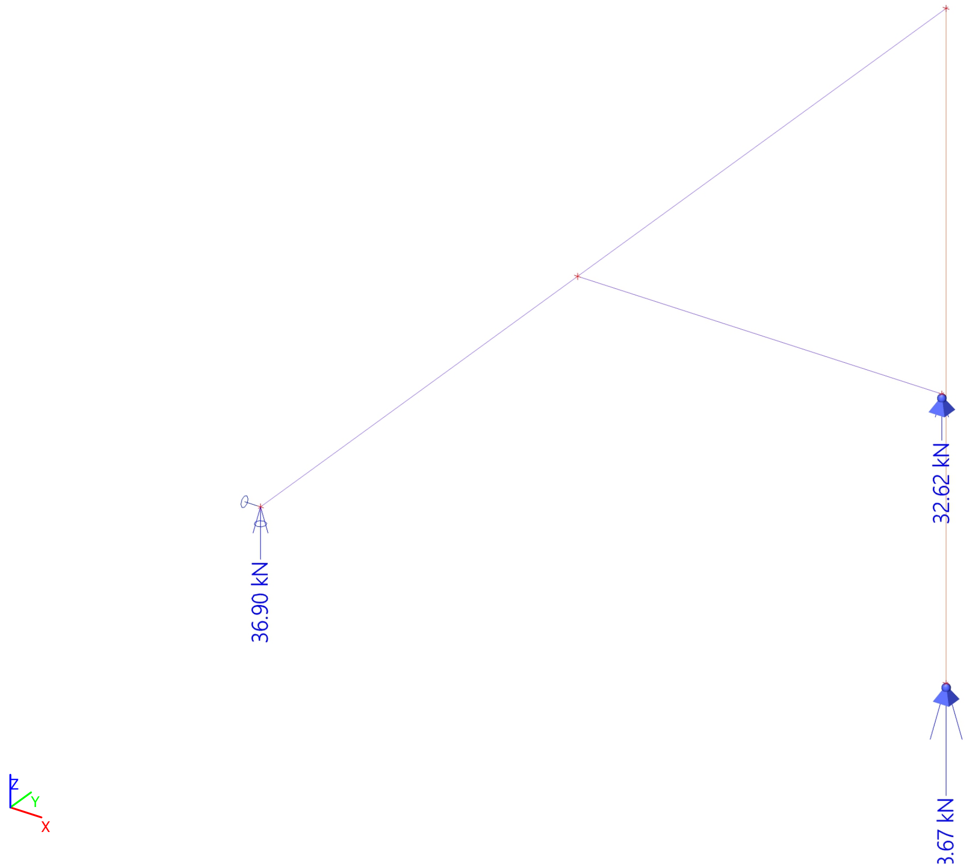
Reactions - Wind load



Reactions ULS

Linear calculation
Class: All ULS
System: Global
Extreme: Member
Selection: All
Nodal reactions

Name	Case	R_x [kN]	R_y [kN]	R_z [kN]	M_x [kNm]	M_y [kNm]	M_z [kNm]	e_x [mm]	e_y [mm]
Sn1/N4	CO2/1	**9.73**	**12.70**	27.12	**0.00**	**0.00**	**0.00**	0.0	0.0
Sn1/N4	CO3/2	5.90	7.71	**32.62**	0.00	0.00	0.00	0.0	0.0
Sn1/N4	CO4/3	**3.62**	**4.73**	**17.71**	0.00	0.00	**0.00**	0.0	0.0
Sn2/N1	CO4/3	**-3.62**	0.00	**21.67**	0.00	0.00	0.00	0.0	0.0
Sn2/N1	CO3/2	-5.90	**0.00**	**36.90**	**0.00**	**0.00**	**0.00**	0.0	0.0
Sn2/N1	CO2/1	**-9.73**	0.00	32.19	0.00	0.00	**0.00**	0.0	0.0
Sn3/N5	CO3/2	**0.00**	**-7.71**	**78.67**	**0.00**	**0.00**	**0.00**	0.0	0.0
Sn3/N5	CO4/4	0.00	**-2.38**	**49.45**	0.00	0.00	0.00	0.0	0.0



Reactions SLS

Linear calculation
Class: All SLS
System: Global
Extreme: Member
Selection: All

Nodal reactions

Name	Case	R_x [kN]	R_y [kN]	R_z [kN]	M_x [kNm]	M_y [kNm]	M_z [kNm]	e_x [mm]	e_y [mm]
Sn1/N4	CO6/1	**8.11**	**10.59**	**22.60**	**0.00**	**0.00**	**0.00**	0.0	0.0
Sn1/N4	CO5/2	**3.62**	**4.73**	**17.71**	0.00	0.00	**0.00**	0.0	0.0
Sn2/N1	CO5/2	**-3.62**	**0.00**	**21.67**	**0.00**	**0.00**	**0.00**	0.0	0.0
Sn2/N1	CO6/1	**-8.11**	0.00	**26.82**	0.00	0.00	**0.00**	0.0	0.0
Sn3/N5	CO6/3	**0.00**	**-5.41**	**55.54**	**0.00**	**0.00**	**0.00**	0.0	0.0
Sn3/N5	CO6/4	0.00	**-3.05**	**49.87**	0.00	0.00	0.00	0.0	0.0

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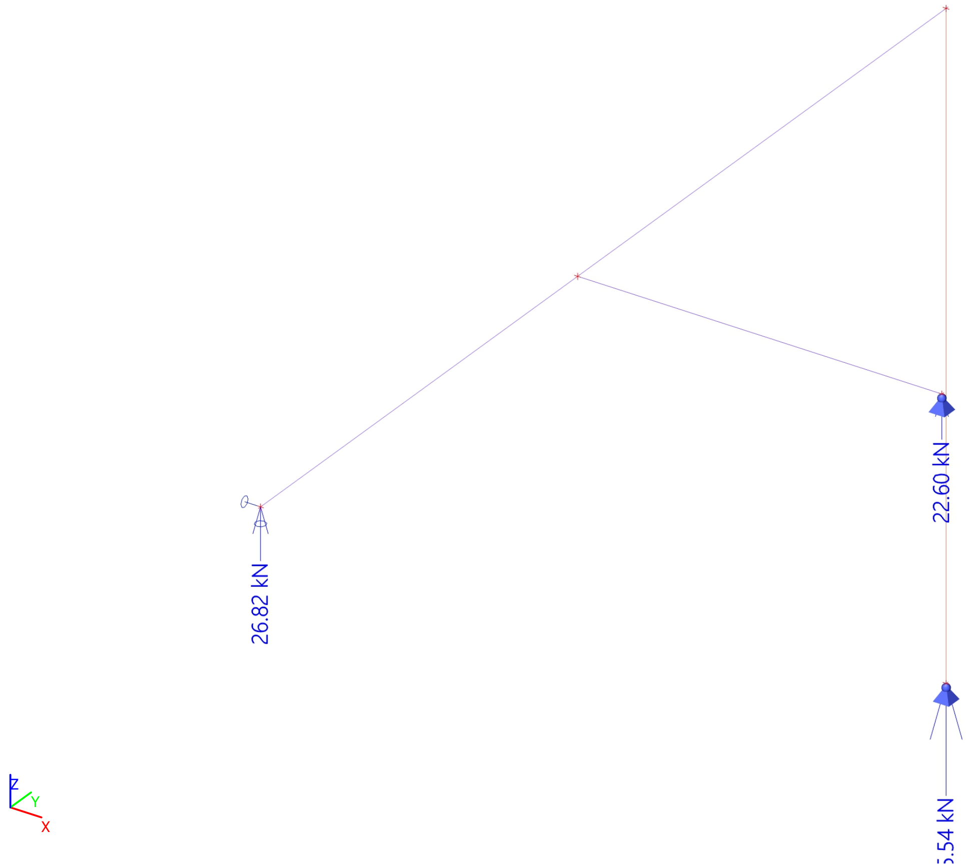
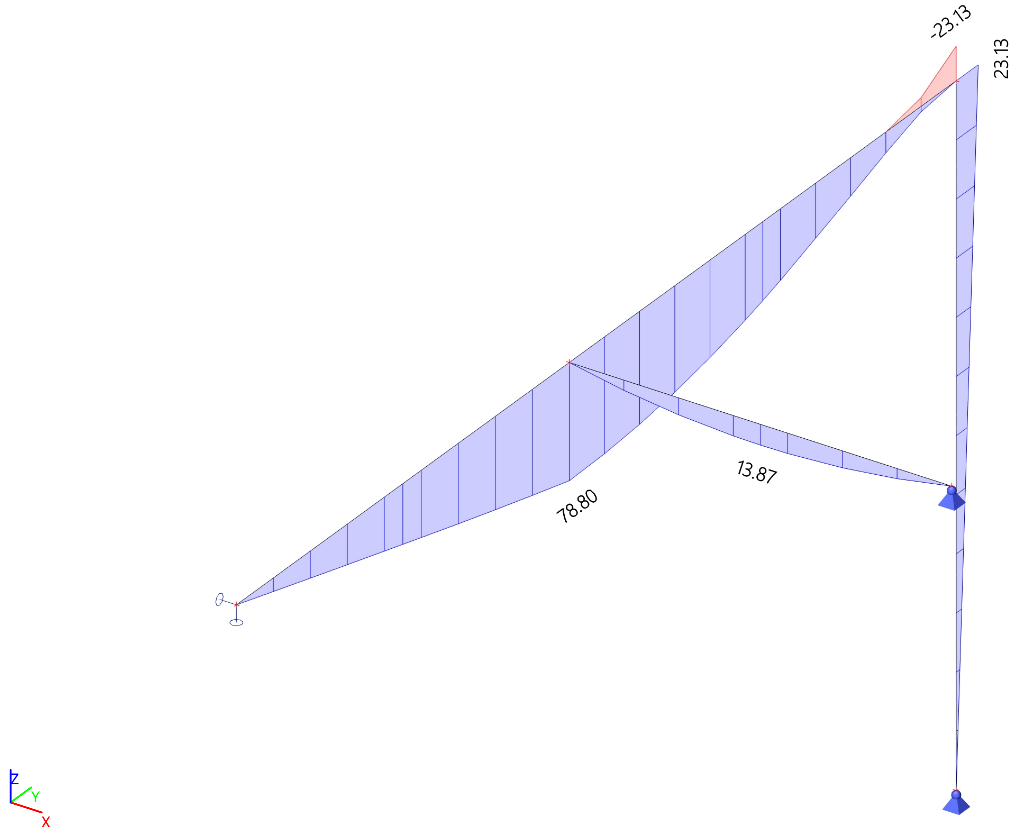


RESULTS - 1D INTERNAL FORCES

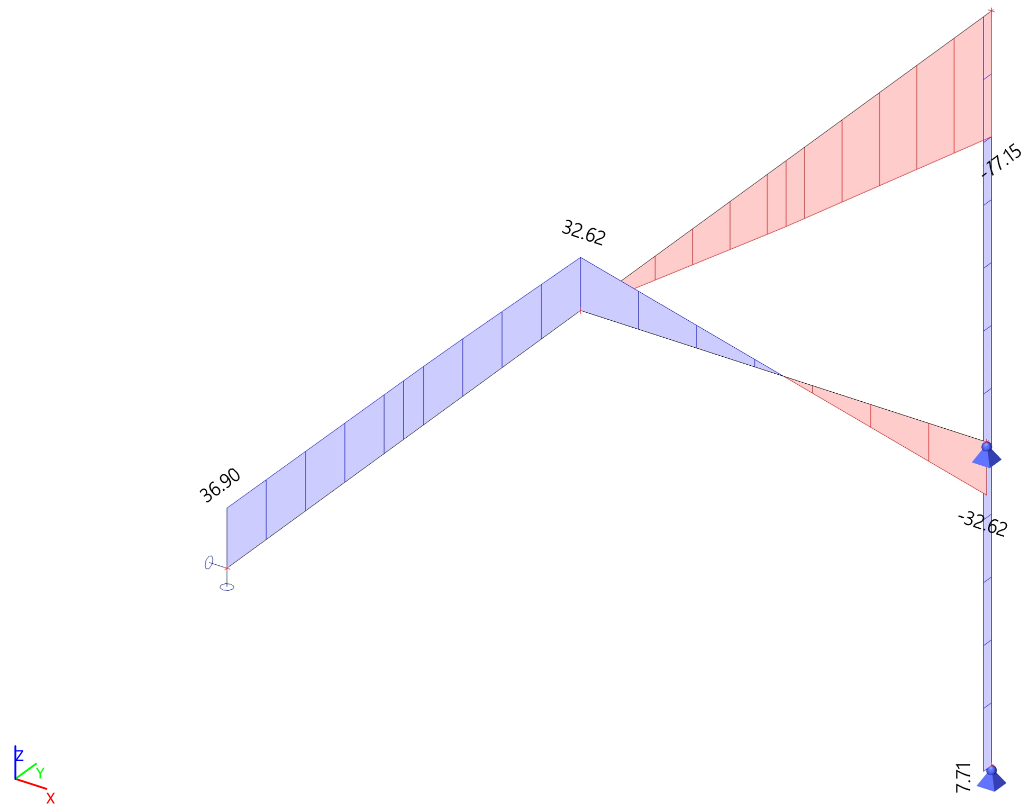
Linear calculation
Class: All ULS
Coordinate system: Principal
Extreme 1D: Member
Selection: All

Name	dx [m]	Case	N [kN]	V_y [kN]	V_z [kN]	M_x [kNm]	M_y [kNm]	M_z [kNm]
SB-4	2.22+	CO2/1	**-12.70**	**0.00**	1.64	0.00	68.91	0.00
SB-4	0.00	CO3/2	**0.00**	5.90	**36.90**	**0.00**	0.00	**0.00**
SB-4	4.80	CO3/2	-7.71	0.00	**-77.15**	0.00	**-23.13**	0.00
SB-4	2.22+	CO3/2	-7.71	0.00	-0.03	0.00	**78.80**	0.00
SB-4	2.22-	CO2/1	0.00	**9.73**	28.76	0.00	68.91	**21.60**
SB-3	0.00	CO2/1	**9.73**	**-12.70**	27.12	0.00	0.00	**21.60**
SB-3	1.70	CO3/2	5.90	-7.71	**-32.62**	0.00	0.00	0.00
SB-3	0.00	CO3/2	5.90	-7.71	**32.62**	**0.00**	**0.00**	13.11
SB-3	0.85+	CO3/2	5.90	-7.71	0.00	0.00	**13.87**	6.55
SB-3	1.70	CO4/3	**3.62**	**-4.73**	-17.71	0.00	0.00	**0.00**
C-1	3.00	CO4/4	**-48.37**	**0.00**	**2.38**	**0.00**	7.13	**0.00**
C-1	3.00	CO3/2	-77.15	0.00	**7.71**	0.00	**23.13**	0.00
C-1	0.00	CO3/2	**-78.67**	0.00	7.71	0.00	**0.00**	**0.00**

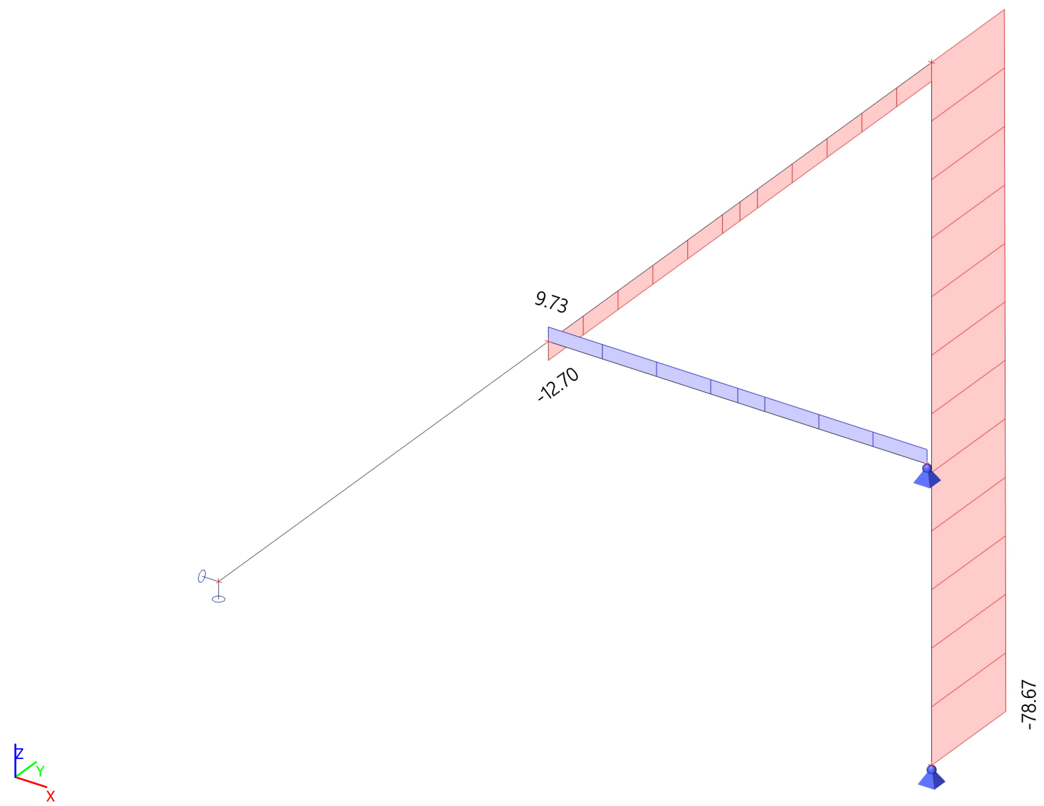
Strength combinations - Moment envelope [kNm]



Strength combinations - Shear envelope [kN]



Strength combinations - Axial force envelope [kN]



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Calc Date	Client	Page No.	Revision

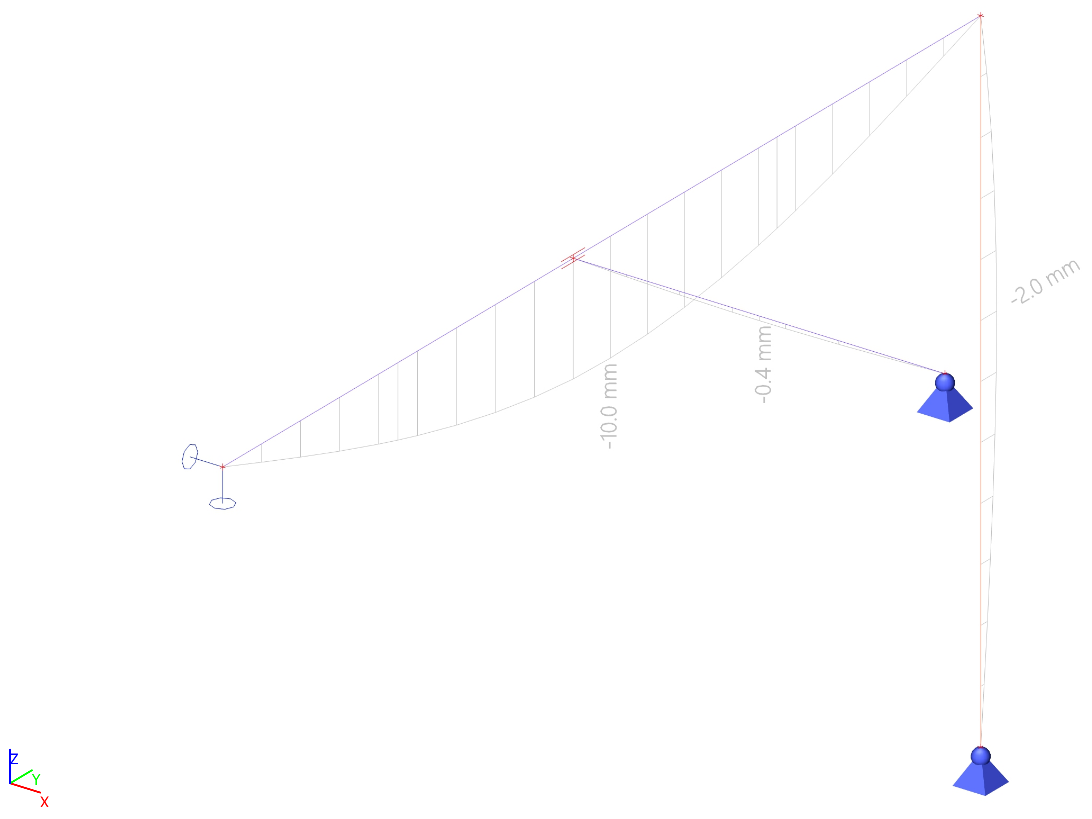
RESULTS - DEFORMATION

Relative deformation

Linear calculation, Extreme : Member, System : Principal
Selection : All
Class : All SLS

Member	dx [m]	Case - combination	uy [mm]	Rel uy [1/xx]	uz [mm]	Rel uz [1/xx]
SB-4	1.23	CO6/1	**-1.3**	**1/1695**	-7.0	1/690
SB-4	4.80	CO6/1	**8.0**	**1/323**	0.0	1/10000
SB-4	0.00	CO5/2	0.0	0	**0.0**	**0**
SB-4	2.45	CO6/1	0.7	1/3554	**-10.0**	**1/478**
SB-3	0.73	CO6/1	**-1.0**	**1/1657**	-0.4	1/4584
SB-3	0.00	CO5/2	**0.0**	**0**	**0.0**	**0**
SB-3	0.85	CO6/1	-1.0	1/1700	**-0.4**	**1/4476**
C-1	0.00	CO5/2	**0.0**	**0**	**0.0**	**0**
C-1	3.00	CO6/1	**8.0**	**1/375**	0.0	0
C-1	1.75	CO6/3	2.4	1/1259	**-2.0**	**1/1487**
C-1	1.75	CO6/1	4.7	1/643	-1.4	1/2158

Relative deformation; uz



STEEL MEMBER DESIGN & CHECK (BS-EN1993)

EC-EN 1993 Steel check ULS

Linear calculation
Class: All ULS
Coordinate system: Principal
Extreme 1D: Cross-section
Selection: All

EN 1993-1-1 Code Check
National annex: British BS-EN NA

Member SB-4	2.220 / 4.800 m	UC203/203/60	S 275	All ULS	0.53 -

Combination key
All ULS / 1.20*LC1 + 1.20*LC2 + 1.20*LC3 + 1.20*LC4

Partial safety factors	
γ_{M0} for resistance of cross-sections	1.00
γ_{M1} for resistance to instability	1.00
γ_{M2} for resistance of net sections	1.10

Material		
Yield strength f_y	275.0	MPa
Ultimate strength f_u	430.0	MPa
Fabrication	Rolled	

...::SECTION CHECK::...

The critical check is on position 2.220 m

Internal forces	Calculated	Unit
N_{Ed}	0.00	kN
$V_{y,Ed}$	9.73	kN
$V_{z,Ed}$	28.76	kN
T_{Ed}	0.00	kNm
$M_{y,Ed}$	68.91	kNm
$M_{z,Ed}$	21.60	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [kN/m ^{2}]	σ_2 [kN/m ^{2}]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	SO	88.0	14.2	-1.255e+05	-2.176e+05								
3	SO	88.0	14.2	-9.433e+04	-2.278e+03								
4	I	160.8	9.4	-9.045e+04	9.045e+04	-1.00		0.50	17.11	66.56	76.73	114.63	1
5	SO	88.0	14.2	1.255e+05	2.176e+05	0.58	0.47	1.00	6.20	8.32	9.24	13.34	1
7	SO	88.0	14.2	9.433e+04	2.278e+03	0.02	1.59	1.00	6.20	8.32	9.24	24.46	1

Note: The Classification limits have been set according to Semi-Comp+.
The cross-section is classified as Class 1

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

$W_{pl,y}$	6.5608e-04	m^3
$M_{pl,y,Rd}$	180.42	kNm
Unity check	0.38	-

Bending moment check for M_z
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

$W_{pl,z}$	3.0534e-04	m^3
$M_{pl,z,Rd}$	83.97	kNm
Unity check	0.26	-

Shear check for V_y
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

η	1.00	
A_v	6.0290e-03	m^2
$V_{pl,y,Rd}$	957.23	kN
Unity check	0.01	-

Shear check for V_z
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

η	1.00	
A_v	2.2184e-03	m^2
$V_{pl,z,Rd}$	352.22	kN
Unity check	0.08	-

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.9.1 and formula (6.41)

$M_{pl,y,Rd}$	180.42	kNm
α	2.00	
$M_{pl,z,Rd}$	83.97	kNm
β	1.00	

Unity check (6.41) = 0.15 + 0.26 = 0.40 -

Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment resistances is neglected.

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design
Decisive position for stability classification: 2.220 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [kN/m ^{2}]	σ_2 [kN/m ^{2}]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	SO	88.0	14.2	-1.255e+05	-2.176e+05								
3	SO	88.0	14.2	-9.433e+04	-2.278e+03								
4	I	160.8	9.4	-9.045e+04	9.045e+04	-1.00		0.50	17.11	66.56	76.73	114.63	1
5	SO	88.0	14.2	1.255e+05	2.176e+05	0.58	0.47	1.00	6.20	8.32	9.24	13.34	1
7	SO	88.0	14.2	9.433e+04	2.278e+03	0.02	1.59	1.00	6.20	8.32	9.24	24.46	1

Note: The Classification limits have been set according to Semi-Comp+.
The cross-section is classified as Class 1

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1 & 6.3.2.3 and formula (6.54)

LTB parameters		
Method for LTB curve	Alternative case	
Plastic section modulus $W_{pl,y}$	6.5608e-04	m^3
Elastic critical moment M_{cr}	1785.11	kNm
Relative slenderness $\lambda_{rel,LT}$	0.32	
Limit slenderness $\lambda_{rel,LT,0}$	0.40	

Note: The slenderness or bending moment is such that Lateral Torsional Buckling effects may be ignored according to EN 1993-1-1 article 6.3.2.2(4).

Mcr parameters		
LTB length L	2.220	m
Influence of load position	no influence	
Correction factor k	1.00	
Correction factor k_w	1.00	
LTB moment factor C_1	1.74	
LTB moment factor C_2	0.01	

Mcr parameters		
LTB moment factor C_3	1.00	
Shear center distance d_z	0.0	mm
Distance of load application z_g	0.0	mm
Mono-symmetry constant β_y	0.0	mm
Mono-symmetry constant z_j	0.0	mm

Note: C parameters are determined according to ECCS 119 2006 / Galea 2002.

Bending and axial compression check
According to EN 1993-1-1 article 6.3.3 and formula (6.61),(6.62)

Bending and axial compression check parameters		
Interaction method	alternative method 1	
Cross-section area A	7.6400e-03	m^2
Plastic section modulus $W_{pl,y}$	6.5608e-04	m^3
Plastic section modulus $W_{pl,z}$	3.0534e-04	m^3
Design compression force N_{Ed}	0.00	kN
Design bending moment (maximum) $M_{y,Ed}$	68.91	kNm
Design bending moment (maximum) $M_{z,Ed}$	21.60	kNm
Characteristic compression resistance N_{Rk}	2101.00	kN
Characteristic moment resistance $M_{y,Rk}$	180.42	kNm
Characteristic moment resistance $M_{z,Rk}$	83.97	kNm
Reduction factor χ_y	1.00	
Reduction factor χ_z	1.00	
Modified reduction factor $\chi_{LT,mod}$	1.00	
Interaction factor k_{yy}	1.00	
Interaction factor k_{yz}	0.59	
Interaction factor k_{zy}	0.58	
Interaction factor k_{zz}	0.79	

Maximum moment $M_{y,Ed}$ is derived from beam SB-4 position 2.220 m.
Maximum moment $M_{z,Ed}$ is derived from beam SB-4 position 2.220 m.

Interaction method 1 parameters		
Critical Euler load $N_{cr,y}$	760.91	kN
Critical Euler load $N_{cr,z}$	12227.53	kN
Elastic critical load $N_{cr,T}$	11280.53	kN
Plastic section modulus $W_{pl,y}$	6.5608e-04	m^3
Elastic section modulus $W_{el,y}$	5.8400e-04	m^3
Plastic section modulus $W_{pl,z}$	3.0534e-04	m^3
Elastic section modulus $W_{el,z}$	2.0100e-04	m^3
Second moment of area I_y	6.1258e-05	m^4
Second moment of area I_z	2.0646e-05	m^4
Torsional constant I_t	4.7200e-07	m^4
Method for equivalent moment factor $C_{my,0}$	Table A.2 Line 2 (General)	
Design bending moment (maximum) $M_{y,Ed}$	68.91	kNm
Maximum relative deflection δ_z	-12.0	mm
Equivalent moment factor $C_{my,0}$	1.00	
Method for equivalent moment factor $C_{mz,0}$	Table A.2 Line 1 (Linear)	
Ratio of end moments ψ_z	0.00	
Equivalent moment factor $C_{mz,0}$	0.79	
Factor μ_y	1.00	
Factor μ_z	1.00	
Factor a_{LT}	0.99	
Critical moment for uniform bending $M_{cr,0}$	1024.70	kNm
Relative slenderness $\lambda_{rel,0}$	0.42	
Limit relative slenderness $\lambda_{rel,0,lim}$	0.26	
Equivalent moment factor C_{my}	1.00	
Equivalent moment factor C_{mz}	0.79	
Equivalent moment factor C_{mLT}	1.00	
Factor b_{LT}	0.01	
Factor c_{LT}	0.13	

Interaction method 1 parameters		
Factor d_{LT}	0.80	
Factor e_{LT}	2.09	
Factor w_y	1.12	
Factor w_z	1.50	
Factor n_{pl}	0.00	
Maximum relative slenderness $\lambda_{rel,max}$	1.66	
Factor C_{yy}	1.00	
Factor C_{yz}	0.93	
Factor C_{zy}	0.90	
Factor C_{zz}	1.00	

Unity check (6.61) = 0.00 + 0.38 + 0.15 = 0.53 -
Unity check (6.62) = 0.00 + 0.22 + 0.20 = 0.42 -

Shear Buckling check
According to EN 1993-1-5 article 5 & 7.1 and formula (5.10) & (7.1)

Shear Buckling parameters		
Buckling field length a	4.800	m
Web	unstiffened	
Web height h_w	181.2	mm
Web thickness t	9.4	mm
Material coefficient ε	0.92	
Shear correction factor η	1.00	

Shear Buckling verification	
Web slenderness h_w/t	19.28
Web slenderness limit	66.56

Note: The web slenderness is such that Shear Buckling effects may be ignored according to EN 1993-1-5 article 5.1(2).

The member satisfies the stability check.

EN 1993-1-1 Code Check
National annex: British BS-EN NA

Member SB-3	**0.000 / 1.700 m**	**UC203/203/46**	**S 275**	**All ULS**	**0.34 -**

Combination key
All ULS / 1.20*LC1 + 1.20*LC2 + 1.20*LC3 + 1.20*LC4

Partial safety factors	
γ_{M0} for resistance of cross-sections	1.00
γ_{M1} for resistance to instability	1.00
γ_{M2} for resistance of net sections	1.10

Material		
Yield strength f_y	275.0	MPa
Ultimate strength f_u	430.0	MPa
Fabrication	Rolled	

...::SECTION CHECK::...

The critical check is on position 0.000 m

Internal forces	Calculated	Unit
N_{Ed}	9.73	kN
$V_{y,Ed}$	-12.70	kN
$V_{z,Ed}$	27.12	kN
T_{Ed}	0.00	kNm
$M_{y,Ed}$	0.00	kNm
$M_{z,Ed}$	21.60	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [kN/m ^{2}]	σ_2 [kN/m ^{2}]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	SO	88.0	11.0	-2.091e+04	-1.437e+05								
3	SO	88.0	11.0	1.759e+04	1.403e+05	0.13	0.54	1.00	8.00	8.32	9.24	14.33	1
4	I	160.8	7.2	-1.656e+03	-1.656e+03								
5	SO	88.0	11.0	1.759e+04	1.403e+05	0.13	0.54	1.00	8.00	8.32	9.24	14.33	1
7	SO	88.0	11.0	-2.091e+04	-1.437e+05								

Note: The Classification limits have been set according to Semi-Comp+.
The cross-section is classified as Class 1

Tension check
According to EN 1993-1-1 article 6.2.3 and formula (6.5)

A	5.8700e-03	m^2
$N_{pl,Rd}$	1614.25	kN
$N_{u,Rd}$	2065.17	kN
$N_{t,Rd}$	1614.25	kN
Unity check	0.01	-

Bending moment check for M_z
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

$W_{pl,z}$	2.3086e-04	m^3
$M_{pl,z,Rd}$	63.49	kNm
Unity check	0.34	-

Shear check for V_y
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

η	1.00	
A_v	4.6045e-03	m^2
$V_{pl,y,Rd}$	731.06	kN
Unity check	0.02	-

Shear check for V_z
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

η	1.00	
A_v	1.6944e-03	m^2
$V_{pl,z,Rd}$	269.02	kN
Unity check	0.10	-

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.9.1 and formula (6.31)

$M_{pl,z,Rd}$	63.49	kNm
Unity check	0.34	-

Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment resistances is neglected.
Note: Since the axial force satisfies criteria (6.35) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment resistance about the z-z axis is neglected.

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design
Decisive position for stability classification: 0.000 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [kN/m ^{2}]	σ_2 [kN/m ^{2}]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	SO	88.0	11.0	-2.091e+04	-1.437e+05								
3	SO	88.0	11.0	1.759e+04	1.403e+05	0.13	0.54	1.00	8.00	8.32	9.24	14.33	1
4	I	160.8	7.2	-1.656e+03	-1.656e+03								
5	SO	88.0	11.0	1.759e+04	1.403e+05	0.13	0.54	1.00	8.00	8.32	9.24	14.33	1
7	SO	88.0	11.0	-2.091e+04	-1.437e+05								

Note: The Classification limits have been set according to Semi-Comp+.
The cross-section is classified as Class 1

Shear Buckling check
According to EN 1993-1-5 article 5 & 7.1 and formula (5.10) & (7.1)

Shear Buckling parameters		
Buckling field length a	1.700	m
Web	unstiffened	
Web height h_w	181.2	mm
Web thickness t	7.2	mm
Material coefficient ε	0.92	
Shear correction factor η	1.00	

Shear Buckling verification	
Web slenderness h_w/t	25.17
Web slenderness limit	66.56

Note: The web slenderness is such that Shear Buckling effects may be ignored according to EN 1993-1-5 article 5.1(2).

The member satisfies the stability check.

EN 1993-1-1 Code Check
National annex: British BS-EN NA

Member C-1	0.000 / 3.000 m	UC152/152/37	S 275	All ULS	0.40 -

Combination key
All ULS / 1.40*LC1 + 1.40*LC2 + 1.60*LC3

Partial safety factors	
γ_{M0} for resistance of cross-sections	1.00
γ_{M1} for resistance to instability	1.00
γ_{M2} for resistance of net sections	1.10

Material		
Yield strength f_y	275.0	MPa
Ultimate strength f_u	430.0	MPa
Fabrication	Rolled	

...::SECTION CHECK::...

The critical check is on position 0.000 m

Internal forces	Calculated	Unit
N_{Ed}	-78.67	kN
$V_{y,Ed}$	0.00	kN
$V_{z,Ed}$	7.71	kN
T_{Ed}	0.00	kNm
$M_{y,Ed}$	0.00	kNm
$M_{z,Ed}$	0.00	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [kN/m ^{2}]	σ_2 [kN/m ^{2}]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	SO	65.6	11.5	1.670e+04	1.670e+04	1.00	0.43	1.00	5.70	8.32	9.24	12.94	1
3	SO	65.6	11.5	1.670e+04	1.670e+04	1.00	0.43	1.00	5.70	8.32	9.24	12.94	1
4	I	123.6	8.0	1.670e+04	1.670e+04	1.00		1.00	15.45	25.88	31.43	35.13	1
5	SO	65.6	11.5	1.670e+04	1.670e+04	1.00	0.43	1.00	5.70	8.32	9.24	12.94	1
7	SO	65.6	11.5	1.670e+04	1.670e+04	1.00	0.43	1.00	5.70	8.32	9.24	12.94	1

Note: The Classification limits have been set according to Semi-Comp+.
The cross-section is classified as Class 1

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

A	4.7100e-03	m^2
$N_{c,Rd}$	1295.25	kN
Unity check	0.06	-

Shear check for V_z
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

η	1.00	
A_v	1.4256e-03	m^2
$V_{pl,z,Rd}$	226.34	kN
Unity check	0.03	-

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design
Decisive position for stability classification: 3.000 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [kN/m ^{2}]	σ_2 [kN/m ^{2}]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	SO	65.6	11.5	-6.225e+04	-6.225e+04								
3	SO	65.6	11.5	-6.225e+04	-6.225e+04								
4	I	123.6	8.0	-4.828e+04	8.103e+04	-0.60		0.64	15.45	46.03	54.46	78.72	1
5	SO	65.6	11.5	9.500e+04	9.500e+04	1.00	0.43	1.00	5.70	8.32	9.24	12.94	1
7	SO	65.6	11.5	9.500e+04	9.500e+04	1.00	0.43	1.00	5.70	8.32	9.24	12.94	1

Note: The Classification limits have been set according to Semi-Comp+.
The cross-section is classified as Class 1

Flexural Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters	yy	zz	
Sway type	sway	non-sway	
System length L	3.000	3.000	m
Buckling factor k	2.42	1.00	
Buckling length L_{cr}	7.270	3.000	m
Critical Euler load N_{cr}	867.06	1626.52	kN
Slenderness λ	106.11	77.47	
Relative slenderness λ_{rel}	1.22	0.89	
Limit slenderness $\lambda_{rel,0}$	0.20	0.20	
Buckling curve	b	c	
Imperfection α	0.34	0.49	
Reduction factor χ	0.47	0.60	
Buckling resistance $N_{b,Rd}$	603.86	783.01	kN

Flexural Buckling verification		
Cross-section area A	4.7100e-03	m^2
Buckling resistance $N_{b,Rd}$	603.86	kN
Unity check	0.13	-

Torsional(-Flexural) Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)
Note: For this I-section the Torsional(-Flexural) buckling resistance is higher than the resistance for Flexural buckling. Therefore Torsional(-Flexural) buckling is not printed on the output.

Bending and axial compression check
According to EN 1993-1-1 article 6.3.3 and formula (6.61),(6.62)

Bending and axial compression check parameters		
Interaction method	alternative method 1	
Cross-section area A	4.7100e-03	m^2
Plastic section modulus $W_{pl,y}$	3.0876e-04	m^3
Design compression force N_{Ed}	78.67	kN
Design bending moment (maximum) $M_{y,Ed}$	23.13	kNm
Design bending moment (maximum) $M_{z,Ed}$	0.00	kNm
Characteristic compression resistance N_{Rk}	1295.25	kN
Characteristic moment resistance $M_{y,Rk}$	84.91	kNm
Reduction factor χ_y	0.47	
Reduction factor χ_z	0.60	
Modified reduction factor $\chi_{LT,mod}$	1.00	
Interaction factor k_{yy}	0.98	
Interaction factor k_{zy}	0.56	

Maximum moment $M_{y,Ed}$ is derived from beam C-1 position 3.000 m.
Maximum moment $M_{z,Ed}$ is derived from beam C-1 position 0.000 m.

Interaction method 1 parameters		
Critical Euler load $N_{cr,y}$	867.06	kN
Critical Euler load $N_{cr,z}$	1626.52	kN
Elastic critical load $N_{cr,T}$	3985.33	kN
Plastic section modulus $W_{pl,y}$	3.0876e-04	m^3
Elastic section modulus $W_{el,y}$	2.7300e-04	m^3
Plastic section modulus $W_{pl,z}$	1.3958e-04	m^3
Elastic section modulus $W_{el,z}$	9.1500e-05	m^3
Second moment of area I_y	2.2109e-05	m^4
Second moment of area I_z	7.0625e-06	m^4
Torsional constant I_t	1.9200e-07	m^4
Method for equivalent moment factor $C_{my,0}$	Table A.2 Line 1 (Linear)	
Ratio of end moments ψ_y	0.00	
Equivalent moment factor $C_{my,0}$	0.78	
Factor μ_y	0.95	
Factor μ_z	0.98	
Factor ε_y	5.07	
Factor a_{LT}	0.99	
Critical moment for uniform bending $M_{cr,0}$	200.36	kNm
Relative slenderness $\lambda_{rel,0}$	0.65	
Limit relative slenderness $\lambda_{rel,0,lim}$	0.26	
Equivalent moment factor C_{my}	0.93	
Equivalent moment factor C_{mLT}	1.00	
Factor b_{LT}	0.00	
Factor d_{LT}	0.00	
Factor w_y	1.13	
Factor w_z	1.50	
Factor n_{pl}	0.06	
Maximum relative slenderness $\lambda_{rel,max}$	1.22	
Factor C_{yy}	0.99	
Factor C_{zy}	0.94	

Unity check (6.61) = 0.13 + 0.27 + 0.00 = 0.40 -
Unity check (6.62) = 0.10 + 0.15 + 0.00 = 0.25 -

Shear Buckling check
According to EN 1993-1-5 article 5 & 7.1 and formula (5.10) & (7.1)

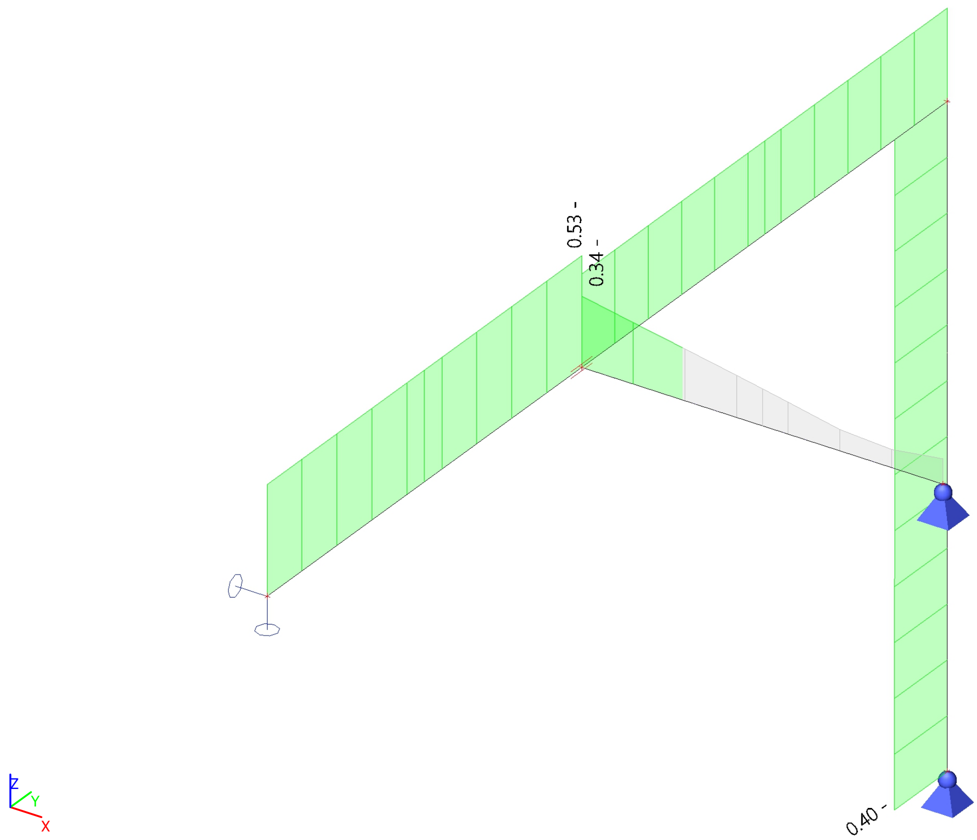
Shear Buckling parameters		
Buckling field length a	3.000	m
Web	unstiffened	
Web height h_w	138.8	mm
Web thickness t	8.0	mm
Material coefficient ε	0.92	
Shear correction factor η	1.00	

Shear Buckling verification	
Web slenderness h_w/t	17.35
Web slenderness limit	66.56

Note: The web slenderness is such that Shear Buckling effects may be ignored according to EN 1993-1-5 article 5.1(2).

The member satisfies the stability check.

ULS Design check - Resultant utilisation

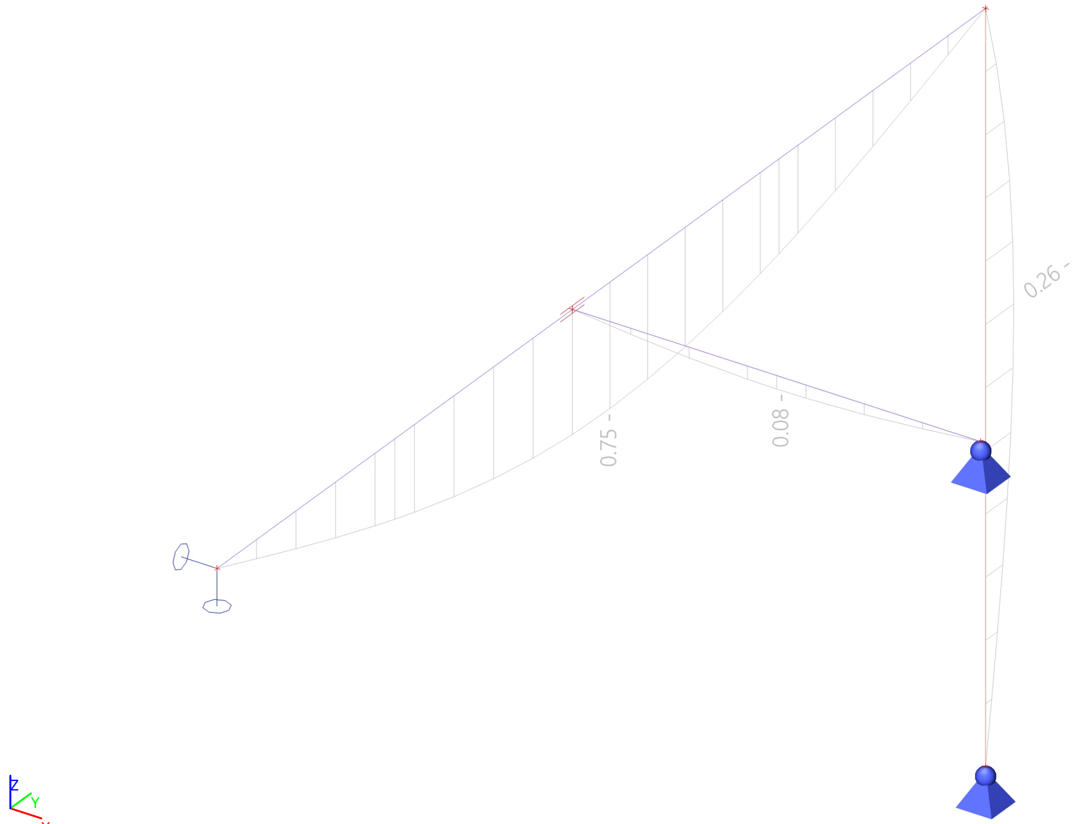


SLS Design check - Resultant utilisation

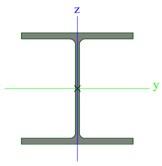
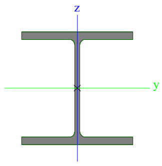
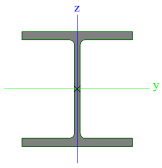
Relative deformation

Linear calculation, Extreme : Member, System : Principal
Selection : All
Class : All SLS

Member	dx [m]	Case - combination	uy [mm]	uz [mm]	Check uy [-]	Check uz [-]
SB-4	1.23	CO6/1	**-1.3**	-7.0	0.21	0.52
SB-4	4.80	CO6/1	**8.0**	0.0	**1.11**	0.00
SB-4	0.00	CO5/2	0.0	**0.0**	**0.00**	**0.00**
SB-4	2.45	CO6/1	0.7	**-10.0**	0.10	**0.75**
SB-3	0.73	CO6/1	**-1.0**	-0.4	**0.22**	0.08
SB-3	0.00	CO5/2	**0.0**	**0.0**	**0.00**	**0.00**
SB-3	0.85	CO6/1	-1.0	**-0.4**	0.21	**0.08**
C-1	0.00	CO5/2	**0.0**	**0.0**	**0.00**	**0.00**
C-1	3.00	CO6/1	**8.0**	0.0	**0.80**	0.00
C-1	1.75	CO6/3	2.4	**-2.0**	0.24	0.20
C-1	1.75	CO6/1	4.7	-1.4	0.47	**0.26**



Cross-sections

Name	Type	Item material	h [mm]	b [mm]	t [mm]	s [mm]	r [mm]	Section
SB-3	UC203/203/46	S 275	203.2	203.6	11.0	7.2	10.2	
SB-4	UC203/203/60	S 275	209.6	205.8	14.2	9.4	10.2	
C-1	UC152/152/37	S 275	161.8	154.4	11.5	8.0	7.6	

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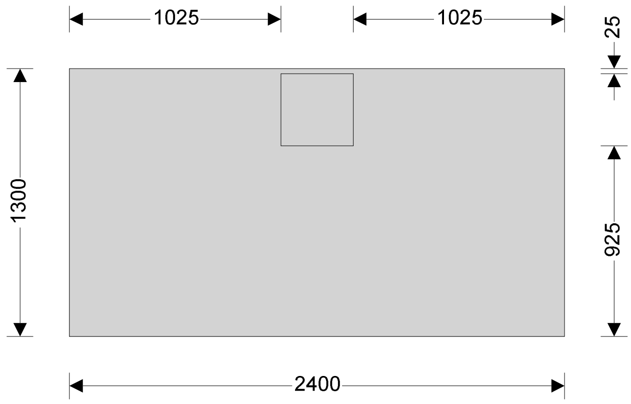
PAD FOOTING 1: *COLUMN C-1*

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

Tedds calculation version 2.0.07



Pad footing details

Length of pad footing $L = \mathbf{2400}$ mm | Width of pad footing $B = \mathbf{1300}$ mm

Depth of pad footing $h = \mathbf{500}$ mm | Depth of soil over pad footing $h_{soil} = \mathbf{200}$ mm

Density of concrete $\rho_{conc} = \mathbf{23.6}$ kN/m^3

Column details

Column base length $l_A = \mathbf{350}$ mm | Column base width $b_A = \mathbf{350}$ mm

Column eccentricity in x $e_{PxA} = \mathbf{0}$ mm | Column eccentricity in y $e_{PyA} = \mathbf{450}$ mm

Soil details

Depth of soil over pad footing $h_{soil} = \mathbf{200}$ mm | Density of soil $\rho_{soil} = \mathbf{20.0}$ kN/m^3

Allowable bearing pressure $P_{bearing} = \mathbf{91}$ kN/m^2

Axial loading on column

Dead axial load $P_{GA} = \mathbf{50.9}$ kN | Imposed axial load $P_{QA} = \mathbf{4.6}$ kN

Wind axial load $P_{WA} = \mathbf{4.2}$ kN | Total axial load $P_A = \mathbf{59.7}$ kN

Foundation loads

Dead surcharge load $F_{Gsur} = \mathbf{3.000}$ kN/m^2 | Imposed surcharge load $F_{Qsur} = \mathbf{2.000}$ kN/m^2

Pad footing self weight $F_{swt} = \mathbf{11.800}$ kN/m^2

Soil self weight $F_{soil} = \mathbf{4.000}$ kN/m^2 | Total foundation load $F = \mathbf{64.9}$ kN

Horizontal loading on pad footing

Dead load in x direction $H_{GxA} = \mathbf{0.0}$ kN | Dead load in y direction $H_{GyA} = \mathbf{4.7}$ kN

Imposed load in x direction $H_{QxA} = \mathbf{0.0}$ kN | Imposed load in y direction $H_{QyA} = \mathbf{0.0}$ kN

Wind load in x direction $H_{WxA} = \mathbf{0.0}$ kN | Wind load in y direction $H_{WyA} = \mathbf{6.9}$ kN

Total load in x direction $H_{xA} = \mathbf{0.0}$ kN | Total load in y direction $H_{yA} = \mathbf{11.6}$ kN

Check stability against sliding

Passive pressure coefficient $K_p = \mathbf{2.464}$

Stability against sliding in y direction

Total sliding resistance in y dir $H_{yres} = \mathbf{65.0}$ kN

PASS - Resistance to sliding is greater than horizontal load in y direction

Check stability against overturning in y direction

Total overturning moment $M_{yOT} = \mathbf{5.795}$ kNm | Total restoring moment $M_{yres} = \mathbf{48.312}$ kNm

PASS - Restoring moment is greater than overturning moment in y direction

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Calculate pad base reaction

Total base reaction $T = 124.6$ kN

Base reaction eccentricity in x $e_{Tx} = 0$ mm

Base reaction eccentricity in y $e_{Ty} = 262$ mm

Base reaction acts outside of middle third of base

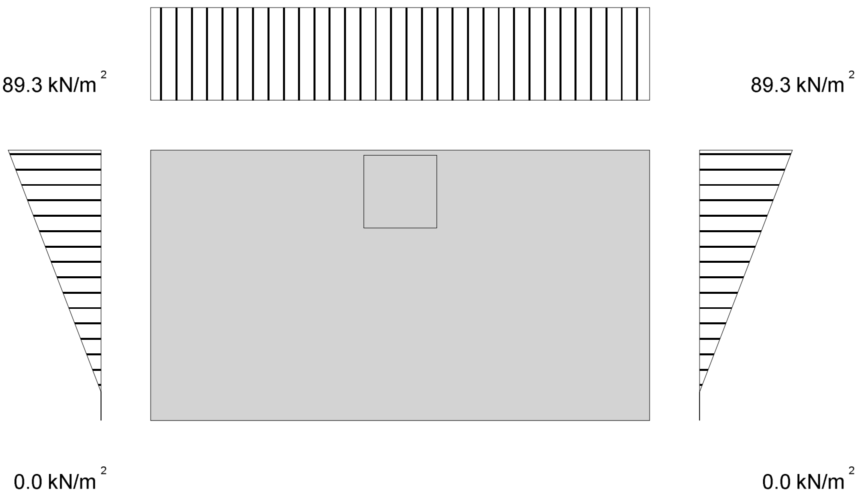
Calculate pad base pressures

$q_1 = 0.000$ kN/m^2 $q_2 = 89.273$ kN/m^2 $q_3 = 0.000$ kN/m^2 $q_4 = 89.273$ kN/m^2

Minimum base pressure $q_{min} = 0.000$ kN/m^2

Maximum base pressure $q_{max} = 89.273$ kN/m^2

PASS - Maximum base pressure is less than allowable bearing pressure



Partial safety factors for loads

Dead loads $\gamma_{fG} = 1.40$

Imposed loads $\gamma_{fQ} = 1.60$

Wind loads $\gamma_{fW} = 0.00$

Ultimate axial loading on column

Ultimate axial load on column $P_{uA} = 78.7$ kN

Ultimate foundation loads

Ultimate foundation load $F_u = 92.1$ kN

Ultimate horizontal loading on column

Ult.horizontal load in x dir $H_{xuA} = 0.0$ kN

Ult. horizontal load in y dir $H_{yuA} = 6.6$ kN

Ultimate moment on column

Ult.moment on column in x dir $M_{xuA} = 0.000$ kNm

Ult.moment on column in y dir $M_{yuA} = 0.000$ kNm

Ultimate pad base reaction

Ultimate base reaction $T_u = 170.8$ kN

Ecc.of ult.base reaction in x $e_{Txu} = 0$ mm

Ecc.of ult.base reaction in y $e_{Tyu} = 227$ mm

Calculate ultimate pad base pressures

$q_{1u} = 0.000$ kN/m^2 $q_{2u} = 112.070$ kN/m^2 $q_{3u} = 0.000$ kN/m^2 $q_{4u} = 112.070$ kN/m^2

Minimum ult.base pressure $q_{minu} = 0.000$ kN/m^2

Maximum ult.base pressure $q_{maxu} = 112.070$ kN/m^2

Library item: Ultimate pressures summaryUltimate moments

Ultimate moment in x dir $M_x = 24.818$ kNm

Ultimate moment in y dir $M_y = 3.680$ kNm

Material details

Char.strength of concrete $f_{cu} = 30$ N/mm^2

Char.strength of reinft $f_y = 500$ N/mm^2

Char.strength of shear reinft $f_{yv} = 500$ N/mm^2

Nom.cover to reinforcement $c_{nom} = 30$ mm

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Moment design in x direction

Tens.reinforcement diameter ϕ_{xB} = **12** mm Tens.reinforcement depth d_x = **464** mm

Design formula for rectangular beams (cl 3.4.4.4) K_x = **0.003** K_x' = **0.156**

K_x < K_x' compression reinforcement is not required

Tens.reinforcement required $A_{s\_x\_req}$ = **129** mm^2 Minimum tens.reinforcement $A_{s\_x\_min}$ = **845** mm^2

Tens.reinforcement provided **8 No. 12 dia. bars btm** $A_{s\_xB\_prov}$ = **905** mm^2

PASS - Tension reinforcement provided exceeds tension reinforcement required

Moment design in y direction

Tens.reinforcement diameter ϕ_{yB} = **12** mm Tens.reinforcement depth d_y = **452** mm

Design formula for rectangular beams (cl 3.4.4.4) K_y = **0.000** K_y' = **0.156**

K_y < K_y' compression reinforcement is not required

Tens.reinforcement required $A_{s\_y\_req}$ = **20** mm^2 Minimum tens.reinforcement $A_{s\_y\_min}$ = **1560** mm^2

Tens.reinforcement provided **15 No. 12 dia. bars btm** $A_{s\_yB\_prov}$ = **1696** mm^2

PASS - Tension reinforcement provided exceeds tension reinforcement required

Calculate ultimate shear force at d from right face of column

Ult.pressure for shear q_{su} = **56.035** kN/m^2

Area loaded for shear A_s = **0.653** m^2 Ult.shear force V_{su} = **17.306** kN

Shear stresses at d from right face of column (cl 3.5.5.2)

Design shear stress v_{su} = **0.029** N/mm^2

Design concrete shear stress v_c = **0.347** N/mm^2 Allowable design shear stress v_{max} = **4.382** N/mm^2

PASS - v_{su} < v_c - No shear reinforcement required

Calculate ultimate punching shear force at face of column

Ult.press.for punching shear q_{puA} = **94.420** kN/m^2 Avg.effective reinf.depth d = **458** mm

Area loaded A_{pA} = **0.123** m^2 Length of shear perimeter u_{pA} = **1400** mm

Ult.punching shear force V_{puA} = **70.728** kN Eff.punching shear force V_{puAeff} = **70.728** kN

Punching shear stresses at face of column (cl 3.7.7.2)

Design shear stress v_{puA} = **0.110** N/mm^2

PASS - Design shear stress is less than allowable design shear stress

Calculate ultimate punching shear force at perimeter of 1.5 d from face of column

Ult.press.for punching shear $q_{puA1.5d}$ = **54.707** kN/m^2 Avg.effective reinf.depth d = **458** mm

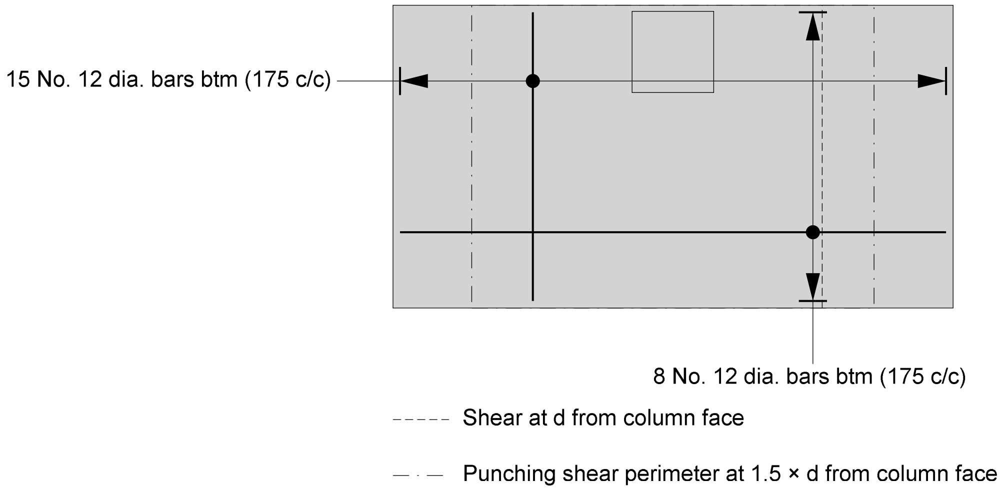
Area loaded $A_{pA1.5d}$ = **2.241** m^2 Length of shear perimeter $u_{pA1.5d}$ = **2600** mm

Ult.punching shear force $V_{puA1.5d}$ = **22.230** kN Eff.punching shear force $V_{puA1.5deff}$ = **27.787** kN

Punching shear stresses at perimeter of 1.5 d from face of column (cl 3.7.7.2)

Design shear stress $v_{puA1.5d}$ = **0.023** N/mm^2

PASS - $v_{puA1.5d}$ < v_c - No shear reinforcement required



15 No. 12 dia. bars btm (175 c/c)
8 No. 12 dia. bars btm (175 c/c)
Shear at d from column face
Punching shear perimeter at 1.5 × d from column face

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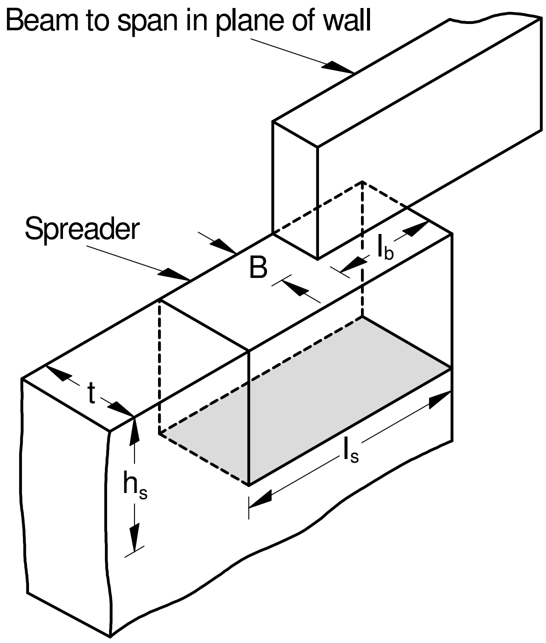
PADSTONE DESIGN SB-3

MASONRY BEARING DESIGN TO BS5628-1:2005

TEDDS calculation version 1.0.05

Masonry details

Masonry type	**Clay or calcium silicate bricks**		
Compressive strength	p_{unit} = **5.0** N/mm^2	Mortar designation	**iv**
Masonry units	**Category II**	Construction control	**Normal**
Partial safety factor	γ_m = **3.5**	Characteristic strength	f_k = **2.2** N/mm^2
Leaf thickness	t = **215** mm	Effective wall thickness	t_{ef} = **215** mm
Wall height	h = **2700** mm	Effective height of wall	h_{ef} = **2500** mm



Bearing details

Beam spanning in plane of wall

Width of bearing	B = **203** mm	Length of bearing	l_b = **200** mm

Loading details

Concentrated dead load	G_k = **18** kN	Concentrated imposed load	Q_k = **5** kN
Design concentrated load	F = **32.6** kN		
Distributed dead load	g_k = **20.4** kN/m	Distributed imposed load	q_k = **5.8** kN/m
Design distributed load	f = **37.7** kN/m		

Masonry bearing type

Bearing type	**Type 2**	Bearing safety factor	γ_{bear} = **1.50**

Check design bearing without a spreader

Design bearing stress	f_{ca} = **0.979** N/mm^2	Allowable bearing stress	f_{cp} = **0.943** N/mm^2

FAIL - Design bearing stress exceeds allowable bearing stress, use a spreader

Spreader details

Length of spreader	l_s = **215** mm	Depth of spreader	h_s = **215** mm

Edge distance s_{edge} = **0** mm

Spreader bearing type

Bearing type **Type 3**

Bearing safety factor γ_{bear} = **2.00**

Check design bearing with a spreader

Loading acts eccentrically within middle third – triangular stress distribution

Design bearing stress f_{ca} = **1.029** N/mm^2

Allowable bearing stress f_{cp} = **1.257** N/mm^2

PASS - Allowable bearing stress exceeds design bearing stress

Check design bearing at 0.4 × h below the bearing level

Design bearing stress f_{ca} = **0.294** N/mm^2

Allowable bearing stress f_{cp} = **0.622** N/mm^2

PASS - Allowable bearing stress at 0.4 × h below bearing level exceeds design bearing stress

Calc By	Project	Job No.	
Calc Date	Client	Page No.	Revision

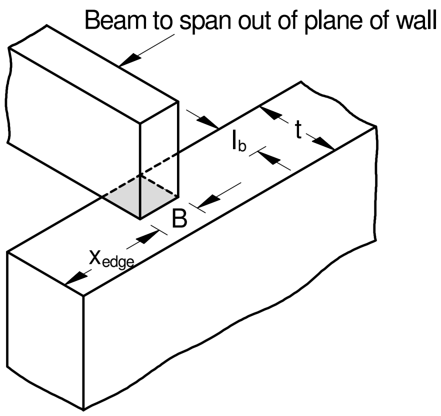
PADSTONE DESIGN SB-4

MASONRY BEARING DESIGN TO BS5628-1:2005

TEDDS calculation version 1.0.05

Masonry details

Masonry type	**Autoclaved aerated concrete blocks**		
Compressive strength	p_{unit} = **7.3** N/mm^2	Mortar designation	**iii**
Least horiz dim of units	l_{unit} = **100** mm	Height of units	h_{unit} = **215** mm
Masonry units	**Category II**	Construction control	**Normal**
Partial safety factor	γ_m = **3.5**	Characteristic strength	f_k = **6.4** N/mm^2
Leaf thickness	t = **100** mm	Effective wall thickness	t_{ef} = **100** mm
Wall height	h = **2700** mm	Effective height of wall	h_{ef} = **2500** mm



Bearing details

Beam spanning out of plane of wall

Width of bearing	B = **203** mm	Length of bearing	l_b = **100** mm
Edge distance	x_{edge} = **1300** mm		

Loading details

Concentrated dead load	G_k = **22** kN	Concentrated imposed load	Q_k = **4** kN
Design concentrated load	F = **36.9** kN		
Distributed dead load	g_k = **5.0** kN/m	Distributed imposed load	q_k = **2.0** kN/m
Design distributed load	f = **10.2** kN/m		

Masonry bearing type

Bearing type	**Type 2**	Bearing safety factor	γ_{bear} = **1.50**

Check design bearing without a spreader

Design bearing stress	f_{ca} = **1.920** N/mm^2	Allowable bearing stress	f_{cp} = **2.743** N/mm^2

PASS - Allowable bearing stress exceeds design bearing stress

Check design bearing at 0.4 × h below the bearing level

Design bearing stress	f_{ca} = **0.258** N/mm^2	Allowable bearing stress	f_{cp} = **1.024** N/mm^2

PASS - Allowable bearing stress at 0.4 × h below bearing level exceeds design bearing stress

FOUNDATIONS NEAR TREES (NHBC)

FOUNDATIONS NEAR TREES

In accordance with Appendix B of NHBC Part 4: Foundations - Chapter 4.2

Tedds calculation version 2.0.02

Site Details

Site location — London

Reduction depth due to climate variations - Fig. 13 — Z_c = **0.00** m

Soil Details

Plasticity index from lab tests — I_p = **40** %

Percentage of particles < 425 μm — p_{425} = **100** %

Modified plasticity index - cl. D5(b) — $I'_p = I_p \times p_{425} / 100\ \%$ = **40** %

Volume change potential - Table 1 — High

Details for Tree - 1

Species of tree — Broad leaf - Beech

The tree is to remain at the site, with no further planting allowed.

Water demand of tree - Table 12 — Moderate

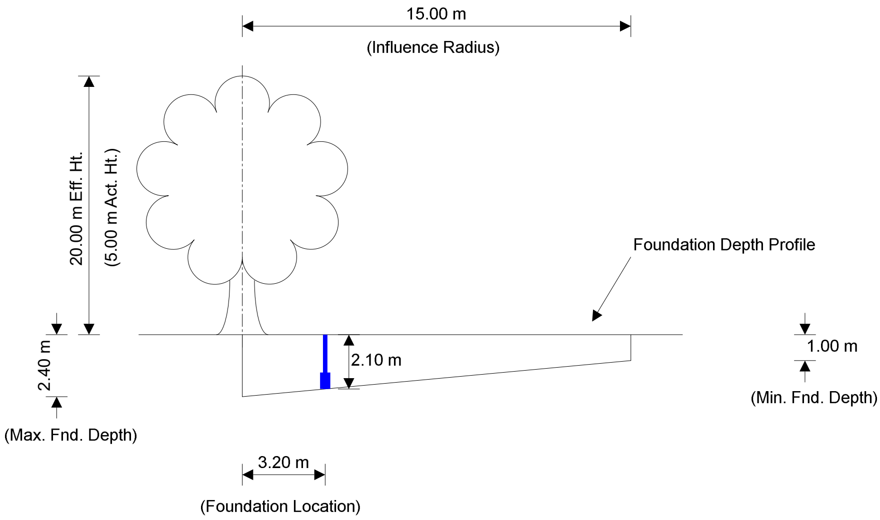
Mature height of tree - Table 12 — H_{m1} = **20.00** m

Influence radius - Table 2 — $r_{inf1} = 0.75 \times H_{m1}$ = **15.00** m

Measured height of tree — H_{act1} = **5.00** m

Distance from centre of tree to face of foundations — D_1 = **3.20** m

Effective height of tree - Fig. 1 — H_{eff1} = **20.00** m



Minimum foundation depth - Table 5 — Z_{min} = **1.00** m

Look up value for foundation depth - Chart 1 Soils with HIGH volume change potential

$Z_{LookUp1}$ = **2.10** m

Required foundation depth — $Z_{req1} = Z_{LookUp1} - Z_c$ = **2.10** m

Details for Tree - 2

Species of tree — Broad leaf - Beech

The tree is to remain at the site, with no further planting allowed.

Water demand of tree - Table 12 — Moderate

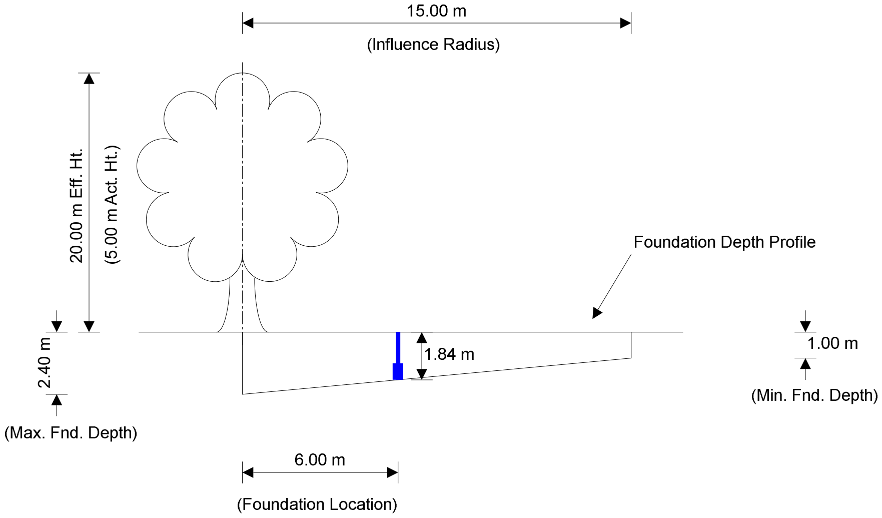
Mature height of tree - Table 12 H_{m2} = **20.00** m

Influence radius - Table 2 $r_{inf2} = 0.75 \times H_{m2}$ = **15.00** m

Measured height of tree H_{act2} = **5.00** m

Distance from centre of tree to face of foundations D_2 = **6.00** m

Effective height of tree - Fig. 1 H_{eff2} = **20.00** m



Minimum foundation depth - Table 5 Z_{min} = **1.00** m

Look up value for foundation depth - Chart 1 Soils with HIGH volume change potential

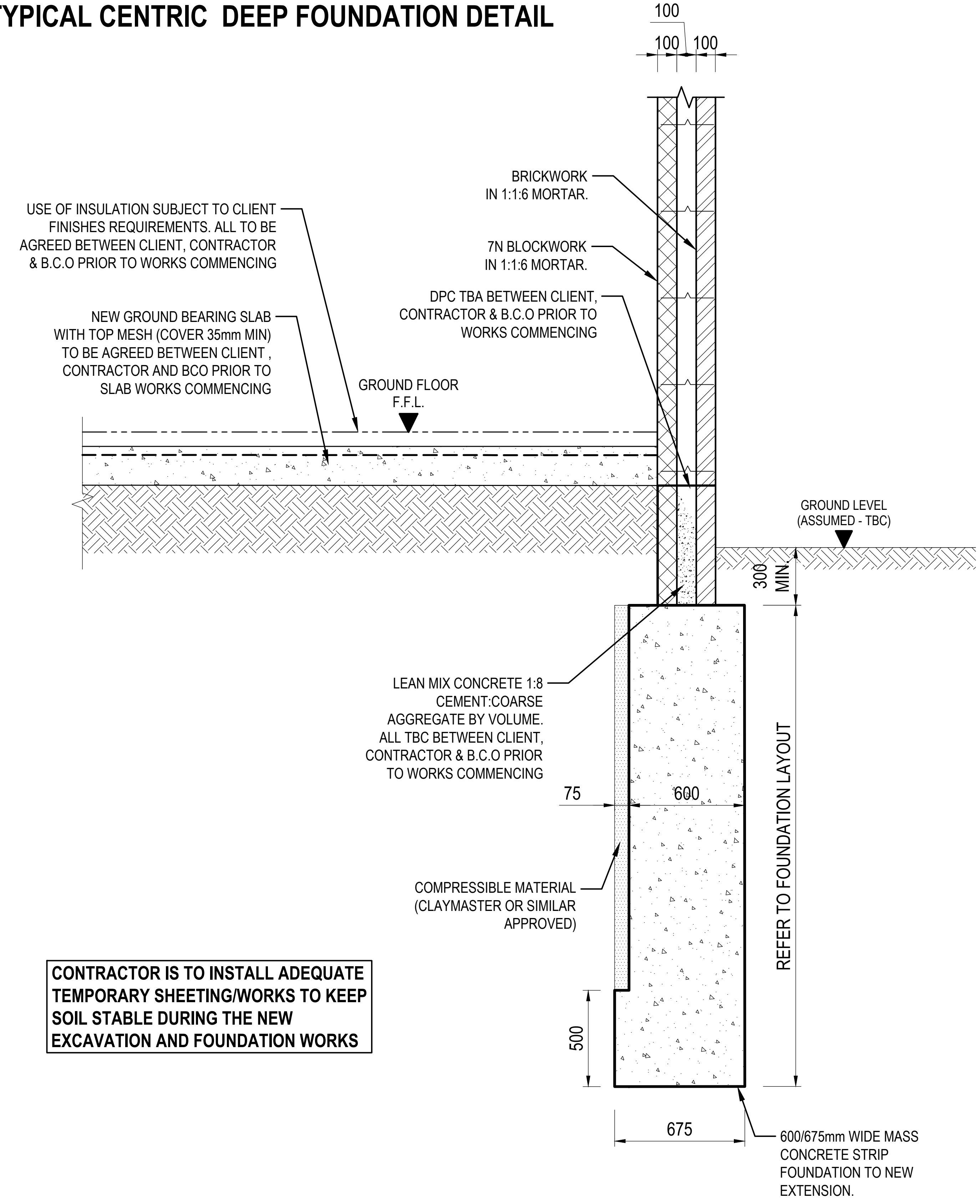
$Z_{LookUp2}$ = **1.84** m

Required foundation depth $Z_{req2} = Z_{LookUp2} - Z_c$ = **1.84** m

Summary Table

Tree	Name	Distance (m)	Measured Height (m)	Effective Height (m)	Tree to be removed	Required Foundation Depth (m)
1	Beech	3.2	5.0	20.0	No	2.10
2	Beech	6.0	5.0	20.0	No	1.84

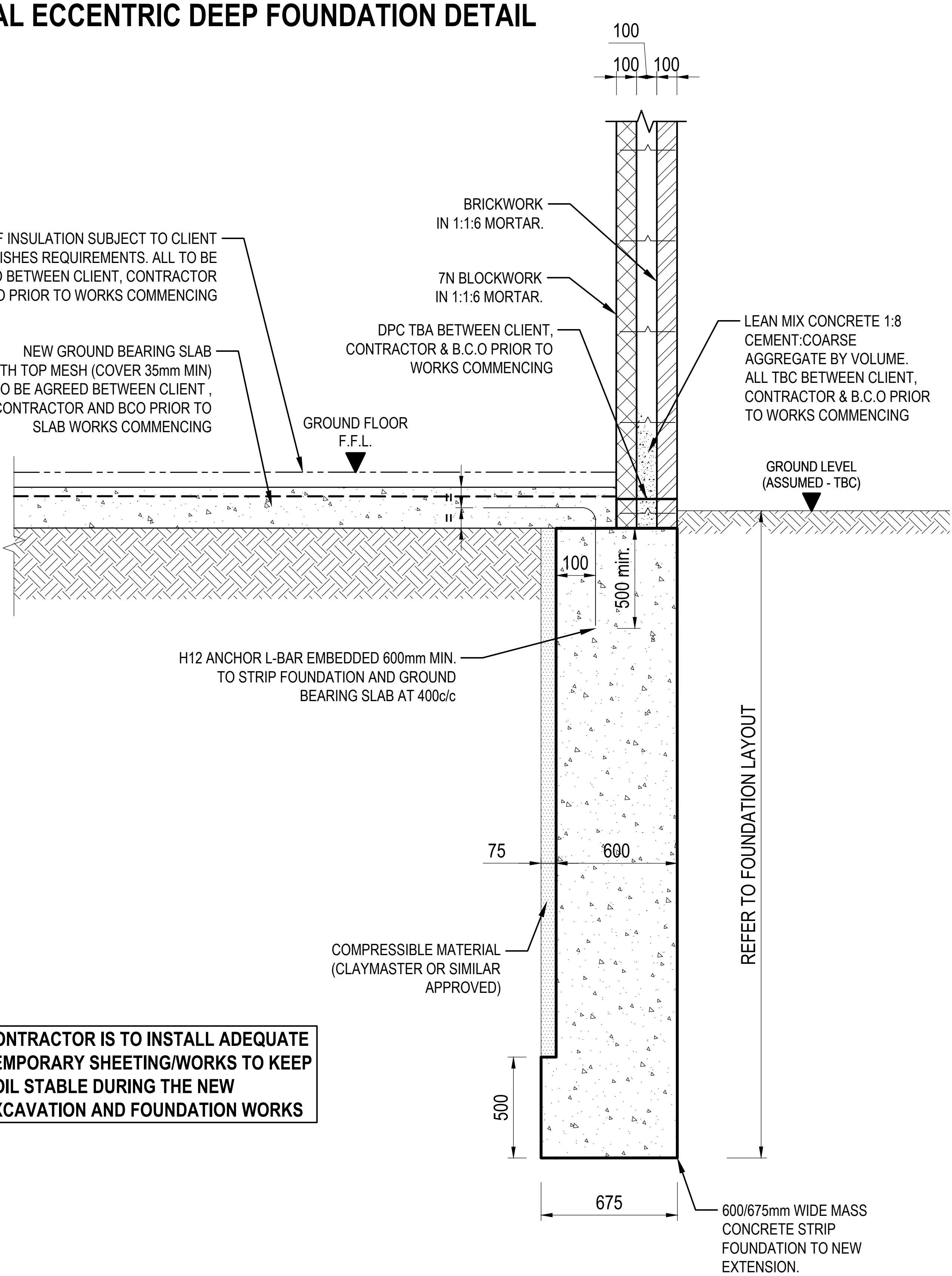
TYPICAL CENTRIC DEEP FOUNDATION DETAIL



CONTRACTOR IS TO INSTALL ADEQUATE TEMPORARY SHEETING/WORKS TO KEEP SOIL STABLE DURING THE NEW EXCAVATION AND FOUNDATION WORKS

PROJECT:	DRAWING:	CLIENT:	DATE:	SCALE @A4:	
			DRG. No.:	REV.:	DATE:

TYPICAL ECCENTRIC DEEP FOUNDATION DETAIL

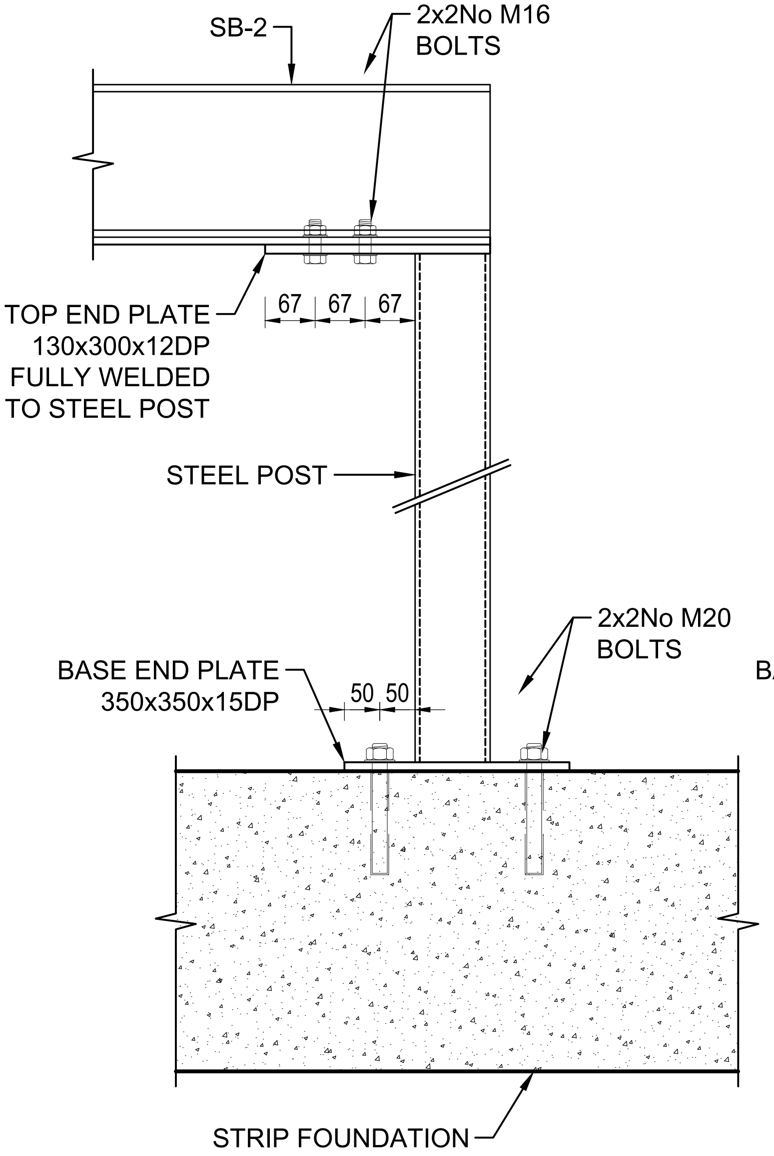


CONTRACTOR IS TO INSTALL ADEQUATE TEMPORARY SHEETING/WORKS TO KEEP SOIL STABLE DURING THE NEW EXCAVATION AND FOUNDATION WORKS

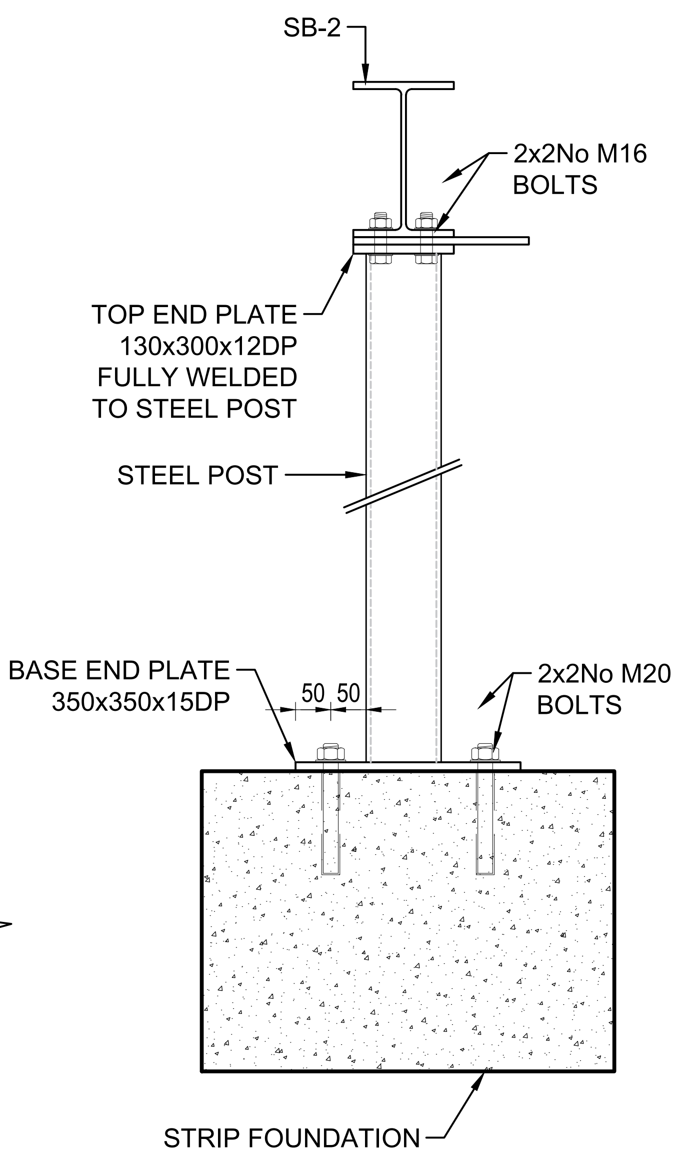
PROJECT:	DRAWING:	CLIENT:	DATE:	SCALE @A4:	
			DRG. No.:	REV.:	DATE:

STEEL POST CONNECTION DETAILS

SIDE ELEVATION



FRONT ELEVATION



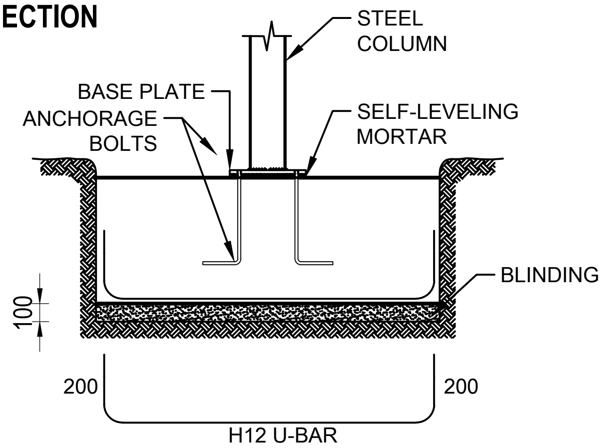
► ***ALL STRUCTURAL MEMBERS MUST BE SIZED ACCORDING TO THE STRUCTURAL CALCULATIONS***

► ***STEEL BEAMS AND STEEL PLATE SHOULD BE WELDED WITH THE STEEL COLUMN BY FULL PENETRATION 6mm WELD AROUND THE PERIMETER CONTACT***

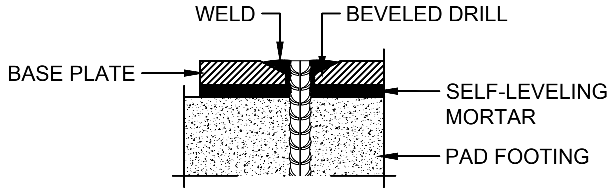
PROJECT:	DRAWING:	CLIENT:	DATE:	SCALE @A4:	
			DRG. No.:	REV.:	DATE:

PAD FOOTING 1 DETAIL

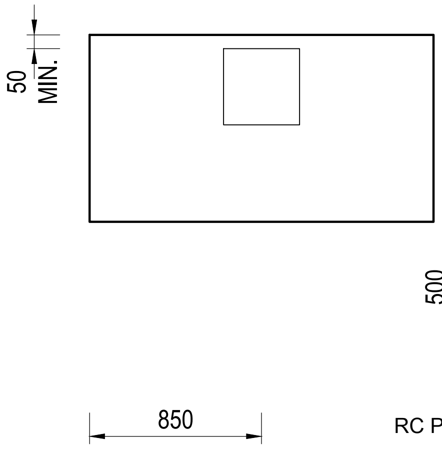
SECTION

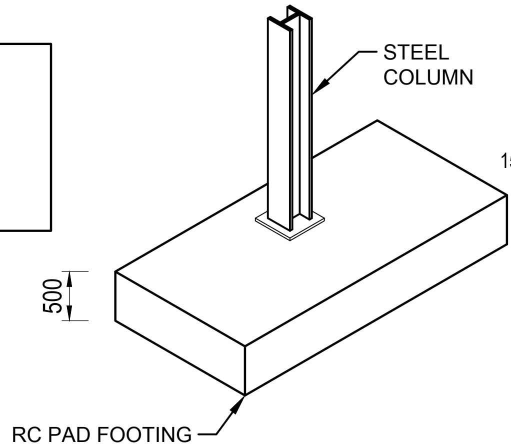


ANCHORAGE SYSTEM DETAIL

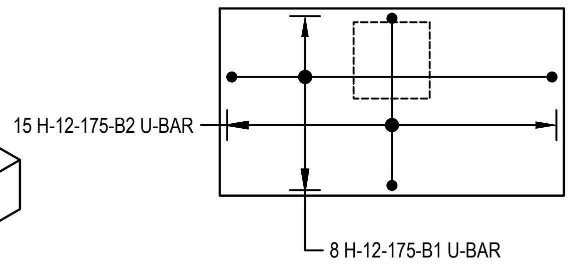


PAD FOOTING PLAN





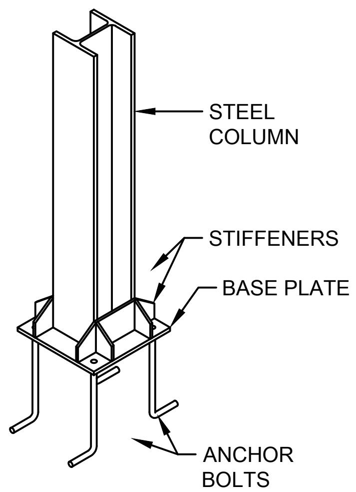
PAD FOOTING REINFORCEMENT PLAN



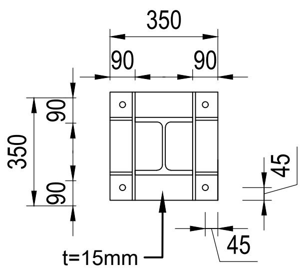
COVER = 50mm

STEEL COLUMN BASE PLATE

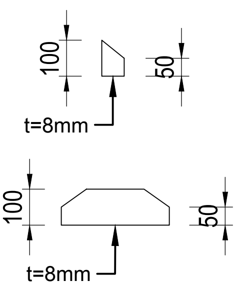
BASE PLATE:	350x350x15DP
ANCHOR BOLTS:	4No M20 GRADE 8.8 MIN.



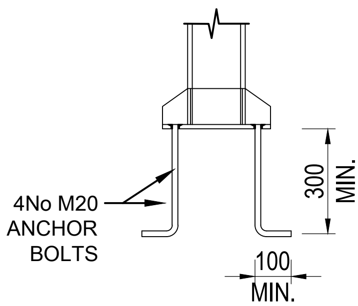
BASE PLATE DETAIL



STIFFENERS DETAIL



ANCHOR BOLT DETAIL

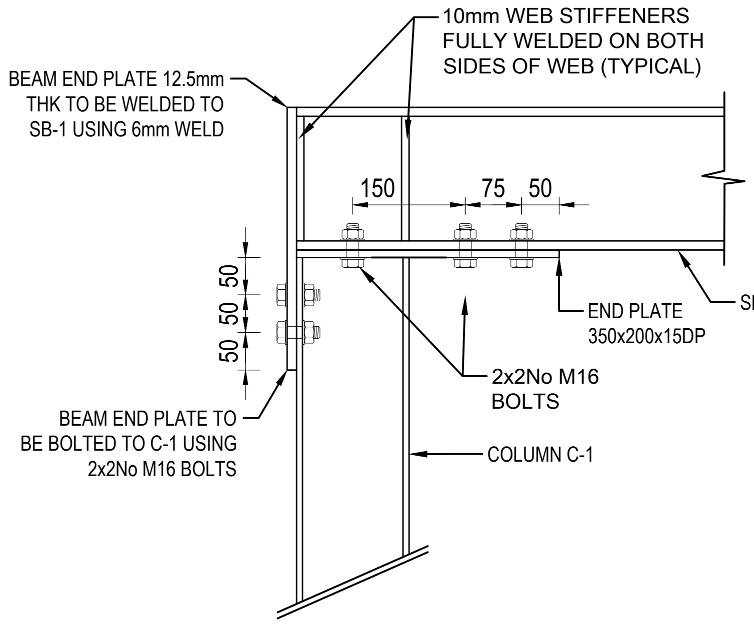


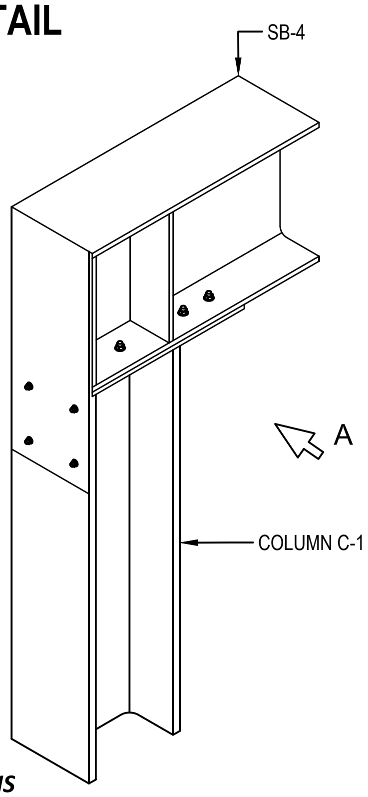
► ***PROFILES, BASE PLATE AND STIFFENERS MUST BE WELDED AROUND THE PERIMETER OF CONTACT***
► ***THE WELDS WILL BE CONTINUOUS WITH FULL PENETRATION***
► ***ALL STRUCTURAL MEMBERS MUST BE SIZED IN ACCORDANCE WITH THE STRUCTURAL CALCULATIONS***

PROJECT:	DRAWING:	CLIENT:	DATE:	SCALE @A4:	
			DRG. No.:	REV.:	DATE:

STEEL BEAM SB-4 TO STEEL COLUMN C-1 CONNECTION DETAIL

ELEVATION 'A'

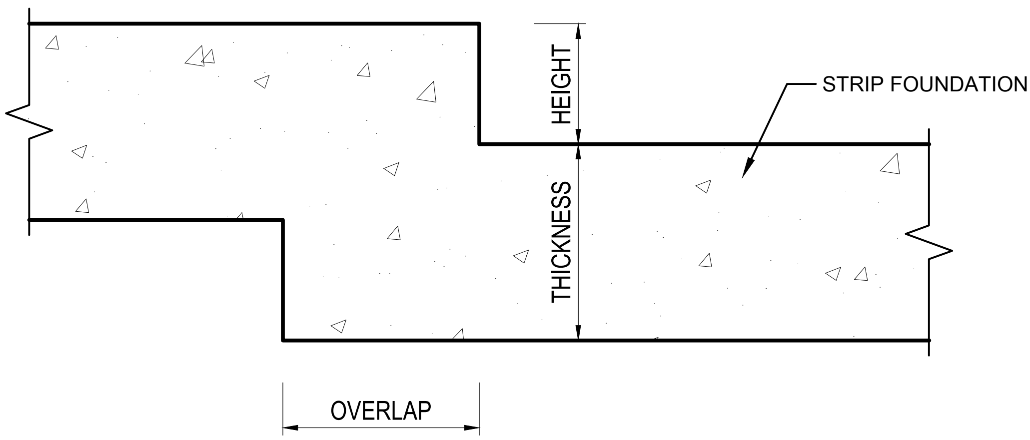




► ***ALL STRUCTURAL MEMBERS MUST BE SIZED ACCORDING TO THE STRUCTURAL CALCULATIONS***

► ***STEEL BEAMS AND STEEL PLATE SHOULD BE WELDED WITH THE STEEL COLUMN BY FULL PENETRATION 6mm WELD AROUND THE PERIMETER CONTACT***

STEPPED FOUNDATION DETAIL

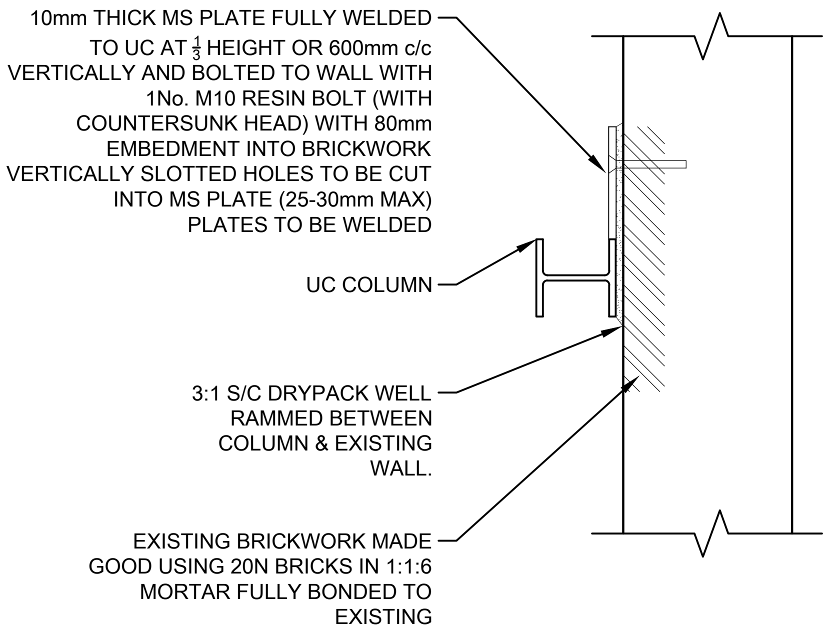


OVERLAP TO BE NOT LESS THAN 2xHEIGHT OR 1m, WHICHEVER IS GREATER.

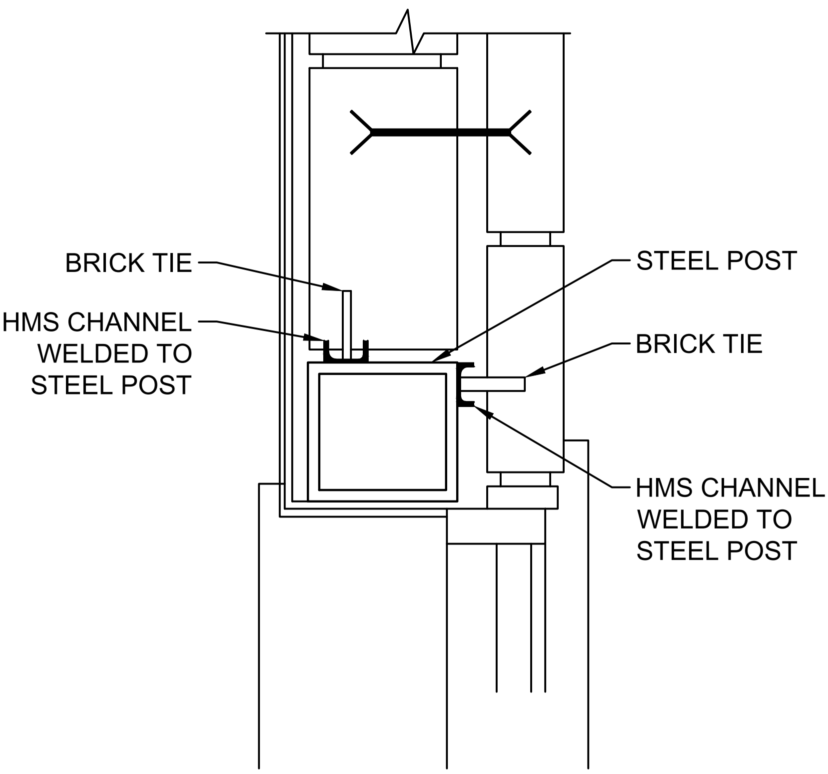
► ***ALL STRUCTURAL MEMBERS MUST BE SIZED ACCORDING TO THE STRUCTURAL CALCULATIONS***

PROJECT:	DRAWING:	CLIENT:	DATE:	SCALE @A4:	
			DRG. No.:	REV.:	DATE:

TYPICAL INTERNAL COLUMN TIE DETAIL



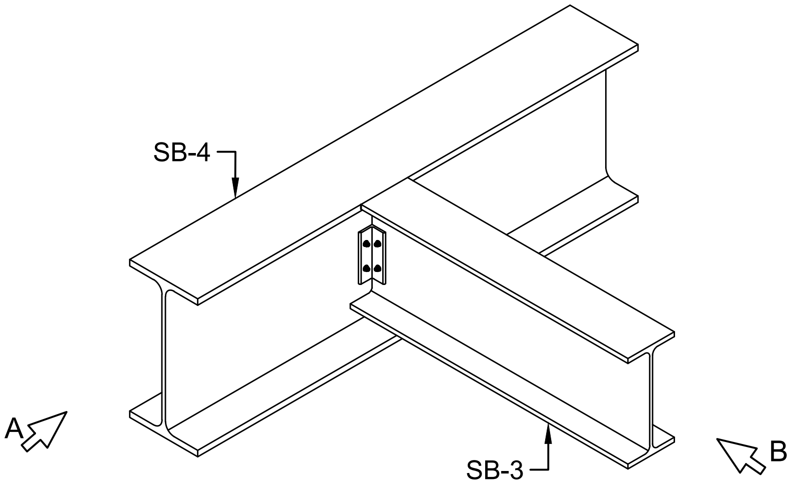
CAVITY WALL TIED TO STEEL POST



- ***ALL STRUCTURAL MEMBERS MUST BE SIZED ACCORDING TO THE STRUCTURAL CALCULATIONS***
- ***USE HALFEN BRICK TIE SYSTEM OR SIMILAR APPROVED***
- ***HMS CHANNEL SHOULD BE WELDED WITH THE STEEP POST BY FULL PENETRATION WELD AROUND THE PERIMETER CONTACT.***

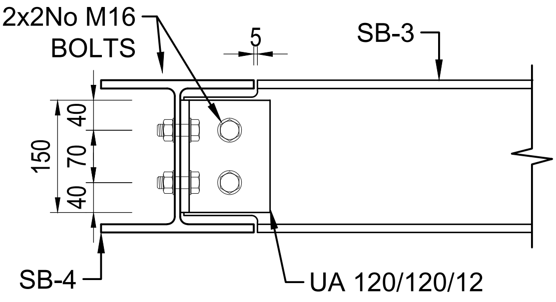
PROJECT:	DRAWING:	CLIENT:	DATE:	SCALE @A4:	
			DRG. No.:	REV.:	DATE:

STEEL BEAM SB-3 TO STEEL BEAM SB-4 CONNECTION DETAIL

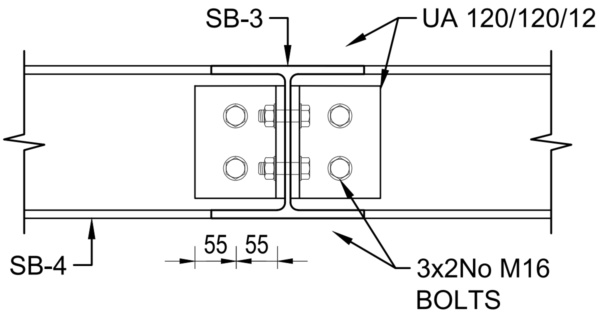


► ***ALL STRUCTURAL MEMBERS MUST BE SIZED ACCORDING TO THE STRUCTURAL CALCULATIONS***

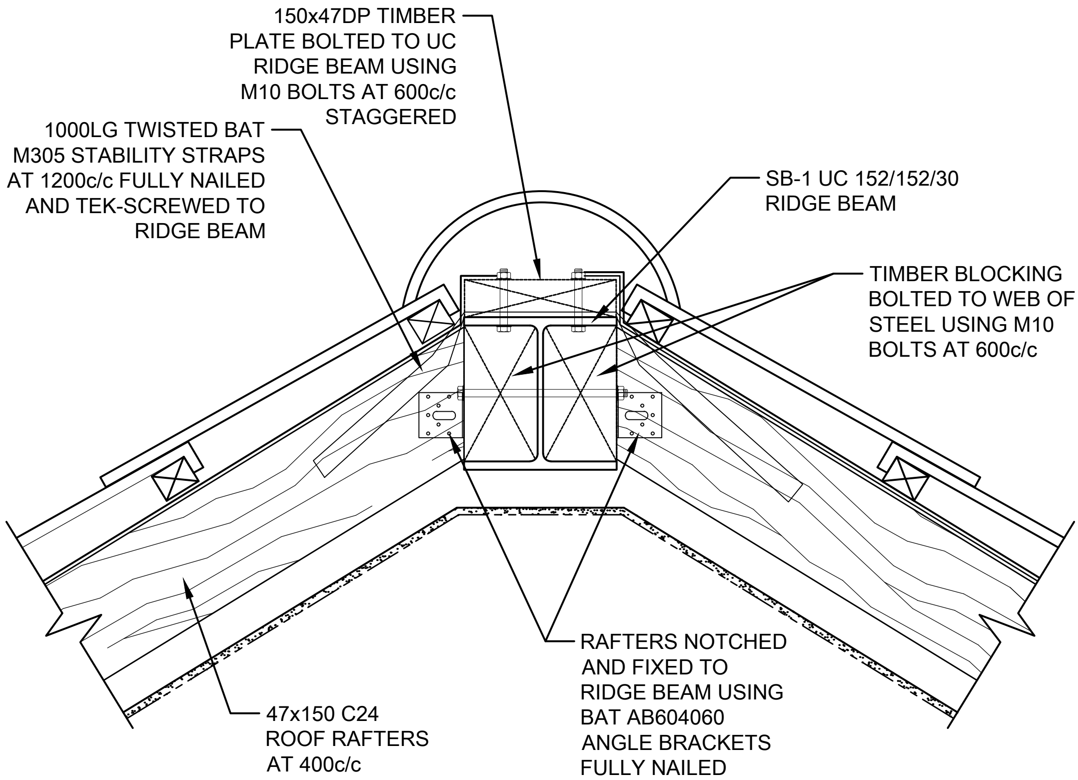
ELEVATION 'A'



ELEVATION 'B'



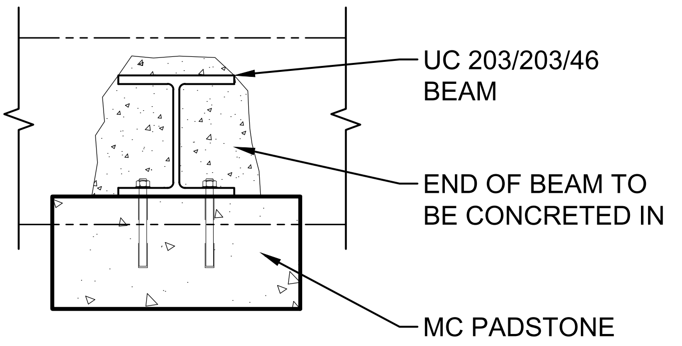
RIDGE BEAM DETAIL

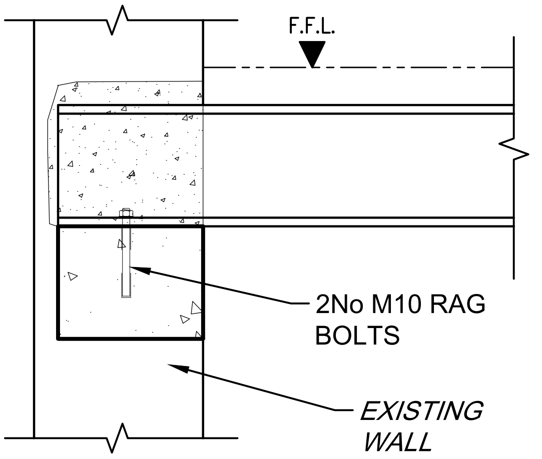


► ***ALL STRUCTURAL MEMBERS MUST BE SIZED ACCORDING TO THE STRUCTURAL CALCULATIONS***

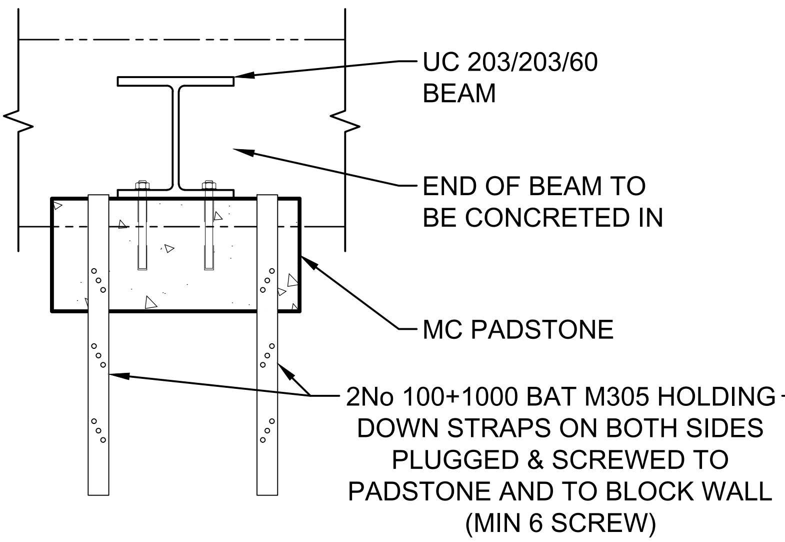
PROJECT:	DRAWING:	CLIENT:	DATE:	SCALE @A4:	
			DRG. No.:	REV.:	DATE:

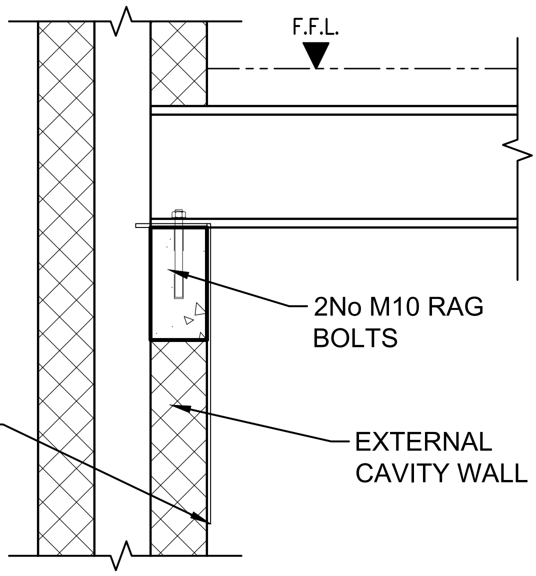
TYPICAL UC BEAM BOLTED TO PADSTONE DETIAL





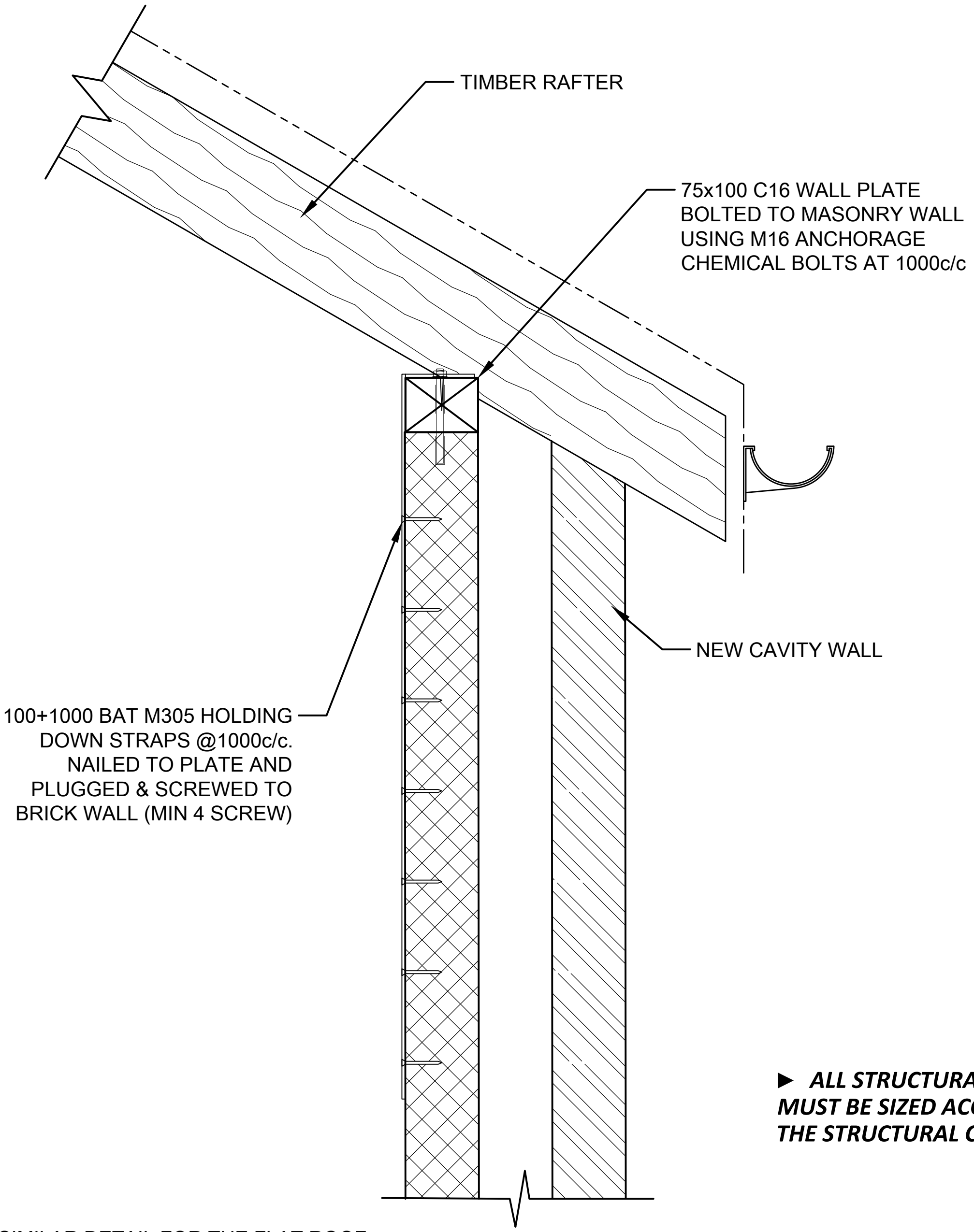
TYPICAL UC/UB BEAM BOLTED TO PADSTONE DETIAL





PROJECT:	DRAWING:	CLIENT:	DATE:	SCALE @A4:	
			DRG. No.:	REV.:	DATE:

HOLDING DOWN STRAP TO WALL PLATE DETAIL

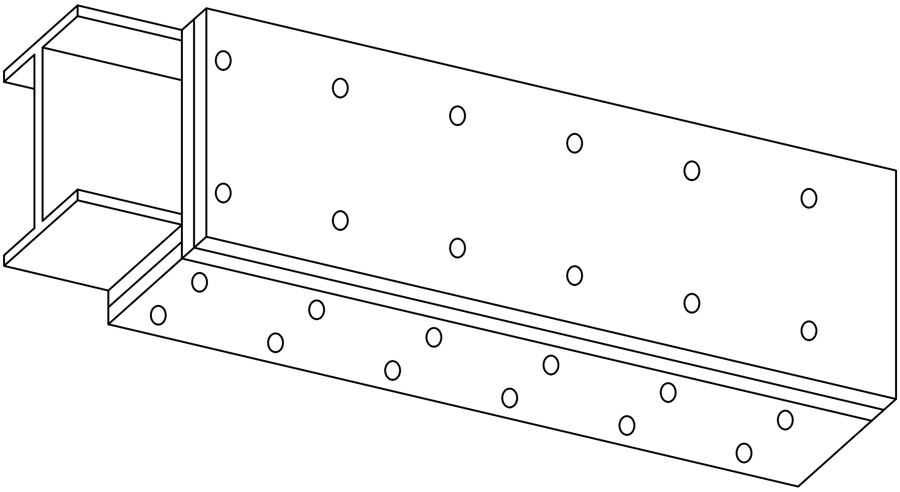


► ***ALL STRUCTURAL MEMBERS MUST BE SIZED ACCORDING TO THE STRUCTURAL CALCULATIONS***

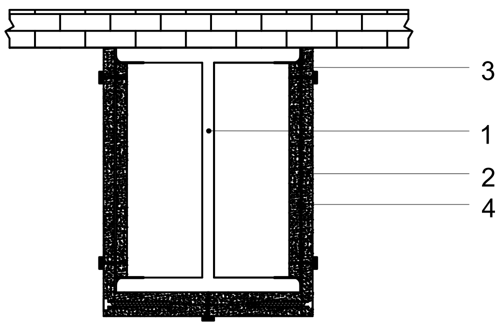
NOTE: PROVIDE SIMILAR DETAIL FOR THE FLAT ROOF

PROJECT:	DRAWING:	CLIENT:	DATE:	SCALE @A4:	
			DRG. No.:	REV.:	DATE:

BRITISH GYPSUM FIRECASE - Fire protection for beams



SECTION

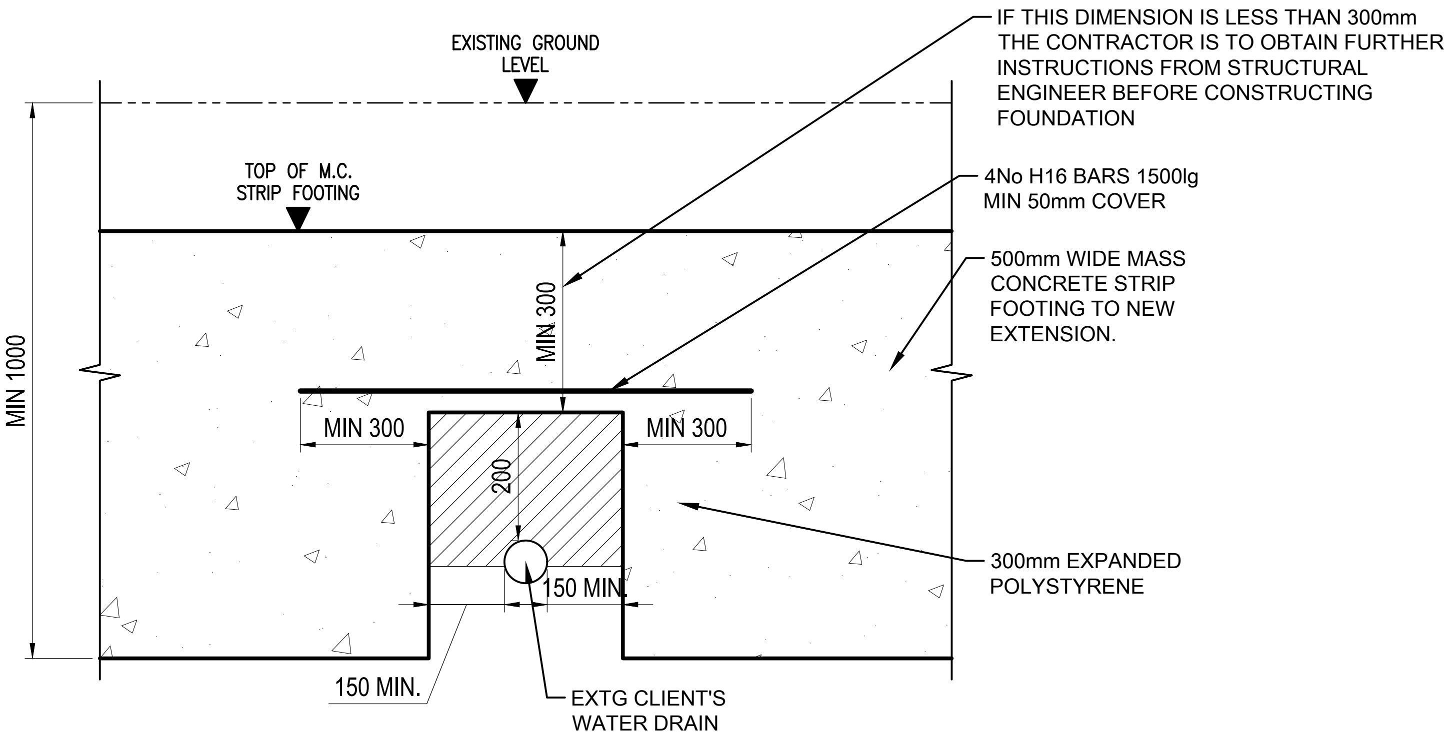


KEY

1. Steel beam
2. Glasroc F Firecase fixed together with Glasroc F Firecase Screws or Glasroc Staples & to FEA1 steel angles with Glasroc F Firecase Screws at 150mm centres.
3. Gypframe FEA1 Steel Angle suitably fixed to beam flange at 600mm centres
4. 60mm wide Glasroc F Firecase backing strip at board joisnts

► STRUCTURAL ELEMENTS MUST BE SIZED ACCORDING TO THE STRUCTURAL CALCULATION

EXISTING DRAIN BRIDGING DETAIL



PROJECT:	DRAWING:	CLIENT:	DATE:	SCALE @A4:	
			DRG. No.:	REV.:	DATE:

METHOD STATEMENT FOR STEEL BEAM INSTALLATION

1. Check the original construction techniques for correct type of temporary works.
2. Check condition of existing brickwork - break out and replace any badly damaged bricks, re-point mortar as required. Ensure any repairs to brickwork is checked and approved by BCO before works commence.
3. Support existing wall above intended beam by Needle Masonry Supports which are supported by Acrow props each 500mm. Install Acrow props support from interior or exterior (not from both side) on the existing (concrete or soil) ground, not on the joist floor. Under Acrow props use wooden or concrete base plate minimum 100x500x500mm.
4. Temporary support installation:

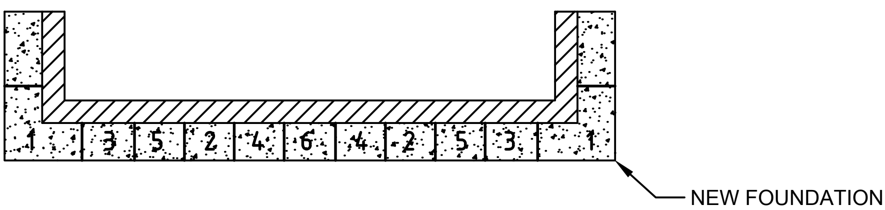
   a) Mark the wall in the correct position where the steel needles are to be inserted
   b) Scrape or grind out the mortar, or remove a brick prior to insertion of the needles in the identified position. Always ensure the underside of the brick to be supported is clean and will sit flat on the blade of the needles.
   c) Insert the blade into the mortar space or brick hole until the blade is at least at the same depth as the rear of the brick on the leaf of the wall is intended to be supported. Where possible the needles should be inserted until the tip of the web is nearly touching the wall.
   d) Ensuring the prop remains completely vertical and in plumb, tighten the collar of the prop until Strongboy and prop are fully engaged with the wall and do not move. Do not over tighten as that may cause the blade to bend or damage the brickwork. Hand tight is generally sufficient.

4. For point load use extra temporary support/s.
5. Make sure the wall is supported properly by temporary support.
6. Knock down wall carefully under temporary support.
7. Install beam from free side of the opening.
8. Make sure that existing wall is supported properly by new steel beam.
9. Remove temporary support.

METHOD STATEMENT FOR POURING STRIP FOUNDATION

1. The contractor to locate and protect any existing buried services.
2. Pouring and excavation sequences are to be not longer than shown on foundation drawing.
3. Poring and excavation is to be carried out in order of shown sequences below starting with no. 1 only if new foundation is to be poured nearby existing neighbor's/adjoining foundation. This to be confirmed by the Building Control Officer on site.
4. Not less than 48 hours after casting blocks (1) excavate adjacent blocks.
5. Continue the above sequence until the perimeter foundation is casted.
6. Under no circumstances shall two simultaneous excavations be made on both sides of a length of wall.
7. Before casting concrete the underside of the already casted footing to be thoroughly cleaned to remove any soil adhering to the underside and then given a coat of rich cement grout to prime the surface.
8. No excavation to be left open for more than three hours before placing the concrete.
9. Not less than 48 hours after pouring the concrete the side sutter may be released.
10. The concrete is to be thoroughly vibrated with a mechanical poker to ensure maximum compaction.
11. If during excavation material is found to be significantly different from exposed in trial pits, the contractor shall inform the Engineer who may visit site to inspect the excavation.
12. Do not use concrete grade less than C20.

GENERIC POURING SCHEDULE



PROJECT:	DRAWING:	CLIENT:	DATE:	SCALE @A4:	
			DRG. No.:	REV.:	DATE:

GENERAL NOTES:

1. THE STRUCTURAL SPECIFICATION WITHIN THIS DOCUMENT IS TO BE READ IN CONJUNCTION WITH ALL RELEVANT ARCHITECTURAL DRAWINGS, THE STANDARD NOTES THAT ARE CONTAINED ON THE DRAWINGS AND ANY OTHER RELEVANT PROJECT INFORMATION.
2. THE DRAWINGS WITHIN THIS DOCUMENT ARE INDICATIVE ONLY, AND REPRESENT DESIGN INTENT ONLY. NO DIMENSIONS ARE TO BE SCALED FROM DRAWINGS CONTAINED WITHIN THIS DOCUMENT. REFER TO THE ARCHITECTURAL DRAWINGS FOR ALL DIMENSIONS. ALL DIMENSIONS SHOULD BE CHECKED ON SITE BY THE CONTRACTOR BEFORE FABRICATION AND ORDERING OF MATERIALS.
3. UNLESS NOTED OTHERWISE, ALL DIMENSIONS ARE IN MILLIMETRES AND ALL LEVELS ARE IN METRES FROM THE SITE DATUM.
4. THE INFORMATION WITHIN THIS DOCUMENT CANNOT BE GUARANTEED AS DIMENSIONALLY EXACT. FIGURED DIMENSIONS MUST BE USED FOR SETTING OUT AND DETAILING.
5. THE CONTRACTOR IS RESPONSIBLE FOR THE DESIGN OF ALL TEMPORARY WORKS, AND IS ALSO RESPONSIBLE FOR THE SAFE MAINTENANCE AND STABILITY OF THE EXISTING BUILDING/S AT ALL TIMES.
6. ALL PARTY WALL AWARDS ARE ENTIRELY THE RESPONSIBILITY OF THE CLIENT
7. **THE CONTRACTOR (OR CLIENT) MUST REPORT ANY DIFFERENCES BETWEEN THE STRUCTURAL DRAWINGS AND SITE CONDITIONS TO THE STRUCTURAL ENGINEER.**
8. **THE CONTRACTOR (OR CLIENT) MUST NOTIFY THE STRUCTURAL ENGINEER OF ANY DESIGN CHANGES THAT COULD AFFECT THE STRUCTURAL SPECIFICATION BEFORE WORK COMMENCES.**
9. THE CONTRACTOR SHALL AT THE OUTSET, ESTABLISH WITH THE LOCAL AUTHORITY THEIR REQUIREMENTS FOR INSPECTING THE WORKS, AND ADHERE TO THESE.
10. ALL DIMENSIONS AND LEVELS SHOWN ON THE DRAWINGS ARE BASED ON SURVEY DRAWINGS AND THE CONTRACTOR IS TO SATISFY HIMSELF THAT DIMENSIONS, LEVELS, ETC., ARE SUFFICIENTLY ACCURATE AND COMPLETE FOR FABRICATION WITHIN THE SPECIFIED TOLERANCES OF ALL PREFABRICATED ELEMENTS.
11. IF IN DOUBT ABOUT THE INFORMATION SHOWN OT THIS DRAWING OR ANY RELATED DRAWINGS - PLEASE ASK.
12. **THE CONTRACTOR HAS SOLE RESPONSIBILITY FOR THE DESIGN OF ALL TEMPORARY WORKS.**
13. ALL LINTELS TO HAVE A MINIMUM 150mm END BEARING INTO NEW/EXISTING WALL.
14. ALL EXISTING LINTELS ARE TO BE INSPECTED AND REPLACED IF THEY SHOW SIGNS OF DETERIORATION, CRACKING OR DISTRESS. THE MATERIAL NATURE OF ANY REPLACEMENT IS TO BE APPROVED BY THE LOCAL AUTHORITY PRIOR TO INSTALLATION.
15. ALL EXISTING WALLS TO BE EXAMINED BY THE CONTRACTOR FOR LACK OF BOND/DELAMINATION, ETC. IF SUCH AREAS ARE NOTED, THE CLIENT'S ENGINEER IS TO BE INFORMED IMMEDIATELY. THE CONTRACTOR SHALL ARRANGE FOR THE DESIGN AND INSTALLATION OF A SUITABLE REMEDIAL TIE SYSTEM.
16. SHOULD ANY EXISTING WALL PROVE TO BE INADEQUATELY RESTRAINED, THE CONTRACTOR ALLOW FOR THE DESIGN AND INSTALLATION OF SUITABLE REMEDIAL WORKS TO PROVIDE ADEQUATE LATERAL RESTRAINT AND SUBMIT THE DESIGN TO THE LOCAL AUTHORITY FOR APPROVAL.
17. THE CONTRACTOR IS TO PROVIDE TEMPORARY WORKS TO PROVIDE LATERAL AND VERTICAL RESTRAINT TO EXISTING WALLS PRIOR TO DEMOLITION OF ANY PART OF THE EXISTING BUILDING COMMENCING.
18. IN ALL CASES THE EXISTING WALLS WILL REQUIRE THE FOLLOWING REMEDIAL WORKS: i) ALL EXISTING STEELWORK AND TIMBERS REQUIRED TO BE CAREFULLY REMOVED. THE CONTRACTOR SHOULD OBTAIN THE APPROVAL OF THE RELEVANT LOCAL AUTHORITIES PRIOR TO THIS WORK BEING CARRIED OUT.

    ii) ALL MINOR CRACKS TO BE REPAIRED USING 1:1:6 MORTAR.

    iii) ANY MORTAR/BRICKWORK THAT IS JUDGED BY THE CONTRACTOR OR THE CLIENT'S ENGINEER OR THE LOCAL AUTHORITY REPRESENTATIVE TO BE CRUMBLY, SOFT, DETERIORATED, ETC., IS TO BE REMOVED AND REBUILT IN BONDED ENGINEERING BRICK, FACING BRICK OR BLOCK AS APPROPRIATE.

    iv) ALL EXISTING INTERNAL MASONRY WALL OPENINGS NO LONGER REQUIRED ARE TO BE IN-FILLED IN BONDED BRICK/BLOCK AS APPROPRIATE.

    v) ALL STRAIGHT JOINTS ARE TO BE TIED ACROSS BY MEANS OF PROPRIETARY GALVANIZED STEEL STRAPPING.

    vi) IF CRACKS WITH AN APERTURE GREATER THAN 2mm ARE UNCOVERED, THE CONTRACTOR SHALL PROVIDE STITCHING USING 900mm LONG x 100mm WIDE x 65mm DEEP PRE-CAST CONCRETE LINTELS AT 900mm CENTRES VERTICALLY. THE LINTELS ARE TO BE INSERTED ON A 1:1:6 MORTAR BED AND PACKED WITH A 1:1:6 SEMI-DRY MORTAR WELL RAMMED.
19. JOISTS ARE TO BE DOUBLED-UP BELOW ALL STUD PARTITIONS. THE JOISTS TO BE BOLTED TOGETHER AT 600mm CENTRES USING M12 GR. 4.6 BOLTS WITH 50mm SQUARE PLATE WASHERS EACH SIDE AND DOUBLE SIDED TOOTHED PLATE CONNECTORS.

FOUNDATION AND GROUND FLOOR NOTES:

1. THE MAIN CONTRACTOR TO ENSURE THAT THE GROUND BEARING STRATA, MINIMUM BEARING DEPTHS AND GENERAL RECOMMENDATIONS OF THE SOIL INVESTIGATION ARE ADHERED TO.
2. THE MAIN CONTRACTOR TO UNDERTAKE EXPLORATORY EXCAVATIONS ADJACENT TO ANY EXISTING BUILDINGS AND SERVICES TO DETERMINE NATURE AND DEPTH OF EXISTING FOOTINGS ETC. CONTRACTOR TO ENSURE THAT ALL FOUNDATIONS TO EXISTING BUILDINGS AND UNDERGROUND SERVICES ARE NOT UNDERMINED OR DISTURBED IN ANY WAY DURING THE CONSTRUCTION WORKS.
3. CONTRACTOR TO BE RESPONSIBLE FOR ALL TEMPORARY WORKS DURING CONSTRUCTION OF THE SUPERSTRUCTURE AND FOUNDATIONS TO ENSURE THAT ALL ADJACENT BUILDINGS, ROADS, FOOTPATHS AND SERVICES ETC. REMAIN STABLE AND FREE FROM DAMAGE.
4. FOUNDATIONS ARE TO BE TAKEN DOWN TO THE DEPTHS SHOWN ON THIS DRAWING. TO BE CHECKED ON-SITE BY BUILDING CONTROL OFFICER/ENGINEER PRIOR TO CONSTRUCTION.
5. UNLESS NOTED OTHERWISE THUS: (600) ALL FOUNDATIONS TO BE A MINIMUM 600MM WIDE, CENTRED ON THE GRID LOCATION OVER.
6. ALL FORMATION LEVELS, SHUTTERS AND REINFORCEMENT TO BE CHECKED ON-SITE BY THE BUILDING CONTROL OFFICER PRIOR TO CONSTRUCTION.
7. ANY EXCAVATIONS LIABLE TO REMAIN EXPOSED TO THE ELEMENTS IN EXCESS OF 24 HOURS PRIOR TO CONCRETING (OR LESS IN INCLEMENT WEATHER) ARE TO RECEIVE 50MM PROTECTIVE BLINDING CONCRETE.
8. IN COHESIVE SOILS THE VERTICAL FACES OF ALL EXCAVATIONS MUST BE SUFFICIENTLY SMOOTH TO ALLOW MOVEMENT OF THE SOIL TO TAKE PLACE WITHOUT DISTURBING THE FOUNDATION.
9. FOUNDATIONS MAY BE TRENCH FILLED BUT THE TOP LEVEL OF CONCRETE MUST SUIT BRICK COURSING AND FINAL GROUND LEVELS.
10. NO SERVICE MAY BE BUILT THROUGH OR CAST INTO FOUNDATIONS WITHOUT THE ENGINEERS APPROVAL.
11. WHERE BLOCKWORK WALLING BELOW GROUND LEVEL IS IN EXCESS OF 900MM HIGH THEN 215MM THICK BLOCK WORK IS TO BE ADOPTED (BLOCKS LAID FLAT AND COURSED TO 225MM VERTICAL CENTERS). BACK FILLING IS TO BE CARRIED OUT EQUALLY EITHER SIDE OF WALLING AND TO BE OF AN APPROVED, WELL GRADED GRANULAR MATERIAL.
12. FOR EXISTING MANHOLES/DRAINRUNS/SERVICES REFERENCE IS TO BE MADE TO THE ARCHITECTURAL DRAWINGS. THE DEPTH AND POSITIONS OF ALL EXISTING SERVICES ARE TO BE CHECKED ON SITE BY THE CONTRACTOR PRIOR TO COMMENCING WORK AND ANY VARIATION WITH THE ARCHITECTURAL DRAWINGS IS TO BE REPORTED TO THEM PRIOR TO PROCEEDING. IF ANY DAMAGE IS CAUSED TO THE EXISTING RUNS/SERVICES DURING THESE WORKS BY THE CONTRACTOR ANY MAKING GOOD WILL BE AT THE CONTRACTORS OWN COST.

STEELWORK NOTES:

1. STRUCTURAL STEELWORK DESIGN TO BS5950-1:2000.
2. DESIGN, FABRICATION AND ERECTION OF STRUCTURAL STEELWORK TO COMPLY WITH THE LATEST EDITION OF THE NATIONAL STRUCTURAL STEELWORK SPECIFICATION (N.S.S.S.).
3. THE ENDS OF ALL BEAMS ARE TO BE PROPERLY SAWN/MACHINED, IN THE FABRICATION WORKSHOP, TO ENSURE A CLOSE BEARING FIT BETWEEN FLANGES AND END PLATES.
4. ALL HOT ROLLED STRUCTURAL STEELWORK TO BE MILD STEEL TO GRADE S275JR TO BS EN 10025:1993, AND HOLLOW SECTIONS GRADE S355J2H TO BS EN 10210-1.
5. <u>INTERNAL STEELWORK</u> - TO BE BLAST CLEANED TO SA$2\frac{1}{2}$ TO RECEIVE SHOP APPLIED ZINC PHOSPHATE EPOXY PRIMER (80 MICRONS).

    <u>EXTERNAL STEELWORK</u> - TO BE EITHER BLAST CLEANED TO SA$2\frac{1}{2}$ AND HOT DIPPED GALVANISED (140 MICRONS) OR CHEMICALLY CLEANED AND HOT DIPPED GALVANISED (85 MICRONS).
6. THE STEELWORK FABRICATOR IS TO CONDUCT AN ACCURATE SITE SURVEY TO DETERMINE FINAL DIMENSIONS FOR ALL NEW STEELWORK PRIOR TO FABRICATION. THE SURVEY SHOULD HIGHLIGHT ANY EXISTING OBSTRUCTIONS, SERVICES, ETC. THAT NEED TO BE ALTERED/RELOCATED TO AVOID CLASH.
7. ALL BOLTS TO BE BE GRADE 8.8 IN ACCORDANCE WITH EN ISO 1461 UNLESS NOTED OTHERWISE ON THE DRAWINGS.
8. WELD TESTS TO BE CARRIED OUT TO COMPLY WITH THE LATEST EDITION OF 'THE NATIONAL STRUCTURAL STEELWORK SPECIFICATION FOR BUILDING CONSTRUCTION'. 'FLUID' IS TO BE PROVIDED WITH COPIES OF ALL WELD TEST RESULTS.
9. ALL STEEL BEAMS AND COLUMNS ARE TO BE FIRE PROTECTED IN ACCORDANCE WITH THE SURVEYORS DETAILS AND CURRENT BUILDING REGULATIONS. ALLOW MINIMUM 60 MINUTES, UNLESS NOTED OTHERWISE.

MASONRY NOTES:

1. ALL MASONRY IS DESIGNED, AND SHALL BE CONSTRUCTED IN ACCORDANCE WITH BS 5628-1:2005 PLUS ALL LATEST AMENDMENTS, INCLUDING ADEQUATE RESTRAINT AS DETAILED IN ANNEX 'D'
2. BRICK AND BLOCK STRENGTHS SHOWN ARE MINIMUM REQUIRED AND SHOULD BE INCREASED AS NECESSARY TO SUIT COURSING. CUT BRICKS/BLOCKS BELOW PADSTONE'S WILL NOT BE ACCEPTED.
3. TEST CERTIFICATES CONFIRMING BRICK AND BLOCK CRUSHING STRENGTHS WILL BE REQUIRED FOR ENGINEERS APPROVAL.
4. WALLS BELOW GROUND LEVEL ARE TO BE BUILT UP IN DENSE CONCRETE BLOCKWORK (MINIMUM 7.0KN/MM²) INNER AND OUTER LEAVES IN 1:$\frac{1}{4}$:3 MORTAR, WITH LEAN MIX CONCRETE CAVITY FILL UP TO 150MM BELOW GROUND LEVEL.
5. WALLS ABOVE GROUND LEVEL TO BE BUILT UP IN 1:1:6 MORTAR (CEMENT:LIME:SAND) OR EQUIVALENT UNLESS SHOWN OTHERWISE.
6. WHERE BLOCKWORK WALLING BELOW GROUND IS IN EXCESS OF 900MM HIGH, THEN 215MM THICK BLOCKWORK IS TO BE ADOPTED, USING BLOCK LAID FLAT AND COURSED AT 225MM VERTICAL CENTERS. BACKFILLING IS TO BE CARRIED OUT EQUALLY EITHER SIDE OF THE WALLING AND IS TO BE OF AN APPROVED, WELL GRADED, GRANULAR MATERIAL.
7. MINIMUM BLOCKWORK STRENGTH TO BE 7.0N/MM², UNLESS STATED OTHERWISE.
8. CONTRACTOR TO OBTAIN CONFIRMATION FROM MANUFACTURERS CONCERNING SUITABILITY OF BLOCKS AND BRICKS FOR USE BELOW DPC.
9. WALL TIES TO BE STAINLESS STEEL STAFIX RT2 AT 450CRS VERTICALLY AND 900CRS HORIZONTALLY (STAGGERED). TIES TO BE POSITIONED AT 225CRS VERTICALLY AROUND ALL OPENINGS.

TIMBER NOTES:

1. GALVANISED STEEL STRAPS TO BE PROVIDED AT NOT GREATER THAN 1000MM CENTERS BETWEEN TIMBER AND WALLS AND OR STEELWORK, COMPLETE WITH SOLID BLOCKING.
2. ALL TIMBER TO BE MINIMUM STRENGTH CLASS C16 UNLESS NOTED OTHERWISE.
3. ALL TIMBERS PROJECTING INTO CAVITY OF CAVITY WALL CONSTRUCTION TO HAVE ENDS LIBERALLY COATED IN 'WOLMANOL' OR SIMILAR APPROVED GRAIN PRESERVATIVE.
4. ALL TIMBER TO BE REGULARISED AND SUPPLIED AT A MOISTURE CONTENT AVERAGE NOT EXCEEDING 18%.
5. ALL CONNECTORS, BOLTS AND WASHERS TO BE GALVANISED.
6. ALL DIMENSIONS AND BEARING POINTS OF THE TRUSSES ARE TO BE CHECKED PRIOR TO FABRICATION.
7. ALL ROOF BRACING IS TO THE DESIGN OF THE SPECIALIST SUPPLIER. HOWEVER, ALL ROOF BRACING MEMBERS ARE TO BE AT LEAST 97X22, GRADE C16 AND ARE TO BE NAILED TO EVERY TRUSSED RAFTER THEY CROSS WITH 2NO. 3.35MM Ø X 65MM LONG GALVANISED ROUND WIRE NAILS. WHERE BRACING MEMBERS ARE PROVIDED IN TWO PIECES THE ARE TO BE LAP JOINTED OVER AT LEAST TWO TRUSSED RAFTERS AND NAILED AS ABOVE.

TIMBER LINTEL SCHEDULE:

OPENINGS UP TO 1100mm - 150x100mm C16 LINTEL

OPENINGS UP TO 1900mm - 200x100mm C16 LINTEL

CONSULT WITH STRUCTURAL ENGINEER IF LARGER OPENINGS REQUIRED IN STUD WALL.

CONSTRUCTION (DESIGN AND MANAGEMENT REGULATIONS NOTES

ALL WORKS AND SITE PROCEDURES MUST BE IN KEEPING WITH THE LATEST VERSION OF THE 'CDM REGULATIONS 2015' AND A PRINCIPAL DESIGNER IS TO BE APPOINTED BY THE CLIENT TO MANAGE AND COORDINATE HEALTH AND SAFETY MATTERS ACCORDINGLY

- <u>TEMPORARY WORKS - AS APPLICABLE</u>
  THE CONTRACTOR IS ENTIRELY RESPONSIBLE FOR MAINTAINING THE STABILITY OF ALL EXISTING BUILDINGS AND STRUCTURES WITHIN, AND ADJACENT TO, THE WORKS, FROM THE DATE OF POSSESSION OF THE SITE UNTIL PRACTICAL COMPLETION OF THE WORKS. THE DESIGN, INSTALLATION (INCLUDING SEQUENCE), MAINTENANCE AND REMOVAL (INCLUDING SEQUENCE) OF THE TEMPORARY WORKS IS ENTIRELY THE RESPONSIBILITY OF THE CONTRACTOR. IN ADDITION TO THE SUPPORT OF THE VERTICAL LOADS, DUE REGARD SHALL BE GIVEN TO THE OVERALL LATERAL STABILITY OF THE STRUCTURES, AND THE LATERAL STABILITY OF WALLS COLUMNS AND PIERS, ESPECIALLY WHERE ADJACENT EXISTING FLOORS AND ROOFS ARE TO BE DEMOLISHED. THE TEMPORARY SUPPORT SYSTEMS ARE TO BE ERECTED OFF A FOUNDATION/SPREADER SYSTEM ADEQUATE FOR THE VERTICAL AND HORIZONTAL LOADS REQUIRED TO BE SUPPORTED. THE DESIGN OF THE TEMPORARY WORKS IS TO BE UNDERTAKEN BY A COMPETENT PERSON.
- THE CONTRACTOR IS TO ENSURE THAT ALL EXISTING CONSTRUCTION IS ADEQUATELY SUPPORTED PRIOR TO COMMENCING DEMOLITION WORKS. IF IN DOUBT ENGINEER TO BE CONSULTED PRIOR TO COMMENCING DEMOLITION.
- BEFORE COMMENCING REMOVAL OF TEMPORARY SUPPORTS THE CONTRACTOR IS TO COMPLETE ERECTION AND CONNECTION OF THE NEW PERMANENT SUPPORTING STRUCTURE. THE CONTRACTOR IS TO ENSURE THAT THE TEMPORARILY PROPPED STRUCTURE IS ADEQUATELY SUPPORTED ON THE NEW PERMANENT WORKS PRIOR TO REMOVAL OF ANY TEMPORARY SUPPORTS. THE CONTRACTOR IS TO ENSURE THAT ALL PARTS OF THE NEW SUPPORTING STRUCTURE HAVE GAINED ADEQUATE STRENGTH PRIOR TO REMOVING TEMPORARY SUPPORTS.
- THE LOADINGS FOR THE TEMPORARY WORKS DESIGN ARE GIVEN BELOW, AND ON THE DRAWINGS AS APPROPRIATE. NOTE: ALL LOADS ARE UNFACTORED. THE TEMPORARY WORKS DESIGNER MUST MULTIPLY THESE LOADS BY THE APPROPRIATE ULTIMATE LOAD FACTORS AS GIVEN IN THE RELEVANT DESIGN STANDARD.

STEEL BEAMS DEAD LOAD	= 40kN/m
STEEL BEAMS IMPOSED LOAD	= 15kN/m

<u>DEMOLITION</u>
THE DEMOLITION WORKS ARE TO BE CARRIED OUT WITH GREAT CARE. IF AT ANY TIME THE CONTRACTOR IS UNSURE IF UNPROPPED STRUCTURAL MEMBERS ARE TAKING SUPPORT OFF CONSTRUCTION PROPOSED TO BE DEMOLISHED, DEMOLITION WORK MUST CEASE IMMEDIATELY AND THE CONTRACTOR IS TO OBTAIN FURTHER ADVICE FROM ENGINEER PRIOR TO CONTINUING.

DEMOLITION WORK OTHER THAN THAT INDICATED ON ENGINEERING/THE ARCHITECTS' DRAWINGS IS NOT PERMITTED. IF THE CONTRACTOR BELIEVES THAT ADDITIONAL DEMOLITION TO THAT SHOWN ON ENGINEERING/THE ARCHITECTS' DRAWINGS IS NECESSARY, THEN FURTHER INSTRUCTIONS ARE TO BE SOUGHT FROM ENGINEER AND THE ARCHITECT PRIOR TO COMMENCING THIS ADDITIONAL DEMOLITION WORK.

THE CONTRACTOR IS TO PRODUCE A METHOD STATEMENT FOR DEMOLITION WORKS AND THIS METHOD STATEMENT IS TO BE SUBMITTED TO THE PRINCIPAL DESIGNER FOR COMMENT NOT LESS THAN 2 WEEKS PRIOR TO COMMENCING THE DEMOLITION WORKS.

<u>EXISTING SERVICES</u>
FOR THE INDICATIVE POSITIONS OF EXISTING BURIED SERVICES REFERENCE IS TO BE MADE TO THE ARCHITECTS DRAWINGS. THE ACCURACY OF THIS INFORMATION CANNOT BE GUARANTEED, AND THEREFORE IT IS THE CONTRACTOR'S RESPONSIBILITY TO ESTABLISH THE POSITIONS AND DEPTHS OF ALL UNDERGROUND SERVICES (INCLUDING THOSE NOT SHOWN ON ENGINEERING DRAWINGS) PRIOR TO THE WORKS COMMENCING. THE CONTRACTOR IS TO ALSO NOTE ANY OVERHEAD CABLES ETC. THAT MAY PRESENT A HAZARD DURING THE WORKS.

<u>SOIL CONDITIONS</u>
EXCAVATION IS TO PROCEED WITH GREAT CARE. AS EXCAVATION PROCEEDS THE CONTRACTOR IS TO MAINTAIN VIGILANCE FOR GROUND CONDITIONS WHICH MAY AFFECT THE STABILITY OF THE EXCAVATIONS, AND THE SAFETY OF OPERATIVES. (E.G WEAK/SOFT SOILS, WATER TABLE, TOXIC SUBSTANCES/GASES BURIED SERVICES ETC,) ADEQUATE SUPPORT TO BE PROVIDED TO SIDES OF EXCAVATIONS AS NECESSARY.

Design Team

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SW9 9AJ

tel: 020 7242 5353

CLIENT:

ARCHITECT: Design Team
342 Clapham Road
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SW9 9AJ

SITE:

TITLE:

SCALE AT A3:	DATE:	DRAWN:	CHECKED:
PROJECT NO:	DRAWING NO:		REVISION:

IN DOUBT? ASK!

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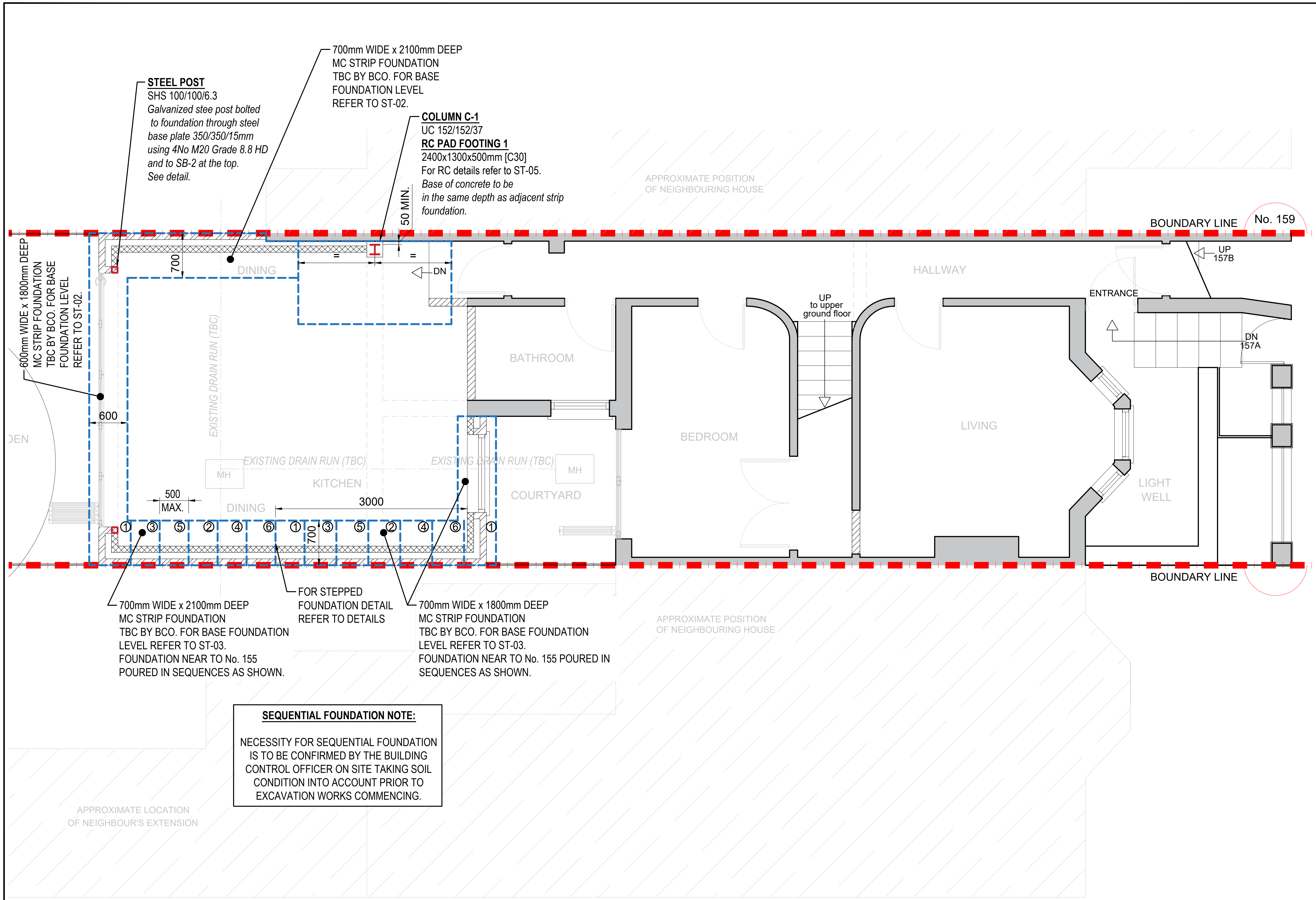

SEQUENTIAL FOUNDATION NOTE:

NECESSITY FOR SEQUENTIAL FOUNDATION IS TO BE CONFIRMED BY THE BUILDING CONTROL OFFICER ON SITE TAKING SOIL CONDITION INTO ACCOUNT PRIOR TO EXCAVATION WORKS COMMENCING.

GROUND FLOOR PLAN

FOUNDATION LAYOUT

(SCALE: NOT TO SCALE)

FOUNDATION NOTE:
BASE OF ALL FOUNDATIONS TO BE 200mm MIN. BELOW BOTTOM OF SEWAGE PIPES WITHIN 600mm FROM FACE OF SEWAGE PIPES.

SSL NOTE:
ALL SSL LEVELS MUST BE REVIEWED AND CONFIRMED BETWEEN THE CLIENT, ARCHITECTS AND CONTRACTOR PRIOR TO WORKS COMMENCING.

SETTING OUT NOTE:
ALL SETTING & LEVELS TO BE CONFIRMED BETWEEN THE CLIENT, ARCHITECTS AND CONTRACTOR PRIOR TO COMMENCING FABRICATION.

NOTE:
REQUIREMENT FOR LOAD BEARING WALLS TO BE CONFIRMED BY CONTRACTOR OR BCO ON SITE

GENERAL NOTES:

1. The structural specification within this document is to be read in conjunction with all relevant architectural drawings, the standard notes that are contained on the drawings and any other relevant project information.
2. The drawings within this document are indicative only, and represent design intent only. No dimensions are to be scaled from drawings contained within this document. Refer to the Architectural drawings for all dimensions. All dimensions should be checked on site by the Contractor before fabrication and ordering of materials.
3. Unless noted otherwise, all dimensions are in millimetres and all levels are in metres from the site datum.
4. The information within this document cannot be guaranteed as dimensionally exact. Figured dimensions must be used for setting out and detailing.
5. The Contractor is responsible for the design of all temporary works, and is also responsible for the safe maintenance and stability of the existing building/s at all times.
6. All party wall awards are entirely the responsibility of the client
7. **The contractor (or Client) must report any differences between the structural drawings and site conditions to the Structural Engineer.**
8. **The Contractor (or Client) must notify the Structural Engineer of any design changes that could affect the structural specification before work commences.**

KEY:

- STANDS EXISTING WALLS TO REMAIN UNALTERED.
- STANDS WALLS UNDER.
- STANDS NEW 20N/mm² BRICKWORK IN 1:1:6 MORTAR (U.N.O.)
- STANDS NEW 7N/mm² BLOCKWORK IN 1:1:6 MORTAR (U.N.O.)
- STANDS NEW TIMBER STUDWORK.
- STANDS DIRECTION OF FLOOR JOISTS SPAN
- STANDS LOCATION OF REVISION.
- XX STANDS REVISION VERSION

REV:	DESCRIPTION:	BY:	DATE:
T1	ISSUED FOR TENDER		

STATUS: ISSUED FOR TENDER

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SITE:

TITLE: GROUND FLOOR PLAN FOUNDATION LAYOUT

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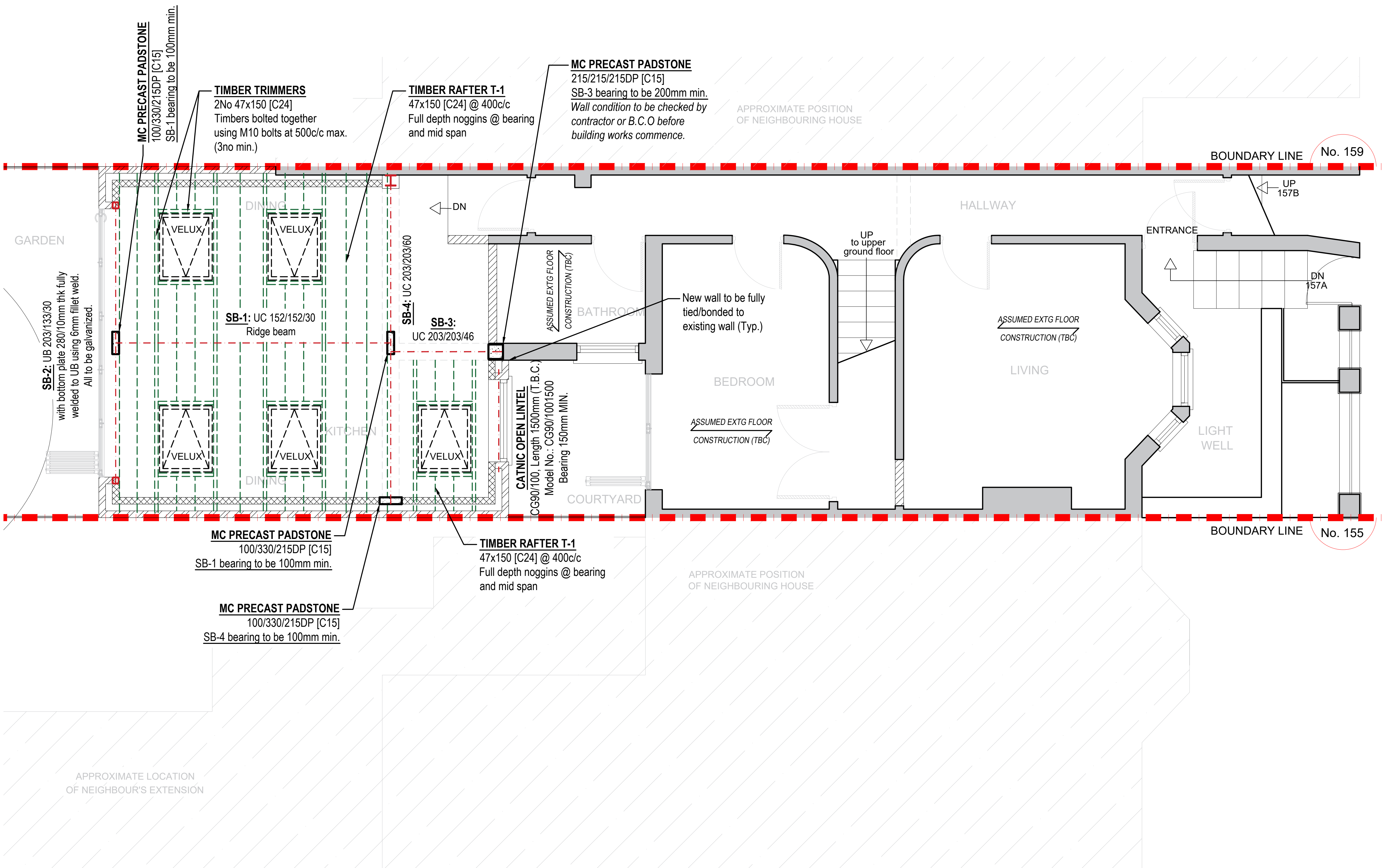

GROUND FLOOR PLAN

STRUCTURE OVER

(SCALE: NOT TO SCALE)

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STRUCTURE OVER

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