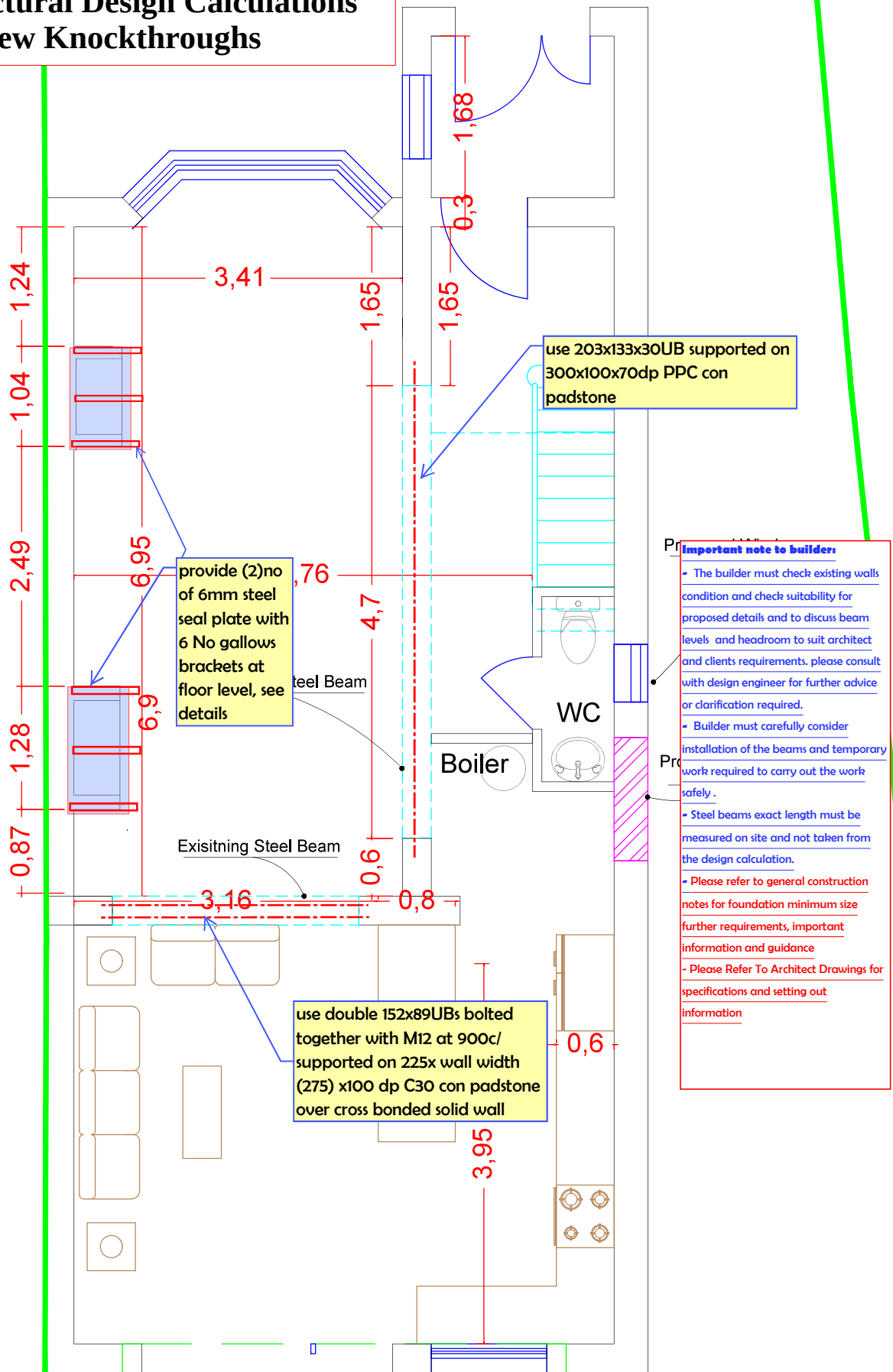


29 Manor Dr, Manchester M21 7GQ

Beams Structural Design Calculations

for New Knockthroughs



Proposal ground floor

Structural Design Calculations

Site address: 29 Manor Dr, Manchester M21 7GQ
 Job: Beams structural design
 Client:

Job number:
 Made by: NK
 Page: 1

internal .ps7

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ProSteel 7 Project Summary

Site address: 2 Manor Dr, Manchester M21 7GQ
 Job: Beams structural design
 Client:
 Job number:

ITEMS:

- 1: Beam: Internal Beam B1
 Span: 4.8 m.
 Reactions (unfactored/factored): R1: 28.20/44.40 kN; R2: 28.20/44.40 kN
 Use 203 x 133 x 30 UB S355
 Bearing R1: 250 x 200 mm padstone
 Bearing R2: As R1

- 2: Beam: Beam B2 - Kitchen Knockthrough -inner
 Span: 3.2 m.
 Reactions (unfactored/factored): R1: 27.04/41.98 kN; R2: 27.04/41.98 kN
 Use 2No 152 x 89 x 16 UB S355
 Bearing R1: 450 x 100 mm padstone
 Bearing R2: As R1

Sections to be bolted together with tube spacers or suitable alternative connection at max 1.5m c/s

Structural Design Calculations

Site address: 29 Manor Dr, Manchester M21 7GQ
Job: Beams structural design
Client:

Job number:
Made by: NK
Page: 2

internal .ps7

Printed 18 Aug 2024 14:04

Beam: Internal Beam B1

Span: 4.8 m.

	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
U L	wall	2.3x2.5	0		L	13.80	13.80	39.7
U D	Floor dead	0.5x3	0		L	3.60	3.60	10.4
U L	Floor live	1.5x3	0		L	10.80	10.80	31.1
Total load (unfactored):						56.4 kN	28.20	28.20
Dead/Permanent (unfactored):						7.2 kN	3.60	3.60
Live/Variable (unfactored):						49.2 kN	24.60	24.60
Total load (factored):						88.8 kN	44.40	44.40

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

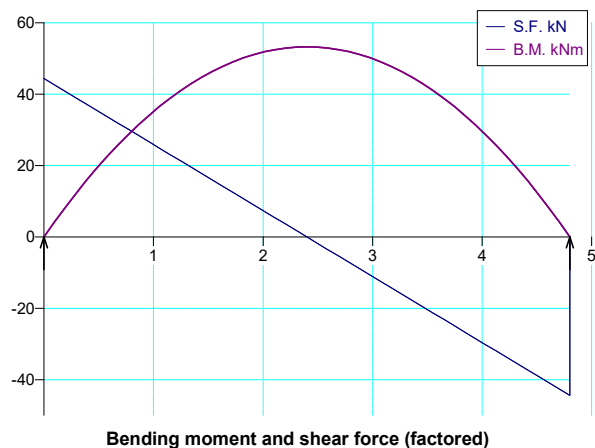
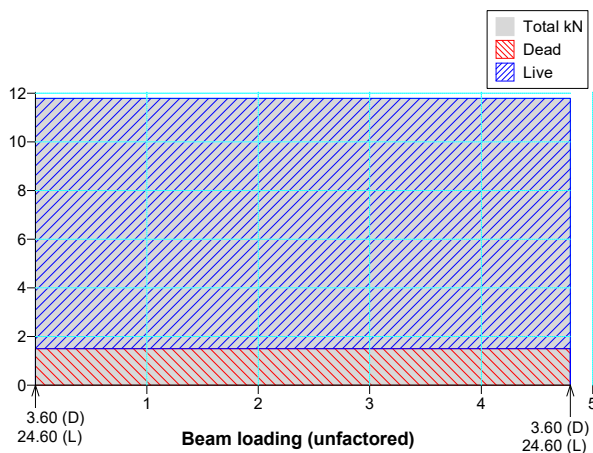
Maximum B.M. = 53.3 kNm (factored) at 2.40 m. from R1

Maximum S.F. = 44.4 kN (factored) at R1

Mid-span deflections: Dead: $10.4 \times 10^8/EI$ (E in N/mm^2 , I in cm^4)

Live: $70.8 \times 10^8/EI$

Total: $81.2 \times 10^8/EI$



Beam calculation to BS5950-1:2000 using S355 steel

SECTION SIZE : 203 x 133 x 30 UB S355 (compact)

D=206.8 mm B=133.9 mm t=6.4 mm T=9.6 mm $I_x=2,900 \text{ cm}^4$ $r_y=3.17 \text{ cm}$ $S_x=314 \text{ cm}^3$ $x=21.5$

Section classification: T = 9.6mm $p_y = 355 \text{ N/mm}^2$ $\varepsilon = \sqrt{(275/355)} = 0.88$

(Table 11)

Flange: $b/T = 66.9/9.6 = 6.97 (<=9\varepsilon)$: Class 1 plastic

Web: $d/t = 172.4/6.4 = 26.9 (<=80\varepsilon)$: Class 1, plastic

For design purposes section classification is Class 2, compact

Shear

Maximum S.F. = 44 kN

Shear capacity = $0.6 p_y \cdot t \cdot D = 0.6 \times 355 \times 6.4 \times 206.8/1000 = 282 \text{ kN OK}$

Bending

Maximum B.M. = 53.3 kNm

Moment capacity, $M_c = p_y \cdot S_x = 355 \times 314/1000 = 111 \text{ kNm OK}$

Lateral-torsional buckling

Beam is laterally restrained at supports only

Restraint condition at R1 and R2: Compression flange laterally restrained. Nominal torsional restraint.

Compression flange partially restrained against rotation on plan (0.85L)

Effective length = $0.85L = 4.08 \text{ m}$. [Table 13]

Bending strength, $p_b = 168 \text{ N/mm}^2$

Structural Design Calculations

Site address: 29 Manor Dr, Manchester M21 7GQ
Job: Beams structural design
Client:

Job number:
Made by: NK
Page: 3

internal .ps7

Printed 18 Aug 2024 14:04

Maximum moment within segment, $M_x = 53.3$ kNm

Equivalent uniform moment factor, $m_{LT} = 0.925$ ($M_2 = 40.0$, $M_3 = 53.3$, $M_4 = 40.0$)

Equivalent uniform moment = $0.925 \times 53.3 = 49.3$ kNm

Buckling resistance moment, $M_b = p_b \cdot S_x = 168 \times 314/1000 = 52.7$ kNm OK

Web capacity

Check unstiffened web capacity with load of 44.4 kN

$C1 = 78.2/0.00$ kN; $C2 = 2.27$ kN/mm; $C4 = 261$; $K = \min\{0.5 + (a_e/1.4d), 1.0\}$; $p_{yw} = 355$ N/mm²

For derivation of C factors see SCI Steelwork Design Guide to BS5950-1:2000 6th ed. (P325)

Minimum required stiff bearing length, $b_1 = 0$ mm; $a_e = 0$ mm; $d = 172.4$ mm ; $K = 0.500$

Bearing capacity, $P_w = C1 + b_1 \cdot C2 = 78.2$ kN

Buckling capacity, $P_x = K \sqrt{C4 \cdot P_w} = 0.500 \sqrt{(261 \times 78.2)} = 71.5$ kN ←

Deflection

LL deflection = $70.8 \times 1e8 / (205,000 \times 2,900) = 11.9$ mm (L/403) OK

TL deflection = $81.2 \times 1e8 / (205,000 \times 2,900) = 13.7$ mm (L/351)

Bearings

203 x 133 x 30 UB stiff bearing length, $b_1 = t + 1.6r + 2T = 37.8$ mm

Masonry: 100mm 3.6N/mm² solid block (SF>2.0), class (iii) mortar, normal const/special mfr, Class 1 bearing

Local design strength (factored) = $3.5 \times 1.25/3.1 = 1.41$ N/mm² (BS5628-1:2005 Table 2d/2e)

R1: 250 x 200 mm Padstone

Factored reaction = $1.4 \times 3.60 + 1.6 \times 24.60 = 44.4$ kN

Factored stress under padstone = $44.4 \times 1000/250 \times 200 = 0.89$ N/mm² OK

R2 as R1

Structural Design Calculations

Site address: 29 Manor Dr, Manchester M21 7GQ
Job: Beams structural design
Client:

Job number:
Made by: NK
Page: 4

internal .ps7

Printed 18 Aug 2024 14:04

Beam: Beam B2 - Kitchen Knockthrough -inner

Span: 3.2 m.

	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
U D	roof	1x4	0		L	6.40	6.40	5.46
U L	roof	0.6x4	0		L	3.84	3.84	3.28
U L	wall	4.2x2.5	0		L	16.80	16.80	14.34
Total load (unfactored):						54.1 kN	27.04	27.04
								23.07
Dead/Permanent (unfactored):					12.8 kN	6.40	6.40	5.46
Live/Variable (unfactored):					41.3 kN	20.64	20.64	17.61
Total load (factored):					84.0 kN	41.98	41.98	

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

Maximum B.M. = 33.59 kNm (factored) at 1.60 m. from R1

Maximum S.F. = 42.0 kN (factored) at R1

Mid-span deflections: Dead: $5.46 \times 10^8/EI$ (E in N/mm^2 , I in cm^4)

Live: $17.61 \times 10^8/EI$

Total: $23.07 \times 10^8/EI$

Beam calculation to BS5950-1:2000 using S355 steel

SECTION SIZE : 2No 152 x 89 x 16 UB S355 (compact)

D=152.4 mm B=88.7 mm t=4.5 mm T=7.7 mm $I_x=834 \text{ cm}^4$ $r_y=2.10 \text{ cm}$ $S_x=123 \text{ cm}^3$ $x=19.6$

Section classification: T = 7.7mm $p_y = 355 \text{ N/mm}^2$ $\varepsilon = \sqrt{(275/355)} = 0.88$

(Table 11) Flange: $b/T = 44.3/7.7 = 5.76$ ($\leq 9\varepsilon$: Class 1 plastic)

Web: $d/t = 121.8/4.5 = 27.1$ ($\leq 80\varepsilon$: Class 1, plastic)

For design purposes section classification is Class 2, compact

Shear

Maximum S.F. = 42 kN

Shear capacity = $0.6 p_y t D = 0.6 \times 355 \times 2 \times 4.5 \times 152.4/1000 = 292 \text{ kN OK}$

Bending

Maximum B.M. = 33.6 kNm

Moment capacity, $M_c = p_y S_x = 355 \times 123 \times 2/1000 = 87.3 \text{ kNm OK}$

Lateral-torsional buckling

Beam is laterally restrained at 2.50m from R1

Restraint condition at R1: Compression flange laterally restrained. Nominal torsional restraint. Compression flange partially restrained against rotation on plan (0.85L)

Segment 0 - 2.50m $L_E = (0.85L + L)/2 = 2.31\text{m}$ [Table 13]

Restraint condition at R2: Compression flange laterally restrained. Nominal torsional restraint. Compression flange partially restrained against rotation on plan (0.85L)

Segment 2.50m - L $L_E = (0.85L + L)/2 = 0.65\text{m}$

Segment	M_x	u	v	λ	β_w	λ_{LT}	p_b	M_b	m_{LT}	M_b/m_{LT}	
0.00-2.50	33.6	0.89	0.79	110.12	1.00	77.26	198.1	48.7	0.92	53.2	OK
2.50-3.20	23.0	0.89	0.97	30.83	1.00	26.62	355.0	87.3	0.65	134.2	OK

Critical segment for buckling lies between 0.00m and 2.50m

Bending strength, $p_b = 198 \text{ N/mm}^2$

Maximum moment within segment, $M_x = 33.6 \text{ kNm}$

Equivalent uniform moment factor, $m_{LT} = 0.916$ ($M_2 = 21.1$, $M_3 = 32.0$, $M_4 = 32.6$)

Equivalent uniform moment = $0.916 \times 33.6 = 30.8 \text{ kNm}$

Buckling resistance moment, $M_b = p_b S_x = 198 \times 123 \times 2/1000 = 48.7 \text{ kNm OK}$

Structural Design Calculations

Site address: 29 Manor Dr, Manchester M21 7GQ
Job: Beams structural design
Client:

Job number:
Made by: NK
Page: 5

internal .ps7

Printed 18 Aug 2024 14:04

Web capacity

Check unstiffened web capacity with load of $42.0/2 = 21.0$ kN

$C1 = 48.9/0.00$ kN; $C2 = 1.60$ kN/mm; $C4 = 129$; $K = \min\{0.5+(a_e/1.4d), 1.0\}$; $p_{yw} = 355$ N/mm²

For derivation of C factors see SCI Steelwork Design Guide to BS5950-1:2000 6th ed. (P325)

Minimum required stiff bearing length, $b_1 = 0$ mm; $a_e = 0$ mm; $d = 121.8$ mm ; $K = 0.500$

Bearing capacity, $P_w = C1 + b_1.C2 = 48.9$ kN

Buckling capacity, $P_x = K\sqrt{(C4.P_w)} = 0.500\sqrt{(129 \times 48.9)} = 39.6$ kN ←

Deflection

LL deflection = $17.61 \times 1e8 / (2 \times 205,000 \times 834) = 5.2$ mm (L/621) OK

TL deflection = $23.07 \times 1e8 / (2 \times 205,000 \times 834) = 6.7$ mm (L/474)

Bearings

152 x 89 x 16 UB stiff bearing length, $b_1 = t + 1.6r + 2T = 32.1$ mm; O/A b_1 taken as 120.8 mm

Masonry: 20N/mm² brick, class (iii) mortar, normal const/normal mfr, Class 1 bearing

Local design strength (factored) = $5 \times 1.25/3.5 = 1.79$ N/mm² (BS5628-1:2005 Table 2a)

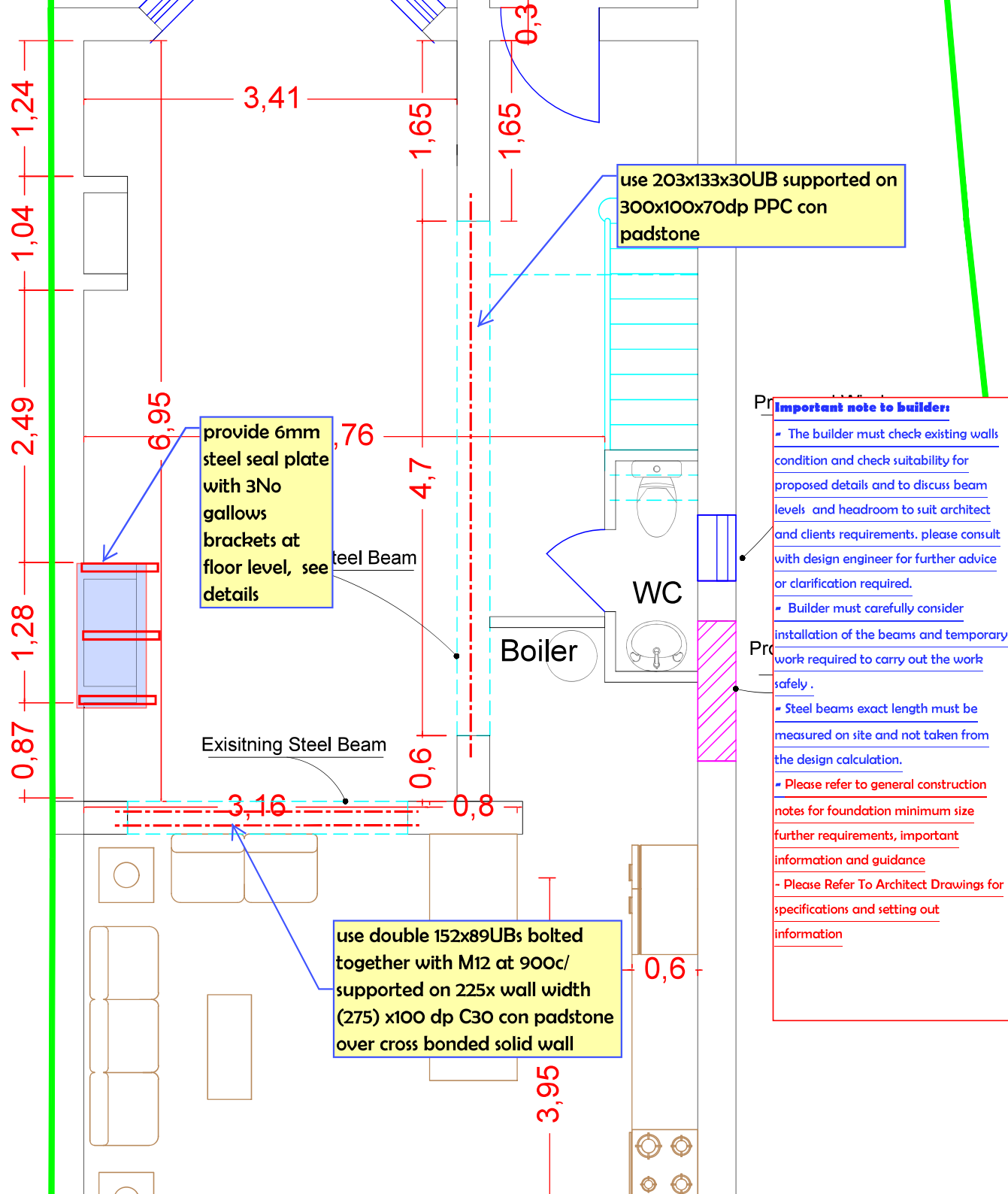
R1: 450 x 100 mm Padstone

Factored reaction = $1.4 \times 6.40 + 1.6 \times 20.64 = 42.0$ kN

Factored stress under padstone = $42.0 \times 1000 / (450 \times 100) = 0.93$ N/mm² OK

R2 as R1

Sections to be bolted together with tube spacers or suitable alternative connection at max 1.5m c/s



**New Support Brackets to partially removed Chimney Support at
2 Manor Dr, Manchester M21 7GQ**

**Check Compliance with LABC guidance for use of gallows
bracket**

1. The party wall supporting the gallows bracket is a minimum of 215mm thick, in brickwork, and in sound condition.

YES

2. The maximum width of the chimney breast is less than 1200mm. For wider chimney breasts a structural engineer should be consulted

YES (1200mm)

3. The chimney breast should project no more than 340mm into the room.

YES

4. The chimney is no more than two storeys high plus roof space.

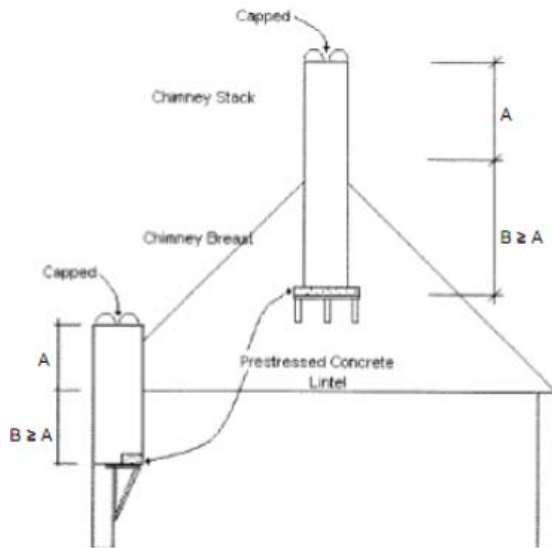
YES.

5. The chimney is no more than two storeys high plus roof space.

YES.

6. The minimum height of the retained chimney breast below the roof line must be equal to or greater than the height of brickwork above the roofline. (See diagram below).

YES (to be confirmed by builder on site)



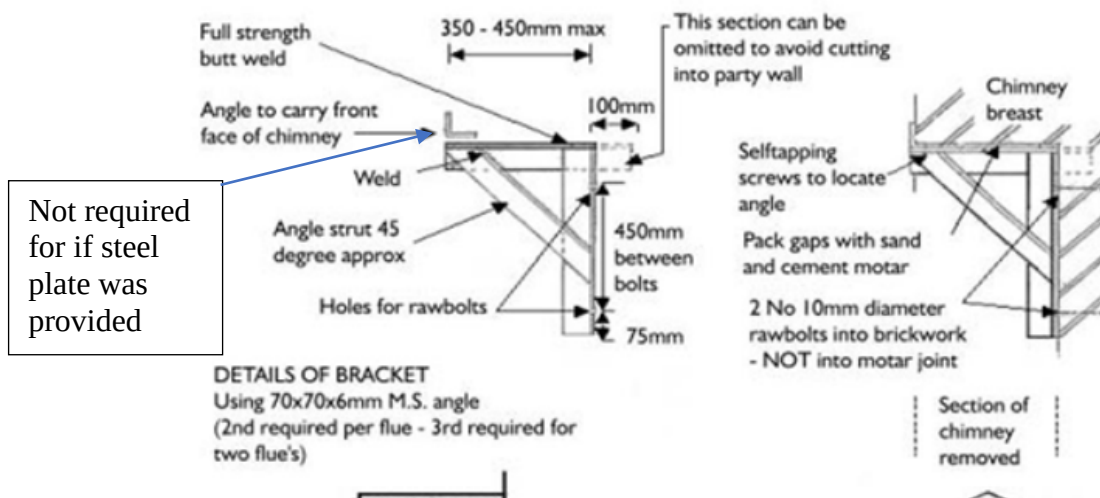
Note: B must be equal to or greater than A
Above: Minimum height of the chimney breast remaining in the house is the height of brickwork in the stack above.

7. The bracket members should be 600mm dp- 70mm x 70mm x 6mm mild steel angles with 6mm fillet shop welded joints with the angles pre-drilled to take a minimum of 2no M12 chemical or resin anchor bolts. (e.g. Chemfix or similar).

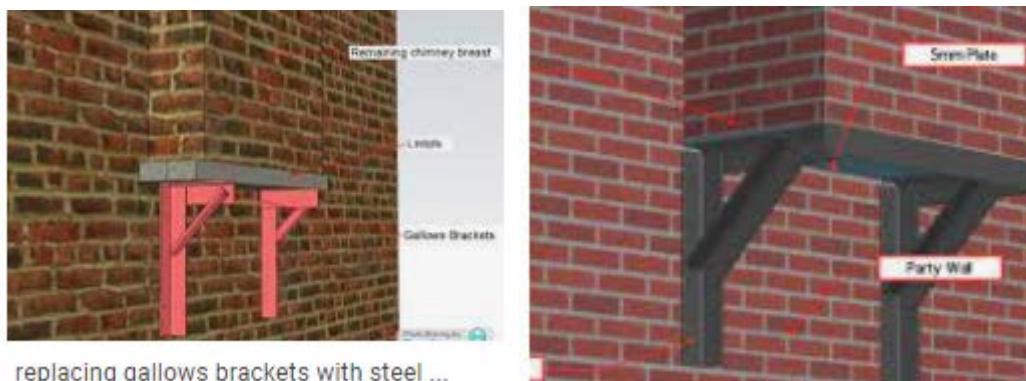
YES 3No



8. The bolts must be drilled into sound brickwork, not mortar joints.
Builder to check and apply
9. A plate (e.g. 5-6mm steel plate) should be placed on the top of the brackets to prevent soot and debris falling from the remaining chimney. See below diagram for minimum dimensions



10. Alternative to steel plate use concrete PPC lintels, see below diagram



Additional notes to the builder

- a. Has the neighbour's part of the stack already been removed.

HSH

Structural design calculations

Address: 29 Manor Dr, Manchester M21 7GQ

Aug 2024

Proposed Alterations

TBC by builder

- b. The relevant notices required under the Party Wall Act 1996 have been served on the adjoining owner (where the chimney is on a party wall).

Builder/client to apply