SKYM WRAPSON JOHNSTON
SWJ

Crowmarsh Parish Council

Crowmarsh Gifford Pavillion

Structural Calculations Package For Removal Of Internal Load Bearing Wall

February 2020

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Structural Calculations For Removal Of Internal Loadbearing Wall

At

Crowmarsh Gifford Pavillion

SWJ Consulting Ltd has prepared these calculations in accordance with the instruction given by Mr MC Crea on Behalf Of Crowmarsh Parsh Council(The Client).

Prepared by:	Sam Johnston *BEng (Hons) MSc*	Signed:	SSchutt	Date:	25/02/21
Checked by:	Sam Johnston *BEng (Hons) MSc*	Signed:	SSchutt	Date:	25/02/21
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Issue

Status Key:

S0 – Preliminary, S1 – Co-ordination, S2 – Information, S3 – Comment, S4 – Building Control Approval

Date	Status	Sheet No	Sets	Recipient

Amendments

Date	Status	Amendments	Recipient

CONTENTS

STRUCTURAL ANALYSIS & DESIGN

DESIGN STANDARDS

The structural design will be undertaken in accordance with Eurocode Design Standards and it has been assumed that all construction work will be undertaken to Execution Class EXC2.

Where no specific guidance is given within the Eurocodes, the design guidance in the British Standards, Building Regulations and NHBC, Premier Guarantee, etc. insurance will be used and referenced as necessary.

CDM

Our service only provides a specification for the structural work required for the project as presented in SWJ's project specific calculations, sketches and specifications. Unfortunately, we are unable to provide 'Principal Designer' duties for this project. This role should be undertaken by the architect or lead designer who will manage the pre-construction phase of the work as required by the Construction Design and Management Regulations 2015. We are however, able to provide additional structural information if requested by the 'Principal Designer'.

SWJ's project specific design, as presented through our calculations, sketches and specifications has been produced on the assumption that the client will appoint a suitably qualified competent contractor who is experienced in working in the field of domestic alterations, refurbishments and new build.

All construction activities however involve risk and it is not possible to eliminate these entirely. We consider buildability, maintenance and demolition aspects of a design assuming that the contractor will use methods of construction and procedures which are generally accepted as safe within the industry, when carried out by appropriately qualified personnel. We do not dictate what these methods or procedures must be as they have to be chosen by the contractor in relation to his own management and construction skills and abilities. We therefore only identify areas that we consider as abnormal hazards and associated risks, where our engineering design or the prevalent site conditions, require an unusual approach or where a design may require special consideration as to how it is built, maintained or demolished.

On that basis we confirm, within the limits of our appointment, we have produced a design which does not introduce any abnormal hazards and associated risks.

GENERAL NOTES

- Until technical approval has been obtained from the relevant Authorities it should be understood that all SWJ sketches and specifications issued are preliminary and NOT for construction. Should the contractor start site work prior to approval been given, it is entirely at his own risk.
- Should there be any discrepancies between details indicated in SWJ sketches and specifications and those indicated on other drawings or conditions found on site the Engineer should be informed PRIOR to construction on site.
- All temporary works requirements are the responsibility of the contractor and are to be assessed on site by the contractor and agreed with the Building Control Inspector. The Engineer should be informed of any concerns PRIOR to construction on site.
- SWJ sketches and specifications are project specific and are not to be used and shall not be copied in whole or in part without the written permission of SWJ Consulting Limited.
- All lengths should be taken from site dimensions and then have the bearing lengths added. You should NOT use the dimensions in these calculations.
- It has been assumed that foundations and bearing walls are in an acceptable condition and suitable for the modified load paths. Investigations should be made by the contractor to verify this. If in doubt, ASK.

MATERIALS

Timber

All timber to be Grade C24 in accordance with BS EN 14081 unless noted otherwise
All timber to be sourced from a sustainable source accredited by the FSC

Masonry

All new load bearing blockwork to be 7.3N blocks in accordance with BS EN 771-3 and have a minimum density of 1500kg/m^3

All masonry above ground to be laid in M4 mortar (1:1:6) in accordance with BS EN 998-1

All masonry below ground to be laid in M12 mortar (1:3) in accordance with BS EN 998-1

Steelwork

All internal steelwork to be Grade S355 JR in accordance with BS EN 10025

All external steelwork to be Grade S355 J2 in accordance with BS EN 10025

All internal steelwork to be protected with two coats of zinc phosphate primer

All external steelwork (including steelwork within an external wall) to be galvanised

All steelwork elements to be fabricated to a required CE Marking Execution Class of EXC2 to BS EN 1090-2.

Concrete

All concrete to be designated mix RC25/30 in accordance with BS 8500-1 unless noted otherwise

DESIGN LOADS

Existing Pitched Roof

- Permanent load - Cambrian Slate Tiles 0.25, timber battens 0.10, felt 0.05, insulation 0.05 on 20 degrees pitch
  - Timber rafters 0.15
  - Plasterboard and skim 0.15 and light services 0.10 on plan

  = 0.910 kN/m2 on plan

- Variable load - Imposed load on the pitched roof 0.75 Light Storage 0.25 = 1.000 kN/m2 on plan

Masonry Wall

- Permanent load - cavitywall 100mm Block 2.10 + 2.10
  - Plasterboard and skim 0.20

  = 4.40kN/m2 on elev

BEAM 1

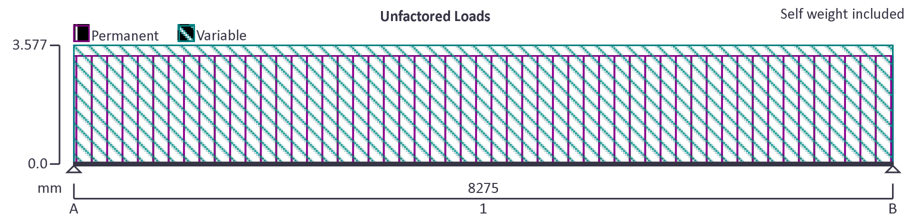
Roof Loads

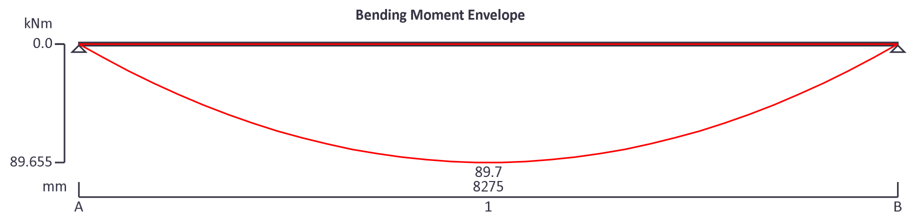
Permanent Load on Beam	g_k	$= 0.910\text{kN/m}^2 \times 3.577\text{m}$	= **3.255** kN/m
Imposed Load on Beam	q_k	$= 1.000\text{kN/m}^2 \times 3.577\text{m}$	= **3.577** kN/m

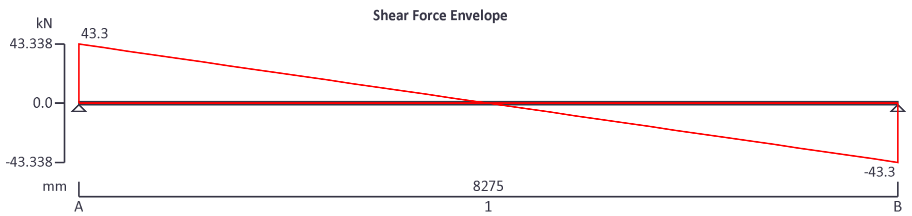
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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 3.255 kN/m Variable full UDL 3.577 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

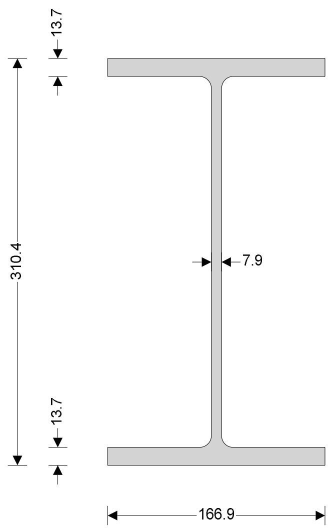
Maximum moment	M_{max} = **89.7** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **43.3** kN	V_{min} = **-43.3** kN
Deflection	δ_{max} = **18.3** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **43.3** kN	R_{A_min} = **43.3** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **15.7** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **14.8** kN	
Maximum reaction at support B	R_{B_max} = **43.3** kN	R_{B_min} = **43.3** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **15.7** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **14.8** kN	

Section details

Section type	**UKB 305x165x54 (Tata Steel Advance)**
Steel grade	**S355**

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

Nominal thickness of element	$t = \max(t_f, t_w)$ = **13.7** 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 $c = d$ = **265.2** mm

$c / t_w = 41.3 * \varepsilon <= 72 * \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 * r) / 2$ = **70.6** mm

$c / t_f = 6.3 * \varepsilon <= 9 * \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 * t_f$ = **283** mm

Shear area factor η = **1.000**

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

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **43.3** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 * b * t_f + (t_w + 2 * r) * t_f, \eta * h_w * t_w)$ = **2656** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v * (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **544.3** kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **89.7** kNm

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} * f_y / \gamma_{M0}$ = **300.4** 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 $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.953**

Poissons ratio ν = **0.3**

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

Unrestrained length $L = 1.0 * L_{s1}$ = **8275** mm

Elastic critical buckling moment $M_{cr} = C_1 * \pi^2 * E * I_z / (L^2 * g) * \sqrt{[I_w / I_z + L^2 * G * I_t / (\pi^2 * E * I_z)]}$ = **126.3** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} * f_y / M_{cr})}$ = **1.542**

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 $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **1.586**

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.410**

Modification factor $f = \min(1 - 0.5 * (1 - k_c) * [1 - 2 * (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **1.000**

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1)$ = **0.410**

Design buckling resistance moment - eq 6.55 $M_{b,Rd} = \chi_{LT,mod} * W_{pl.y} * f_y / \gamma_{M1}$ = **123** 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 $\delta_{lim} = L_{s1} / 360$ = **23** mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **18.299** mm

PASS - Maximum deflection does not exceed deflection limit

BEAM 1 LHS

MASONRY BEARING DESIGN

In accordance with EN1996-1-1:2005 + A1:2012, incorporating Corrigenda February 2006 and July 2009 and the UK National Annex.

Tedds calculation version 1.0.11

Summary table

Load	Local concentration		Spreader		Utilisation	
	Design force	Resistance	Design stress	Resistance		
1	43.4 kN	47.7 kN	N/A	N/A	0.909	Pass

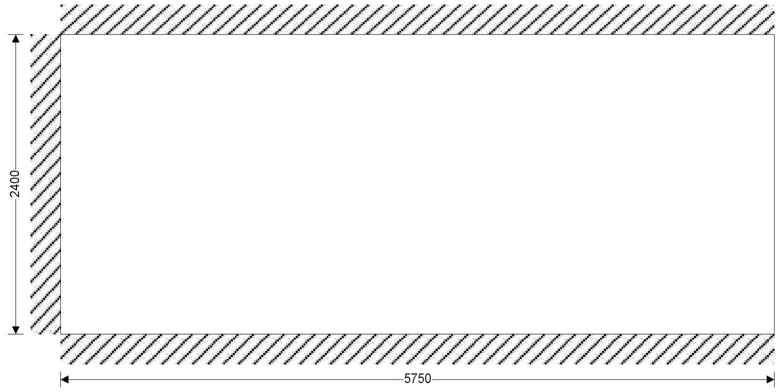
Masonry panel details

Panel length L = **5750** mm

Panel height h = **2400** mm

Panel support conditions

Top, bottom and one vertical edge supported



Effective height of masonry wall - Section 5.5.1.2

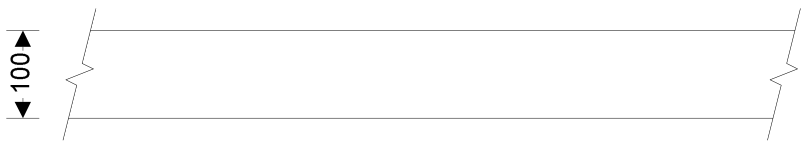
Vertical restraints L >= 15 * t_{ef} (Wall restrained at top and bottom only)

Horizontal restraints **Simply supported**

Reduction factor - eq. 5.5 ρ_2 = **1.00**

Effective height of wall - eq. 5.2 $h_{ef} = \rho_2 \times h$ = **2400** mm

Wall construction details



Wall type **Single leaf panel**

Overall wall thickness t = **100.0** mm

Effective thickness of masonry wall - Section 5.5.1.3

Effective thickness $t_{ef} = t$ = **100.0** mm

Masonry material details

Unit type **Aggregate concrete - Group 1**

Compressive strength of masonry unit f_c = **7.3** N/mm^2

Height of unit h_u = **215** mm

Width of unit w_u = **100** mm

Conditioning factor k = **1.0**

- Conditioning to the air dry condition in accordance with cl.7.3.2

Shape factor - Table A.1 d_{sf} = **1.38**

Mean compressive strength of masonry unit $f_b = f_c * k * d_{sf}$ = **10.07** N/mm^2

Specific weight of units γ = **18** kN/m^3

Mortar type **M4 - General Purpose**

Compressive strength of mortar f_m = **4.0** N/mm^2

Compressive strength factor - Tbl. NA 4 K = **0.75**

Characteristic compressive strength - eq. 3.1 $f_k = K * f_b^{0.7} * f_m^{0.3}$ = **5.73** N/mm^2

Short term secant modulus of elasticity factor K_E = **1000**

Modulus of elasticity - cl.3.7.2 $E_w = K_E \times f_k$ = **5727** N/mm^2

Design compressive strength of masonry

Category of manufacturing control **Category II**

Class of execution control **Class 2**

Partial factor for compressive strength γ_M = **3.00**

Cross-sectional area of wall $A = L \times t$ = **0.58** m^2

Design compressive strength of masonry $f_d = f_k / \gamma_M$ = **1.91** N/mm^2

Partial safety factors for design loads

Partial safety factor for permanent load γ_{fG} = **1.35**

Partial safety factor for variable load γ_{fQ} = **1.50**

Superimposed vertical loading details

Permanent UDL at top of wall g_k = **0.00** kN/m

Variable UDL at top of wall q_k = **0.00** kN/m

Eccentricity of permanent UDL load e_{gu} = **0** mm

Eccentricity of variable UDL load e_{qu} = **0** mm

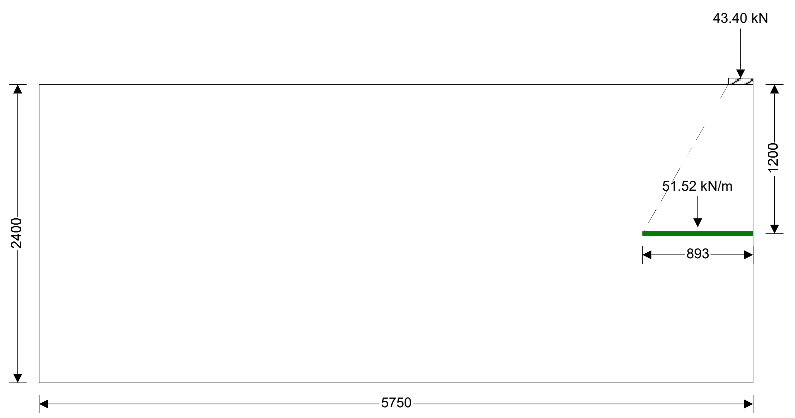
Slenderness ratio of masonry wall - Section 5.5.1.4

Slenderness ratio limit λ_{lim} = **27**

Slenderness ratio $\lambda = h_{ef} / t_{ef}$ = **24.0**

PASS - Slenderness ratio is less than slenderness limit

Concentrated Load 1 details



Permanent concentrated load G_{kc1} = **15.70** kN

Variable concentrated load Q_{kc1} = **14.80** kN

Eccentricity of concentrated load	e_{c1} = **0** mm
Length of concentrated load	L_{c1} = **200** mm
Width of concentrated load	w_{c1} = **100** mm
Height of concentrated load	h_{c1} = **2400** mm
Distance of load to right vertical edge	r_{11} = **0** mm
Distance of load to nearest vertical edge	a_{11} = **0** mm

Walls subjected to concentrated loads - Section 6.1.3

Eccentricity check — $e_{c1} <= t / 4$

PASS - Eccentricity of load is less than t/4

Area of bearing	$A_{b1} = L_{c1} * w_{c1}$ = **20000** mm^2
Effective length of bearing at mid-height	$l_{efm1} = L_{c1} + h_{c1} / 2 * \tan(30) + r_{11}$ = **893** mm
Effective bearing area	$A_{ef1} = l_{efm1} * t$ = **89282** mm^2
Bearing area ratio check	$A_{ratio1} = Min(A_{b1} / A_{ef1}, 0.45)$ = **0.22**
Initial enhancement factor	$\beta_{init1} = Max((1 + 0.3 * a_{11} / h_{c1}) * (1.5 - 1.1 * A_{ratio1}), 1.0)$ = **1.25**
Maximum enhancement factor	$\beta_{max1} = Min(1.25 + a_{11} / (2 * h_{c1}), 1.5)$ = **1.25**
Enhancement factor for concentrated loads	$\beta_1 = Min(\beta_{init1}, \beta_{max1})$ = **1.25**
Design value of the concentrated load	$N_{Edc1} = G_{kc1} * \gamma_{fG} + Q_{kc1} * \gamma_{fQ}$ = **43.40** kN
Design value concentrated load resistance	$N_{Rdc1} = \beta_1 * A_{b1} * f_d$ = **47.72** kN

PASS - Design resistance exceeds applied concentrated load

Walls subjected to mainly vertical loading - Section6.1.2

Eccentricity of permanent UDL at mid-height below concentrated load

$e_{gmu1} = e_{gu} * h_{c1} / (2 * h)$ = **0.0** mm

Eccentricity of variable UDL at mid-height below concentrated load

$e_{qmu1} = e_{qu} * h_{c1} / (2 * h)$ = **0.0** mm

Eccentricity of concentrated load at mid-height	$e_{mc1} = e_{c1} / 2$ = **0.0** mm
Initial eccentricity - cl.5.5.1.1(4)	$e_{init} = h / 450$ = **5.3** mm
Concentrated load at mid-height as UDL	$N_{mc1} = N_{Edc1} / l_{efm1}$ = **48.60** kN/m
Vertical load at mid-height	$N_{Ed1} = (g_k + \gamma * t * (h - h_{c1} / 2)) * \gamma_{fG} + q_k * \gamma_{fQ} + N_{mc1}$ = **51.52** kN/m
Design moment at mid-height	$M_{Ed1} = g_k * \gamma_{fG} * e_{gmu1} + q_k * \gamma_{fQ} * e_{qmu1} + N_{mc1} * e_{mc1}$ = **0.00** kNm/m
Eccentricities due to loads - eq. 6.7	$e_{m1} = Abs(M_{Ed1}) / N_{Ed1} + e_{init}$ = **5.3** mm
Slenderness ratio limit for creep eccentricity	λ_c = **27**
Eccentricity due to creep	e_{k1} = **0.0** mm
Eccentricity at mid-height - eq. 6.6	$e_{mk1} = Max(e_{m1} + e_{k1}, 0.05 \times t)$ = **5.3** mm
From eq. G2	$A_{11} = 1 - 2 * e_{mk1} / t$ = **0.89**
From eq. G3	$u_1 = (h_{ef} / t_{ef} * (1 / K_E)^{1/2} - 0.063) / (0.73 - 1.17 * e_{mk1} / t)$ = **1.04**
Capacity reduction factor - eq. G1	$\Phi_{m1} = A_{11} * \exp(-(u_1^2) / 2)$ = **0.52**
Design vertical resistance of panel - eq.6.2	$N_{Rd1} = \Phi_{m1} * t * f_d$ = **99.05** kN/m

PASS - Design value of vertical resistance exceeds applied vertical load

BEAM 1 RHS

MASONRY BEARING DESIGN

In accordance with EN1996-1-1:2005 + A1:2012, incorporating Corrigenda February 2006 and July 2009 and the UK National Annex.

Tedds calculation version 1.0.11

Summary table

Load	Local concentration		Spreader		Utilisation	
	Design force	Resistance	Design stress	Resistance		
1	43.4 kN	26.5 kN	1.00 N/mm^2	2.86 N/mm^2	0.348	Pass

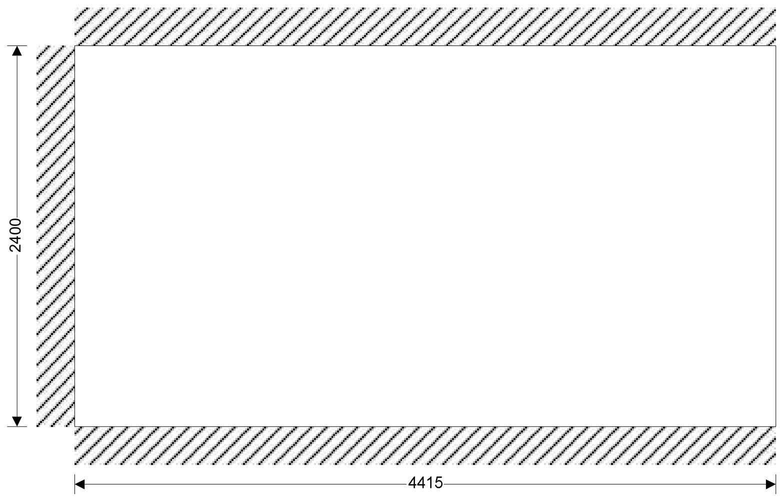
Masonry panel details

Panel length — L = **4415** mm

Panel height — h = **2400** mm

Panel support conditions

Top, bottom and one vertical edge supported



Effective height of masonry wall - Section 5.5.1.2

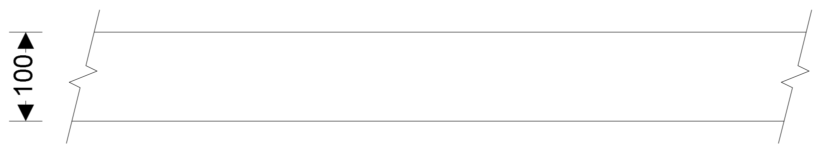
Vertical restraints — L >= 15 * t_{ef} (Wall restrained at top and bottom only)

Horizontal restraints — **Simply supported**

Reduction factor - eq. 5.5 — ρ_2 = **1.00**

Effective height of wall - eq. 5.2 — $h_{ef} = \rho_2 \times h$ = **2400** mm

Wall construction details



Wall type — **Single leaf panel**

Overall wall thickness — t = **100.0** mm

Effective thickness of masonry wall - Section 5.5.1.3

Effective thickness — t_{ef} = t = **100.0** mm

Masonry material details

Unit type — **Aggregate concrete - Group 1**

Compressive strength of masonry unit — f_c = **7.3** N/mm^2

Height of unit — h_u = **215** mm

Width of unit — w_u = **100** mm

Conditioning factor — k = **1.0**

- Conditioning to the air dry condition in accordance with cl.7.3.2

Shape factor - Table A.1 d_{sf} = **1.38**
Mean compressive strength of masonry unit $f_b = f_c * k * d_{sf}$ = **10.07** N/mm^2
Specific weight of units γ = **18** kN/m^3
Mortar type **M4 - General Purpose**
Compressive strength of mortar f_m = **4.0** N/mm^2
Compressive strength factor - Tbl. NA 4 K = **0.75**
Characteristic compressive strength - eq. 3.1 $f_k = K * f_b^{0.7} * f_m^{0.3}$ = **5.73** N/mm^2
Short term secant modulus of elasticity factor K_E = **1000**
Modulus of elasticity - cl.3.7.2 $E_w = K_E \times f_k$ = **5727** N/mm^2

Design compressive strength of masonry
Category of manufacturing control **Category II**
Class of execution control **Class 2**
Partial factor for compressive strength γ_M = **3.00**
Cross-sectional area of wall $A = L \times t$ = **0.44** m^2
Design compressive strength of masonry $f_d = f_k / \gamma_M$ = **1.91** N/mm^2

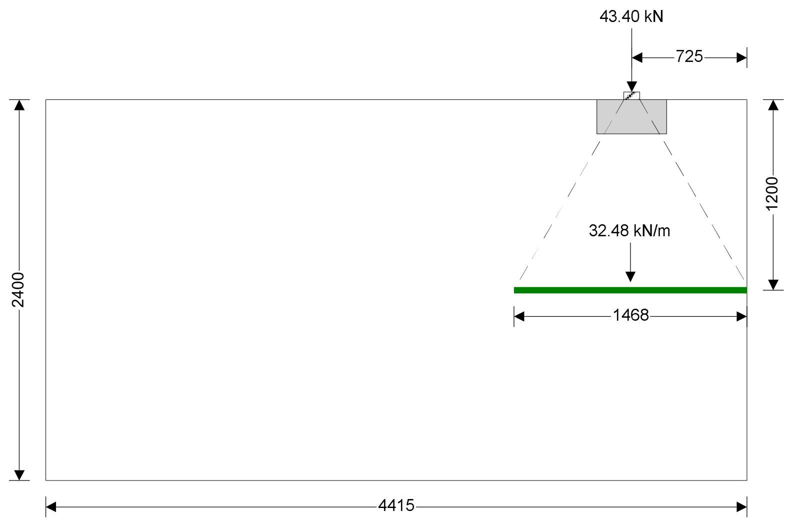
Partial safety factors for design loads
Partial safety factor for permanent load γ_{fG} = **1.35**
Partial safety factor for variable load γ_{fQ} = **1.50**

Superimposed vertical loading details
Permanent UDL at top of wall g_k = **0.00** kN/m
Variable UDL at top of wall q_k = **0.00** kN/m
Eccentricity of permanent UDL load e_{gu} = **0** mm
Eccentricity of variable UDL load e_{qu} = **0** mm

Slenderness ratio of masonry wall - Section 5.5.1.4
Slenderness ratio limit λ_{lim} = **27**
Slenderness ratio $\lambda = h_{ef} / t_{ef}$ = **24.0**

PASS - Slenderness ratio is less than slenderness limit

Concentrated Load 1 details



Permanent concentrated load G_{kc1} = **15.70** kN
Variable concentrated load Q_{kc1} = **14.80** kN

Eccentricity of concentrated load	e_{c1} = **0** mm
Length of concentrated load	L_{c1} = **100** mm
Width of concentrated load	w_{c1} = **100** mm
Height of concentrated load	h_{c1} = **2400** mm
Distance of load to right vertical edge	r_{11} = **675** mm
Distance of load to nearest vertical edge	a_{11} = **675** mm

Walls subjected to concentrated loads - Section 6.1.3

Eccentricity check	$e_{c1} <= t / 4$
	PASS - Eccentricity of load is less than t/4
Area of bearing	$A_{b1} = L_{c1} * w_{c1}$ = **10000** mm^2
Effective length of bearing at mid-height	$l_{efm1} = L_{c1} + h_{c1} / 2 * \tan(30) + r_{11}$ = **1468** mm
Effective bearing area	$A_{ef1} = l_{efm1} * t$ = **146782** mm^2
Bearing area ratio check	$A_{ratio1} = Min(A_{b1} / A_{ef1}, 0.45)$ = **0.07**
Initial enhancement factor	$\beta_{init1} = Max((1 + 0.3 * a_{11} / h_{c1}) * (1.5 - 1.1 * A_{ratio1}), 1.0)$ = **1.55**
Maximum enhancement factor	$\beta_{max1} = Min(1.25 + a_{11} / (2 * h_{c1}), 1.5)$ = **1.39**
Enhancement factor for concentrated loads	$\beta_1 = Min(\beta_{init1}, \beta_{max1})$ = **1.39**
Design value of the concentrated load	$N_{Edc1} = G_{kc1} * \gamma_{fG} + Q_{kc1} * \gamma_{fQ}$ = **43.40** kN
Design value concentrated load resistance	$N_{Rdc1} = \beta_1 * A_{b1} * f_d$ = **26.55** kN
	Applied concentrated load exceeds design resistance, spreader required!

Design of spreader beam

Type of spreader	**Concrete padstone**
Type of bearing onto spreader	**Point load**
Location of load from RHS of spreader	P_{11} = **220** mm
Length of spreader	L_{sp1} = **440** mm
Height of spreader	h_{sp1} = **215** mm
Width of spreader	w_{sp1} = **100** mm
Eccentricity of load on spreader	e_{sp1} = **0** mm
Modulus of elasticity	E_{sp1} = **29962** N/mm^2
Second moment of area	$I_{sp1} = 1/12 * w_{sp1} * h_{sp1}^3$ = **82819792** mm^4
Modulus of the wall	$k_0 = E_w / h$ = **2.39** N/mm^2/mm
Winkler's constant	$K_{c1} = k_0 * w_{sp1}$ = **238.62** N/mm/mm
Characteristic of the system	$\alpha_1 = (K_{c1} / (4 * E_{sp1} * I_{sp1}))^{1/4}$ = **0.00221** mm^{-1}
Classification of spreader	$\alpha L_1 = \alpha_1 * L_{sp1}$ = **0.97 Medium**
Krilov's functions for the spreader length	$B_{\alpha l1} = 1/2 * (\cosh(\alpha L_1) * \sin(180 * \alpha L_1 / \pi) + \sinh(\alpha L_1) * \cos(180 * \alpha L_1 / \pi))$ = **0.95**
	$C_{\alpha l1} = 1/2 * \sinh(\alpha L_1) * \sin(180 * \alpha L_1 / \pi)$ = **0.47**
	$D_{\alpha l1} = 1/4 * (\cosh(\alpha L_1) * \sin(180 * \alpha L_1 / \pi) - \sinh(\alpha L_1) * \cos(180 * \alpha L_1 / \pi))$ = **0.15**
Krilov's functions at the point load	$A_{\alpha P11} = \cosh(\alpha_1 * P_{11}) * \cos(180 * \alpha_1 * P_{11} / \pi)$ = **0.99**
	$B_{\alpha P11} = 1/2 * (\cosh(\alpha_1 * P_{11}) * \sin(180 * \alpha_1 * P_{11} / \pi) + \sinh(\alpha_1 * P_{11}) * \cos(180 * \alpha_1 * P_{11} / \pi))$ = **0.49**
Using method of initial conditions	
Initial moment of LH edge	M_{01} = **0** kNm
Initial shear of LH edge	V_{01} = **0** kN
Which gives	$(4 * \alpha_1^2 * C_{\alpha l1} * \delta_{01} + 4 * \alpha_1 * D_{\alpha l1} * \Phi_{01}) * E_{sp1} * I_{sp1} - B_{\alpha P11} / \alpha_1 * N_{Edc1}$ = **0.00** kNm

and	$(4 * \alpha_1^3 * B_{\alpha l1} * \delta_{01} + 4 * \alpha_1^2 * C_{\alpha l1} * \Phi_{01}) * E_{sp1} * I_{sp1} - A_{\alpha P11} * N_{Edc1}$ = **0.00** kN
Therefore,	
Initial deflection of LH edge	δ_{01} = **0.40638** mm
Initial rotationof LH edge	Φ_{01} = **0.000070**
Location of maximum deflection	x_{def1} = **220** mm
Krilov's functions at the spreader length	$A_{\alpha xdef1} = \cosh(\alpha_1 * x_{def1}) * \cos(180 * \alpha_1 * x_{def1} / \pi)$ = **0.99**
	$B_{\alpha xdef1} = 1/2 * (\cosh(\alpha_1 * x_{def1}) * \sin(180 * \alpha_1 * x_{def1} / \pi) + \sinh(\alpha_1 * x_{def1}) * \cos(180 * \alpha_1 * x_{def1} / \pi))$ = **0.49**
Distance of point load right of loaction	p_{1def1} = **0** mm
Krilov's functions at the spreader length	$D_{\alpha p1def1} = 1/4 * (\cosh(\alpha_1 * p_{1def1}) * \sin(180 * \alpha_1 * p_{1def1} / \pi) - \sinh(\alpha_1 * p_{1def1}) * \cos(180 * \alpha_1 * p_{1def1} / \pi))$ = **0.00**
Particular integral due to load	$\delta'_1 = D_{\alpha p1def1} / \alpha_1^3 * N_{Edc1} / (I_{sp1} * E_{sp1})$ = **0.000** mm
Maximum deflection	$\delta_{max1} = A_{\alpha xdef1} * \delta_{01} + B_{\alpha xdef1} * \Phi_{01} / \alpha_1 + \delta'_1$ = **0.418** mm
Location of maximum moment	x_{M1} = **220** mm
Krilov's functions at the spreader length	$C_{\alpha xM1} = 1/2 * \sinh(\alpha_1 * x_{M1}) * \sin(180 * \alpha_1 * x_{M1} / \pi)$ = **0.12**
	$D_{\alpha xM1} = 1/4 * (\cosh(\alpha_1 * x_{M1}) * \sin(180 * \alpha_1 * x_{M1} / \pi) - \sinh(\alpha_1 * x_{M1}) * \cos(180 * \alpha_1 * x_{M1} / \pi))$ = **0.02**
Distance of point load right of loaction	p_{1M1} = **0** mm
Krilov's functions at the spreader length	$B_{\alpha p1M1} = 1/2 * (\cosh(\alpha_1 * p_{1M1}) * \sin(180 * \alpha_1 * p_{1M1} / \pi) + \sinh(\alpha_1 * p_{1M1}) * \cos(180 * \alpha_1 * p_{1M1} / \pi))$ = **0.00**
Particular integral due to load	$M'_1 = -B_{\alpha p1M1} / \alpha_1 * N_{Edc1}$ = **0.00** kNm
Maximum moment	$M_{Edsp1} = (4 * \alpha_1^2 * C_{\alpha xM1} * \delta_{01} + 4 * \alpha_1 * D_{\alpha xM1} * \Phi_{01}) * (I_{sp1} * E_{sp1}) + M'_1$ = **2.37** kNm
Location of maximum shear	x_{V1} = **220** mm
Krilov's functions at the spreader length	$B_{\alpha xV1} = 1/2 * (\cosh(\alpha_1 * x_{V1}) * \sin(180 * \alpha_1 * x_{V1} / \pi) + \sinh(\alpha_1 * x_{V1}) * \cos(180 * \alpha_1 * x_{V1} / \pi))$ = **0.49**
	$C_{\alpha xV1} = 1/2 * \sinh(\alpha_1 * x_{V1}) * \sin(180 * \alpha_1 * x_{V1} / \pi)$ = **0.12**
Distance of point load right of loaction	p_{1V1} = **0** mm
Krilov's functions at the spreader length	$A_{\alpha p1V1} = \cosh(\alpha_1 * p_{1V1}) * \cos(180 * \alpha_1 * p_{1V1} / \pi)$ = **1.00**
Particular integral due to load	$V'_1 = -A_{\alpha p1V1} * N_{Edc1}$ = **-43.39** kN
Shear at concentrated point load	$V_1 = (4 * \alpha_1^3 * B_{\alpha xV1} * \delta_{01} + 4 * \alpha_1^2 * C_{\alpha xV1} * \Phi_{01}) * (I_{sp1} * E_{sp1}) + V'_1$ = **-21.70** kN
Maximum shear	$V_{Edsp1} = Max(Abs(V_1), N_{Edc1} - Abs(V_1))$ = **21.70** kN
Maximum allowable stress under spreader	$\sigma_{Rdsp1} = 1.5 * f_d$ = **2.86** N/mm^2
Maximum reaction	$N_{Edsp1} = K_{c1} * \delta_{max1}$ = **99.73** kN/m
Design stress	$\sigma_{Edsp1} = N_{Edsp1} / w_{sp1}$ = **1.00** N/mm^2

PASS - Design stress under spreader is less than the allowable bearing stress

Walls subjected to mainly vertical loading - Section6.1.2

Eccentricity of permanent UDL at mid-height below concentrated load

$e_{gmu1} = e_{gu} * h_{c1} / (2 * h)$ = **0.0** mm

Eccentricity of variable UDL at mid-height below concentrated load

	$e_{qmu1} = e_{qu} * h_{c1} / (2 * h) =$ **0.0** mm
Eccentricity of concentrated load at mid-height	$e_{mc1} = e_{c1} / 2 =$ **0.0** mm
Initial eccentricity - cl.5.5.1.1(4)	$e_{init} = h / 450 =$ **5.3** mm
Concentrated load at mid-height as UDL	$N_{mc1} = N_{Edc1} / l_{efm1} =$ **29.56** kN/m
Vertical load at mid-height	$N_{Ed1} = (g_k + \gamma * t * (h - h_{c1} / 2)) * \gamma_{fG} + q_k * \gamma_{fQ} + N_{mc1} =$ **32.48** kN/m
Design moment at mid-height	$M_{Ed1} = g_k * \gamma_{fG} * e_{gmu1} + q_k * \gamma_{fQ} * e_{qmu1} + N_{mc1} * e_{mc1} =$ **0.00** kNm/m
Eccentricities due to loads - eq. 6.7	$e_{m1} = Abs(M_{Ed1}) / N_{Ed1} + e_{init} =$ **5.3** mm
Slenderness ratio limit for creep eccentricity	$\lambda_c =$ **27**
Eccentricity due to creep	$e_{k1} =$ **0.0** mm
Eccentricity at mid-height - eq. 6.6	$e_{mk1} = Max(e_{m1} + e_{k1}, 0.05 \times t) =$ **5.3** mm
From eq. G2	$A_{11} = 1 - 2 * e_{mk1} / t =$ **0.89**
From eq. G3	$u_1 = (h_{ef} / t_{ef} * (1 / K_E)^{1/2} - 0.063) / (0.73 - 1.17 * e_{mk1} / t) =$ **1.04**
Capacity reduction factor - eq. G1	$\Phi_{m1} = A_{11} * \exp(-(u_1^2) / 2) =$ **0.52**
Design vertical resistance of panel - eq.6.2	$N_{Rd1} = \Phi_{m1} * t * f_d =$ **99.05** kN/m

PASS - Design value of vertical resistance exceeds applied vertical load

LINTEL DESIGN

New Window To Meeting Room

Opening Width	=	2.25	m
Effective Length	=	2.475	m
Loading Area	=	1.53	m^2

Additional Load within Loading/Interaction Zone:

First Floor	=	0	m
Balcony	=	0	m
Distance to Loading Zone,A	=	0.250	m
Length of Loading Zone,B	=	1.975	m
Length of Interaction Zone,D	=	0.106	m

See Figure A for Zonal Diagram

Lintel Supplier	=	IG Lintel	
Lintel Type	=	L1/S 75	
Length	=	2.55	m
Capacity	=	27	kN

Masonry Outer Leaf Load	=	2.2	kN/m^2
Masonry Inner Leaf Load	=	2.2	kN/m^2
Load acting in loading/interaction zone:			
Roof Load	=	1.91	kN/m^2
	=	0	kN/m^2
UDL in loading/interaction zone	=	0.00	kN/m
Masonry Outer Leaf Load	=	3.37	kN
Masonry Inner Leaf Load	=	3.37	kN
Additional Load from Loading Zone	=	0.00	kN
Additional Load from Interaction Zone	=	0.00	kN

Total UDL	=	6.74	kN	OK
Ratio	=	1.00	:1	OK

New Door To Meeting Room

Opening Width	=	1	m		Masonry Outer Leaf Load	=	2.2	kN/m²	
Effective Length	=	1.100	m		Masonry Inner Leaf Load	=	2.2	kN/m²	
Loading Area	=	0.30	m²		Load acting in loading/interaction zone:				
Additional Load within Loading/Interaction Zone:					Roof Load	=	1.91	kN/m²	
						=	0	kN/m²	
Roof Load	=	1.87	m		UDL in loading/interaction zone	=	3.57	kN/m	
	=		m						
Distance to Loading Zone,A	=	0.100	m		Masonry Outer Leaf Load	=	0.67	kN	
Length of Loading Zone,B	=	0.900	m		Masonry Inner Leaf Load	=	0.67	kN	
Length of Interaction Zone,D	=	0.042	m		Additional Load from Loading Zone	=	3.21	kN	
Lintel Supplier	=	IG			Additional Load from Interaction Zone	=	0.12	kN	
Lintel Type	=	L1/HD 75			**Total UDL**	=	4.67	kN	OK
Length = 1.35 m	Capacity =	22	kN		**Ratio**	=	6.02	:1	OK

See Figure A for Zonal Diagram

Opening To WC & Drinks Servery

Opening Width	=	3	m		Masonry Outer Leaf Load	=	2.2	kN/m²	
Effective Length	=	3.300	m		Masonry Inner Leaf Load	=	2.2	kN/m²	
Loading Area	=	2.72	m²		Load acting in loading/interaction zone:				
Additional Load within Loading/Interaction Zone:					roof Load	=	1.91	kN/m²	
						=		kN/m²	
Roof	=	1.8	m		UDL in loading/interaction zone	=	3.44	kN/m	
	=		m						
Distance to Loading Zone,A	=	0.100	m		Masonry Outer Leaf Load	=	5.99	kN	
Length of Loading Zone,B	=	3.100	m		Masonry Inner Leaf Load	=	5.99	kN	
Length of Interaction Zone,D	=	0.042	m		Additional Load from Loading Zone	=	10.66	kN	
Lintel Supplier	=	IG			Additional Load from Interaction Zone	=	0.12	kN	
Lintel Type	=	L1/HD 75			**Total UDL**	=	22.76	kN	OK
Length = 3.30 m	Capacity =	32	kN		**Ratio**	=	2.80	:1	OK

See Figure A for Zonal Diagram

New opening To Disabled WC

Opening Width	=	1	m		Masonry Outer Leaf Load	=	2.2	kN/m²	
Effective Length	=	1.100	m		Masonry Inner Leaf Load	=	2.2	kN/m²	
Loading Area	=	0.30	m²		Load acting in loading/interaction zone:				
Additional Load within Loading/Interaction Zone:					Roof Load	=	1.91	kN/m²	
						=		kN/m²	
First Floor	=	1.8	m		UDL in loading/interaction zone	=	3.44	kN/m	
Balcony	=		m						
Distance to Loading Zone,A	=	0.100	m		Masonry Outer Leaf Load	=	0.67	kN	
Length of Loading Zone,B	=	0.900	m		Masonry Inner Leaf Load	=	0.67	kN	
Length of Interaction Zone,D	=	0.042	m		Additional Load from Loading Zone	=	3.09	kN	
Lintel Supplier	=	IG Lintel			Additional Load from Interaction Zone	=	0.12	kN	
Lintel Type	=	L1/HD 75			**Total UDL**	=	4.55	kN	OK
Length = 1.35 m	Capacity =	22	kN		**Ratio**	=	5.83	:1	OK

See Figure A for Zonal Diagram

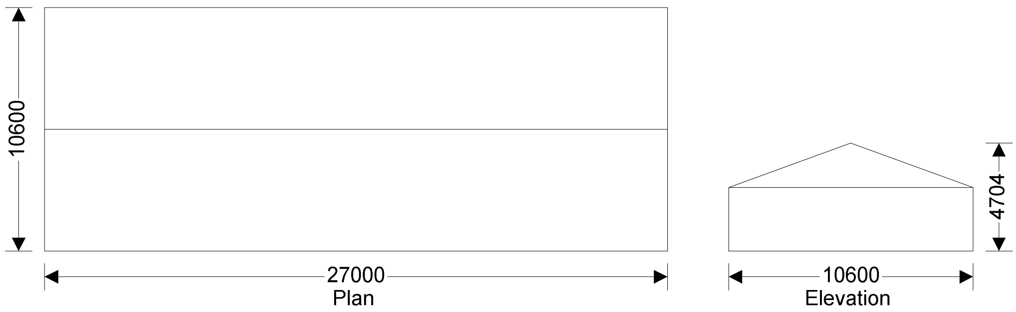
LATERAL MASONRY ASSESMENT

The proposal to remove the buttress wall between the two changing could potentially compromise the ability of the remaining masonry to laterally support wind loads applied to it. A new wall between the drinks/servery and the meeting room will act as a new buttress assuming it is sufficient tied. The following assessment assumes this to be case.

WIND LOADING

In accordance with EN1991-1-4:2005+A1:2010 and the UK national annex

Tedds calculation version 3.0.23



Building data

Type of roof;	Duopitch
Length of building;	L = **27000** mm
Width of building;	W = **10600** mm
Height to eaves;	H = **2775** mm
Pitch of roof;	α_0 = **20.0** deg
Total height;	h = **4704** mm

Basic values

Location;	Oxford
Wind speed velocity (FigureNA.1);	$v_{b,map}$ = **21.3** m/s
Distance to shore;	L_{shore} = **108.00** km
Altitude above sea level;	A_{alt} = **64.0**m
Altitude factor;	$c_{alt} = A_{alt}/1m * 0.001 + 1$ = **1.064**
Fundamental basic wind velocity;	$v_{b,0} = v_{b,map} * c_{alt}$ = **22.7** m/s
Direction factor;	c_{dir} = **1.00**
Season factor;	c_{season} = **1.00**
Shape parameter K;	K = **0.2**
Exponent n;	n = **0.5**
Air density;	ρ = **1.226** kg/m^3
Probability factor;	$c_{prob} = [(1 - K * \ln(-\ln(1-p)))/(1 - K * \ln(-\ln(0.98)))]^n$ = **1.00**
Basic wind velocity (Exp. 4.1);	$v_b = c_{dir} * c_{season} * v_{b,0} * c_{prob}$ = **22.7** m/s
Reference mean velocity pressure;	$q_b = 0.5 * \rho * v_b^2$ = **0.315** kN/m^2

Orography

Orography factor not significant;	c_o = 1.0
Terrain category;	Country
Displacement height (sheltering effect excluded);	h_{dis} = 0mm

The velocity pressure for the windward face of the building with a 0 degree wind is to be considered as 1 part as the height h is less than b (cl.7.2.2)

The velocity pressure for the windward face of the building with a 90 degree wind is to be considered as 1 part as the height h is less than b (cl.7.2.2)

Peak velocity pressure - windward wall - Wind 0 deg and roof

Reference height (at which q is sought);	z = **2775**mm
Displacement height (sheltering effects excluded);	h_{dis} = **0** mm
Exposure factor (Figure NA.7);	c_e = **1.58**

Peak velocity pressure; $q_p = c_e * q_b =$ **0.50** kN/m^2

Structural factor

Structural damping; $\delta_s =$ **0.100**

Height of element; $h_{part} =$ **2775** mm

Size factor (Table NA.3); $c_s =$ **0.890**

Dynamic factor (Figure NA.9); $c_d =$ **1.000**

Structural factor; $c_sc_d = c_s \times c_d =$ **0.890**

Peak velocity pressure - windward wall - Wind 90 deg and roof

Reference height (at which q is sought); z = **4704**mm

Displacement height (sheltering effects excluded); $h_{dis} =$ **0** mm

Exposure factor (Figure NA.7); $c_e =$ **1.87**

Peak velocity pressure; $q_p = c_e * q_b =$ **0.59** kN/m^2

Structural factor

Structural damping; $\delta_s =$ **0.100**

Height of element; $h_{part} =$ **4704** mm

Size factor (Table NA.3); $c_s =$ **0.924**

Dynamic factor (Figure NA.9); $c_d =$ **1.012**

Structural factor; $c_sc_d = c_s \times c_d =$ **0.935**

Structural factor - roof 0 deg

Structural damping; $\delta_s =$ **0.100**

Height of element; $h_{part} =$ **4704** mm

Size factor (Table NA.3); $c_s =$ **0.888**

Dynamic factor (Figure NA.9); $c_d =$ **1.000**

Structural factor; $c_sc_d = c_s \times c_d =$ **0.888**

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.); $q_{p,i} =$ **0.59** kN/m^2

Pressures and forces

Net pressure; $p = c_sc_d * q_p * c_{pe} - q_{p,i} * c_{pi}$;

Net force; $F_w = p_w * A_{ref}$;

Roof load case 1 - Wind 0, c_{pi} 0.20, -c_{pe}

Zone	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (kN/m^2)	Net pressure p (kN/m^2)	Area A_{ref} (m^2)	Net force F_w (kN)
F (-ve)	-0.90	0.59	-0.59	4.71	-2.77
G (-ve)	-0.70	0.59	-0.48	22.32	-10.81
H (-ve)	-0.33	0.59	-0.29	125.25	-36.60
I (-ve)	-0.50	0.59	-0.38	125.25	-47.53
J (-ve)	-1.17	0.59	-0.73	27.03	-19.69

Total vertical net force; $F_{w,v} =$ **-110.32** kN

Total horizontal net force; $F_{w,h} =$ **5.83** kN

Walls load case 1 - Wind 0, c_{pi} 0.20, -c_{pe}

Zone	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (kN/m^2)	Net pressure p (kN/m^2)	Area A_{ref} (m^2)	Net force F_w (kN)
A	-1.20	0.59	-0.75	5.87	-4.38
B	-0.80	0.59	-0.54	30.21	-16.23
C	-0.50	0.59	-0.38	3.57	-1.36
D	0.73	0.50	0.20	74.93	15.23
E	-0.35	0.50	-0.27	74.93	-20.48

Overall loading

Equiv leeward net force for overall section; $F_l = F_{w,wE}$ = **-20.5** kN

Net windward force for overall section; $F_w = F_{w,wD}$ = **15.2** kN

Lack of correlation (cl.7.2.2(3) – Note); f_{corr} = **0.85**; as h/W is 0.444

Overall loading overall section; $F_{w,D} = f_{corr} * (F_w - F_l + F_{w,h})$ = **35.3** kN

Roof load case 2 - Wind 90, c_{pi} -0.30, + c_{pe}

Zone	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (kN/m^2)	Net pressure p (kN/m^2)	Area A_{ref} (m^2)	Net force F_w (kN)
F (+ve)	0.30	0.59	0.34	4.71	1.61
G (+ve)	0.27	0.59	0.32	5.31	1.72
H (+ve)	0.23	0.59	0.31	42.45	12.96
I (+ve)	0.20	0.59	0.29	251.50	72.16

Total vertical net force; $F_{w,v}$ = **83.12** kN

Total horizontal net force; $F_{w,h}$ = **0.00** kN

Walls load case 2 - Wind 90, c_{pi} -0.30, + c_{pe}

Zone	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (kN/m^2)	Net pressure p (kN/m^2)	Area A_{ref} (m^2)	Net force F_w (kN)
A	-1.20	0.50	-0.38	5.22	-1.99
B	-0.80	0.50	-0.20	20.89	-4.07
C	-0.50	0.50	-0.06	48.82	-2.72
D	0.70	0.59	0.56	39.64	22.29
E	-0.30	0.59	0.01	39.64	0.45

Overall loading

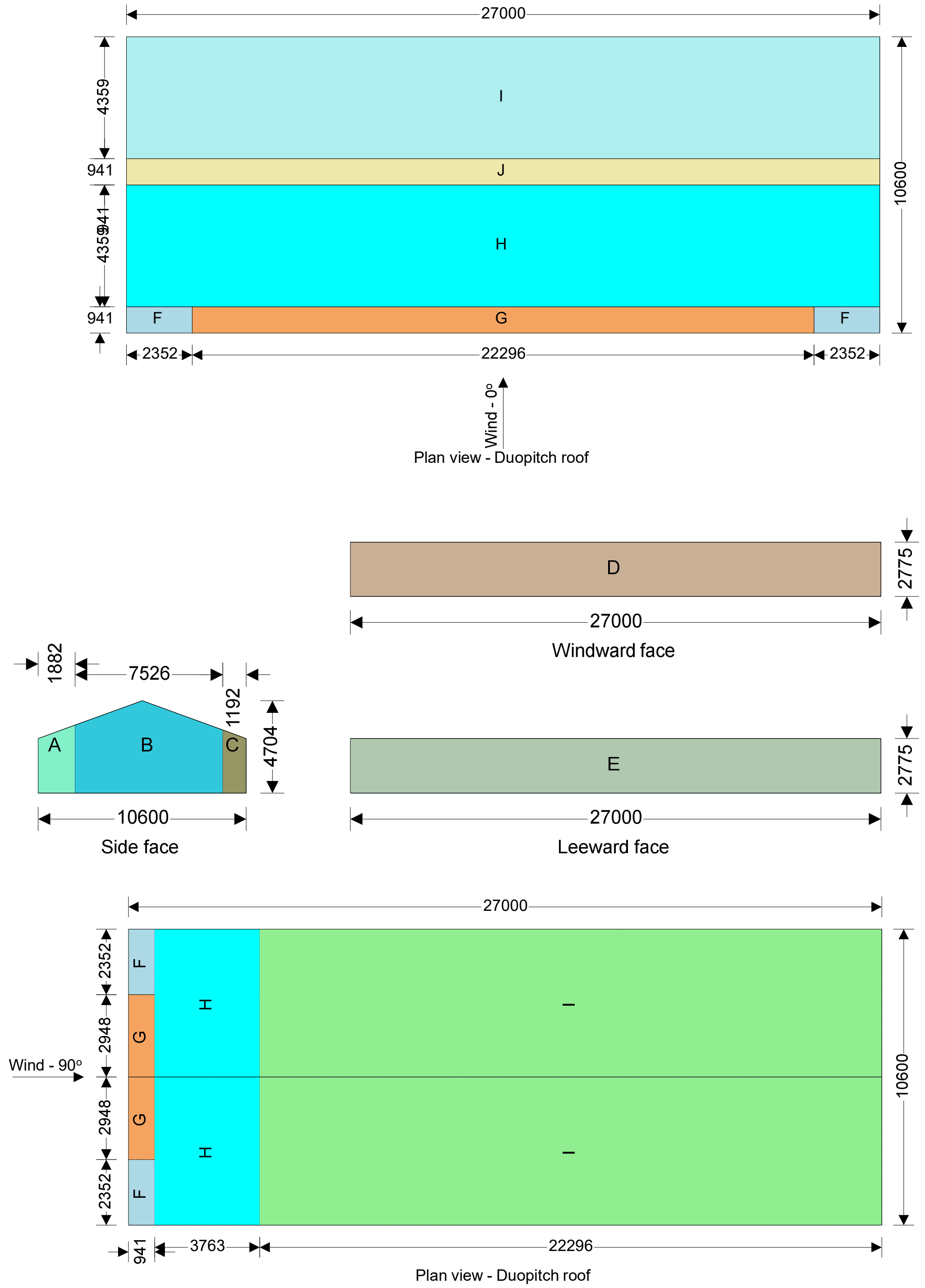
Equiv leeward net force for overall section; $F_l = F_{w,wE}$ = **0.5** kN

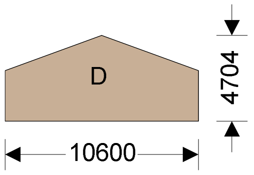
Net windward force for overall section; $F_w = F_{w,wD}$ = **22.3** kN

Lack of correlation (cl.7.2.2(3) – Note); f_{corr} = **0.85**; as h/L is 0.174

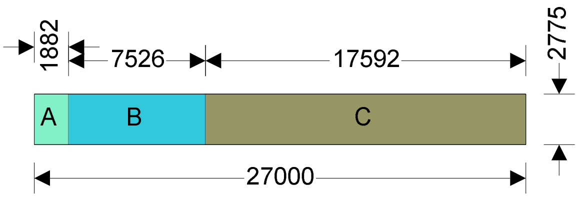
Overall loading overall section; $F_{w,D} = f_{corr} * (F_w - F_l + F_{w,h})$ = **18.6** kN

SWJ Consulting Albion House, Compton Way Witney, Oxon, OX28 3AB T:01993 225 085 E: mail@swjconsulting.co.uk	Project Crowmarsh Gifford Pavillion				Job Ref. 01410	
	Section Structural Calculation Package				Sheet no./rev. 25	
	Calc. by SAJ	Date 25/02/2021	Chk'd by SAJ	Date 25/02/2021	App'd by SAJ	Date 25/02/2021

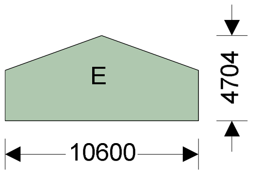




Windward face



Side face



Leeward face

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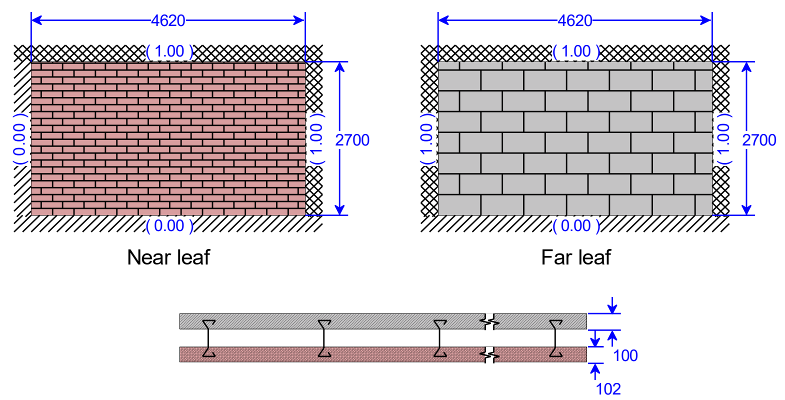
SWJ Consulting Albion House, Compton Way Witney, Oxon, OX28 3AB T:01993 225 085 E: mail@swjconsulting.co.uk	Project Crowmarsh Gifford Pavillion			Job Ref. 01410	
	Section Structural Calculation Package			Sheet no./rev. 27	
	Calc. by SAJ	Date 25/02/2021	Chk'd by SAJ	Date 25/02/2021	App'd by SAJ	Date 25/02/2021



Near leaf

Far leaf

Sectional plan

Data item	*Near*	*Far*
Bed joint reinforcement	No	No
DPC - top	No	No
DPC - bottom	No	No

Note:
1.For details of masonry units, bed joint reinforcement and damp proof course refer to 'Material and design data'
2.For details of wall ties (if specified) refer to 'Concise' or 'Detailed' report.
3.Figures in brackets eg: (1.00) shown in elevation indicate user defined proportional fixity/continuity factor applied before allowing for openings.

Masonry characteristic strengths

Description	*Near*	*Far*	*Units*	*Description*	*Near*	*Far*	*Units*
Design code	EN 1996-1-1:2005			Shear			
Compression				Shear without compression	0.20	0.15	N/mm²
Factor for collar joint	1.00	1.00	-	Shear friction coefficient	0.40	0.40	-
Compression on bed joints	5.68	5.73	N/mm²	Limiting shear	1.15	0.65	N/mm²
Compression // bed joints	5.68	5.73	N/mm²	Vertical shear- bonded			N/mm²
Flexure				Elastic modulus			
Horizontal span	0.90	0.60	N/mm²	Short term	5.68	5.73	kN/mm²
Vertical span	0.30	0.25	N/mm²	Long term	2.27	2.29	kN/mm²

Characteristic vertical loads

Load category name	Near		Far	
	Load (kN/m)	Ecc. mm	Load (kN/m)	Ecc. mm

Characteristic lateral wind pressure

Category name	Dyn. pr. kN/m²	Coeff. Near	Coeff. Far	Net coeff.	Res. pr. kN/m²
Wind far	0.590	-1.000	0.200	-1.200	-0.708
Wind near	0.590	0.800	-0.300	1.100	0.649

Characteristic lateral line load

Category name	Load kN/m	Height from bottom mm

Note: For details of more than two loads please refer detailed report

Summary results (critical load combinations)

Description	*Wall*	*Status*	*Units*	*Description*	*Near*	*Far*	*Status*	*Units*
Lateral load capacity	1.136		kN/m²	Max. slenderness	27	27		
Design uniform load	-1.062		kN/m²	Actual	18.564	18.564		
Utilisation	0.934	Pass		Utilisation	0.688	0.688	Pass	
Load combination	0.90D+1.50Wf							
Limiting dimension / area								
Allowable	75		mm					
Actual	100		mm					
Utilisation	0.750	Pass						

Therefore panel is acceptable when new buttress wall is effectively tied. Use ancnon wall starter system to join new wall to inner leaf of exernal cavity wall.