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CALCULATIONS

PROJECT: Grays - IOS

REF: 06977E

DATE: March 2025

RECEIVED
By Tom.Anderton at 1:29 pm, Jul 29, 2025

CALCULATIONS	PROJECT: Grays, IOS	PROJECT No:	06977E
		DATE:	March 25
		CALCULATIONS BY:	MDH

STRUCTURAL SUMMARY

We were commissioned to carry out a structural condition survey for this property due to cracking being discovered in areas when a renovation was underway.

The original part of the property in formed in solid stone, with a rear extension carried out in the 1970's. The extension was formed in cavity blockwork and infilled a previous courtyard, as such elements of the original stonework wall have been used.

The stonework throughout was found to be of a reasonable condition, however the workmanship of the cavity blockwork construction and the supporting systems was not of a reasonable standard. Including a proprietary lintel being installed upside down, and an acrow prop being used as permanent works. As well as the external proprietary lintels rusting and delaminating.

As such the following calculations are for replacement steelwork to remove the above elements. The steelwork will be galvanised and much more robust than the proprietary lintels, which suits the environment and proximity to the sea.

These works do not bring the building completely in line with today's standards but do bring the building to a quality that is structurally sound. We are satisfied that the building will be adequately stable once the works are carried out.

All steelwork is to be measured on site, the measurements in the calculations are just for analysis purposes only.

STRUCTURAL STABILITY

The masonry returns and buttresses are adequate in providing the overall stability.

ASSUMPTIONS

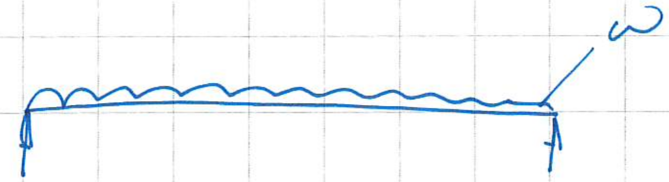
None.

The works are to be carried out by a competent contractor. Who will have responsibility of the temporary works and stability during construction.

DESIGN REFERENCES

Eurocode 1: Actions on structures (EN 1991)
Eurocode 2: Design of concrete structures (EN 1992)
Eurocode 3: Design of steel structures (EN 1993)
Eurocode 5: Design of timber structures (EN 1995)
Eurocode 6: Design of masonry structures (EN 1996)

Designing B1



3500

w:

Roof:	$\frac{1.0}{0.6} \times \frac{3.6}{2}$	1.8	1.1
Wall:	$18 \times 0.2 \times 6.0$	21.6	
Floor:	$\frac{0.5}{1.5} \times \frac{7.0}{2}$	1.8	5.3
		25.2	6.4

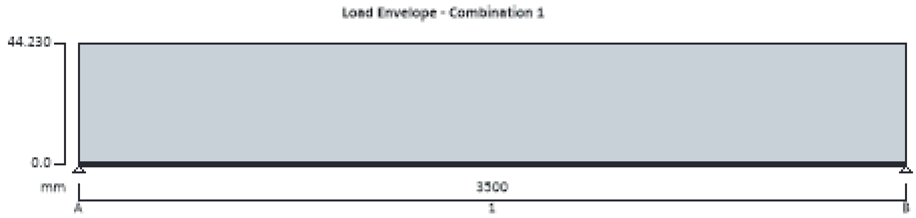
From TEDDS, use 203UC46

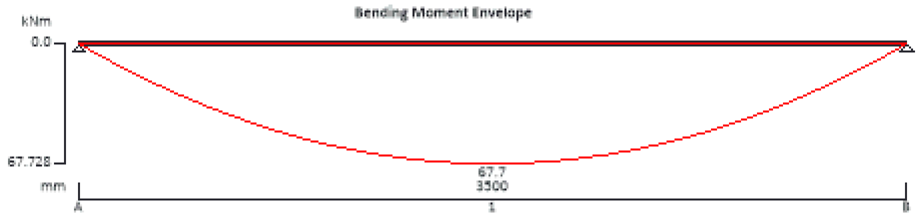
Project				Job no.	
Grays, IOS				06977E	
Calcs for				Start page no./Revision	
B1				2	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
MDH	28/03/2025				

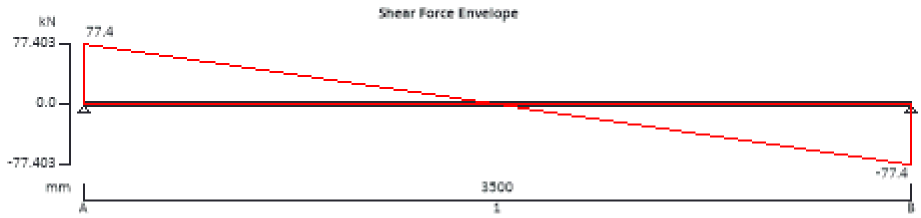
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14







Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Permanent self weight of beam × 1
	Permanent full UDL 25.2 kN/m
	Variable full UDL 6.4 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
		Permanent × 1.35
		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50

Analysis results

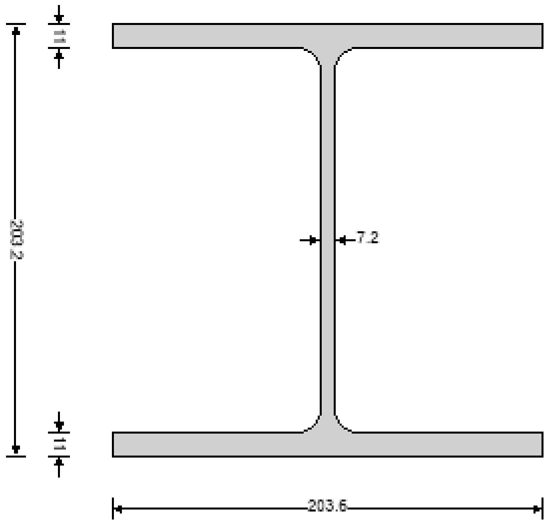
Maximum moment	M_{max} = **67.7** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **77.4** kN	V_{min} = **-77.4** kN
Deflection	δ_{max} = **6.5** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **77.4** kN	R_{A_min} = **77.4** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **44.9** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **11.2** kN	
Maximum reaction at support B	R_{B_max} = **77.4** kN	R_{B_min} = **77.4** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **44.9** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **11.2** kN	

Section details

Section type	**UKC 203x203x46 (Tata Steel Advance)**
Steel grade	**S275**

EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element	$t = \max(t_f, t_w)$ = **11.0** mm
Nominal yield strength	f_y = **275** N/mm^2
Nominal ultimate tensile strength	f_u = **410** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1.00**
Resistance of members to instability	γ_{M1} = **1.00**
Resistance of tensile members to fracture	γ_{M2} = **1.10**

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	K_y = **1.000**
Effective length factor in minor axis	K_z = **1.000**
Effective length factor for torsion	$K_{LT.A}$ = **1.000**
	$K_{LT.B}$ = **1.000**

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]}$ = **0.92**

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section — $c = d$ = **160.8** mm

$c / t_w = 24.2 \times \varepsilon <= 72 \times \varepsilon$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2$ = **88** mm

$c / t_f = 8.7 \times \varepsilon <= 9 \times \varepsilon$ — Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f$ = **181.2** mm

Shear area factor — η = **1.000**

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force — $V_{Ed} = max(abs(V_{max}), abs(V_{min}))$ = **77.4** kN

Shear area - cl 6.2.6(3) — $A_v = max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1698** mm^2

Design shear resistance - cl 6.2.6(2) — $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **269.5** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment — $M_{Ed} = max(abs(M_{s1\_max}), abs(M_{s1\_min}))$ = **67.7** kNm

Design bending resistance moment - eq 6.13 — $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **136.8** kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — k_c = **0.94**

$C_1 = 1 / k_c^2$ = **1.132**

Curvature factor — $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.813**

Poissons ratio — ν = **0.3**

Shear modulus — $G = E / [2 \times (1 + \nu)]$ = **80769** N/mm^2

Unrestrained length — $L = 1.0 \times L_{s1}$ = **3500** mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **462.2** kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.544**

Limiting slenderness ratio — $\bar{\lambda}_{LT,0}$ = **0.4**

$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5 — b

Imperfection factor - Table 6.3 — α_{LT} = **0.34**

Correction factor for rolled sections — β = **0.75**

LTB reduction determination factor — $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.635**

LTB reduction factor - eq 6.57 — $\chi_{LT} = min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.942**

Modification factor — $f = min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.974**

Modified LTB reduction factor - eq 6.58 — $\chi_{LT,mod} = min(\chi_{LT} / f, 1)$ = **0.967**

Design buckling resistance moment - eq 6.55 — $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **132.3** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

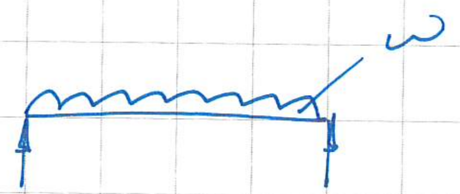
Consider deflection due to permanent and variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 360 =$ **9.7** mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **6.529** mm

PASS - Maximum deflection does not exceed deflection limit

Designing B2



W: Wall: 18 × 0.1 × 3.0 — 5.4

Floors: 0.5 × 6.0/2 × 2 — 3.0

1.5 × 6.0/2 — 4.5

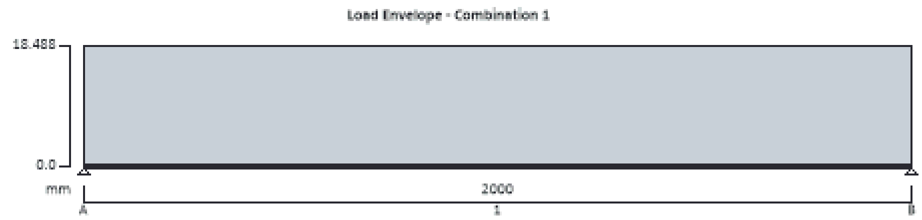
5.4	
3.0	
	4.5
8.4	4.5

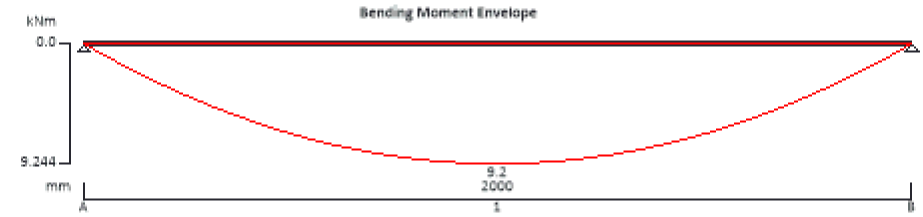
From TEDDS, use 152 UC 30

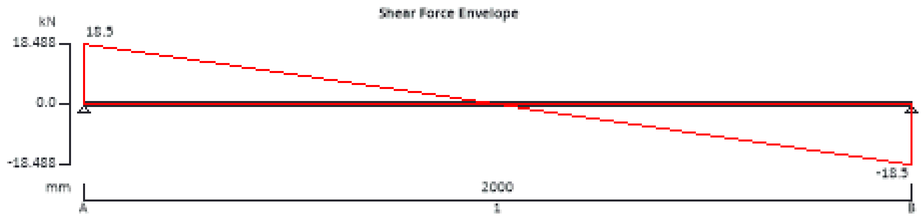
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14







Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Permanent self weight of beam × 1
	Permanent full UDL 8.4 kN/m
	Variable full UDL 4.5 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
		Permanent × 1.35
		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50

Analysis results

Maximum moment	M_{max} = **9.2** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **18.5** kN	V_{min} = **-18.5** kN
Deflection	δ_{max} = **0.7** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **18.5** kN	R_{A_min} = **18.5** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **8.7** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **4.5** kN	
Maximum reaction at support B	R_{B_max} = **18.5** kN	R_{B_min} = **18.5** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **8.7** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **4.5** kN	

Section details

Section type **UKC 152x152x30 (Tata Steel Advance)**

Steel grade **S355**

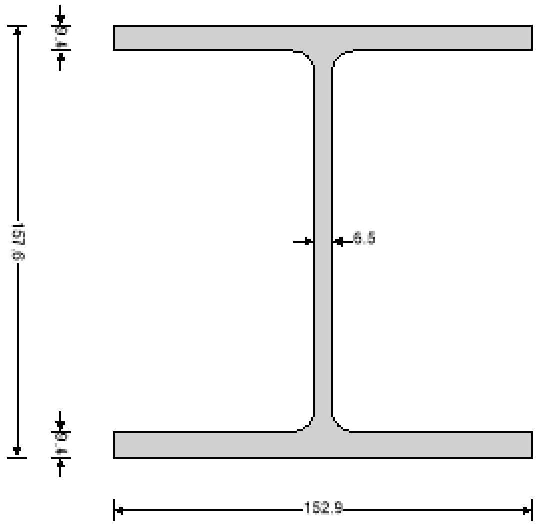
EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element $t = max(t_f, t_w)$ = **9.4** mm

Nominal yield strength f_y = **355** N/mm^2

Nominal ultimate tensile strength f_u = **470** N/mm^2

Modulus of elasticity E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections γ_{M0} = **1.00**

Resistance of members to instability γ_{M1} = **1.00**

Resistance of tensile members to fracture γ_{M2} = **1.10**

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis K_y = **1.000**

Effective length factor in minor axis K_z = **1.000**

Effective length factor for torsion $K_{LT.A}$ = **1.000**

$K_{LT.B}$ = **1.000**

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.81**

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section — $c = d$ = **123.6** mm

$c / t_w = 23.4 \times \varepsilon <= 72 \times \varepsilon$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2$ = **65.6** mm

$c / t_f = 8.6 \times \varepsilon <= 9 \times \varepsilon$ — Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f$ = **138.8** mm

Shear area factor — η = **1.000**

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force — $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **18.5** kN

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1156** mm^2

Design shear resistance - cl 6.2.6(2) — $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **236.9** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment — $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **9.2** kNm

Design bending resistance moment - eq 6.13 — $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **87.9** kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — k_c = **0.94**

$C_1 = 1 / k_c^2$ = **1.132**

Curvature factor — $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.824**

Poissons ratio — ν = **0.3**

Shear modulus — $G = E / [2 \times (1 + \nu)]$ = **80769** N/mm^2

Unrestrained length — $L = 1.0 \times L_{s1}$ = **2000** mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **365.8** kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.49**

Limiting slenderness ratio — $\bar{\lambda}_{LT,0}$ = **0.4**

$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5 — b

Imperfection factor - Table 6.3 — α_{LT} = **0.34**

Correction factor for rolled sections — β = **0.75**

LTB reduction determination factor — $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.605**

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.964**

Modification factor — $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.976**

Modified LTB reduction factor - eq 6.58 — $\chi_{LT,mod} = \min(\chi_{LT} / f, 1)$ = **0.988**

Design buckling resistance moment - eq 6.55 — $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **86.9** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

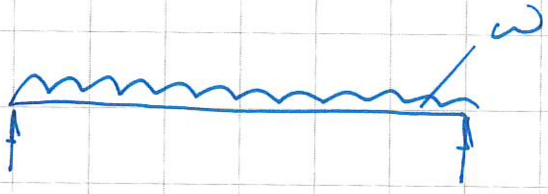
Consider deflection due to permanent and variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 360 =$ **5.6** mm

Maximum deflection span 1 $\delta = max(abs(\delta_{max}), abs(\delta_{min})) =$ **0.749** mm

PASS - Maximum deflection does not exceed deflection limit

Designing B3



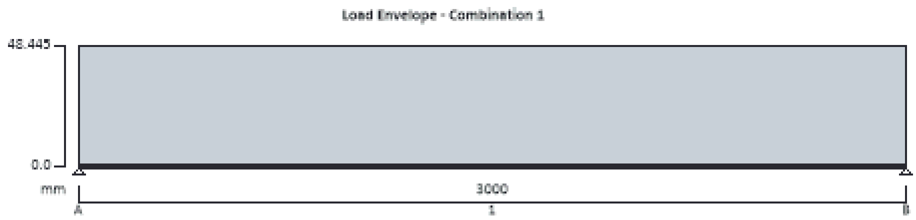
w:	Roof	$\frac{1.0}{0.6} \times \frac{7.0}{2}$	3.50	2.10
	Wall	18×0.2×6.0	21.60	
	Floors	$\frac{0.5}{1.5} \times \frac{6.0}{2} \times 2$	3.00	4.50
			28.1	6.60

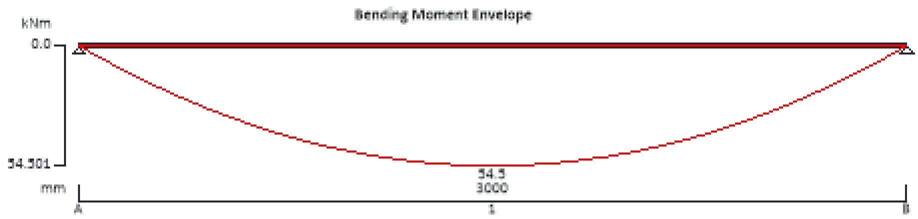
From TEDDS, use 203UC46

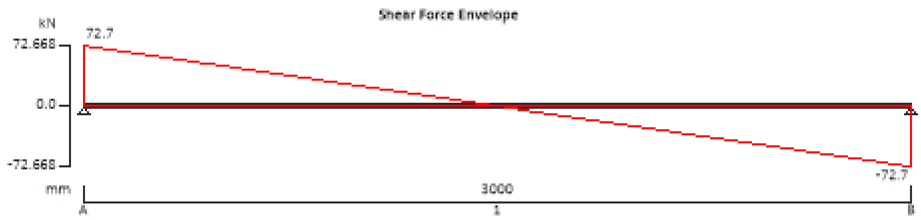
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14







Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Permanent self weight of beam × 1
	Permanent full UDL 28.1 kN/m
	Variable full UDL 6.6 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
		Permanent × 1.35
		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50

Analysis results

Maximum moment	M_{max} = **54.5** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **72.7** kN	V_{min} = **-72.7** kN
Deflection	δ_{max} = **3.9** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **72.7** kN	R_{A_min} = **72.7** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **42.8** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **9.9** kN	
Maximum reaction at support B	R_{B_max} = **72.7** kN	R_{B_min} = **72.7** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **42.8** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **9.9** kN	

Section details

Section type — **UKC 203x203x46 (Tata Steel Advance)**

Steel grade — **S275**

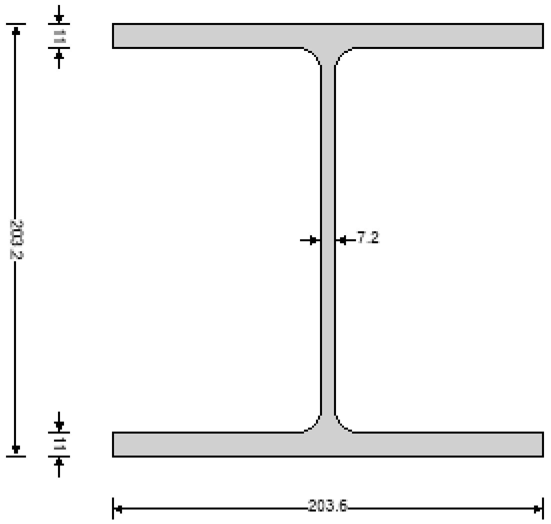
EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element — $t = \max(t_f, t_w)$ = **11.0** mm

Nominal yield strength — f_y = **275** N/mm^2

Nominal ultimate tensile strength — f_u = **410** N/mm^2

Modulus of elasticity — E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections — γ_{M0} = **1.00**

Resistance of members to instability — γ_{M1} = **1.00**

Resistance of tensile members to fracture — γ_{M2} = **1.10**

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis — K_y = **1.000**

Effective length factor in minor axis — K_z = **1.000**

Effective length factor for torsion — $K_{LT.A}$ = **1.000**

$K_{LT.B}$ = **1.000**

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = \mathbf{0.92}$

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section — $c = d = \mathbf{160.8}$ mm

$c / t_w = 24.2 \times \varepsilon <= 72 \times \varepsilon$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2 = \mathbf{88}$ mm

$c / t_f = 8.7 \times \varepsilon <= 9 \times \varepsilon$ — Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f = \mathbf{181.2}$ mm

Shear area factor — $\eta = \mathbf{1.000}$

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force — $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{72.7}$ kN

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{1698}$ mm^2

Design shear resistance - cl 6.2.6(2) — $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = \mathbf{269.5}$ kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment — $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = \mathbf{54.5}$ kNm

Design bending resistance moment - eq 6.13 — $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = \mathbf{136.8}$ kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — $k_c = \mathbf{0.94}$

$C_1 = 1 / k_c^2 = \mathbf{1.132}$

Curvature factor — $g = \sqrt{[1 - (I_z / I_y)]} = \mathbf{0.813}$

Poissons ratio — $\nu = \mathbf{0.3}$

Shear modulus — $G = E / [2 \times (1 + \nu)] = \mathbf{80769}$ N/mm^2

Unrestrained length — $L = 1.0 \times L_{s1} = \mathbf{3000}$ mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = \mathbf{592.5}$ kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})} = \mathbf{0.48}$

Limiting slenderness ratio — $\bar{\lambda}_{LT,0} = \mathbf{0.4}$

$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5 — b

Imperfection factor - Table 6.3 — $\alpha_{LT} = \mathbf{0.34}$

Correction factor for rolled sections — $\beta = \mathbf{0.75}$

LTB reduction determination factor — $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = \mathbf{0.600}$

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = \mathbf{0.968}$

Modification factor — $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \mathbf{0.976}$

Modified LTB reduction factor - eq 6.58 — $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.992}$

Design buckling resistance moment - eq 6.55 — $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = \mathbf{135.7}$ kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

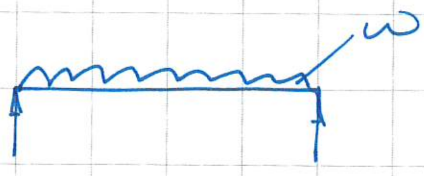
Consider deflection due to permanent and variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 360 =$ **8.3** mm

Maximum deflection span 1 $\delta = max(abs(\delta_{max}), abs(\delta_{min})) =$ **3.865** mm

PASS - Maximum deflection does not exceed deflection limit

Designing B5



W:	Roof:	1.0 × 9.0/2	4.5	
		0.6		2.7
	Wall:	18×0.2×1.0	3.6	
			8.1	2.7

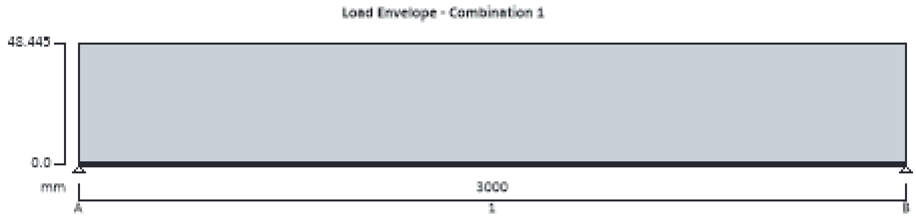
From TEDDS, use 152 UC 23

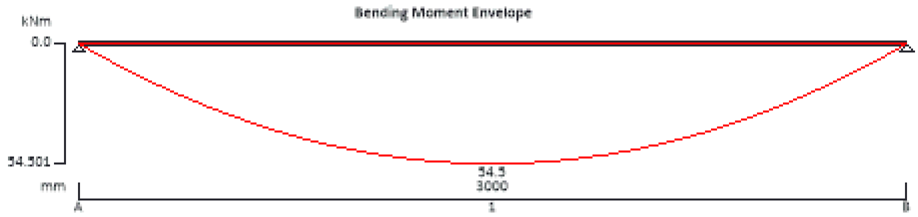
Project				Job no.	
Grays, IOS				06977E	
Calcs for				Start page no./Revision	
B5				17	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
MDH	28/03/2025				

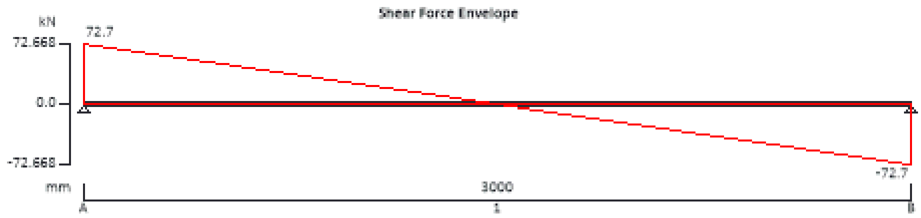
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14







Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Permanent self weight of beam × 1
	Permanent full UDL 28.1 kN/m
	Variable full UDL 6.6 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
		Permanent × 1.35
		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50

Analysis results

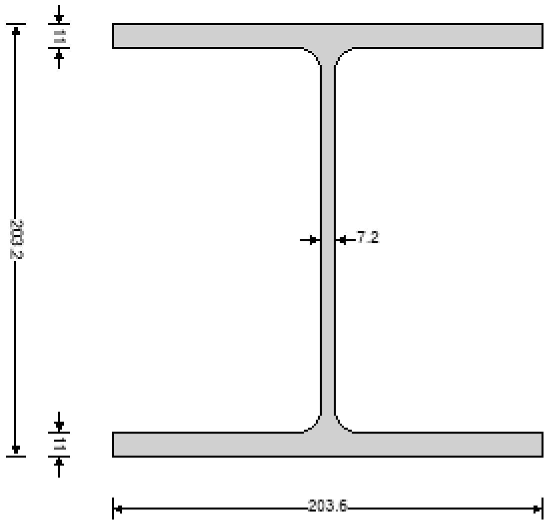
Maximum moment	M_{max} = **54.5** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **72.7** kN	V_{min} = **-72.7** kN
Deflection	δ_{max} = **3.9** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **72.7** kN	R_{A_min} = **72.7** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **42.8** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **9.9** kN	
Maximum reaction at support B	R_{B_max} = **72.7** kN	R_{B_min} = **72.7** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **42.8** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **9.9** kN	

Section details

Section type	**UKC 203x203x46 (Tata Steel Advance)**
Steel grade	**S275**

EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element	$t = \max(t_f, t_w)$ = **11.0** mm
Nominal yield strength	f_y = **275** N/mm^2
Nominal ultimate tensile strength	f_u = **410** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1.00**
Resistance of members to instability	γ_{M1} = **1.00**
Resistance of tensile members to fracture	γ_{M2} = **1.10**

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	K_y = **1.000**
Effective length factor in minor axis	K_z = **1.000**
Effective length factor for torsion	$K_{LT.A}$ = **1.000**
	$K_{LT.B}$ = **1.000**

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.92**

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section — $c = d$ = **160.8** mm

$c / t_w = 24.2 \times \varepsilon <= 72 \times \varepsilon$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2$ = **88** mm

$c / t_f = 8.7 \times \varepsilon <= 9 \times \varepsilon$ — Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f$ = **181.2** mm

Shear area factor — η = **1.000**

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force — $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **72.7** kN

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1698** mm²

Design shear resistance - cl 6.2.6(2) — $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **269.5** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment — $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **54.5** kNm

Design bending resistance moment - eq 6.13 — $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **136.8** kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — k_c = **0.94**

$C_1 = 1 / k_c^2$ = **1.132**

Curvature factor — $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.813**

Poissons ratio — ν = **0.3**

Shear modulus — $G = E / [2 \times (1 + \nu)]$ = **80769** N/mm²

Unrestrained length — $L = 1.0 \times L_{s1}$ = **3000** mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **592.5** kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.48**

Limiting slenderness ratio — $\bar{\lambda}_{LT,0}$ = **0.4**

$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5 — b

Imperfection factor - Table 6.3 — α_{LT} = **0.34**

Correction factor for rolled sections — β = **0.75**

LTB reduction determination factor — $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.600**

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.968**

Modification factor — $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.976**

Modified LTB reduction factor - eq 6.58 — $\chi_{LT,mod} = \min(\chi_{LT} / f, 1)$ = **0.992**

Design buckling resistance moment - eq 6.55 — $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **135.7** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 360 =$ **8.3** mm

Maximum deflection span 1 $\delta = max(abs(\delta_{max}), abs(\delta_{min})) =$ **3.865** mm

PASS - Maximum deflection does not exceed deflection limit

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General Notes.

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This drawing is not to be scaled. Work to figured dimensions only.

CDM NOTES

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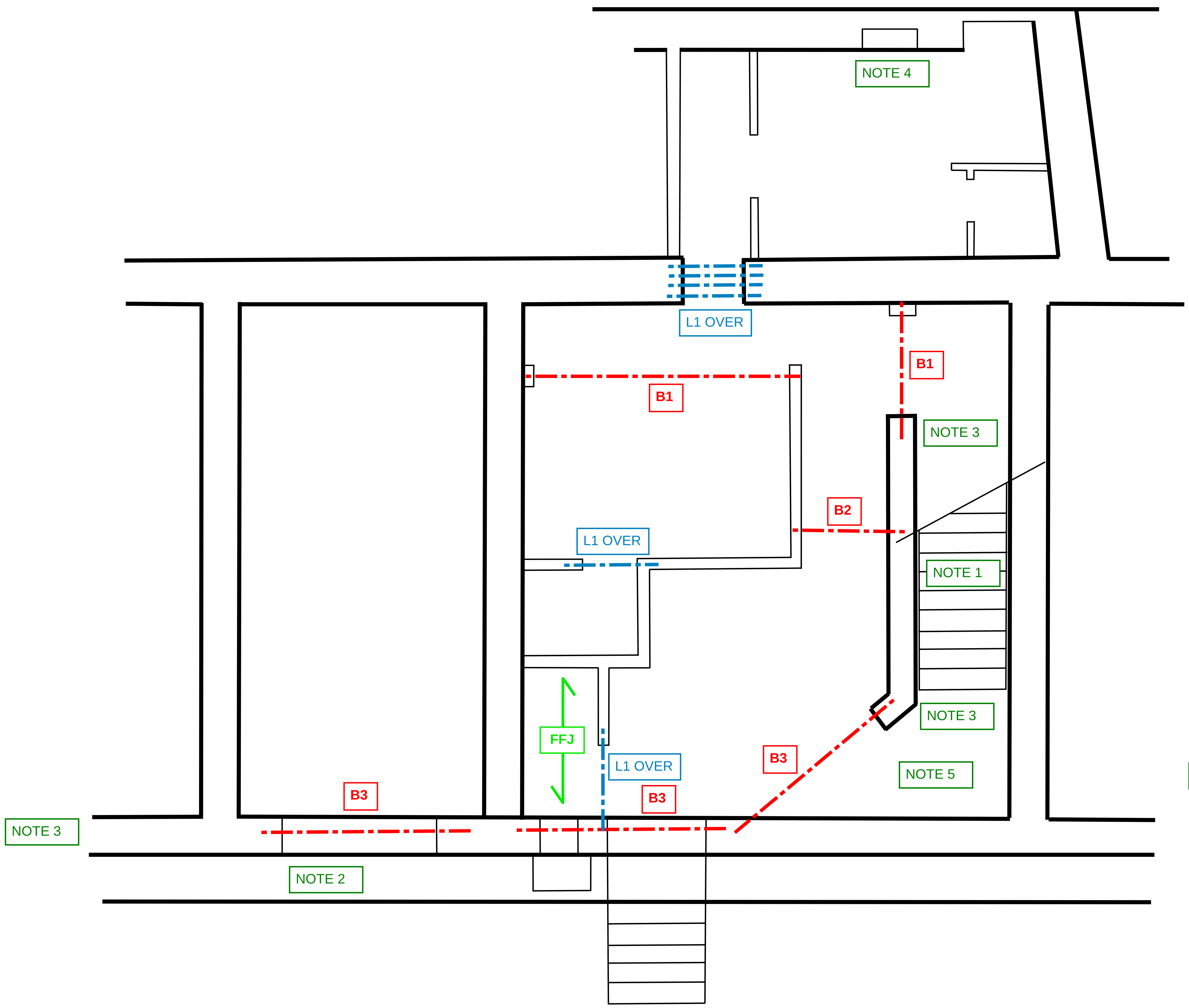


Structural Specification

B1 203 UC 46 + 10mm Top Plate (All Galvanised)

B2 152 UC 30

B3 203 UC 46 + 10mm Bottom Plate (All Galvanised)

L1 100 x 140mm deep RC lintels, where wall is thicker than 100mm then use several lintels (i.e. 4No. 100mm for 400mm wide wall)

FFJ Replace concrete floor with 50 x 200mm C24 joists at 400mm c/c's with 18mm OSB glued and screwed

NOTE - Install concrete RC20 padstones under all steels and lintel bearings and check existing masonry brick / block / stone under and repair as necessary

NOTE 1 STAIRS - Consider replacing the external stair case with a steel or new hard wood timber stair

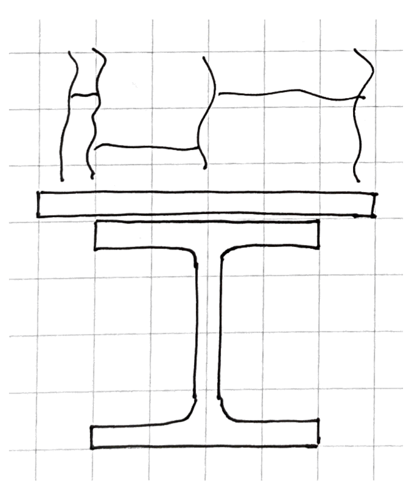
NOTE 2 SEWER - Consider repairing cracks or replacing concrete plinth over walkway/sewer but consult authority prior to works

NOTE 3 EXTERNAL FACADE - Where the external leaf has cracking then install crack stitching in accordance with manufacturer details

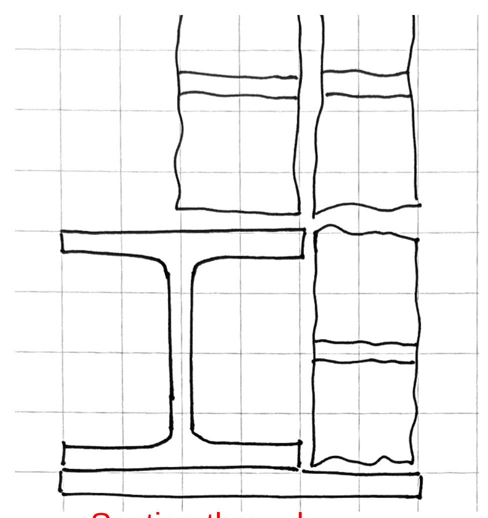
EXTERNAL FACADE - Consider removing cement render to and replace with lime render

NOTE 4 FLUE - Consider opening up for ventilation

NOTE 5 RWP - Consider replacement of rainwater pipes throughout



Section through...
B1



Section through...
B3

REV.	DATE	DETAILS	DRAWN	CHECKED
A	12.03.25	Construction Issue	MDH	PRS
-	29.01.25	Tender Issue	MDH	PRS

CONSTRUCTION

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Grays, Isles of Scilly

Lower Ground Floor - SE Requirements

Drawing Number 06977E_SK_001 — Revision A

Drawn by MDH, Date 24.01.25 — Checked by PRS, Date 29.01.25

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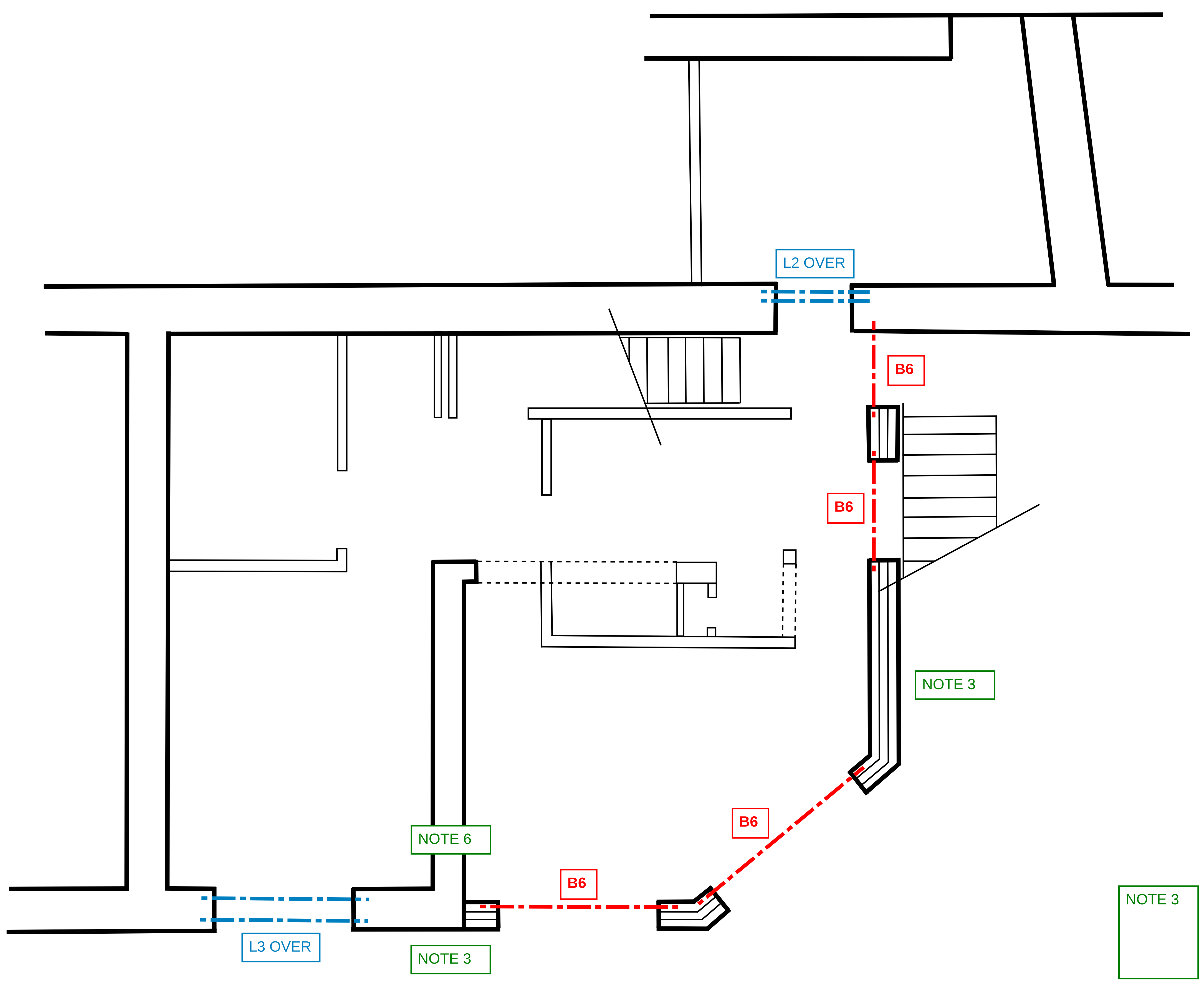


Structural Specification

B5 152 UC 23 + 10mm Top Plate

B6 203UC 46 + Bottom Plate (All Galvanised)

L1 100 x 140mm deep RC lintels, where wall is thicker than 100mm then use several lintels (i.e. 4No. 100mm for 400mm wide wall)

L2 Replace timber beam with 2No. 100 x 140mm deep RC lintels

L3 Replace existing lintels with 100 x 220mm deep RC lintels or steel B6

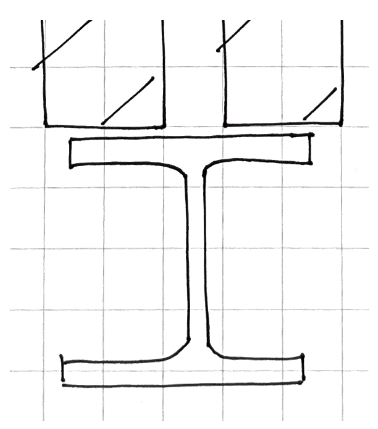
L4 Investigate and replace lintel possibly

NOTE - Install concrete RC20 padstones under all steels and lintel bearings and check existing masonry brick / block / stone under and repair as necessary

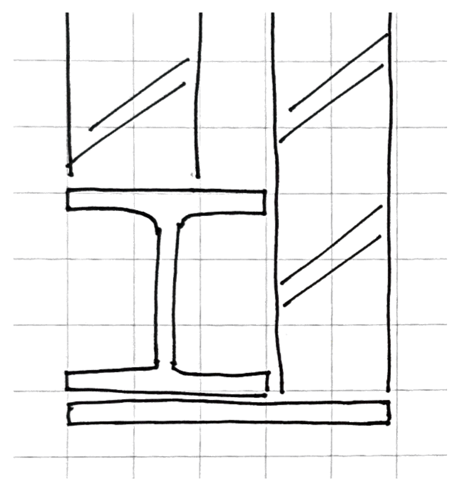
NOTE 3 EXTERNAL FACADE - Where the external leaf has cracking then install crack stitching in accordance with manufacturer details

EXTERNAL FACADE - Consider removing cement render to and replace with lime render

NOTE 6 Internal crack stitching to manufacturers details



Section through...
B5



Section through...
B6

REV.	DATE	DETAILS	DRAWN	CHECKED
A	12.03.25	Construction Issue	MDH	PRS
-	29.01.25	Tender Issue	MDH	PRS

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Ground Floor - SE Requirements

Drawing Number 06977E_SK_002

Revision A

Drawn by MDH
Date 24.01.25

Checked by PRS
Date 29.01.25

CONSTRUCTION

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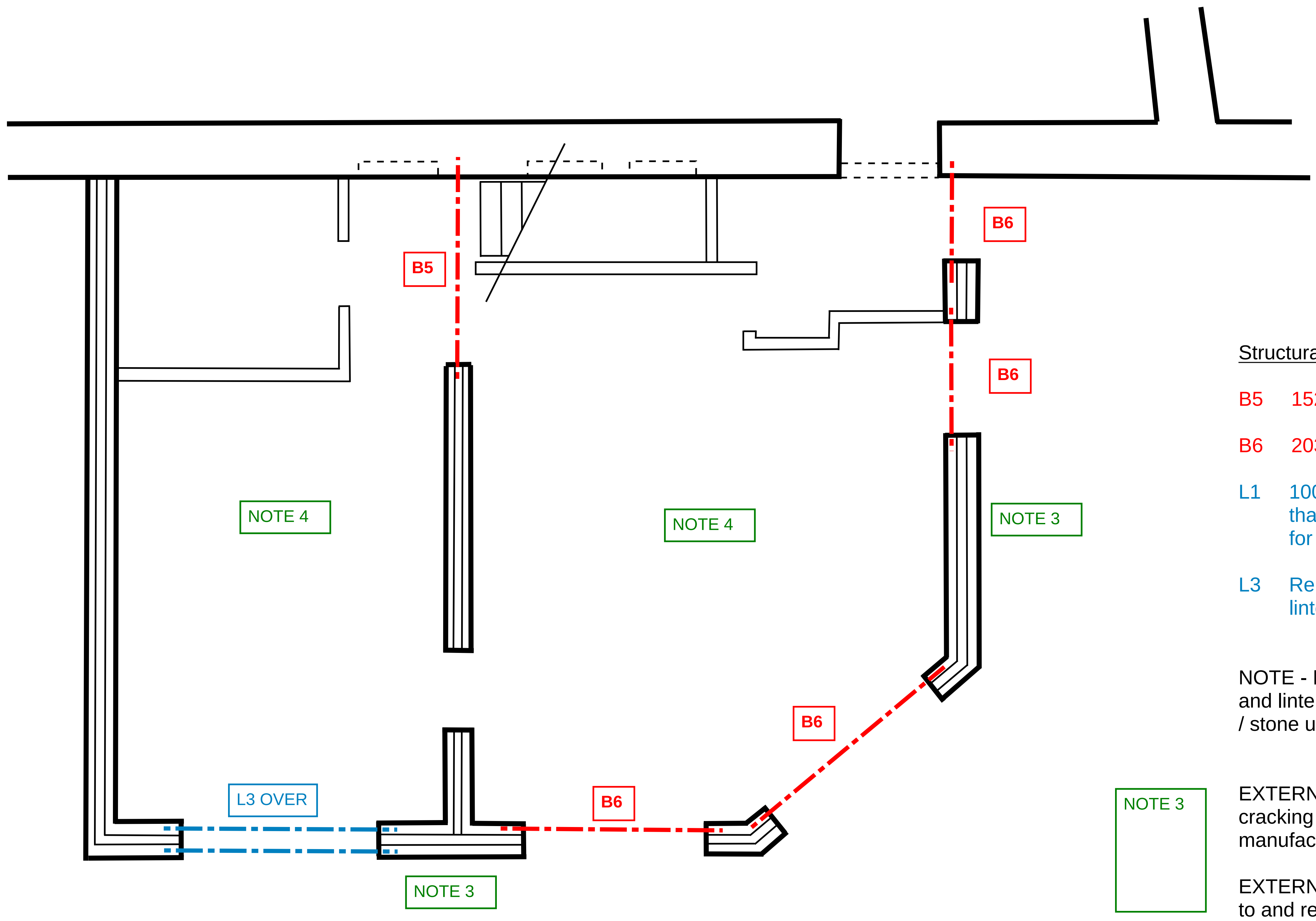


<u>Structural Specification</u>

B5 152 UC 23 + 10mm Top Plate

B6 203UC 46 + Bottom Plate (All Galvanised)

L1 100 x 140mm deep RC lintels, where wall is thicker than 100mm then use several lintels (i.e. 4No. 100mm for 400mm wide wall)

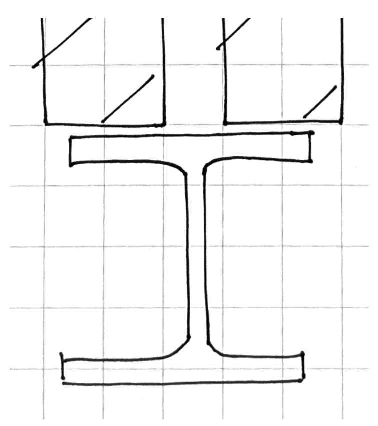
L3 Replace existing lintels with 100 x 220mm deep RC lintels or steel B6

NOTE - Install concrete RC20 padstones under all steels and lintel bearings and check existing masonry brick / block / stone under and repair as necessary

NOTE 3 EXTERNAL FACADE - Where the external leaf has cracking then install crack stitching in accordance with manufacturer details

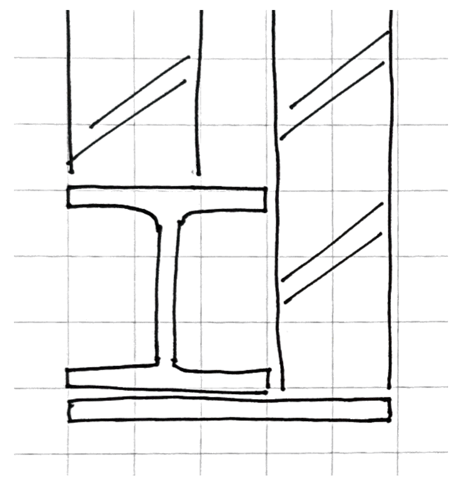
EXTERNAL FACADE - Consider removing cement render to and replace with lime render

NOTE 4 Expose and check roof structure



Section through...

B5



Section through...

B6

REV.	DATE	DETAILS	DRAWN	CHECKED
A	12.03.25	Construction Issue	MDH	PRS
-	29.01.25	Tender Issue	MDH	PRS

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First Floor - SE Requirements

Drawing Number 06977E_SK_003 — Revision A

Drawn by MDH Date 24.01.25 — Checked by PRS Date 29.01.25

CONSTRUCTION