

PLAN IT JUPITER

STRUCTURAL
DESIGN & DETAIL

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112 Warminster Road, Westbury, BA13 3PR **Structural Calcs**

2026-112WRW-01- 1 -

May 2026

Structural Calculations.

Architectural drawings and setting out information provided by others.

The dimensions used within these calculations are for design purposes only. Detailed measurements are to be taken from site by the building contractor and in accordance with the Architects Drawings. Should any major inconsistencies occur then please inform us.

The contractor is to act as the temporary works coordinator and should be competent and experienced enough to carry out the design and installation of temporary works to BS 5975 in order to maintain stability of the building and excavations.

If these calculations are undertaken at "Planning stage" and prior to Architectural Building Regulation approval being available, and as such some detail may need to be clarified with the Architect and building control prior to construction. The designs herein should be adequate for the Architect to develop their proposals, Tender process and for Building Regulation submission related to structure only. It is advised that calculations are checked and signed off by building control prior to commencement on site.

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Design Brief.



EXISTING
SOUTH WEST ELEVATION (Partial)
SCALE 1:50



EXISTING
SOUTH EAST ELEVATION
SCALE 1:50

Rebuilding of previously taken down chimney stack.

Calculations prepared by; Julian Swift

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Loadings**External Chimney, taken as the following****Dead Load**

Masonry (Outer Leaf);	= 2.20 kN/m ²
Insulation, flu and linings as necessary;	= 0.30 kN/m ²
Total Dead Load;	= 2.50 kN/m ²

Wind Loading**Dynamic pressure;**

Say 0.7;	= 0.70 kN/m ²
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Key Minimum Structural Requirements (Approved Document A)

Height-to-Width Ratio: If the chimney is not adequately secured, its total height (from the highest point of intersection with the roof) should not exceed 4.5 times its least horizontal dimension.

Support & Anchorage: Chimneys must be securely tied to the roof structure to prevent lateral movement.

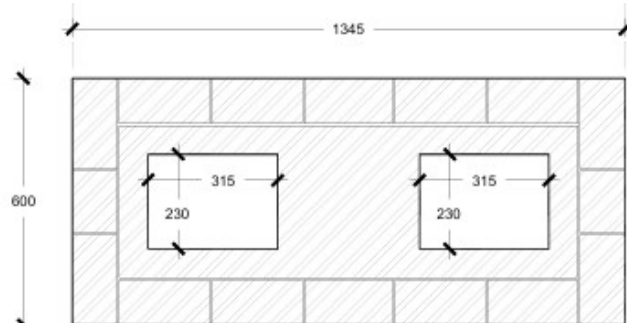
Masonry Density: Masonry used for the stack should generally have a minimum density of 1500kg/m³.



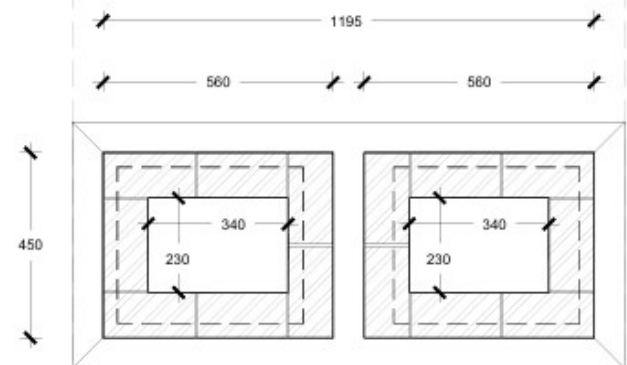
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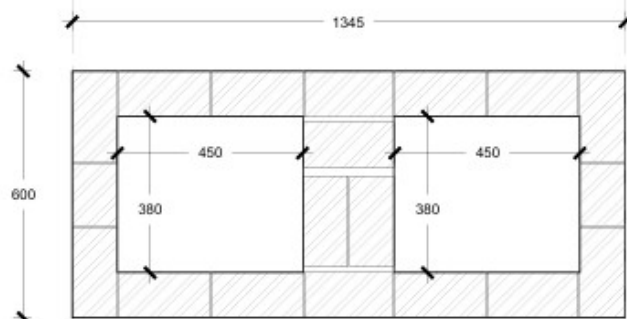
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PLAN AT HORIZONTAL SECTION C-C
SCALE 1:10



PLAN AT HORIZONTAL SECTION B-B
SCALE 1:10



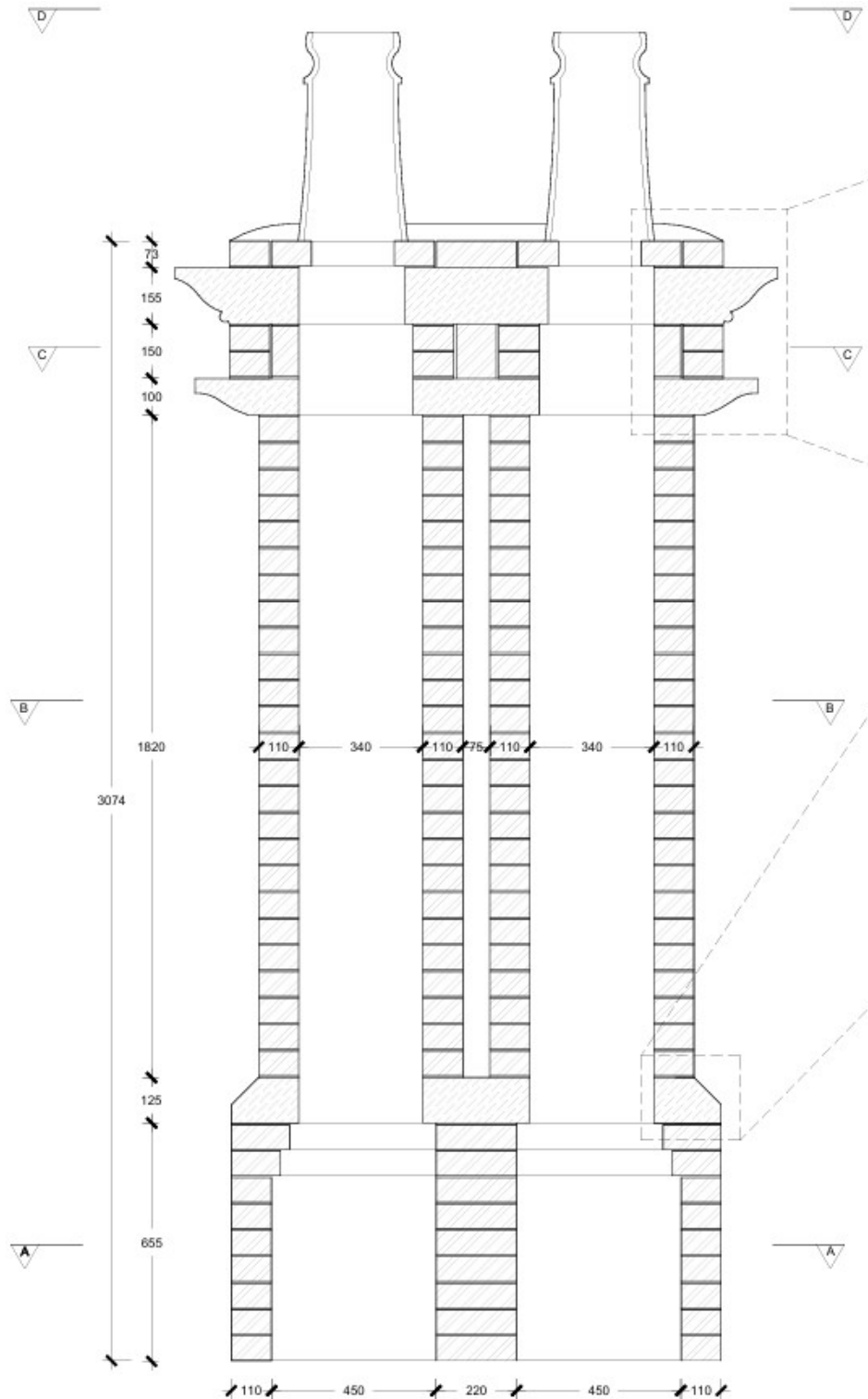
PLAN AT HORIZONTAL SECTION A-A
SCALE 1:10



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Vertical Section Through Chimney
 SCALE 1:10



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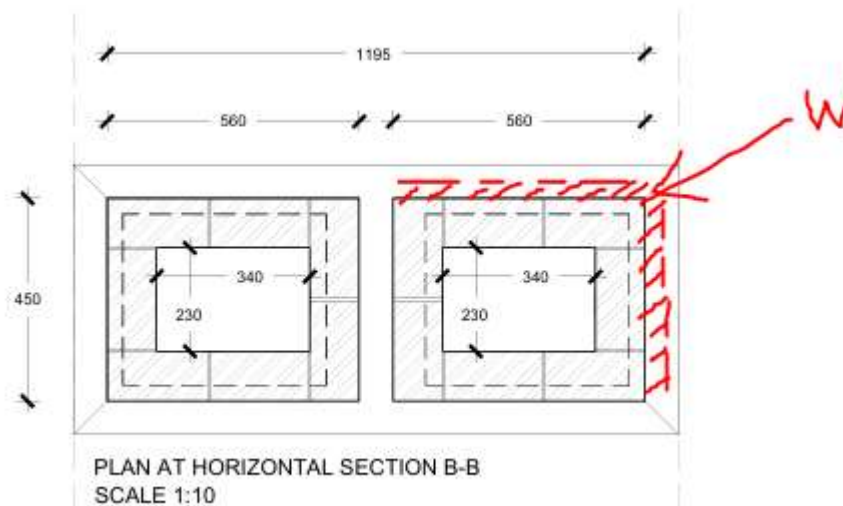
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Taking height as being 2.4m then $2400/4.5 = 537\text{mm}$ as a minimum width requirement to satisfy the building regulations.

This is then acceptable for the stack where the top and bottom are combined.

in one direction, however, where the stacks are built with a gap then one can argue that the minimum width is 450mm so max allowable height taken as 2025mm.

The stack is in reality a hollow column, and when dealing with wind, one can argue that any effect tends to be spread through all four faces,



Whereby panel is buttressed by the returns. The taller the stack the more reliant it is on resisting overturning from bed joints only.

Check related to wind as a universal pressure.

MASONRY WALL PANEL DESIGN

In accordance with BS5628-1:2005

Masonry panel details

side of chimney flue - Unreinforced masonry wall without openings

Panel length; $L = 450 \text{ mm}$

Panel height; $h = 2400 \text{ mm}$

Panel support conditions

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Effective panel length; $L_{ef} = 1.0 \times L = 450 \text{ mm}$

Effective panel height; $h_{ef} = 2.0 \times h = 4800 \text{ mm}$

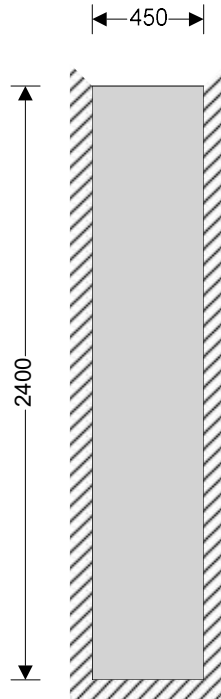
Right, left and bottom supported, bottom continuous



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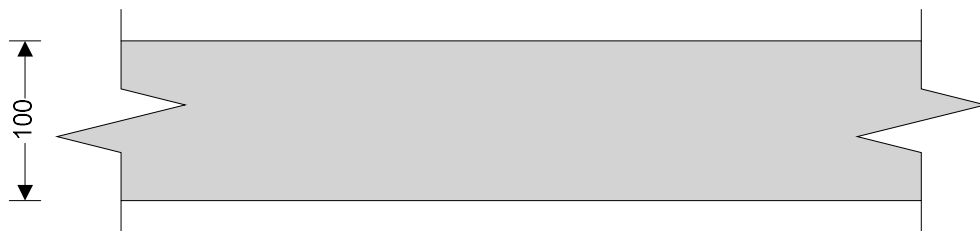
Single-leaf wall construction details

Wall thickness;

$t = 100 \text{ mm}$

Effective wall thickness;

$t_{ef} = t = 100 \text{ mm}$



Masonry details

Masonry type;

12%

Clay bricks having a water absorption between 7% and

Compressive strength of unit;

$p_{unit} = 5.0 \text{ N/mm}^2$

Mortar strength Class/Designation;

M6 / (ii);

Height of masonry units;

$h_b = 65 \text{ mm}$

Density of masonry;

$\gamma = 18.0 \text{ kN/m}^3$

From BS5628-1 Table 2a - Characteristic compressive strength of masonry

Characteristic compressive strength;

$f_k = (0.7 + 1.5 \times t \times L / 1 \text{ m}^2) \times 2.5 \text{ N/mm}^2 = 1.92 \text{ N/mm}^2$

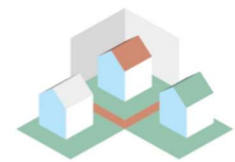
From BS5628-1 Table 3 - Characteristic flexural strength of masonry

Plane of failure parallel to bed joints;

$f_{kx_para} = 0.40 \text{ N/mm}^2$

Plane of failure perpendicular to bed joints;

$f_{kx_perp} = 1.10 \text{ N/mm}^2$



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Lateral loading details

Characteristic wind load on panel; $W_k = 1.000 \text{ kN/m}^2$

Partial safety factors for material strength

Category of manufacturing control; **Category II**

Category of construction control; **Normal**

Partial safety factor for masonry in compression; $\gamma_{mc} = 3.50$

Partial safety factor for masonry in flexure; $\gamma_{mf} = 3.00$

Partial safety factor for masonry in shear; $\gamma_{mv} = 2.50$

Horizontal loading (cl 32)

Limiting dimensions (cl 32.3)

Area of panel; $A_p = h \times L = 1.1 \text{ m}^2$

Limiting area of panel; $A_{max} = 2025 \times t_{ef}^2 = 20.3 \text{ m}^2$

PASS - Area of panel does not exceed limiting area of panel

Limiting panel dimension; $L_{max} = 50 \times t_{ef} = 5000 \text{ mm}$

PASS - Limiting panel dimension is not exceeded

Partial safety factors for design loads

Partial safety factor for design wind load; $\gamma_{fW} = 1.40$

Partial safety factor for design dead load; $\gamma_{fG} = 0.90$

Design moments of resistance in panels (cl 32.4.2)

Self weight of wall; $S_{wt} = 0.5 \times h \times t \times \gamma = 2.16 \text{ kN/m}$

Design vertical compressive stress; $g_d = \gamma_{fG} \times (G_k + S_{wt}) / t = 0.02 \text{ N/mm}^2$

Enhanced flexural strength of masonry; $f_{ka_para} = f_{kx_para} + \gamma_{mf} \times g_d = 0.46 \text{ N/mm}^2$

Section modulus of wall; $Z = t^2 / 6 = 1666667 \text{ mm}^3/\text{m}$

Elastic design moment of resistance; $M_d = f_{kx_perp} \times Z / \gamma_{mf} = 0.611 \text{ kNm/m}$

Design moment in panels (cl 32.4.2)

Using elastic analysis to determine bending moment coefficients for a horizontally spanning panel

Bending moment coefficient; $\alpha = 0.125$

Design moment in wall; $M = \alpha \times W_k \times \gamma_{fW} \times L^2 = 0.035 \text{ kNm/m}$

PASS - Resistance moment exceeds design moment

Therefore, 450mm width is acceptable for 2.4m height.

Check applied wind load as an eccentric moment at the top of a 450mm wide panel that is 2.4m high.

VERTICAL LOADING RECTANGULAR COLUMN (BS5628-1:2005)

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

Mortar designation; Mortar = "iii"

Brick compressive strength; $p_{unit} = 5.0 \text{ N/mm}^2$

Characteristic compressive strength; $f_k = 2.50 \text{ N/mm}^2$

Column width; $b = 450 \text{ mm}$



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Column thickness;	$t = 100 \text{ mm}$
Column height;	$h = 2.40 \text{ m}$
Column slenderness - minor axis (Clause 24.1);	$\lambda_t = h/t = 24.00$
Column slenderness - major axis (Clause 24.1);	$\lambda_b = h/b = 5.33$
Maximum slenderness;	$\lambda_{\max} = \max(\lambda_t, \lambda_b) = 24.00$

Slenderness < 27 - OK

Partial safety factor for material (Table 4);	$\gamma_m = 3.5$
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Load eccentricity

Eccentricity of applied load about minor axis;	$e_{xt} = 30.0 \text{ mm}$
Eccentricity of applied load about major axis;	$e_{xb} = 0.0 \text{ mm}$

Capacity reduction factor: minor axis

Eccentricity due to slenderness;	$e_{at} = \max(0 \text{ mm}, t \times (((h/t)^2/2400) - 0.015)) = 22.5 \text{ mm}$
Design eccentricity;	$e_{tt} = 0.6 \times \max(\text{abs}(e_{xt}), 0.05 \times t) + e_{at} = 40.5 \text{ mm}$
	$e_{mt} = \max(\text{abs}(e_{xt}), e_{tt}) = 40.5 \text{ mm}$;
Capacity reduction factors;	$\beta_{tcalc} = \max(0, 1.1 \times (1 - (2 \times e_{mt} / t))) = 0.21$
	$\beta_{\max} = 1.0$
	$\beta_t = \min(\beta_{tcalc}, \beta_{\max}) = 0.21$

Capacity reduction factor: major axis

Eccentricity due to slenderness;	$e_{ab} = \max(0 \text{ mm}, b \times (((h/b)^2/2400) - 0.015)) = 0.0 \text{ mm}$
Design eccentricity;	$e_{tb} = 0.6 \times \max(\text{abs}(e_{xb}), 0.05 \times b) + e_{ab} = 13.5 \text{ mm}$
	$e_{mb} = \max(\text{abs}(e_{xb}), e_{tb}) = 13.5 \text{ mm}$;
Capacity reduction factors;	$\beta_{bcalc} = \max(0, 1.1 \times (1 - (2 \times e_{mb} / b))) = 1.03$
	$\beta_{\max} = 1.0$
	$\beta_b = \min(\beta_{bcalc}, \beta_{\max}) = 1.00$
Minimum capacity reduction factor;	$\beta_{\min} = \min(\beta_t, \beta_b) = 0.21$

Design vertical load resistance

Compressive strength correction factor	
Plan area of column;	$A = t \times b = 0.05 \text{ m}^2$
For small plan area (Clause 19.1.2);	$c = \min(1.0, 0.7 + (1.5 \text{ m}^{-2}) \times A) = 0.77$
Design vertical load resistance;	$DVLR = \beta_{\min} \times t \times b \times c \times f_k / \gamma_m = 5.156 \text{ kN}$
Applied factored vertical load on column;	$V = 5.000 \text{ kN}$

Column - OK

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Therefore, also acceptable.

We therefore conclude that the as existing arrangement of chimney stack is within acceptable limits, even though it initially exceeds the parameters required by part A of the building Regulations.



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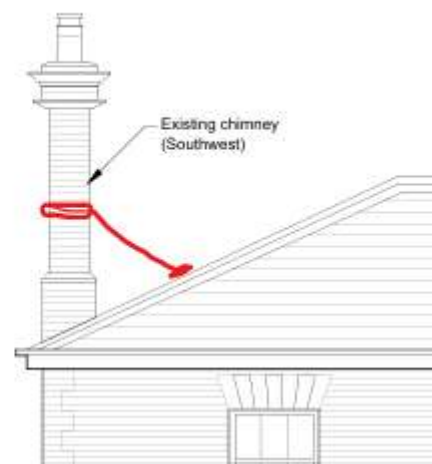
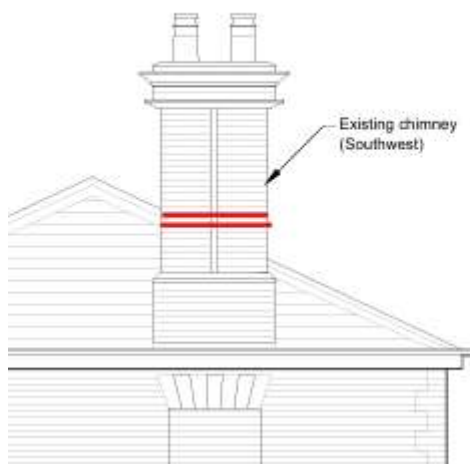
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With any listed building we do suggest that a quinquennial inspection of the roof should be carried out, and this should also include the chimneys. It is important that the integrity of mortar, bond and condition of bricks are checked and maintained with any repairs to mortar to match the existing lime-based mortars, and brickwork of a similar quality.

Potentially leaning can occur, and therefore as a potential mitigation method thought to bracing, to reduce the effective height of the stack may be considered.



Obviously, these would need to be to the approval of conservation officer and also follow the general guidelines of Historic England, whereby a restraint girdle of fabricated steel is braced back to main roof via tie rods.

This tends to be on the weaker axis only.

Again, this is not required based on the calculations above, but potential mitigation for any future movement should in the unlikely event it may occur due to reduction of strength to mortar or if pointing is compromised.



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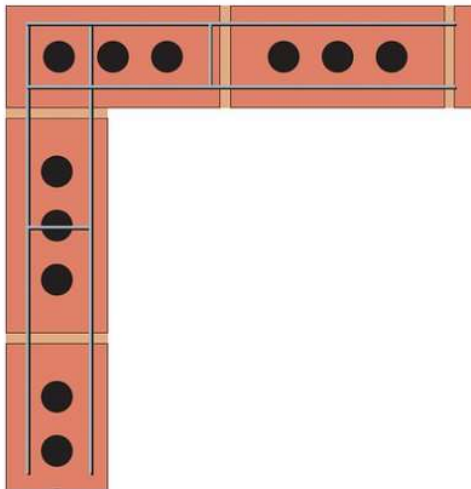
There are other alternatives, but these would need to be run past the conservation team.

Option B:

Can be to reinforce the masonry with Ancon or similar bedjoint reinforcement.

Ancon AMR Corner Units

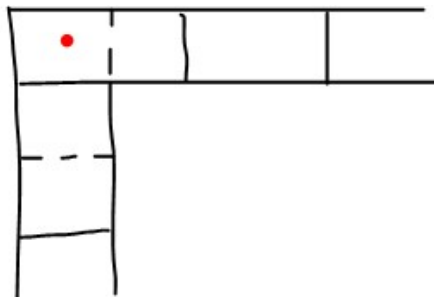
Prefabricated corner units can be manufactured to provide true continuity of reinforcement.



102mm Brick use 60mm AMR x3.0mm thk. If using corner units then stagger these / hand them accordingly to suit the configuration of the chimney. This will increase the flexial strength of the bed joints and therefore assist with any side ways sway.

Option C:

Depending upon the type of bricks used (conservation discretion) potential to use resin anchored H10 bars to the corner bricks if frogs not used. Ideally the existing corners of the chimney would need to be carefully removed, to allow predrilling of the existing to line up with new masonry holes and to anchor the rebar into the existing.



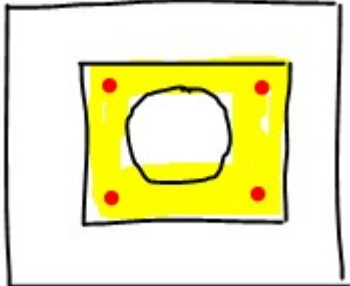
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Option D:

This option depends on if the stack is to be lined. Cast-in-Place (Concrete) Liners best suited for Older historic stacks or severely deteriorated chimneys needing major structural reinforcement. An inflatable rubber tube is lowered down the chimney and a specialized, heat-resistant cement is pumped into the void between the tube and the brick, filling all cracks. Again H10 rebar can be included into the corners.

Once cured, the tube is removed, leaving behind a seamless, rigid, insulated structural liner.

**Calculation End.**