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CONCRETE BLOCK ASSOCIATION

Unreinforced Masonry Diaphragm Walls

Data Sheet 10
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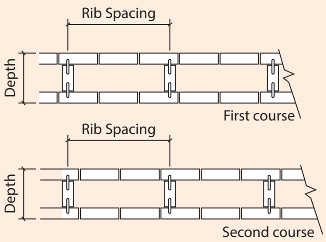
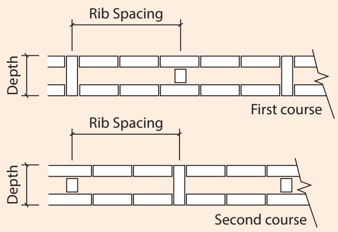
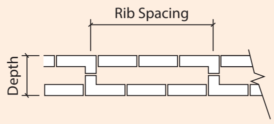


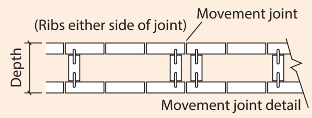
Tied Diaphragm Wall Fig 1A



Bonded Diaphragm Wall Fig 1B



Quoin Bonded Diaphragm Wall Fig 1C



Movement Joint Detail Fig 1D

Introduction

A blockwork diaphragm wall is a wide cavity wall with two leaves of concrete blockwork bonded together with blockwork cross ribs. These ribs are bonded by steel ties or block bonding to allow the wall cross section to act integrally as a series of box sections producing a high section modulus and radius of gyration.

Various forms of wall are available depending upon the bonding pattern of the cross ribs. Some typical forms include tied, block bonded and quoin bonded as indicated within Figure 1.

The high section modulus produces a relatively large bending resistance for concrete blockwork walls whilst the high radius of gyration allows the walls to carry significantly enhanced vertical loading in relation to that of a traditionally constructed blockwork cavity wall.

These enhancements have made concrete blockwork diaphragm walls very economical for tall single storey structures or free standing walls and retaining walls where the high bending resistance is utilised to resist lateral loading and the large radius of gyration permits tall unrestrained vertical support to roof structures.

Diaphragm walls offer the benefit of utilising the architectural external enclosure as the structural element eliminating the need for a steel or concrete framework. They are ideally suited to single storey structures such as sports halls, swimming pools, theatres, cinemas and the like, which require large unobstructed internal areas.

In mass masonry stems to cantilevered gravity retaining structures solid sections are traditionally utilised to provide adequate flexural strength. The use of diaphragm walls in these situations minimised the use of materials by intelligent use of structural geometry. Where additional mass is required for global stability it is possible to fill the pockets of the diaphragm wall with a suitable inert filler.

Benefits include:

- Elimination of specialist off-site fabrication.
- Minimal lead time required as off-site prefabrication is not required.
- Excellent dimensional tolerance due to on-site construction.
- Variety of quality surface finishes can easily be achieved.
- External and internal surfaces may be flush.
- Excellent fire resistance.
- Excellent durability.
- No expensive formwork or cranage required.

This data sheet provides basic information on the structural design of concrete blockwork diaphragm walls.

Movement joint detailing

When positioning movement joints, ensure that the joint is suitably restrained at either side to provide adequate stability at the joint position. This can be achieved with wall ties or by providing additional ribs to stabilise the joint. (See detail above).

Advice should be sought from a Structural Engineer for the positioning of movement joints in diaphragm walls.

Concrete Block Association, The Old Rectory
8 Main Street, Glenfield LE3 8DG
Tel: 0116 232 5165
www.cba-blocks.org.uk

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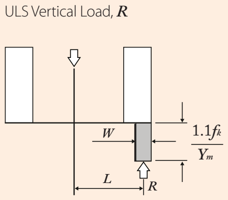
Design applications

Equation 1

$$M_r = \left(\frac{f_{kx}}{\gamma_m} + g_d \right) Z$$

Fig 2



Equation 2

$$M_r = \frac{R}{2} \left(D - \frac{R\gamma_m}{1.1 f_k} \right)$$

Equation 3

$$V = \frac{V}{D t_w}$$

Equation 4

$$K_v = \frac{t_w v s \gamma_m}{f_y}$$

Size of Tie	Kv
35 x 5	14.6
30 x 5	12.5
25 x 5	10.4
20 x 5	8.3
25 x 4	8.3
20 x 4	6.6
15 x 4	5
20 x 3	5
15 x 3	3.8
10 x 3	2.5
10 x 2	1.6

Table 1

Design method

The design of concrete blockwork diaphragm walls is governed by BS 5628 : Pt 1 : 2005 'Use of Masonry' which adopts an ultimate limit state design method. Serviceability Limit State is unlikely to be a governing design condition in blockwork diaphragm walls due to their high second moment of area that limits deflection. Generally, flexural cracking is likely to occur before significant wall deflection. Free standing diaphragm walls are generally designed as simple vertical cantilevers under lateral load.

External diaphragm walls to a single storey building are designed as propped cantilevers under lateral load. The bending moment assumed at the base, however, is the lesser of the fixed end elastic moment of a propped cantilever or the cracked section moment of resistance in accordance with Equation 2. BS5628-1:2005 clause 32.9 offers direct guidance of these walls.

The prop to the diaphragm wall is usually provided by the roof structure acting as horizontal bracing which transfers the load to the adjacent side shear walls.

A diaphragm wall should be designed to resist vertical and horizontal bending induced by lateral load, compression induced by axial load and shear. If steel ties are utilised to form the bond between the ribs and the leaves these are required to resist the shear force at the rib to leaf interface.

The flexural capacity of a propped diaphragm wall in a single storey building calculated from equation 1 should be greater than the actual bending moment with partial factors of safety of 0.90 and 1.4 on the dead load and wind load respectively at all locations except the base of the wall where the cracked section capacity must be used. Additionally the cracked moment capacity of the section must be greater than the applied bending moment at all locations under the application of a partial factor of safety of 1 on both dead and wind load.

Design for vertical bending

The bending resistance of a diaphragm wall is calculated by two methods depending on the condition.

- Based on the flexural resistance of the masonry
- Based on gravity stability utilising a cracked section

Flexural resistance

At sections other than the base of the wall the flexural resistance of the wall may be calculated using equations 1 which utilises the flexural strength of the masonry providing no dpc is present at the location being considered.

Cracked section

It may be assumed that the wall cracks at the tensile face with the bending resistance of the wall being calculated from the wall's intrinsic stability moment. Providing the vertical load at the base of the wall may be assumed to act along the wall centre line the cracked moment of resistance may be calculated from equation 2. The point of rotation is generally taken as the distance between the centroid of the vertical load and the centre of a rectangular stress block as indicated above . BS5628-1:2005 clause 32.5.3 provides additional guidance on the cracked section flexural resistance of masonry. The stress block utilised is presented in Appendix B B.2 to the standard.

Design for shear

The shear stresses within a diaphragm wall may be calculated on a similar basis to that of an 'I' section steel beam apportioning the shear force between the ribs.

The shear stress calculated using this formula should be less than 0.35 N/mm^2 divided by the appropriate partial factor of safety (γ_{mv})for M12 and M6 mortar strength designations. No allowance should be made for vertical compression as this is not present when considering the complimentary vertical shear stresses within the ribs. Alternatively, an elastic shear stress distribution may be adopted resulting in a more rigorous analysis.

Wall tie design

Where the ribs are not fully bonded to the internal and external flanges of the diaphragm wall, steel ties have to be used to transfer the shear force across the connection.

The mechanism for the failure of the wall ties is generally by the formation of two plastic hinges along the length of the tie, one at the tie 's embedment into the flange and the other at the embedment into the web. The distance between these two hinges is taken as 6 times the thickness of the tie.

The full plastic section modulus of the tie may be utilised to calculate the magnitude of the plastic moment. BS5628-1:2005 clause 32.9.4.1 provides guidance on the design of ties in this condition, however, the most convenient method of expressing the adequacy of the ties is by calculating the required tie coefficient developed from this assumption and given by the equation 4 opposite. Ties should not be spaced greater than 450mm vertically.

This should be less than the actual tie coefficient as given within Table 1 above.

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$$M = \frac{wB^2}{10}$$

Equation 5

$$DVLR = \frac{\beta f_k A}{\gamma_m}$$

Equation 6

Design for horizontal bending

The flanges of the diaphragm wall have to span horizontally between the internal ribs under the action of the lateral pressure.

Since the flange is continuous over the ribs the applied bending moment may be taken as:

This should be less than the flexural moment of resistance of the wall (failure perpendicular to the bed joints) as given in clause 32.4.3 in BS 5628 : Pt 1 : 2005 and given here as equation 1. Within equation 1 g_d should be taken as zero in this condition.

To allow full utilisation of the flange in using the box section analysis of the diaphragm wall the centres of the ribs should generally not exceed 12 times the thickness of the flange plus the thickness of the web in accordance with the clause 32.4.3 of BS 5628 : Pt 1 : 2005.

Design for vertical load

The vertical load resistance of a diaphragm wall is calculated in accordance with clause 28.2.2 of BS 5628 : Pt 1 : 2005 as represented by equation 6 above.

The capacity reduction factor may be taken from table 2 taking effective thickness of the diaphragm as the actual thickness.

Alternatively, the designer may wish to use table 7 of BS 5628 : Pt 1 : 2005 with the effective thickness being based on the equivalent thickness of a section with the same radius of gyration as the diaphragm wall under consideration.

The vertical load resistance of a diaphragm wall is seldom critical in the standard design case of a single storey building.

Detail consideration

Durability

Environmental conditions will need to be considered when selecting mortar and block type.

Detailed provisions on durability are not given here.

Reference should be made to BS 5628 : Pt 3.

Movement

Movement within the wall may occur due to temperature or moisture variation or horizontal movement within the roof structure. Provision for such movement should be via vertical movement joints at centres indicated in BS 5628 : Pt 3 or consult the block manufacturer.

Construction

Diaphragm walls to buildings gain their stability from the restraint provided by the roof structure at their head. During construction such restraint is not available; provision should therefore be made to provide temporary lateral restraint during construction.

Means of providing stability to the diaphragm wall should be noted to the contractor so that any requirements for temporary bracing may be considered.

Insulation

Diaphragm walls have large cavities, which can incorporate insulation as required. Alternatively , walls can be insulated either internally or externally.

The U value for the overall wall construction should be calculated using the proportional area method given in BS EN ISO 6946.

SR h/D	ECCENTRICITY AT TOP OF DIAPHRAGM WALL, e_x up to 0.05D	0.1D	0.2D	0.3D	0.4D	0.5D
0	1.00	0.90	0.74	0.60	0.38	0
6	1.00	0.90	0.74	0.60	0.38	0
8	1.00	0.90	0.74	0.60	0.38	0
10	0.98	0.90	0.74	0.60	0.38	0
12	0.95	0.89	0.74	0.60	0.38	0
14	0.91	0.85	0.74	0.60	0.38	0
16	0.86	0.81	0.72	0.60	0.38	0
18	0.80	0.75	0.68	0.60	0.38	0
20	0.75	0.72	0.63	0.55	0.38	0
22	0.71	0.67	0.59	0.49	0.28	0
24	0.66	0.62	0.53	0.36	0.13	0
26	0.60	0.56	0.43	0.21	0	0
27	0.57	0.52	0.34	0.11	0	0
28	0.53	0.47	0.26	0.03	0	0
30	0.42	0.30	0.07	0	0	0

Table 2 Capacity Reduction Factor, β

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Geometrical properties of diaphragm walls

TIED BLOCKWORK WALLS					
block width (m)	depth (m)	rib spacing (m)	area (m^2/m)	I value (m^4/m)	Z value (m^3/m)
0.10	0.44	0.45	0.253	0.0062	0.0282
0.10	0.44	0.68	0.235	0.0061	0.0278
0.10	0.44	0.90	0.227	0.0061	0.0276
0.10	0.44	1.13	0.221	0.0060	0.0275
0.10	0.44	1.35	0.218	0.0060	0.0274
0.10	0.66	0.45	0.302	0.0176	0.0535
0.10	0.66	0.68	0.268	0.0170	0.0516
0.10	0.66	0.90	0.251	0.0167	0.0508
0.10	0.66	1.13	0.241	0.0166	0.0502
0.10	0.66	1.35	0.234	0.0164	0.0498
0.10	0.89	0.45	0.353	0.0375	0.0842
0.10	0.89	0.68	0.301	0.0354	0.0795
0.10	0.89	0.90	0.277	0.0344	0.0773
0.10	0.89	1.13	0.261	0.0338	0.0759
0.10	0.89	1.35	0.251	0.0334	0.0751

Table 3

BONDED WALLS					
block width (m)	depth (m)	rib spacing (m)	area (m^2/m)	I value (m^4/m)	Z value (m^3/m)
0.10	0.44	0.73	0.233	0.0061	0.0277
0.10	0.44	1.18	0.220	0.0060	0.0275
0.10	0.55	0.73	0.248	0.0108	0.0392
0.10	0.55	1.18	0.230	0.0106	0.0385
0.10	0.78	0.73	0.279	0.0255	0.0654
0.10	0.78	1.18	0.249	0.0247	0.0632

Table 4

QUION BONDED WALLS					
block width (m)	depth (m)	rib spacing (m)	area (m^2/m)	I value (m^4/m)	Z value (m^3/m)
0.10	0.44	0.45	0.253	0.0062	0.0282
0.10	0.44	0.90	0.227	0.0061	0.0276
0.10	0.44	1.35	0.218	0.0060	0.0274
0.10	0.67	0.45	0.304	0.0183	0.0547
0.10	0.67	0.90	0.252	0.0174	0.0519
0.10	0.67	1.35	0.235	0.0171	0.0509

Table 5

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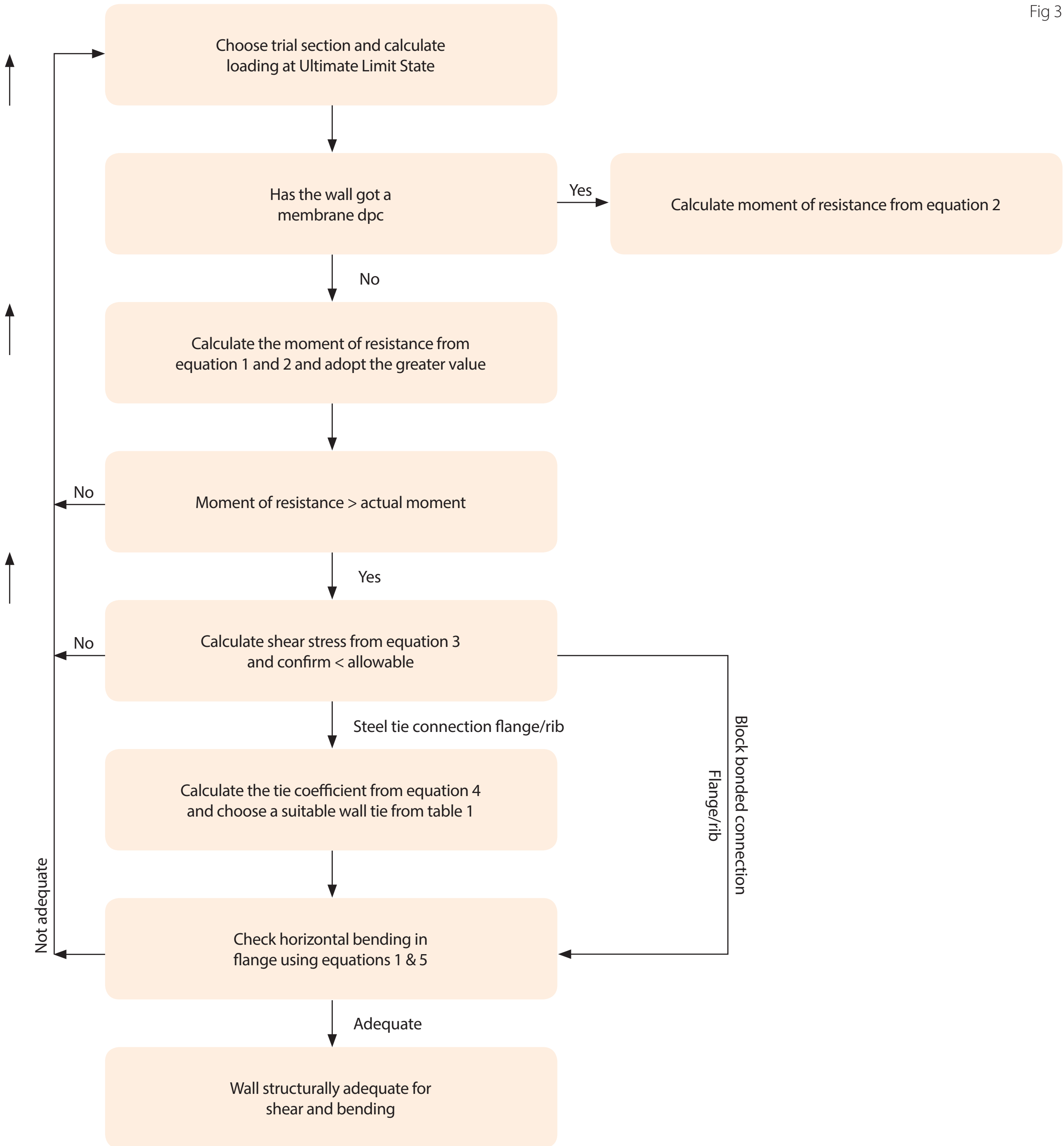
Design of freestanding diaphragm walls

Fig 3



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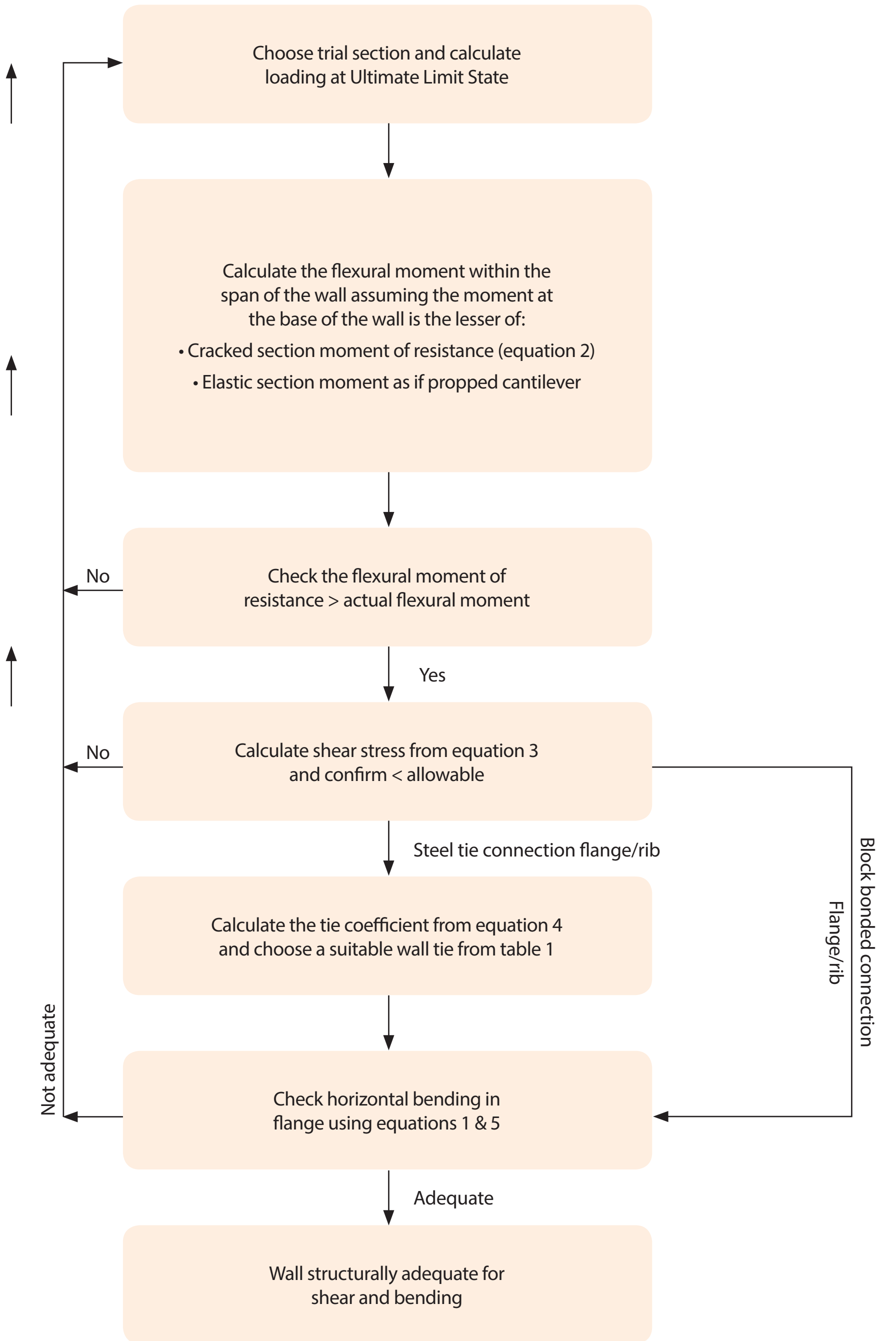
Design of single storey propped diaphragm wall

Fig 4



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Design example

- Design an 8 metre tall diaphragm wall to a single storey building.
- The characteristic wind pressure is 0.70 KN/m^2 and the blockwork unit weight may be taken as 16.68 kN/m^3.
- Assume the weight of the roof structure and capping beam balances any wind uplift.

Reference	Calculation	Output
	Wall loading	**w = 0.98 kN/m^2**
BS 5628	Ultimate limit state wind loading = 0.70 x 0.98 = 1.4 kN/m^2 Elastic moment at the base of a propped cantilever $M = wh^2/8$ = 0.98 x 8^2/8 = 7.84 kNm	M = 7.84 kNm
	Try 660 x 900mm tied diaphragm wall constructed from 100mm7N/mm^2 blocks set in class III mortar	
Table 3 BS 5268 BS 5268	A = 0.251 m^2 Z = 0.508m^3 D = 0.66m t_w = 100mm f_k = 6.4 N/mm^2 f_{kxperp} = 0.6 N/mm^2 f_{kxpar} = 0.25 N/mm^2 Y_m = 3.5 Y_{mv} = 2.5 Vertical load at base due to self weight = 0.251 x 8 x 16.68 = 33.49 kN At ultimate limit state vertical load = 33.49 x 0.9 = 30.14 kN	R = 30.14 kN
	Cracked section moment of resistance	
Equation 2	$M_r = (R/2)\{D - (RY_m / 1.1f_k)\}$ = (30.14/2) {0.66 – (30.14 x 3.5/1.1 x 6.4 x 10^3)} = 9.72 kNm Since 7.84 kNm < 9.72 kNm adopt 7.84 kNm as base moment	M = 9.72 kNm Base moment = 7.84 kNm
	Vertical flexural bending moment	
	Prop force at head of wall = 0.5 (8 x 0.98) – (7.84 / 8) = 2.94 kN Position of zero shear force = 2.94 / 0.98 = 3m Max flexural bending moment = (3 x 2.94) – (3^2 x 0.98 / 2) = 4.41 kNm	Max flexural M = 4.41kNm
	Vertical flexural resistance	
Equation 1	Vertical load at max flexural moment = 3 / 8 x 30.14 = 11.3 kN g_d = 11.3 / 0.251 x 10^3 = 0.045 N/mm^2 $M_r = [(f_{kx}/Y_m) + g_d]\ Z$ = [(0.25/3.5) + 0.045] X 0.0508 X 10^3 = 5.91 kNm <u>5.91 kNm > 4.41 kNm therefore wall adequate in vertical bending</u> Check the stability of the wall assuming stability entirely from gravity. Design wind pressure = 0.70 x 1.0 = 0.70kN/m^2 Elastic bending moment at base = 0.70 x 8^2/ 8 = 5.60kNm Since the elastic moment at the base is less than the cracked section moment of resistance at the base take the base moment equal to the elastic moment. Prop force at roof level = (0.70 x 8 / 2) – (5.6 / 8) = 2.1Kn Position of zero shear = 2.1 / 7.0 = 3m Maximum moment = (2.1 x 3) – (0.70 x 3^2 / 2) = 3.15kNm Design weight of wall at position of zero shear = 0.251 x 3 x 16.68 x 1.0 = 12.56 kN/m M_r = (12.56 /2) (0.66 – (12.56 x 3.5 / (1.1 x 6.4 x 10^3))) = 4.1kNm Since M_r > 3.15 kNm the wall is stable under gravity alone	M_r = 5.91 kNm
	Shear capacity	
Equation 3 BS 5628	Maximum shear per metre = 0.5 (8 x 0.98) + (7.84/8) = 4.9 kN Shear per rib V = 0.9 x 4.9 = 4.41 kN $v = V/Dt_w$ = 4.41 x 10^3 / (660 x 100) = 0.07 N/mm^2 fv / Y_{mv} = 0.35/2.5 = <u>0.14N/mm > 0.07 N/mm^2 therefore shear adequate</u>	V = 4.41 kN v = 0.07 N/mm^2
	Tie design	
Equation 4 Table 1	Assume vertical tie spacing = 225mm Y_m = 1.15 f_y = 250 N/mm^2 $K_v = t_w\, v\, s\, Y_m / f_y$ = 100 x 0.07 x 225 x 1.15/250 = 7.25 For 20 x 5 mild steel ties K_v = 8.3 > 7.25 therefore OK <u>Use 20 x 5 mild steel ties at 225mm vertical centres</u>	K_v = 7.25
	Horizontal flexural bending	
Equation 5 Equation 1	$M = wB^2 / 10$ = 0.98 x 0.9^2 / 10 = 0.08 kNm Z = 1000 x 100^2 / 6 = 1.667 x 10^{6}mm^3 gd =0 $M_r = (f_{kx} / Y_m)\ Z$ = (0.60 / 3.5) x 1.667 = 0.29 kNm <u>0.29 kNm > 0.08 kNm therefore horizontal flexural bending adequate</u> <u>Conclusions – Wall adequate</u>	M = 0.08 kNm M_r = 0.29 kNm

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Glossary of terms

A	Area of wall per unit length
B	Rib spacing
β	Capacity reduction factor
D	Overall depth of wall
$DVLR$	Design vertical load resistance
fk	Characteristic of compressive strength
fkx	Characteristic of flexural strength (failure plan noted)
$fkxpar$	Characteristic Flexural Strength Plane of failure parallel to bed joint
$fkxperp$	Characteristic Flexural Strength Plane of failure perpendicular to bed joint
fy	Yield strength of tie
gd	Design vertical load per unit area
I	Second moment of area of wall per unit length
M_r	Moment of resistance
R	Vertical load in wall
s	Tie spacing
t_w	Width of rib
V	Shear force
v	Shear stress
w	Uniformly distributed loading on wall
Y_m	Partial factor of safety on material
Z	Section modulus of wall per unit length

References:

BS 5628 : Part 1 : 2005
'Structural Use of Unreinforced Masonry'

BS 5628 : Part 3 : 2005
'Use of Masonry' Materials & Components
Design and Workmanship'

ACBA – The Design of Concrete Blockwork Diaphragm Walls,
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mpa British Precast

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