# PART B

# Chapter 6

## Horizontal Loads

This chapter provides the design requirements for masonry subject to horizontal loads – either out-of-plane pressures or in-plane shears due generally to wind or earthquakes.

# Contents

# 6.1 BASIS OF DESIGN

### 6.1.1 WIND LOADS

Australian designers have for many years been required to design buildings to withstand wind loads. The experience of cyclonic winds, commencing with Cyclone Tracey in 1974, has led to much research and innovation in the design and detailing of masonry structures for wind loads and the adaptation of reinforced masonry for Australian conditions. Wind loads can be manifested as uplift on bond beams and lintels (described in **Part B:Chapter 5** of this manual) or as horizontal loads – either out-of-plane or in-plane shear (described in this chapter). However, despite this activity, the rational design of unreinforced and reinforced masonry for wind loads is still not widespread, particularly in the southern states.

### 6.1.2 EARTHQUAKE LOADS

Long experience in many parts of the world has led designers to the conclusion that unreinforced brickwork does not behave well when subjected to the horizontal loads resulting from earthquakes. The brittle, low tensile strength of the medium leads to cracking and collapse. In many parts of the world where severe earthquakes are common, hollow concrete blockwork reinforced with close-spaced reinforcement is used to provide a ductile medium capable of withstanding repeated load reversals without significant loss of strength. Australia does not have a history of severe earthquakes and the use of unreinforced brickwork has become widespread. However, the 1989 Newcastle earthquake demonstrated the possible risks associated with the collapse of unreinforced walls under the action of even moderate earthquakes. Thus the introduction of some quantity reinforcement to moderately increase ductility and strength is considered appropriate. It is unlikely that the Australian public will accept the costs associated with the widespread substitution of "close-spaced" reinforced hollow blockwork for unreinforced brickwork. The use of "wide-spaced" reinforced masonry provides considerable improvement of strength and ductility at a more reasonable cost and is therefore considered more appropriate.

### 6.1.3 ADVANTAGES OF REINFORCED MASONRY

The effectiveness of reinforced concrete blockwork when compared with unreinforced masonry is demonstrated by the Modified Mercalli Scale which is reproduced in part in **Table 6.1**.

It can be seen that an earthquake classified as MM8 on the Mercalli Scale in which "alarm may approach panic", masonry that has not been designed to withstand lateral loads, unreinforced masonry or poorly constructed masonry are in various stages of destruction while reinforced masonry which has been designed to withstand lateral forces of 0.1g remains "undamaged".

### 6.1.4 BENDING AND SHEAR IN UNREINFORCED MASONRY

**Bending in Unreinforced Masonry**

When unreinforced masonry walls built in stretcher bond and laterally supported on two or more adjacent edges are subjected to horizontal out-of-plane pressures (due to wind, earthquake or some other load), they may collapse only after the masonry units have rotated relative to the units immediately above and below. AS 3700 includes a method of assessing the resistance to horizontal pressure based on the virtual work involved in causing this rotation to take place. The method results from extensive research sponsored, in part, by the Concrete Masonry Association of Australia at the Universities of New South Wales and Melbourne, Deakin University and CSIRO. The basis of the empirical method is set out in the Commentary to AS 3700 and in other published papers.

Test data indicate that three primary types of failure develop in unreinforced masonry panels subject to horizontal out-of-plane pressure. For each particular masonry panel, the failure pattern and capacity depends on the type of edge support (ie no support, lateral support or rotational restraint), the number of edges supported and the height-to-length proportions of the wall, **Figure 6.1**.

**Table 6.1** *Effects of Earthquake Intensity Based on the Modified Mercalli Scale*

| | | | ***EFFECT ON:* Masonry structures** | | | |
|---|---|---|---|---|---|---|
| | | | **Reinforced** | | **Unreinforced** | |
| **Earthquake Intensity** (Mercalli scale) | ***EFFECT ON:* People** | ***EFFECT ON:* Non-masonry structures** | Designed for lateral loads | Not designed for lateral loads | Normal workmanship | Poor workmanship |
| MM1 | Not felt, but may cause dizziness and nausea | | | | | |
| MM2 | Felt by a few persons at rest indoors | | | | | |
| MM3 | Felt indoors but not identified as an earthquake by all | | | | | |
| MM4 | Generally noticed indoors but not outside | | | | | |
| MM5 | Generally noticed outdoors | | | | | |
| MM6 | Felt by all. People (and animals) alarmed | | | | | Slight damage |
| MM7 | General alarm. Difficulty standing | | | | A few instances of damage. Loose brickwork dislodged | Cracked and damaged |
| MM8 | Alarm may approach panic | Panel walls thrown out of frame structures | Undamaged | Damaged in some cases | Damaged with partial collapse. Some brick veneers damaged | |
| MM9 | General panic | Frame structures racked and distorted | | Seriously damaged | Heavily damaged, sometimes collapsing completely. Brick veneer fails | Destroyed |
| MM10 | | Some well-built wooden buildings seriously damaged | Most masonry structures destroyed together with their footings | | | |
| MM11 | | Wooden-frame structures destroyed | | | | |
| MM12 | | Damage virtually total | | | | |

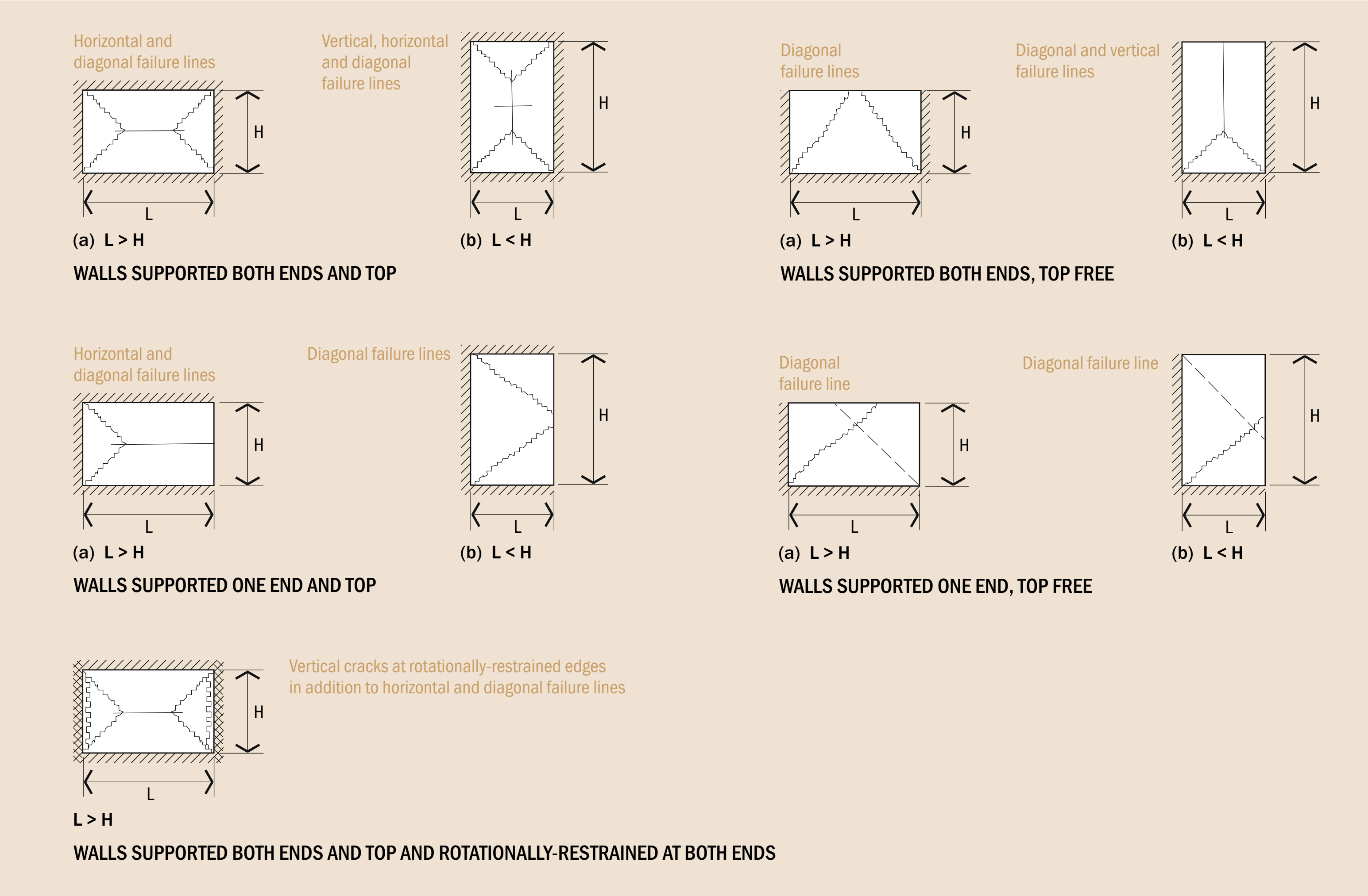



**Figure 6.1** *Summary of Observed Failure Patterns*

**Horizontal Failure Line** A horizontal failure will occur when the vertical bending capacity (influenced by bond strength, section modulus and compressive load) is exceeded. If a wall is relatively long compared to its height and the top edge is supported, a horizontal crack may appear in mortar joints near the mid-height. This is usually the first crack to appear, is often not noticeable and does not constitute a structural failure. A horizontal failure must also develop at or near the base of the wall before collapse can occur. It is normal to assume that the wall is rotationally unrestrained (due to lack of bond strength).

**Vertical Failure Line** A vertical failure line will occur when the horizontal bending capacity (influenced by bond strength and section modulus of perpendicular joints and the lateral modulus of rupture of units) is exceeded. A vertical failure may manifest either as a zigzag pattern around the line of the joints, or as a vertical crack passing alternately through the perpendicular joint and masonry unit. If a wall is relatively high compared to its length, a vertical failure line will appear first. If a wall is continuous past a vertical support, a vertical failure line will develop before collapse occurs.

**Diagonal Failure Line** A diagonal failure line radiates out from any corner where both vertical and horizontal edges are supported and forms as the units rotate relative to the adjacent units. For structural collapse to occur, these diagonal failure lines must cause a mechanism. The slope of the diagonal failure lines is governed by length-to-height proportions of the masonry units. A diagonal failure line will occur when the diagonal bending capacity (influenced by equivalent characteristic torsional strength, related to bond strength, and the equivalent torsional section modulus) is exceeded.

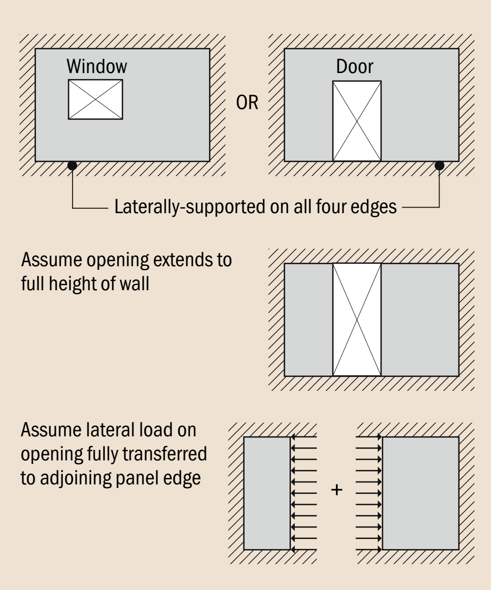



**Figure 6.2** *Assumptions for Walls with Openings*

### Walls With Openings

Walls with openings are considered to form sub-panels either side of the opening, **Figure 6.2**. The edges of the sub-panels adjacent to the opening are regarded as being unsupported (ie no lateral support or rotational restraint) with the remaining edges being supported. To simplify the calculations, the openings are assumed to extend for the full height of the wall. The pressure on the opening (ie on the door panel or window glazing) is considered to be fully transferred to the edge of the two adjoining masonry sub-panels. These are checked for flexural capacity as panels supported top, bottom and at one end, subjected to a horizontal line load at the other end and a uniform horizontal pressure.

AS 3700 does not give guidance on the permissible size of small openings that may be ignored. In the absence of data to the contrary, it is suggested that openings whose maximum dimension is less than one fifth of the height or length of the panel (whichever is the lesser) be ignored.

When a long window or door is to be supported by a short length of masonry, care must be taken to ensure that the masonry is built into the supports or is continuous past the supports so there is sufficient rotational resistance to support the load from the window or door.

### Shear in Unreinforced Masonry

The shear resistance of unreinforced masonry is influenced by two components, the shear bond strength (the ability of the mortar to bind the masonry units to each other and to their supports) and the shear friction strength (the frictional resistance to sliding once the bond is broken). When the masonry is subject to earthquake loading, the vertical movement of the structure relieves the gravity load, thus reducing friction resistance.

### Increasing the Capacity of Unreinforced Masonry

It is increasingly the practice in southern Australia to provide lateral support to unreinforced masonry subject to lateral earthquake, wind or fire loads by building-in galvanised steel mullions. Although this is a convenient practice, it is significantly more expensive than reinforced masonry. Furthermore, the stiffness of the mullion is considerably less than the stiffness of the masonry, which will possibly experience some cracking under extreme load. For typical details, see **Part C:Clause 3.3.2** and for capacities see **Part B:Clause 6.3**.

When cavity walls are subject to lateral earthquake, wind or fire load, the strength of the wall may be increased by tying the two leaves together monolithically, using ties together with either masonry units or mortar packing. This will provide stiffness as well as strength, but is not considered to be ductile. For typical details, see **Part C:Clause 3.3.3** and for capacities see **Part B:Clause 6.3**.

## 6.1.5 BENDING AND SHEAR IN REINFORCED MASONRY

### Bending in Reinforced Masonry

When reinforced masonry is subjected to bending, the moment resistance is provided by a combination of the reinforcement in tension and a width of concrete face shell in compression.

Vertical reinforcement placed in the cores of hollow concrete blockwork spans vertically between horizontal supports and provides strength enhancement to large wall panels.

- If vertical reinforcement is spaced at 800 mm or less, the masonry is regarded as 'close-spaced reinforced masonry', and may be considered ductile. This will have advantages in respect of reduced earthquake loads and increased strength.
- If vertical reinforcement is spaced at 2.0 m centres or less, (but wider than 800 mm), the masonry is regarded as 'wide-spaced reinforced masonry', with some advantages in respect of increased strength.
- If vertical reinforcement is spaced further apart than 2.0 m, the masonry is regarded as 'mixed construction', consisting of unreinforced masonry supported between the vertically-reinforced masonry elements.

It is common to lap the vertical bars with starter bars set in the slab or footings below, thus providing increased shear resistance and perhaps some moment resistance at the base. If the masonry supports a concrete slab, it may also be preferable to continue the wall reinforcement into the slab above. For typical details, see **Part C:Clause 3.5.2** and for capacities see **Part B:Clause 6.3**.

### Shear in Reinforced Masonry

The considerable overseas research into the behaviour of masonry with close-spaced reinforcement subject to cyclical in-plane shear loads, formed the basis of AS 3700 Clause 8.6.2, for shear walls. However there is little corresponding research for wide-spaced reinforcement subjected to in-plane cyclical loading simulating the action of shear walls during an earthquake.

If vertical reinforcement is placed at 2.0 m centres or closer and horizontal steel at 3.0 m centres or closer, the masonry wall is classified as a reinforced masonry shear wall.

Reinforced masonry shear walls are a combination of bond beams and vertically-reinforced masonry. All reinforcement must be correctly anchored to ensure that the wall remains intact when being subjected to in-plane shear. At corners of the wall and at openings, vertical reinforcement should be lapped with starter bars at the base and cogged into the bond beams and thus lapped with the bond beam reinforcement at the top. For typical details, see **Part C:Clause 3.5.1** and for capacities see **Part B:Clause 6.3**.

# 6.2 DESIGN REQUIREMENTS

## 6.2.1 AUSTRALIAN STANDARDS

This manual is based on the loads and load combinations of AS/NZS 1170.0, AS/NZS 1170.2 and AS 1170.4.

AS 3700 provides rules for masonry design and construction, including capacity reduction factors, geometric parameters (eg bedded areas), steel, block and mortar properties and detailing provisions (eg cover). The magnitude of the seismic loads attracted to a masonry wall will depend on its ductility. Unreinforced masonry being non-ductile attracts higher loads than ductile reinforced masonry. Guidance on the quantity and disposition of reinforcement to achieve structure and member ductility is given in AS 3700 Section 10 (see **Table 6.2**) and AS 1170.4 Table 6.5A.

**Table 6.2** *Structural Ductility Factor* ($\mu$) *and Structural Performance Factor* ($S_p$)
[From AS 3700 Table 10.1]

| Description of structure | $\mu$ | $S_p$ |
|---|---|---|
| Close-spaced reinforced masonry in accordance with Section 8, as appropriate | 2.00 | 0.77 |
| Wide-spaced reinforced masonry in accordance with Section 8.3.6, as appropriate | 1.50 | 0.77 |
| Prestressed masonry in accordance with Section 9, as appropriate | 1.50 | 0.77 |
| Unreinforced masonry in accordance with Section 7, as appropriate | 1.25 | 0.77 |

Notes:
A lower $\mu$ value than is specified in the above Table may be used.

In all cases, the structure shall be detailed to achieve the level of ductility assumed in the design and in accordance with this Standard.

## 6.2.2 SPECIFIC REQUIREMENTS FOR EARTHQUAKE LOADING TO AS 1170.4)

### Background

Until the 1970's, there was no comprehensive design standard controlling the design of masonry buildings. These were designed by a combination of engineering judgement, rule of thumb and experience. Over the last thirty years, piecemeal introduction of loading rules and capacity rules has led to considerable confusion concerning the suitability of masonry, particularly unreinforced masonry, in the context of design for Australian earthquakes.

It is well recognised throughout the world that unreinforced masonry cracks and may collapse under the significant movement associated with severe earth tremors and earthquakes. On the other hand, reinforced masonry building components (incorporating sufficient reinforcement to ensure ductility) deflect and absorb energy.

The fundamental question is, from the point of view of a regulator or standards writer, is, "*What probability of failure under earthquake loading is tolerable in Australia?*" This leads to a further question, "*Should Australian regulators require design to the same principles as are applied in countries prone to severe earthquakes?*"

Under the provisions of the AS 3700 *Masonry structures* Section 10, loadbearing unreinforced may be incorporated into buildings up to 15 m high in some circumstances. AS 3700 Table 10.3 (**Table 6.3**) provides height limits that range between 10 m and 15 m, depending on the site sub-soil classification and the hazard factor, Z. AS 3700 Clause 10.4 also makes provision for loadbearing unreinforced masonry walls of plant rooms, mezzanine floors and the like in higher buildings under some circumstances.

**Table 6.3** *Height Limits for Buildings with Loadbearing Unreinforced Masonry*
[From AS 3700 Table 10]

| Hazard Factor, Z (AS 1170.4) | Height limits (m) for Sub-soil Classification (AS 1170.4) | | | | |
|---|---|---|---|---|---|
| | A | B | C | D | E |
| ≥ 0.11 | 12 | 12 | 12 | 10 | 10 |
| 0.10 | 15 | 12 | 12 | 12 | 10 |
| 0.09 | 15 | 15 | 12 | 12 | 12 |
| 0.08 | 15 | 15 | 15 | 12 | 12 |
| 0.07 | 15 | 15 | 15 | 15 | 12 |
| ≤ 0.06 | 15 | 15 | 15 | 15 | 12 |

Notes:
These limits are not intended to apply to small, loadbearing, unreinforced masonry structures (such as plantrooms and mezzanine floors) contained within larger buildings, which may be over the prescribed heights.

These limits are not intended to apply to non-loadbearing masonry walls.

The particular restrictions are reproduced below.

*AS 3700 Clause 10.4 RESTRICTIONS ON THE USE OF LOADBEARING UNREINFORCED MASONRY*

*Buildings with heights greater than the values shown in Table 10.3 shall not incorporate loadbearing unreinforced masonry elements, except where the masonry complies with the following:*

*(a) The loadbearing masonry supports only a trafficable or non-trafficable roof or mezzanine floor which exerts a maximum of 10 kN/m permanent load to the masonry;*

*(b) All isolated masonry piers are constructed of reinforced masonry and designed accordingly;*

*(c) The area supported by the unreinforced masonry is less than 25 % of the plan area of the structure on which it is supported;*

*(d) The unreinforced masonry is not within 3.0 metres of the edge of the structure on which it is supported; and*

*(e) The loadbearing masonry components are designed for loads derived from AS 1170.4.*

In addition to these restrictions, such buildings must also be designed for earthquake loads derived from AS 1170.4.

The worked example sets out the calculations that must be carried out to determine the compliance with AS 3700 of a typical 15 m high building, subject to Australian earthquake actions determined from AS 1170.4, and incorporating both reinforced and unreinforced concrete masonry.

Failure is assumed to occur when the ultimate capacity of any masonry element is exceeded by the design action calculated using the Equivalent Static Force method. This approach is considered to be conservative, ignoring the ability of the structure to absorb energy and tolerate movement of various masonry elements without causing building collapse.

## Assumptions of Material Properties and Behaviour

### Equivalent Static Analysis

This analysis used the equivalent static method, ignores cyclic reversal of loads and the short period of application of loads. The Equivalent Static Analysis method ignores the displacement history under load, and is considered to be conservative in many cases, particularly for high frequency earthquakes that produce small displacements.

### Analysis

The method used in this Guide is based on AS 1170.4 Clause 5.2.4, treating the masonry as part of the structure. Clause 8.3 also makes provision for the design of walls as "parts" by a simplified method.

### Reference Period

The reference period (design life) used in the analysis is 50 years. Longer periods can be checked if required, and would increase the probability of failure.

### Hazard Factor

Hazard Factors, Z, used to describe ground acceleration for a particular geographical location, are determined from AS 1170.4 Table 3.2 (see **Table 6.4**).

**Table 6.4** *Typical Hazard Factors*

| Hazard Factor, Z | Location |
|---|---|
| 0.03 | Hobart |
| 0.04 | Launceston |
| 0.05 | Brisbane, Gold Coast |
| 0.06 | Cairns |
| 0.07 | Tamworth, Townsville |
| 0.08 | Sydney, Melbourne, Canberra, Alice Springs, Rockhampton |
| 0.09 | Perth, Darwin, Wollongong, Gosford |
| 0.10 | Adelaide |
| 0.11 | Newcastle, Bundaberg |
| 0.12 | Broome, Dampier |
| 0.20 | Meckering, Dowerin |

### Site Sub-soil Class

Site Sub-soil Classes, used with Hazard Factor to determine the equivalent ground acceleration for a particular soil type, are as designated in AS 1170.4 Clause 4.2 (see **Table 6.5**).

**Table 6.5** *Site Sub-Soil Classes*

| Sub-soil Class | Soil type |
|---|---|
| $A_e$ | Strong rock |
| $B_e$ | Rock |
| $C_e$ | Shallow soil |
| $D_e$ | Deep or soft soil |
| $E_e$ | Very soft soil |

### Fundamental Period

Buildings up to 15.0 m high have been assumed to be stiff, elastic, brittle, structures with a value of $k_t = 0.05$. Buildings over 15.0 m high have been assumed to be moment-resisting concrete frames with $k_t = 0.075$.

The Fundamental Period of the Building, $T_1$, is determined using AS 1170.4 Clause 6.2.3, by the maximum of:

$T_1 = 1.25\ k_t\ h_n^{0.75}$ and

$T_1 = 0.4$ s for site sub-soil class A, B or C, or

$T_1 = 0.6$ s for site sub-soil class D, or

$T_1 = 1.0$ s for site sub-soil class E.

### Spectral Shape Factors

Spectral Shape Factors $C_h(T)$, used in the determination of acceleration at the centre of weight of the building, are determined from AS 1170.4 Table 6.4 (see **Table 6.6**).

### Structural Ductility Factor and Structural Performance Factor

The ductility of the building affects the acceleration at which it vibrates. The following values have been used in accordance with AS 1170.4 Clause 6.5 and Table 6.5A.

Masonry buildings up to the heights in **Table 6.3** are assumed, for purposes of this Guide, to have an earthquake-resisting system that is provided by the unreinforced masonry, with or without contribution by a concrete shear core (e.g. four storey brickwork home units),

Structural Ductility Factor: $\mu = 1.25$

Structural Performance Factor: $S_p = 0.77$

Masonry buildings over the heights in **Table 6.3** are assumed, for purposes of this Guide, to have an earthquake-resisting system that consists essentially of reinforced concrete frames and the isolated unreinforced masonry, such that the masonry does not provide a substantial contribution to the earthquake-resisting system (e.g. high rise concrete framed buildings with or without shear walls and with isolated masonry partitions or cladding),

Structural Ductility Factor: $\mu = 2.00$

Structural Performance Factor: $S_p = 0.77$

**Table 6.6** *Spectral Shape Factor, $C_h(T)$* [From AS 1170.4 Table 6.4]

| | Sub-soil Classification | | | | |
|---|---|---|---|---|---|
| **Period (seconds)** | **A (Strong rock)** | **B (Rock)** | **C (Shallow soil)** | **D (Deep or soft soil)** | **E (Very soft soil)** |
| $0 < T \leq 0.1$ | 0.8 + 15.5T | 1.0 + 19.4T | 1.3 + 23.8T | 1.1 + 25.8 T | 1.1 + 25.8T |
| $0.1 < T \leq 1.5$ | Minimum (0.704/T, 2.35) | Minimum (0.88/T, 2.94) | Minimum (1.25/T, 3.68) | Minimum (1.98/T, 3.68) | Minimum (3.08/T, 3.68) |
| $T > 1.5$ | $1.056/T^2$ | $1.32/T^2$ | $1.874/T^2$ | $2.97/T^2$ | $4.62/T^2$ |

Note:
T = the calculated period of vibration

### Seismic Weight

For face loads on masonry walls, the Seismic Weight, $W_t$, is taken as the mean weight of the particular masonry leaf.

It has been assumed in this Guide that the external leaf of cavity masonry is fixed directly to each concrete floor slab or bears on a shelf angle at each floor. This is often the case in external cavity walls of high rise construction, but is not common in medium rise construction. In order to justify this assumption, this detail should become common practice. Appropriate details are given in AS 3700 Clause 10.3.

If the external leaf of cavity walls is not connected directly to each concrete floor slab or bears on a shelf angle at each floor, the face loads resulting from the weight of the external leaf must be resisted by the shear strength of the joints at the top and bottom of the internal leaf.

For shear at the base of the building, the Seismic Weight, $W_t$, is taken as the mean weight of the floors above, factored as follows.

Permanent load factor: 1.0

Imposed load factor: 0.3

### Base Shear

The Base Shear, V, is given by AS 1170.4 Clause 6.2.1,

$$V = k_p \, Z \, C_h(T_1) \, \frac{S_p}{\mu} \, W_t$$

### Acceleration at Various Heights

The Force at any floor is given by AS 1170.4 Clause 6.3.

$$F_i = \frac{W_i \, h_i^k}{\sum_{i=1}^{n} (W_i \, h_i^k)} V$$

Where the exponent is dependent on structure period.

When $T_1 \leq 0.5$ — $k = 1.0$

When $0.5 < T_1 < 2.5$ — k is linearly-interpolated between 1.0 and 2.0

When $T_1 \geq 2.5$ — $k = 2.0$

A similar (although slightly different) approach is given in ISO/DIS 3010 Appendix D[note 2].

*"a) For very low and stiff buildings, whole parts from the top to the base move along with the ground motion. In this case the distribution of seismic forces is uniform…*

*b) For low-rise buildings, the distribution of seismic forces becomes similar to the inverted triangle…*

*c) For high-rise buildings, seismic forces at the upper part become larger because of higher mode effect. If the building is assumed to be a uniform shear type elastic body fixed at the base and subject to white noise excitation, the distribution of seismic shear forces becomes a parabola whose vertex locates at the top…"*

- Very low buildings ($v = 0$) are defined as *"up to two-storey buildings, or structures for which $T \le 0.2$ s"*
- Low-rise buildings ($v = 0$ to 1) are defined as *"three to five-storey buildings, or structures for which 0.2 s < T <= 0.5 s"*
- Intermediate buildings ($v = 1$ to 2) are defined as *"structures for which $0.5\ s < T \le 1.5\ s$"*

Notes:
2 It is recognised that ISO/DIS 3010 Appendix D is not intended to refer to "Parts". However, it is reasoned that the walls of a building behave more like a part of the structure than as an "Attached Part".

### 6.2.3 DESIGN OF MASONRY WALLS FOR OUT-OF-PLANE EARTHQUAKE LOADS

AS 1170.4 requires that walls be designed for out-of-plane loading.

Other design considerations would be:

- a more accurate assessment of height amplification required in AS 1170.4, particularly in its application to low-rise buildings, and
- modification of the out-of-plane load capacity of unreinforced masonry in two-way bending to account for vertical compression, eg due to a number of floors above. This is provided for in AS 3700 Clause 7.4.4.3

### 6.2.4 BENDING IN UNREINFORCED MASONRY

**Vertical Bending Strength**

The vertical bending moment capacity is given by the least of:

$M_{cv} = \phi\ k_{mt}\ f'_{mt}\ Z_d + f_d\ Z_d$ Nmm/m
representing a combination of flexural bond strength and compression,

$M_{cv} = 3.0\ f\ k_{mt}\ f'_{mt}\ Z_d$ Nmm/m
representing an upper bound on flexural bond strength and compression,

$M_{cv} = f_d\ Z_d$ Nmm/m
representing the compression where $f'_{mt} = 0$ (ie at a damp-proof course or interface with another material).
$f_d$ shall not be taken as greater than 0.36 MPa.

**Horizontal Bending Strength**

The horizontal bending moment capacity is given by the least of:

$M_{ch} = 2.0\ \phi\ k_p \sqrt{f'_{mt}}\ (1 + f_d / f'_{mt})\ Z_d$ Nmm/m
representing a zigzag failure around the mortar joints with a combination of torsional bond strength and compression,

$M_{ch} = 4.0\ \phi\ k_p \sqrt{f'_{mt}}\ Z_d$ Nmm/m
representing an upper bound on the zigzag failure with torsional bond strength and compression,

$M_{ch} = \phi\ (0.44\ f'_{ut}\ Z_u + 0.56\ f'_{mt}\ Z_u)$ Nmm/m
representing a straight vertical failure lternating through masonry unit and mortar.

**Diagonal Bending Strength**

The diagonal bending capacity is given by:

$M_{cd} = \phi\ f'_t\ Z_t$ Nmm/m

AS 3700 gives formulae for calculating the torsional section modulus of various types of masonry units.

### 6.2.5 SHEAR IN UNREINFORCED MASONRY

For loads other than earthquake loads,

$V_d = V_o + V_1$

$= \phi\ f'_{ms}\ A_{dw} + k_v\ f_d\ A_{dw}$

For earthquake loads,

$V_d = V_o + V_{le}$

$= \phi\ f'_{ms}\ A_{dw} + 0.9\ k_v\ f_{de}\ A_{dw}$

where $f_{de} = G_g / A_{dw}$

The gravitational force acting vertically on a wall consists of two components: the self weight of the wall and attachments (which contribute to both the out-of-plane earthquake load on the wall and its frictional resistance), and the weight of other parts of the structure (which are supported laterally by a shear core, shear walls or structural frame and therefore do not contribute to the out-of-plane earthquake load on the wall). Gravitational force $G_g$ should be factored down as follows:

- the self weight of the wall and attachments is not factored, since it contributes to both load and resistance
- the weight of other parts of the structure are factored by 0.8, since they contribute only to resistance.

### 6.2.6 BENDING IN REINFORCED MASONRY

When reinforced masonry is subjected to bending, the moment resistance is provided by a combination of the reinforcement in tension and a width of concrete face shell in compression. AS 3700 permits a width of $2t_w$ on either side of the reinforcement for vertically-reinforced masonry and $1.5t_w$ on either side of the reinforcement for horizontally-reinforced masonry. AS 3700 limits the area of tensile reinforcement used for design purposes to a `balanced failure' value. This does not mean that more reinforcement can not be placed in the wall, only that it can not all be used for design to resist bending. The unconfined masonry compressive strength, $f'_m$, significantly underestimates the crushing strength of reinforced masonry and there is little likelihood of brittle failure due to over-reinforcement. The limiting quantity of tensile reinforcement for design purposes is given by:

$A_{sd} = (0.29)\ 1.3\ f'_m\ b\ d\ /\ f_{sy}$

Because the compressive strength of masonry is based on unconfined prisms and the corresponding concrete strength is based on confined cylinders, the strength of masonry against which the tensile forces are balanced must be adjusted to give $1.3f'_m$ corresponding to $f'_c$ in reinforced concrete design.

The ultimate bending moment capacity for reinforced masonry in bending is given by:

$M_d = \phi\ f_{sy}\ A_{sd}\ d\ (1 - 0.6\ f_{sy}\ A_{sd}\ /\ 1.3\ f'_m\ b\ d)$

### 6.2.7 SHEAR IN REINFORCED MASONRY

The in-plane shear resistance of reinforced shear walls, with a height/length ratio (H/L) less than 2.3 and specified quantities of reinforcement crossing the potential crack lines is given by:

$V_d = \phi\ (f_{vr}\ A_d + 0.8\ f_{sy}\ A_s)$.

This includes the shear strength of the masonry (enhanced by the confining action of the reinforcement, diminishing from a theoretical maximum of 1.5 MPa to a limit of 0.35 MPa at H/L = 2.3) and 0.8 times the tensile strength of the reinforcement crossing the potential crack planes. In the worked examples, the formula has been modified to give reduced shear capacity, accounting for the fact that not all of the steel present crosses potential shear cracks.

The spacing of the reinforcement is limited to 2.0 metres horizontal spacing of vertical reinforcement and 3.0 metres vertical spacing of horizontal reinforcement.

Walls which are more slender than H/L = 2.3 will behave in a manner similar to beams, without any enhancement of the masonry strength due to confinement by the reinforcement. The shear capacity is given by:

$V_d = \phi\ (f_{vm}\ b_w\ d + f_{vs}\ A_{st} + f_{sy}\ A_{sv}\ d\ /\ s)$.

Their strength relies on:

- The shear strength of the masonry
- The dowel action of the main tensile reinforcement
- The tensile force in any stirrups closer together than 0.75D.

Shear walls and lightly loaded piers must be considered for stability and may require starter bars to anchor the member to the structure. Stability should be checked using:

$V_d = \phi\ [k_{sw}\ P_v\ L\ /\ 2 + f_{sy}\ A_{sv}\ (L - 2\ l')]\ /\ H$.

The first term defines the resistance due to vertical load while the second term defines the resistance due to the anchorage of heel reinforcement. The reduction factor ($k_{sw}$) accounts for toe crushing in shear walls under heavy vertical loads.

### 6.2.8 TIES AND CONNECTORS

Ties and connectors that fix a masonry wall to the supporting structure shall be capable of transmitting the loads imposed on the wall by wind or earthquake to the supports. These requirements are covered by three Australian Standards:
AS/NZS 1170.2 *Wind actions* Clause 2.5.5.

AS 1170.4 *Earthquake actions in Australia* Clause 5.2.2, Clause 5.2.4, Clause 5.4.6 and Clause 8.1.3.

AS 3700 *Masonry structures*, including Clauses 4.11.4 and 10.4.

The loads required to be transmitted vary with building location, soil type, use to which the building is put, elevation, shielding from wind and topography.

**AS/NZS 1170.2 Requirements**
AS/NZS 1170.2 does not specifically mention ties and connections except that, in Tropical Cyclone Regions C and D, Clauses 2.5.5 require "cladding" connections to be designed to resist fatigue loading.

**AS 1170.4 Requirements**
The use of AS 1170.4 is complicated by the fact that the requirements for connections depend on the building location, soil type, use to which the building is put, elevation and whether it is ductile or non-ductile.

AS 1170.4 Clause 5.2.4 states:

> *Walls shall be anchored to the roof and restrained at all floors which provide horizontal support for the wall. Walls shall be designed for in-plane and out-of-plane forces. Out-of-plane forces on walls shall be designed in accordance with Section 8.*

AS 1170.4 Clauses 5.3, 5.4 and 5.5 provide the rules relevent to Earthquake Design Categories EDC **I**, EDC **II** and EDC **III**.

For EDC **I**, the out-of-plane load is 10% of the wall weight.

For EDC **II**, the out-of-plane load is:

$$F_i = K_s\,(k_p\, Z \frac{S_p}{\mu})W_i$$

For EDC **III**, carry out dynamic analysis in accordance with AS 1170.4.

AS 1170.4 makes it permissible to rely on friction calculated in accordance with AS 3700 to transfer horizontal loads to and from masonry loadbearing walls. Therefore it is **not** necessary to provide ties or connectors at the top or bottom of loadbearing walls.

However, it is required to provide ties or connectors at the top of non-loadbearing walls and it would appear to be the intention of the Standard that connectors (other than friction) be required at the base of non-loadbearing masonry walls.

**Figure 6.3** summarises these requirements.

For loadbearing masonry structures, slabs should be supported on a series of walls at right angles to each other to avoid the possibility of the slab being dislodged from its supporting wall.

**AS 3700 Requirements**

AS 3700 Clause 2.6.3 requires that the ultimate design load on any supporting members be the greater of:

> *The sum of the simple static reactions to the total applied horizontal forces for the appropriate load combination and 2.5% of the vertical load that the masonry member is designed to carry.* (Note: In this manual the additional 2.5% of vertical loads has not been added to the connection loads for earthquake derived using AS 1170.4 because AS 1170.4 Table 5.4 already requires design for twice the calculated lateral load).
>
> *0.4 kPa acting on the appropriate tributary area* of supported masonry.

AS 3700 Clause 2.6.4 requires that the ultimate design load on any connection to a supporting members be the load calculated from Clause 2.6.4 multiplied by 1.25.

AS 3700 Clause 10.2.5 expand the AS 1170.4 requirements set out above.

**Performance of Head Ties and Connectors**

Many commercially available head ties do not have sufficient shear resistance to support large wall panels. The designer should carefully check the shear capacity using the tie characteristic shear strength provided by the tie manufacturer.

$$V_{cap} = \phi_{tie} F_{tie} / S$$

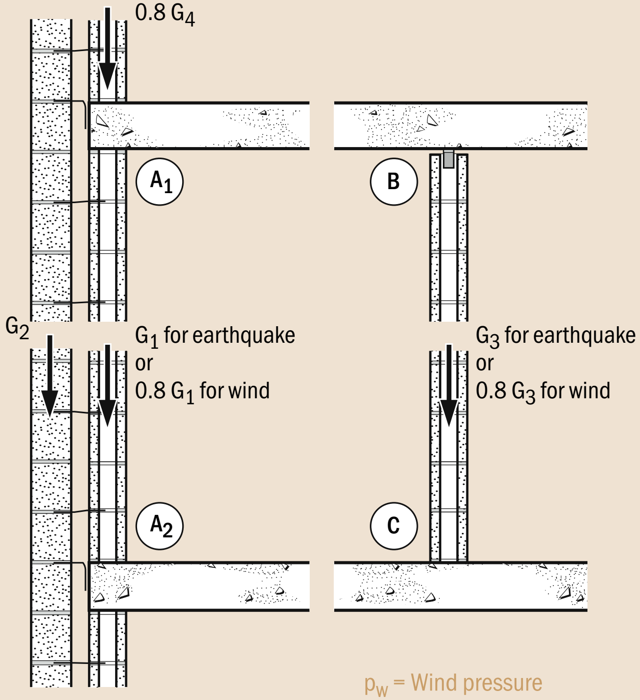
where:

$V_{cap}$ = ultimate capacity of ties kN/m length of wall

$\phi_{tie}$ = capacity reduction factor, taken as 0.75

$F_{tie}$ = characteristic shear strength of a single tie, provided by the tie manufacturer

$S$ = proposed spacing of head ties, to correspond with perpendicular joints.




$p_w$ = Wind pressure

A = Conrtibutory area per 1-m length (= H/2)

S = Tie or connector spacing (< 1.2 m)

$F_{tie}$ = Tie or connector capacity

$\phi_{tie}$ = Capacity reduction factor for ties (= 0.75)

$G_1$ = Self weight of internal leaf

$G_2$ = Self weight of external leaf

$G_3$ = Self weight of internal wall

$G_4$ = Self weight of other parts of structure

Notes:

A In this manual the additional 2.5% of vertical loads has not been added to the connection loads for earthquake derived using AS 1170.4.

### $A_1$/$A_2$ – TOP AND BOTTOM OF LOADBEARING WALL

**Capacity** (kN/m) **> Loads** (kN/m)

**Earthquake:**

$$\phi f'_{ms} A_{dw} + 0.9 k_v f_{dc} A_{dw} > K_s \left(\frac{k_p Z S_p}{\mu}\right)(G_1 + G_4) A \quad \text{(Note A)}$$

$$> 0.4 A$$

**Wind:**

$$\phi f'_{ms} A_{dw} + k_v f_{dc} A_{dw} > 1.25 p_w A + 0.025[1.2(G_1 + G_4) + 1.5Q]$$

$$> 0.4 A$$

### B – TOP OF NON-LOADBEARING WALL

**Capacity** (kN/m) **> Loads** (kN/m)

$$\phi_{tie} \frac{F_{tie}}{S} > K_s \left(\frac{k_p Z S_p}{\mu}\right) G_3 A$$

$$> 1.25 p_w A$$

$$> 0.4 A$$

### C – BOTTOM OF NON-LOADBEARING WALL

If slip joint material is used, design connection as for top
If slip joint material is not used, design using friction, ignoring bond

**Capacity** (kN/m) **> Loads** (kN/m)

$$\phi_{tie} \frac{F_{tie}}{S} \text{ or } \phi f'_{ms} A_{dw} > K_s \left(\frac{k_p Z S_p}{\mu}\right) G_3 A$$

$$> 1.25 p_w A + 0.025(1.2 G_3)$$

$$> 0.4 A$$

**Figure 6.3** *Summary of Wall Tie/Connector Requirements*

## 6.3 STANDARD DESIGNS

### 6.3.1 GENERAL

**Design and detailing**

All design and detailing shall comply with the requirements of AS 3700 and, where appropriate, AS/NZS 1170.

It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties. Control joints and openings must be treated as free ends as specified by AS 3700.

**Masonry properties**

The standard designs in this chapter are based on minimum masonry properties complying with the General Specification set out in **Part C:Chapter 2**, modified as noted on the standard design chart and as noted below.

**Hollow concrete blocks**

Width 90 mm, 110 mm, 140 mm and 190 mm

Height 190 mm

Length 390 mm

Face-shell bedded

Minimum face-shell thickness,
$t_s$ = 25 mm for 90 mm, 110 mm and 140 mm units
$t_s$ = 30 mm for 190 mm units

Minimum characteristic compressive strength,
$f'_{uc}$ =15 MPa

Minimum characteristic lateral modulus of rupture,
$f'_{ut}$ = 0.8 MPa

**Solid or cored concrete bricks**

Width 110 mm

Height 76 mm

Length 230 mm

Fully bedded

Minimum characteristic compressive strength,
$f'_{uc}$ =10 MPa

Minimum characteristic lateral modulus of rupture,
$f'_{ut}$ = 0.8 MPa

**Mortar joints**

Mortar type M3 (or M4)

Joint thickness 10 mm

**Concrete grout**

Minimum characteristic compressive strength,
$f'_c$ = 20 MPa

Minimum cement content 300 kg/m$^3$

**Steel reinforcement**

N12, N16 or N20, as noted, complying with AS 3700, Section 8.5.

### 6.3.2 STANDARD DESIGN CHARTS

**How to Read**

The general procedure with most charts is as follows:

- Select the required wall thickness (and, if appropriate, the reinforcement arrangement).
- Select the appropriate support conditions (eg supported on four sides).
- Project the length of the wall between vertical supports and the height of wall between horizontal supports to determine the design point.
- Select a curve which is above or to the right of the design point. Read off the load capacity corresponding to the selected curve. If necessary, interpolate between curves.
- Check that the masonry wall is adequate for other loadings, design requirements and construction requirements. On some charts, the robustness requirements for the same conditions have been superimposed.

### 6.3.3 INDEX TO DESIGN CHARTS

## GALVANISED STEEL MULLIONS – Moment and Shear Capacities

See Part C:Clause 3.3.2 for typical details

| Section[1] | Grade[2] MPa | Orientation[3] (depth through wall) | End connection type[4] | Shear capacity[5] kN | Moment capacity[6] kN.m |
|---|---|---|---|---|---|
| 150 x 50 x 5.0 RHS | C450LO | 150 | 2-M12, 8 | 15.8 | 31.90 |
| 150 x 50 x 4.0 RHS | C450LO | 150 | 2-M12, 8 | 15.8 | 26.50 |
| 125 x 75 x 6.0 RHS | C450LO | 75 | 2-M12, 8 | 15.8 | 23.90 |
| 125 x 75 x 5.0 RHS | C450LO | 75 | 2-M12, 8 | 15.8 | 20.50 |
| 75 x 75 x 6.0 SHS | C450LO | 75 | 2-M10, 8 | 12.6 | 15.60 |
| 75 x 75 x 5.0 SHS | C450LO | 75 | 2-M10, 8 | 12.6 | 13.60 |
| 75 x 75 x 4.0 SHS | C450LO | 75 | 2-M10, 8 | 12.6 | 11.40 |
| 100 x 50 x 6.0 RHS | C450LO | 50 | 2-M10, 8 | 12.6 | 11.20 |
| 100 x 50 x 5.0 RHS | C450LO | 50 | 2-M10, 8 | 12.6 | 9.88 |
| 100 x 50 x 4.0 RHS | C450LO | 50 | 2-M10, 8 | 12.6 | 8.23 |
| 100 x 50 x 3.5 RHS | C450LO | 50 | 2-M10, 8 | 12.6 | 6.92 |
| 100 x 50 x 3.0 RHS | C450LO | 50 | 2-M10, 8 | 12.6 | 5.63 |
| 100 x 50 x 2.5 RHS | C450LO | 50 | 2-M10, 8 | 12.6 | 4.22 |
| 50 x 50 x 5.0 SHS | C450LO | 50 | 2-M10, 8 | 12.6 | 5.33 |
| 170 x 10 FMS | 250 | 170 | Nil | Nil | 10.80 |
| 120 x 10 FMS | 250 | 120 | Nil | Nil | 5.40 |
| 90 x 10 FMS | 250 | 90 | Nil | Nil | 3.00 |
| 70 x 10 FMS | 250 | 70 | Nil | Nil | 1.80 |

**Notes:**

1 All hollow sections are BHP Duragal.

2 Capacities of all hollow sections are based on Grade C450LO in accordance with AS 1163.

All other sections are based on Grade 250 in accordance with AS 3679.

3 The orientation shows the dimension of the steel section when measured through the wall. For square hollow sections, this value is the same as the side of the section. For rectangular hollow sections, this value is the same as the smaller of the two sides of the section. It is important to ensure that the steel section will fit into the cores of the blocks.

4 The end connections indicated are the ones most likely to lead to efficient design and construction, although other end connections can be used with each section.
The nomenclature is as follows:

| Designation | Number of Anchors | Anchor type | Plate thickness (mm) |
|---|---|---|---|
| *2-M12, 8* | *2* | *M12 Dynabolts* | *8* |

5 Shear capacity is based on the connection shear capacity, using the shear values provided by Ramset Fasteners (Aust) Pty Ltd for 20 MPa concrete.Because there are no end plates on plate mullions, there is no contribution to shear capacity.

6 Moment capacities of Duragal hollow sections are based on values provided by BHP in Duragal design capacity tables for steel hollow sections by Tubemakers, June 1996.Moment capacities of plate mullions are calculated using AS 4100 assuming continuous lateral bracing by the adjacent masonry.

7 Blocks must be of a type and size to enable the mullions to be built into the masonry and the cores packed with mortar.

## COMPOSITE MASONRY MULLIONS – Moment and Shear Capacities

See Part C:Clause 3.3.3 for typical details

| Inner leaf mm(1) | Cavity width mm(2) | Outer leaf mm(3) | Web width mm(4) | Intermediate or End(5) | Wall 2700 mm high: Total width mm(6) | Wall 2700 mm high: Shear capacity kN(7) | Wall 2700 mm high: Moment capacity (kN/m)(8) | Wall 3900 mm high(9): Total width mm(6) | Wall 3900 mm high(9): Shear capacity kN(7) | Wall 3900 mm high(9): Moment capacity (kN/m)(8) |
|---|---|---|---|---|---|---|---|---|---|---|
| 110 | 50 | 110 | 300 | I | 840 | 1.56 | 1.49 | 1080 | 2.86 | 2.08 |
| 110 | 50 | 110 | 300 | E | 570 | 1.09 | 1.01 | 690 | 1.88 | 1.32 |
| 90 | 50 | 110 | 300 | I | 840 | 1.55 | 1.26 | 1080 | 2.84 | 1.75 |
| 90 | 50 | 110 | 300 | E | 570 | 1.08 | 0.85 | 690 | 1.86 | 1.11 |
| 90 | 50 | 90 | 300 | I | 840 | 1.30 | 1.08 | 1080 | 2.37 | 1.50 |
| 90 | 50 | 90 | 300 | E | 570 | 0.91 | 0.73 | 690 | 1.56 | 0.95 |

**Notes:**

1 An inner leaf of 110-mm brickwork has been common for many years, although increasingly 90-mm is being used because of the potential savings in both cost and floor space. Concrete blocks 90 x 119 x 290 mm and 90 x 162 x 290 are available. The 119-mm heights corresponds to 1.5 courses of 76-mm-high brickwork, whilst 162 mm corresponds to two courses of 76-mm-high brickwork.

2 These tables are based on the most common cavity width of 50 mm. Capacities may be increased by increasing the cavity width.

3 An outer leaf of 110-mm brickwork is common. However, split, ribbed polished or fair-face 90-mm concrete blockwork is sometimes used to provide an attractive economical external face.

4 These tables are based on a web width of 300 mm. This can be achieved using a mortar column tied within the leaves by cavity ties. A similar result could be achieved using masonry units bonded to form a diaphragm. In both cases, rainwater must be prevented from crossing the cavity via the diaphragm.

5 End mullions are placed near the end of a wall and have masonry cavity walls extending on one side only. Intermediate mullions are placed within a length of wall and have masonry cavity walls extending on both sides.

6 The calculation of the effective width of the composite mullion (ie the width of each leaf which acts compositely with a web) is six times the width of the leaf based on AS 3700 Clause 4.5.2. For an end mullion, the effective width is the web width plus up to six times the minimum leaf width on one side only. For an intermediate mullion, the effective width is the web width plus six times the minimum leaf width on both sides of the web (ie up to twelve times).

**Notes:**

7 The shear capacities given in the table is based on the following:
- a characteristic shear strength at the interface of the supporting concrete slab, $f'_{ms}$, of zero
- a shear factor, $k_v$, of 0.3
- self weight for a wall 2.7 m and 3.9 m high
- no additional applied vertical load
- the formula in AS 3700 Clause A8.3 for shear arising from earthquake loads.

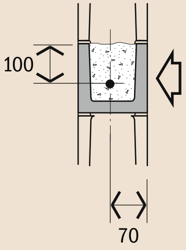
For other circumstances, the shear capacity may be increased.
- If the wall transfers shear load across an interface confined by reinforcement, the characteristic shear strength, $f'_{ms}$, may be taken as 0.35 MPa.
- The shear factor, $k_v$, of 0.3 is appropriate to mortar joints, concrete interface and bitumen-coated aluminium or embossed polyethylene damp-proof-courses and flashings. For other interface materials, 0.3 may not be appropriate.
- Vertical loads as may be applied by supported floor slabs will increase shear capacity.
- If the shear load is not caused by earthquake, the component of capacity which is derived from vertical load may be increased by 11%. See AS 3700 Clause 7.5.1.

The shear capacity is given for the length over which the composite mullion extends.

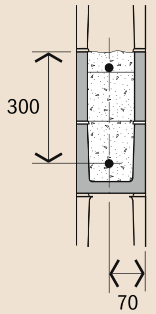
8 The moment capacities given in the tables are based on a characteristic tensile strength, $f'_{mt}$, of 0.2 MPa and the section modulus based on composite action and self weight based on a wall height of 2.7 m or 3.9 m.

9 Walls higher than the value 3.9 m used in these tables will have shear and moment resistance higher than the tabulated values.

## HORIZONTALLY-REINFORCED BOND BEAMS



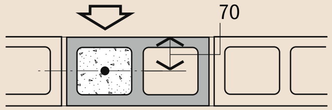

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 5.1 | 2.6 |
| N16 | 6.3 | 3.9 |




| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 10.2 | 4.9 |
| N16 | 12.6 | 5.7 |

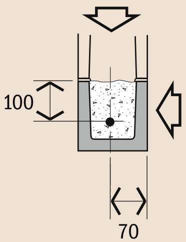
## VERTICALLY-REINFORCED CORES IN MID-WALL

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 5.1 | 2.7 |
| N16 | 6.3 | 4.5 |



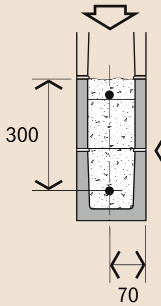

## HORIZONTALLY-REINFORCED LINTELS

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 5.1 | 2.6 |
| N16 | 6.3 | 2.6 |




| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 5.1 | 2.5 |
| N16 | 6.3 | 2.9 |

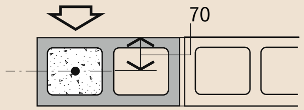
| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 12.5 | 11.4 |
| N16 | 13.7 | 19.4 |




| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 10.2 | 4.7 |
| N16 | 12.6 | 4.7 |

## VERTICALLY-REINFORCED CORES ADJACENT TO OPENINGS

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 5.1 | 2.5 |
| N16 | 6.3 | 3.5 |




## NOTES

$V_c$ = Shear capacity (kN)

$M_c$ = Moment capacity (kN.m)

Wall thickness, 140 mm

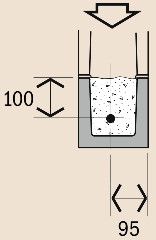
Mortar type, M3

Block characteristic compressive strength, $f'_{uc}$ = 15 MPa

Grout compressive strength, $f'_c$ = 20 MPa

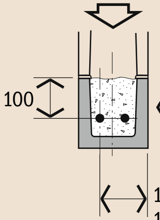
## HORIZONTALLY-REINFORCED LINTELS

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 6.4 | 3.4 |
| N16 | 7.6 | 3.6 |
| N20 | 9.1 | 3.6 |



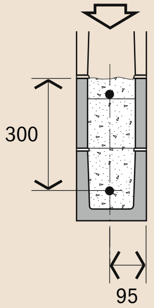

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 6.4 | 3.6 |
| N16 | 7.6 | 6.0 |
| N20 | 9.1 | 6.5 |

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 7.9 | 3.6 |
| N16 | 10.2 | 3.6 |
| N20 | 13.1 | 3.6 |



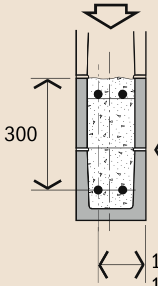

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 8.5 | 5.2 |
| N16 | 9.6 | 8.8 |
| N20 | 10.9 | 12.2 |

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 16.4 | 11.7 |
| N16 | 17.6 | 20.2 |
| N20 | 19.0 | 29.4 |



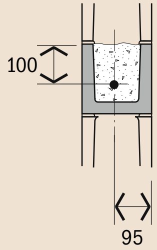

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 12.9 | 6.9 |
| N16 | 15.2 | 9.9 |
| N20 | 18.1 | 9.9 |

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 17.9 | 22.0 |
| N16 | 20.2 | 32.2 |
| N20 | 23.1 | 32.2 |

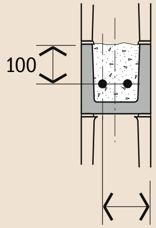



| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 17.0 | 10.2 |
| N16 | 19.1 | 16.9 |
| N20 | 21.8 | 18.6 |

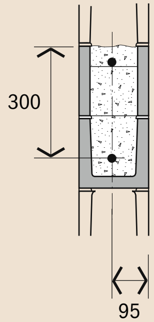
## HORIZONTALLY-REINFORCED BOND BEAMS



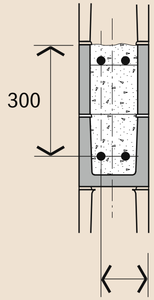

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 6.4 | 3.7 |
| N16 | 7.6 | 6.4 |
| N20 | 9.1 | 9.2 |




| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 8.5 | 5.3 |
| N16 | 9.6 | 9.1 |
| N20 | 10.9 | 13.3 |



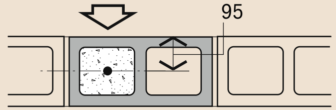

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 12.9 | 7.2 |
| N16 | 15.2 | 12.0 |
| N20 | 18.1 | 13.1 |




| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 17.0 | 10.4 |
| N16 | 19.1 | 17.6 |
| N20 | 21.8 | 24.5 |

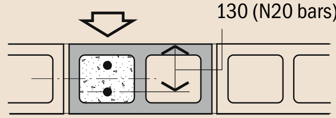
## VERTICALLY-REINFORCED CORES IN MID-WALL

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 6.4 | 3.7 |
| N16 | 7.6 | 6.6 |
| N20 | 9.1 | 9.7 |



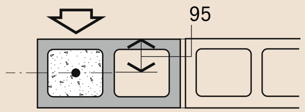

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 8.5 | 5.4 |
| N16 | 9.6 | 9.3 |
| N20 | 10.9 | 13.8 |

134 (N12 bars)
132 (N16 bars)
130 (N20 bars)



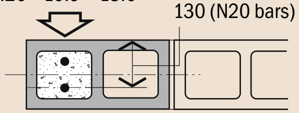
## VERTICALLY-REINFORCED CORES ADJACENT TO OPENINGS

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 6.4 | 3.6 |
| N16 | 7.6 | 6.2 |
| N20 | 9.1 | 8.2 |




| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 8.5 | 5.3 |
| N16 | 9.6 | 9.0 |
| N20 | 10.9 | 13.0 |

134 (N12 bars)
132 (N16 bars)
130 (N20 bars)



## NOTES

$V_c$ = Shear capacity (kN)

$M_c$ = Moment capacity (kN.m)

Wall thickness, 190 mm

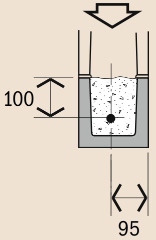
Mortar type, M3

Block characteristic compressive strength, $f'_{uc}$ = 15 MPa

Grout compressive strength, $f'_c$ = 20 MPa

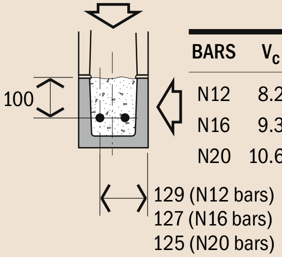
## HORIZONTALLY-REINFORCED LINTELS

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 6.4 | 3.4 |
| N16 | 7.6 | 3.6 |
| N20 | 9.1 | 3.6 |



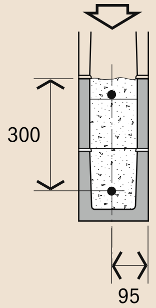

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 6.4 | 3.6 |
| N16 | 7.6 | 6.0 |
| N20 | 9.1 | 6.5 |

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 7.9 | 3.6 |
| N16 | 10.2 | 3.6 |
| N20 | 13.1 | 3.6 |



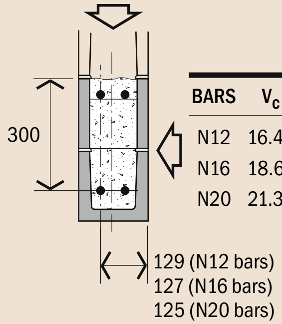

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 8.2 | 5.0 |
| N16 | 9.3 | 8.4 |
| N20 | 10.6 | 11.3 |

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 16.4 | 11.7 |
| N16 | 17.6 | 20.2 |
| N20 | 19.0 | 29.4 |



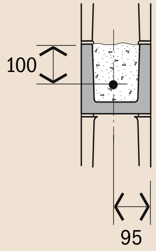

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 12.9 | 6.9 |
| N16 | 15.2 | 9.9 |
| N20 | 18.1 | 9.9 |

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 17.9 | 22.0 |
| N16 | 20.2 | 32.2 |
| N20 | 23.1 | 32.2 |

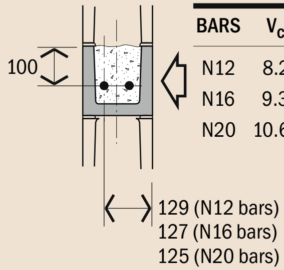



| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 16.4 | 9.8 |
| N16 | 18.6 | 16.1 |
| N20 | 21.3 | 17.2 |

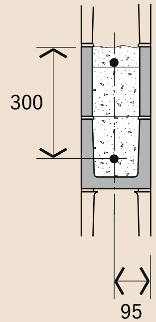
## HORIZONTALLY-REINFORCED BOND BEAMS



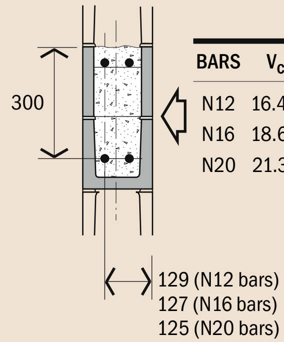

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 6.4 | 3.7 |
| N16 | 7.6 | 6.4 |
| N20 | 9.1 | 9.2 |




| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 8.2 | 5.1 |
| N16 | 9.3 | 8.8 |
| N20 | 10.6 | 12.7 |



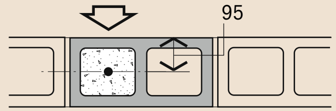

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 12.9 | 7.2 |
| N16 | 15.2 | 12.0 |
| N20 | 18.1 | 13.1 |



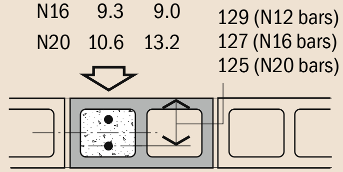

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 16.4 | 10.0 |
| N16 | 18.6 | 16.8 |
| N20 | 21.3 | 22.6 |

## VERTICALLY-REINFORCED CORES IN MID-WALL

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 6.4 | 3.7 |
| N16 | 7.6 | 6.6 |
| N20 | 9.1 | 9.7 |

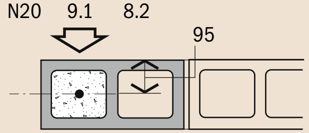



| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 8.2 | 5.2 |
| N16 | 9.3 | 9.0 |
| N20 | 10.6 | 13.2 |

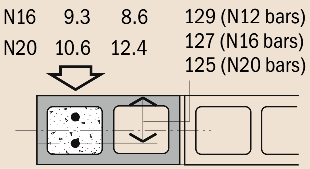



## VERTICALLY-REINFORCED CORES ADJACENT TO OPENINGS

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 6.4 | 3.6 |
| N16 | 7.6 | 6.2 |
| N20 | 9.1 | 8.2 |




| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 8.2 | 5.1 |
| N16 | 9.3 | 8.6 |
| N20 | 10.6 | 12.4 |




## NOTES

$V_c$ = Shear capacity (kN)

$M_c$ = Moment capacity (kN.m)

Wall thickness, 190 mm

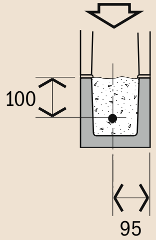
Mortar type, M3

Block characteristic compressive strength, $f'_{uc}$ = 15 MPa

Grout compressive strength, $f'_c$ = 20 MPa

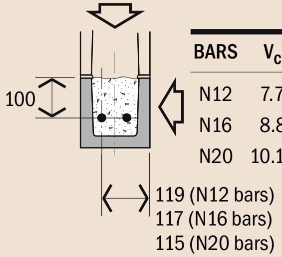
## HORIZONTALLY-REINFORCED LINTELS

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 6.4 | 3.4 |
| N16 | 7.6 | 3.6 |
| N20 | 9.1 | 3.6 |



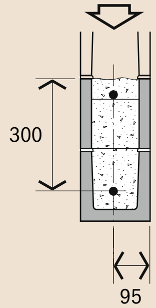

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 6.4 | 3.6 |
| N16 | 7.6 | 6.0 |
| N20 | 9.1 | 6.5 |

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 7.9 | 3.6 |
| N16 | 10.2 | 3.6 |
| N20 | 13.1 | 3.6 |



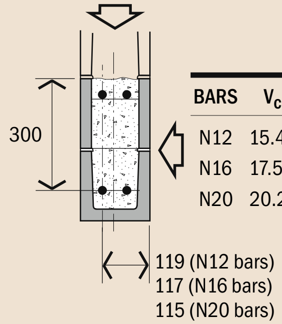

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 7.7 | 4.6 |
| N16 | 8.8 | 7.7 |
| N20 | 10.1 | 9.6 |

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 16.4 | 11.7 |
| N16 | 17.6 | 20.2 |
| N20 | 19.0 | 29.4 |



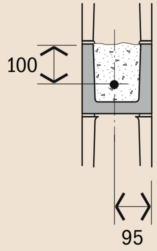

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 12.9 | 6.9 |
| N16 | 15.2 | 9.9 |
| N20 | 18.1 | 9.9 |

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 17.9 | 22.0 |
| N16 | 20.2 | 32.2 |
| N20 | 23.1 | 32.2 |

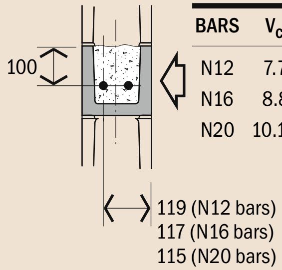



| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 15.4 | 8.9 |
| N16 | 17.5 | 14.6 |
| N20 | 20.2 | 14.6 |

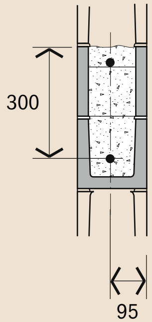
## HORIZONTALLY-REINFORCED BOND BEAMS



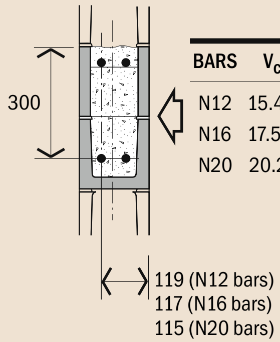

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 6.4 | 3.7 |
| N16 | 7.6 | 6.4 |
| N20 | 9.1 | 9.2 |




| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 7.7 | 4.2 |
| N16 | 8.8 | 8.0 |
| N20 | 10.1 | 11.6 |



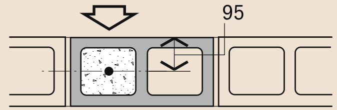

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 12.9 | 7.2 |
| N16 | 15.2 | 12.0 |
| N20 | 18.1 | 13.1 |



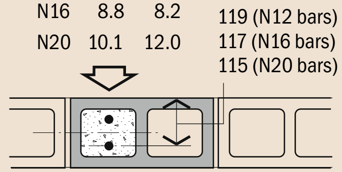

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 15.4 | 9.1 |
| N16 | 17.5 | 15.3 |
| N20 | 20.2 | 19.2 |

## VERTICALLY-REINFORCED CORES IN MID-WALL

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 6.4 | 3.7 |
| N16 | 7.6 | 6.6 |
| N20 | 9.1 | 9.7 |

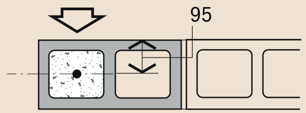



| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 7.7 | 4.7 |
| N16 | 8.8 | 8.2 |
| N20 | 10.1 | 12.0 |

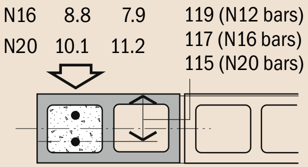



## VERTICALLY-REINFORCED CORES ADJACENT TO OPENINGS

| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 6.4 | 3.6 |
| N16 | 7.6 | 6.2 |
| N20 | 9.1 | 8.2 |




| BARS | $V_c$ | $M_c$ |
|---|---|---|
| N12 | 7.7 | 4.6 |
| N16 | 8.8 | 7.9 |
| N20 | 10.1 | 11.2 |




## NOTES

$V_c$ = Shear capacity (kN)

$M_c$ = Moment capacity (kN.m)

Wall thickness, 190 mm

Mortar type, M3

Block characteristic compressive strength, $f'_{uc}$ = 15 MPa

Grout compressive strength, $f'_c$ = 20 MPa

## REINFORCED CONCRETE MASONRY SHEAR WALLS – Shear Capacities

**SHEAR CAPACITY** (kN) **for 140-mm THICK WALL with 1-N16 BAR per END CORE**[1]

| Height of wall (m) | Length of wall (m) 0.4 | 0.6 | 0.8 | 1.0 | 1.2 | 1.4 | 1.6 | 1.8 | 2.0 | 2.2 | 2.4 | 2.6 | 2.8 | 3.0 | 3.2 | 3.4 | 3.6 | 3.8 | 4.0 | 4.2 | 4.4 | 4.6 | 4.8 | 5.0 | 5.2 | 5.4 | 5.6 | 5.8 | 6.0 | Height of wall (m) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 7.0 | 2.3 | 4.6 | 6.9 | 9.3 | 11.7 | 14.1 | 16.6 | 19.2 | 21.7 | 24.6 | 27.3 | 30.0 | 32.7 | 35.5 | 38.3 | 41.2 | 44.1 | 47.1 | 50.0 | 53.6 | 56.7 | 59.8 | 63.0 | 66.2 | 69.4 | 72.7 | 76.0 | 79.4 | 82.8 | 7.0 |
| 6.0 | 2.7 | 5.3 | 8.0 | 10.7 | 13.5 | 16.3 | 19.1 | 22.0 | 24.9 | 28.2 | 31.2 | 34.3 | 37.4 | 40.5 | 43.7 | 46.9 | 50.2 | 53.5 | 56.9 | 60.8 | 64.2 | 67.7 | 71.2 | 74.8 | 78.4 | 82.1 | 85.7 | 89.5 | 93.2 | 6.0 |
| 5.0 | 3.2 | 6.3 | 9.5 | 12.7 | 16.0 | 19.3 | 22.6 | 26.0 | 29.4 | 33.2 | 36.7 | 40.3 | 43.9 | 47.5 | 51.2 | 55.0 | 58.7 | 62.5 | 66.4 | 70.8 | 74.7 | 78.7 | 82.8 | 86.8 | 91.0 | 95.1 | 99.3 | 103.5 | 107.8 | 5.0 |
| 4.0 | 3.9 | 7.8 | 11.7 | 15.7 | 19.7 | 23.8 | 27.9 | 32.0 | 36.2 | 40.7 | 45.0 | 49.3 | 53.7 | 58.1 | 62.5 | 67.0 | 71.5 | 76.1 | 80.7 | 85.8 | 90.5 | 95.3 | 100.1 | 104.9 | 109.8 | 114.7 | 119.6 | 124.6 | 129.6 | 4.0 |
| 3.0 | 5.2 | 10.3 | 15.5 | 20.7 | 26.0 | 31.3 | 36.6 | 42.0 | 47.5 | 53.2 | 58.7 | 64.3 | 69.9 | 75.6 | 81.3 | 87.0 | 92.8 | 98.6 | 104.4 | 110.9 | 116.8 | 122.8 | 128.9 | 134.9 | 141.1 | 147.2 | 153.4 | 159.7 | 165.9 | 3.0 |
| 2.7 | 5.7 | 11.4 | 17.2 | 22.9 | 28.2 | 34.6 | 40.5 | 46.5 | 52.5 | 58.8 | 64.8 | 71.0 | 77.1 | 83.4 | 89.6 | 95.9 | 102.2 | 108.6 | 115.0 | 122.0 | 128.5 | 135.0 | 141.6 | 148.3 | 155.0 | 161.7 | 168.4 | 175.2 | 182.1 | 2.7 |

1 Remainder of wall reinforced with 1 vertical N16 at 2.0 m centres and 1 horizontal N16 at 3.0 m centres. See Part C:Chapter 3, Detail G1 for details.

**SHEAR CAPACITY** (kN) **for 190-mm THICK WALL with 1-N20 BAR per END CORE**[1]

| Height of wall (m) | Length of wall (m) 0.4 | 0.6 | 0.8 | 1.0 | 1.2 | 1.4 | 1.6 | 1.8 | 2.0 | 2.2 | 2.4 | 2.6 | 2.8 | 3.0 | 3.2 | 3.4 | 3.6 | 3.8 | 4.0 | 4.2 | 4.4 | 4.6 | 4.8 | 5.0 | 5.2 | 5.4 | 5.6 | 5.8 | 6.0 | Height of wall (m) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 7.0 | 3.5 | 7.1 | 10.6 | 14.2 | 17.9 | 21.6 | 25.4 | 29.2 | 33.0 | 37.3 | 41.3 | 45.3 | 49.4 | 53.6 | 57.8 | 62.0 | 66.3 | 70.6 | 75.0 | 80.2 | 84.7 | 89.3 | 93.9 | 98.6 | 103.3 | 108.1 | 112.9 | 117.8 | 122.7 | 7.0 |
| 6.0 | 4.1 | 8.2 | 12.3 | 16.4 | 20.7 | 24.9 | 29.2 | 33.6 | 38.0 | 42.9 | 47.4 | 52.0 | 56.7 | 61.4 | 66.1 | 70.9 | 75.7 | 80.6 | 85.6 | 91.3 | 96.4 | 101.5 | 106.7 | 112.0 | 117.2 | 122.6 | 128.0 | 133.4 | 138.9 | 6.0 |
| 5.0 | 4.9 | 9.7 | 14.6 | 19.5 | 24.5 | 29.6 | 34.7 | 39.8 | 45.0 | 50.6 | 55.9 | 61.3 | 66.8 | 72.2 | 77.8 | 83.3 | 89.0 | 94.6 | 100.4 | 106.9 | 112.7 | 118.6 | 124.6 | 130.6 | 136.7 | 142.8 | 149.0 | 155.2 | 161.4 | 5.0 |
| 4.0 | 6.0 | 12.0 | 18.1 | 24.2 | 30.4 | 36.6 | 39.9 | 47.8 | 55.5 | 62.3 | 68.8 | 75.3 | 81.9 | 88.5 | 95.2 | 102.0 | 108.8 | 115.6 | 122.5 | 130.2 | 137.2 | 144.3 | 151.4 | 158.6 | 165.8 | 173.1 | 180.4 | 187.8 | 195.2 | 4.0 |
| 3.0 | 8.0 | 15.9 | 23.9 | 30.3 | 33.6 | 44.9 | 56.4 | 64.6 | 72.9 | 81.6 | 90.1 | 98.6 | 107.1 | 115.7 | 124.3 | 133.0 | 141.7 | 150.5 | 159.3 | 169.0 | 177.9 | 187.0 | 196.0 | 205.1 | 214.3 | 223.5 | 232.8 | 242.1 | 251.5 | 3.0 |
| 2.7 | 8.8 | 17.6 | 26.5 | 30.4 | 36.0 | 53.4 | 62.4 | 71.5 | 80.7 | 90.3 | 99.6 | 108.9 | 118.3 | 127.7 | 137.2 | 146.8 | 156.4 | 166.0 | 175.7 | 186.2 | 196.0 | 205.9 | 215.8 | 225.8 | 235.9 | 245.9 | 256.1 | 266.3 | 276.5 | 2.7 |

1 Remainder of wall reinforced with 1 vertical N20 at 2.0 m centres and 1 horizontal N16 at 3.0 m centres. See Part C:Chapter 3, Detail G2 for details.

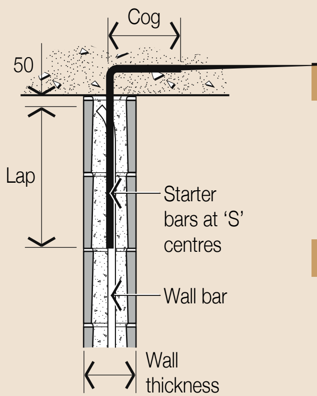
### SHEAR CAPACITY (kN) for 190-mm THICK WALL with 2-N20 BARS per END CORE[1]

| Height of wall (m) | Length of wall (m) 0.4 | 0.6 | 0.8 | 1.0 | 1.2 | 1.4 | 1.6 | 1.8 | 2.0 | 2.2 | 2.4 | 2.6 | 2.8 | 3.0 | 3.2 | 3.4 | 3.6 | 3.8 | 4.0 | 4.2 | 4.4 | 4.6 | 4.8 | 5.0 | 5.2 | 5.4 | 5.6 | 5.8 | 6.0 | Height of wall (m) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 7.0 | 3.9 | 12.8 | 20.6 | 27.5 | 33.0 | 36.2 | 39.5 | 42.7 | 45.9 | 56.0 | 59.2 | 62.4 | 65.6 | 68.9 | 83.5 | 100.8 | 117.7 | 130.4 | 138.1 | 146.6 | 154.5 | 162.4 | 170.3 | 178.3 | 186.4 | 194.5 | 202.6 | 210.8 | 219.1 | 7.0 |
| 6.0 | 4.6 | 14.8 | 23.9 | 29.8 | 33.1 | 36.3 | 39.6 | 42.8 | 46.0 | 56.1 | 59.3 | 62.6 | 80.3 | 98.1 | 115.3 | 132.1 | 141.6 | 150.4 | 159.2 | 168.8 | 177.8 | 186.8 | 195.9 | 205.0 | 214.1 | 223.3 | 232.6 | 241.9 | 251.2 | 6.0 |
| 5.0 | 5.4 | 17.7 | 26.7 | 29.9 | 33.2 | 36.4 | 39.7 | 42.9 | 46.2 | 58.4 | 77.6 | 95.9 | 113.5 | 130.6 | 147.2 | 157.7 | 168.0 | 178.3 | 188.7 | 199.9 | 210.4 | 220.9 | 231.6 | 242.2 | 252.9 | 263.7 | 274.5 | 285.4 | 296.3 | 5.0 |
| 4.0 | 6.7 | 22.1 | 26.8 | 30.1 | 33.3 | 36.6 | 39.9 | 47.8 | 65.8 | 94.5 | 112.6 | 130.0 | 146.8 | 163.2 | 179.3 | 195.0 | 207.6 | 220.2 | 232.9 | 246.4 | 259.3 | 272.1 | 285.1 | 298.1 | 311.1 | 324.2 | 337.4 | 350.6 | 363.8 | 4.0 |
| 3.0 | 8.9 | 23.6 | 27.0 | 30.3 | 33.6 | 44.9 | 63.8 | 81.6 | 98.5 | 130.8 | 147.8 | 164.3 | 180.4 | 196.1 | 211.6 | 226.9 | 242.0 | 257.0 | 306.6 | 324.0 | 340.7 | 357.5 | 374.3 | 391.1 | 408.1 | 425.0 | 442.1 | 459.1 | 476.3 | 3.0 |
| 2.7 | 9.8 | 23.7 | 27.0 | 30.4 | 36.0 | 56.0 | 74.4 | 91.8 | 108.4 | 141.7 | 158.4 | 174.7 | 190.5 | 206.1 | 221.4 | 236.6 | 302.8 | 321.0 | 339.3 | 358.4 | 376.9 | 395.4 | 413.9 | 432.5 | 451.1 | 469.8 | 488.6 | 507.4 | 526.2 | 2.7 |

[1] Remainder of wall reinforced with 1 vertical N20 at 2.0 m centres and 1 horizontal N16 at 3.0 m centres. See Part C:Chapter 3, Detail G3 for details.

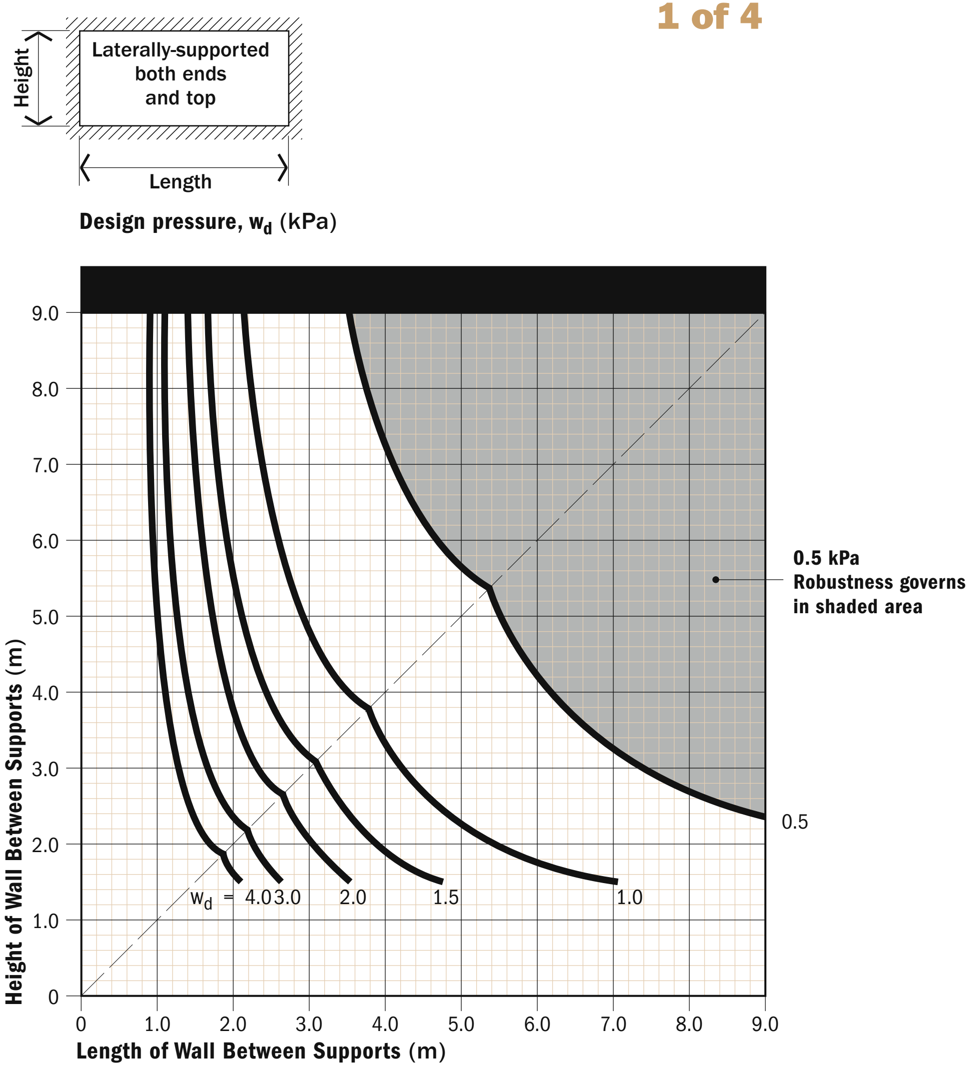
### OUT-OF-PLANE SHEAR CAPACITY OF STARTER-BAR CONNECTIONS – kN/connection and (kN/metre length of wall)

Arrangement[2]

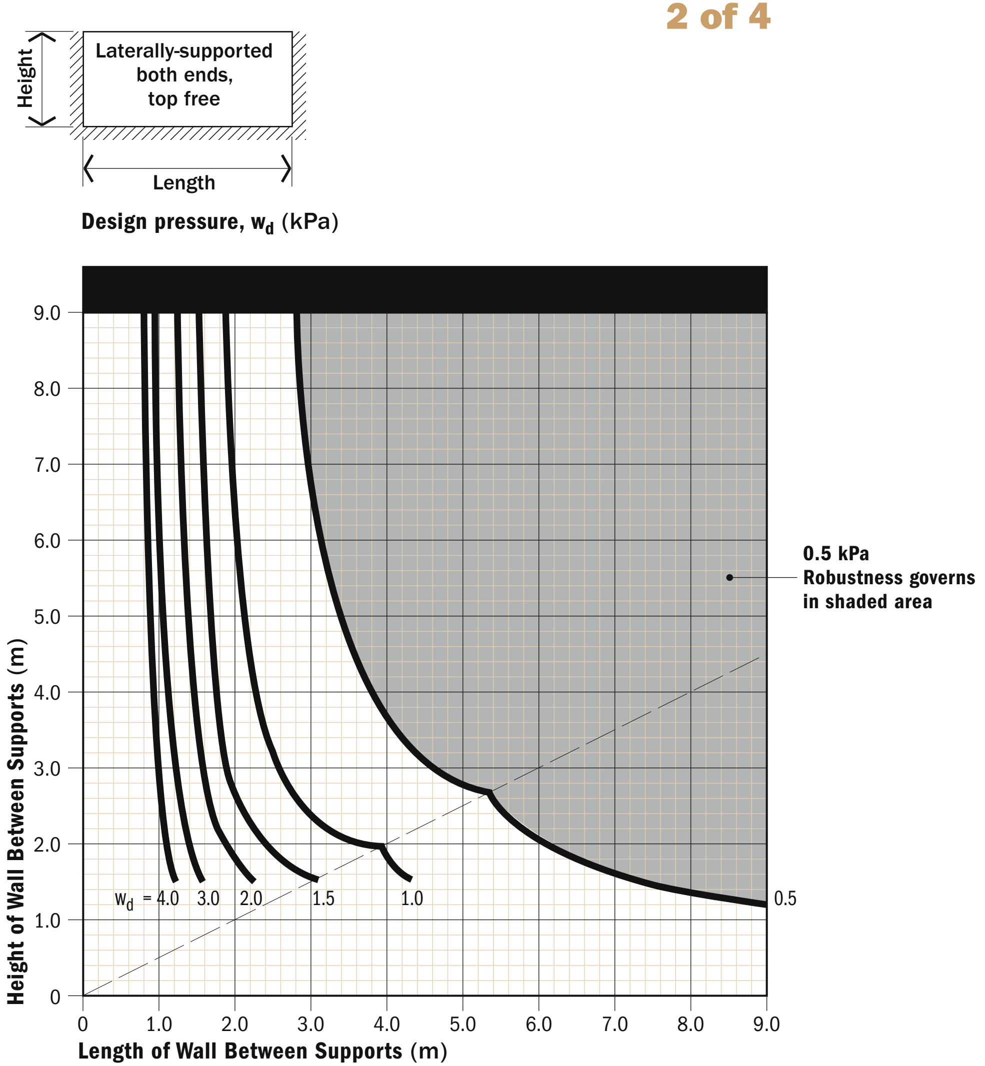



| Centres, S (m) | Details of connection: 1-N12 starter bar Cog 200/lap 450 | | 1-N16 starter bar Cog 240/lap 700 | | 1-N20 starter bar Cog 290/lap 1000 | |
|---|---|---|---|---|---|---|
| **190-mm hollow blockwork** | | | | | | |
| 2.0 | 29.1 | (14.5) | 30.2 | (15.1) | 31.6 | (15.8) |
| 1.6 | 24.0 | (15.0) | 25.2 | (15.7) | 26.6 | (16.6) |
| 1.2 | 19.0 | (15.8) | 20.1 | (16.8) | 21.5 | (20.6) |
| 0.8 | 14.0 | (17.4) | 15.1 | (18.9) | 16.5 | (20.6) |
| **140-mm hollow blockwork** | | | | | | |
| 2.0 | 24.0 | (12.0) | 25.1 | (12.6) | 26.5 | (13.3) |
| 1.6 | 19.8 | (12.4) | 20.9 | (13.1) | 22.3 | (14.0) |
| 1.2 | 15.6 | (13.0) | 16.7 | (13.9) | 18.1 | (15.1) |
| 0.8 | 11.4 | (14.2) | 12.5 | (15.7) | 13.9 | (17.4) |

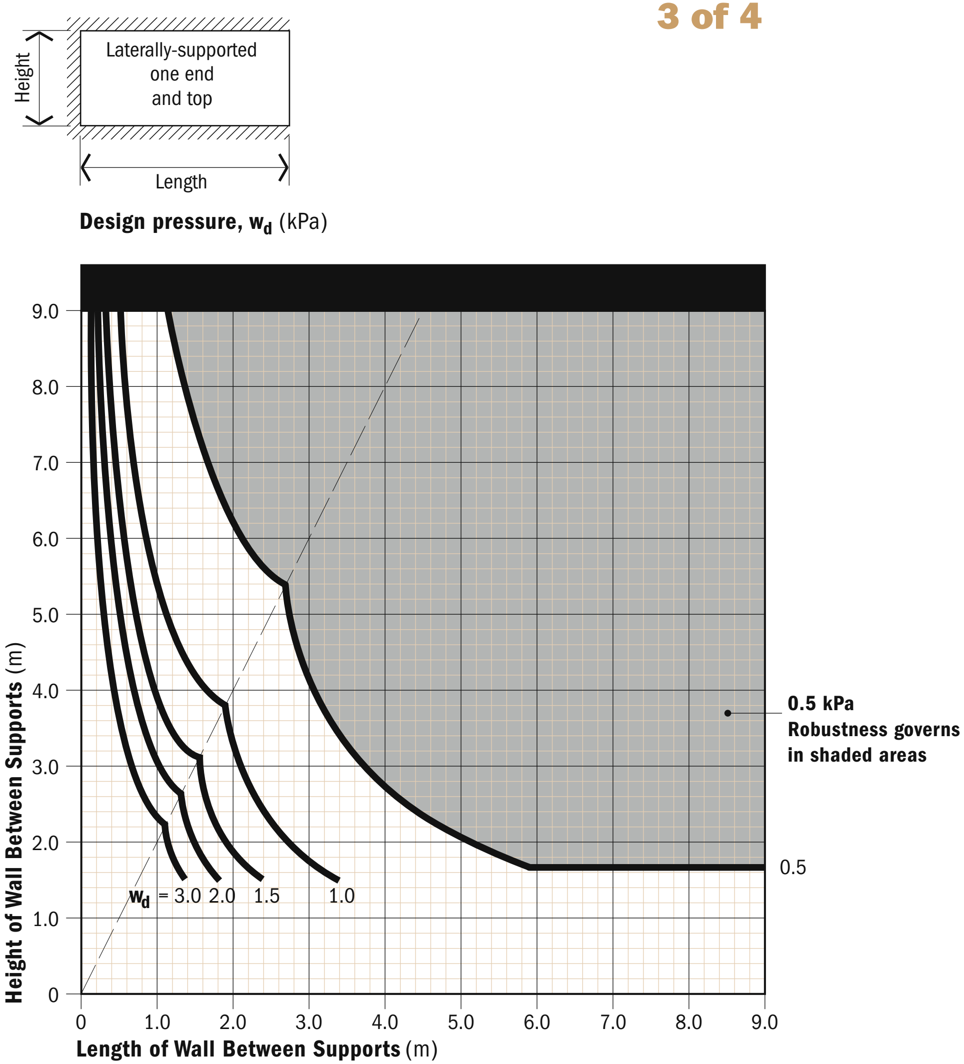
[2] For actual details of the connections given above see Part C:Chapter 3, Details H1, H2 and H3 for details.



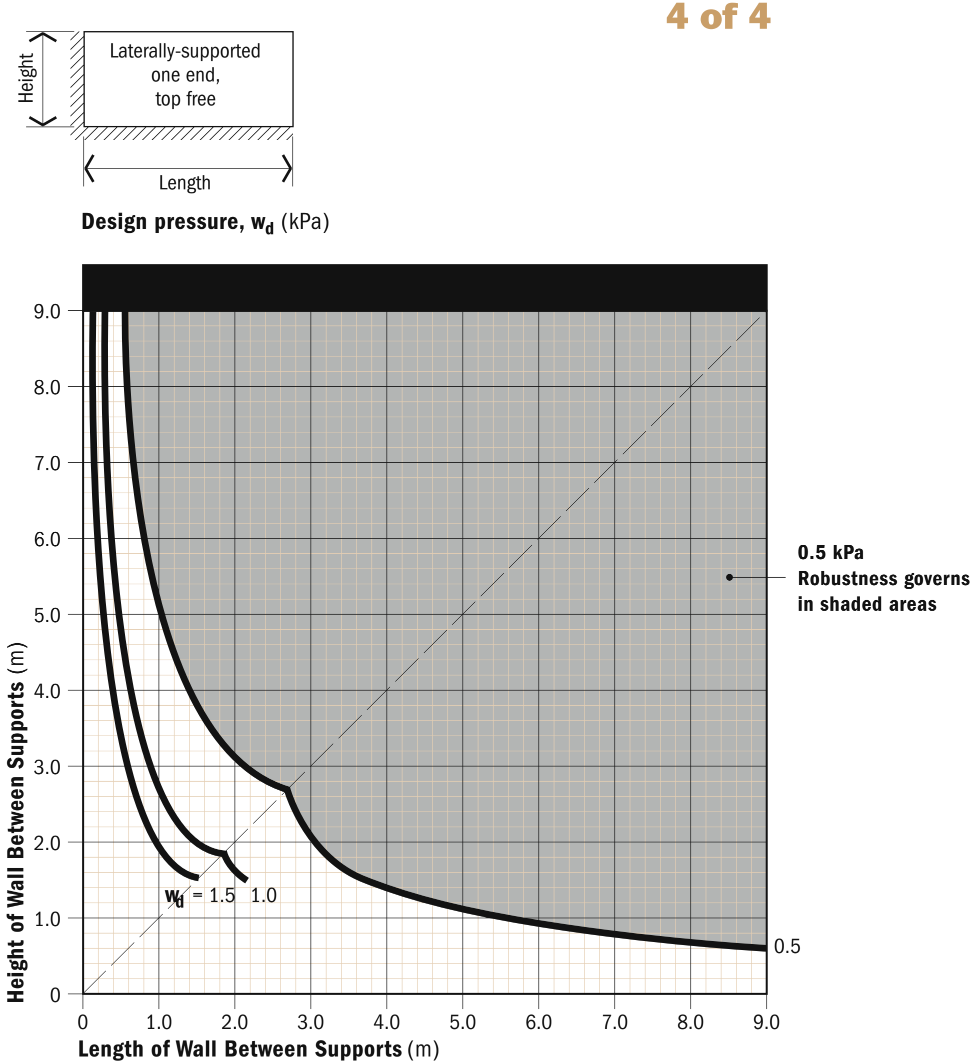



NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties





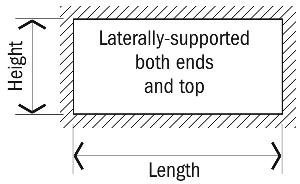




NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties

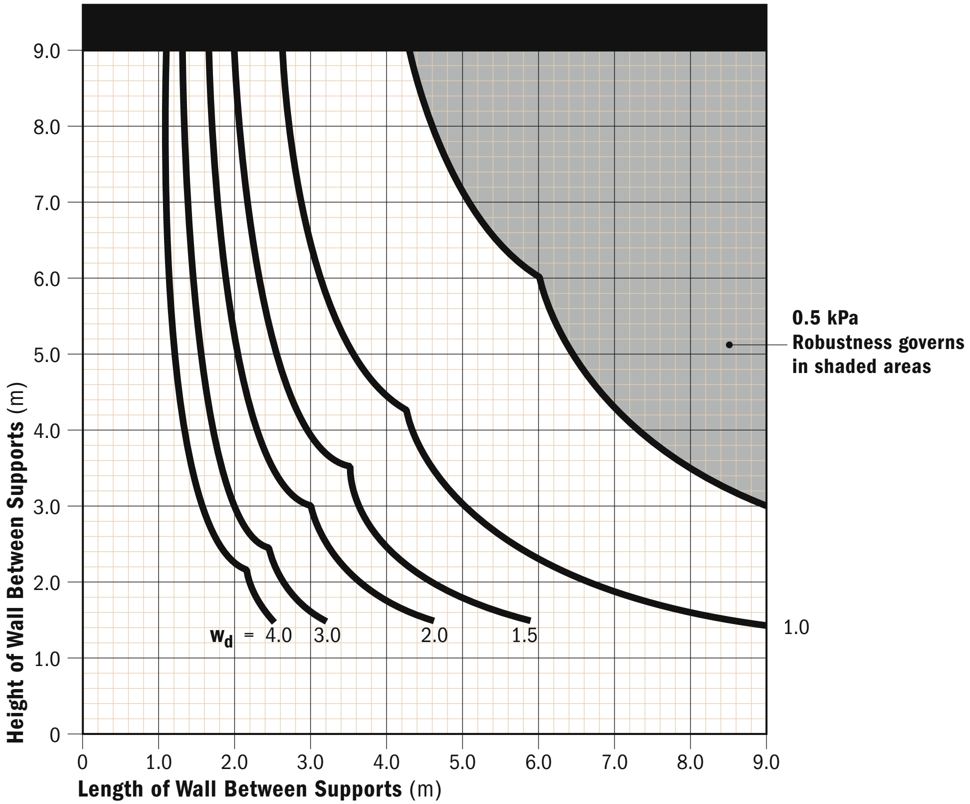
## UNREINFORCED MASONRY – without openings

## 110-mm leaf (390 x 190 units 25 mm face-shell bedded)

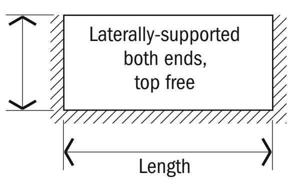





**Design pressure, $w_d$** (kPa)



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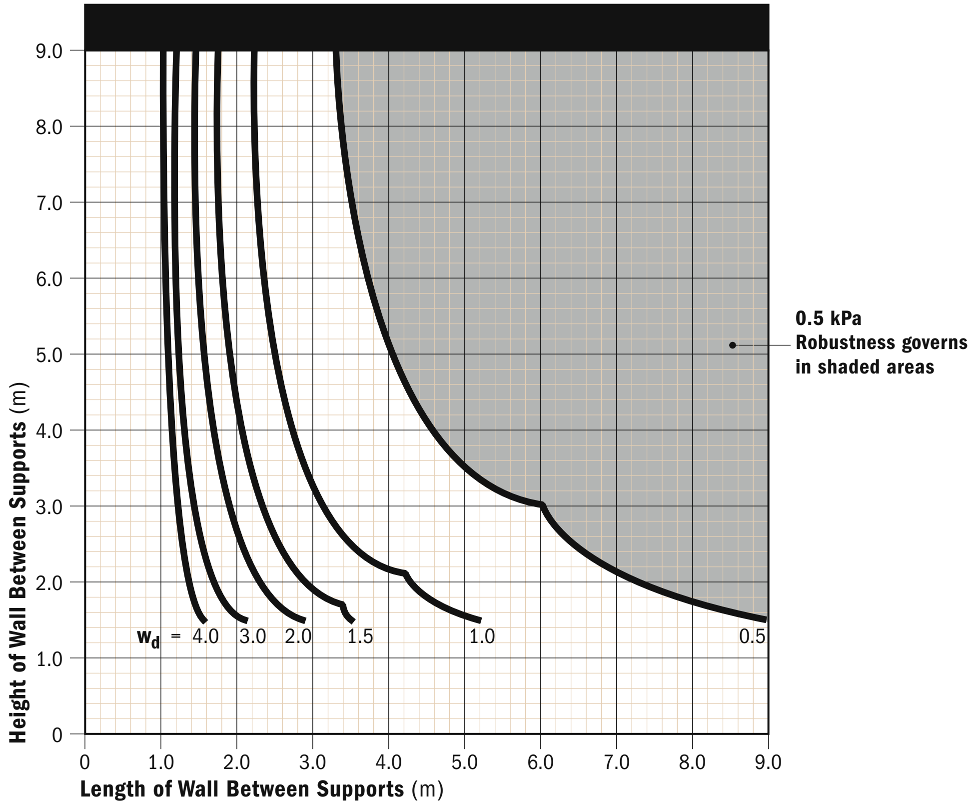
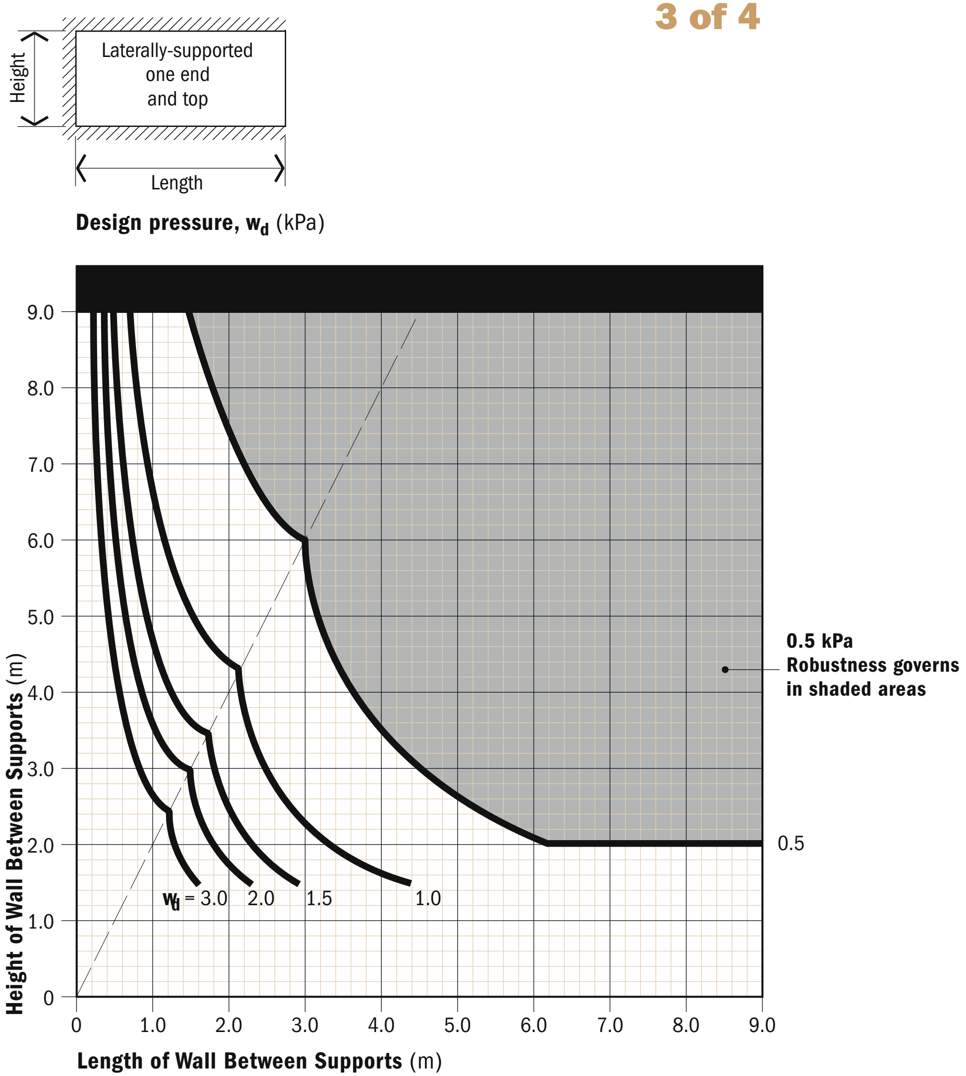
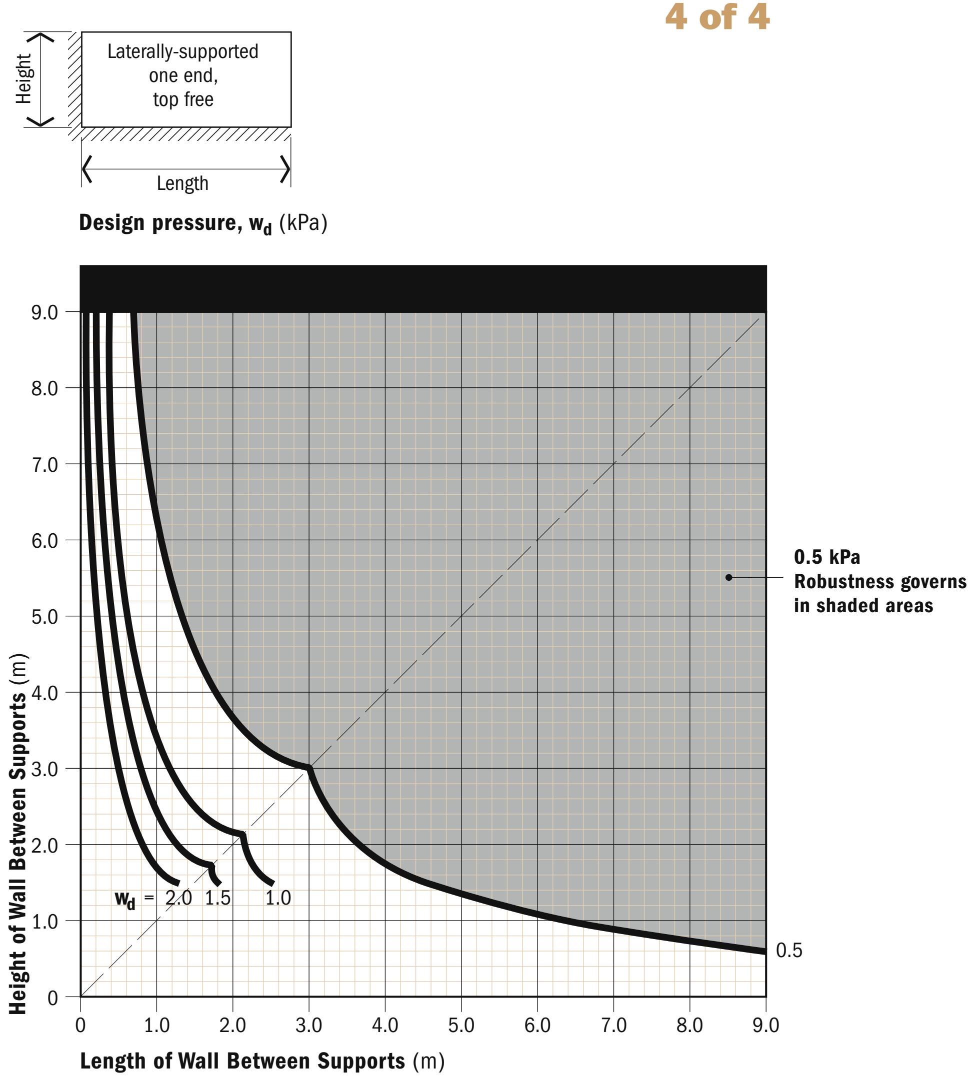


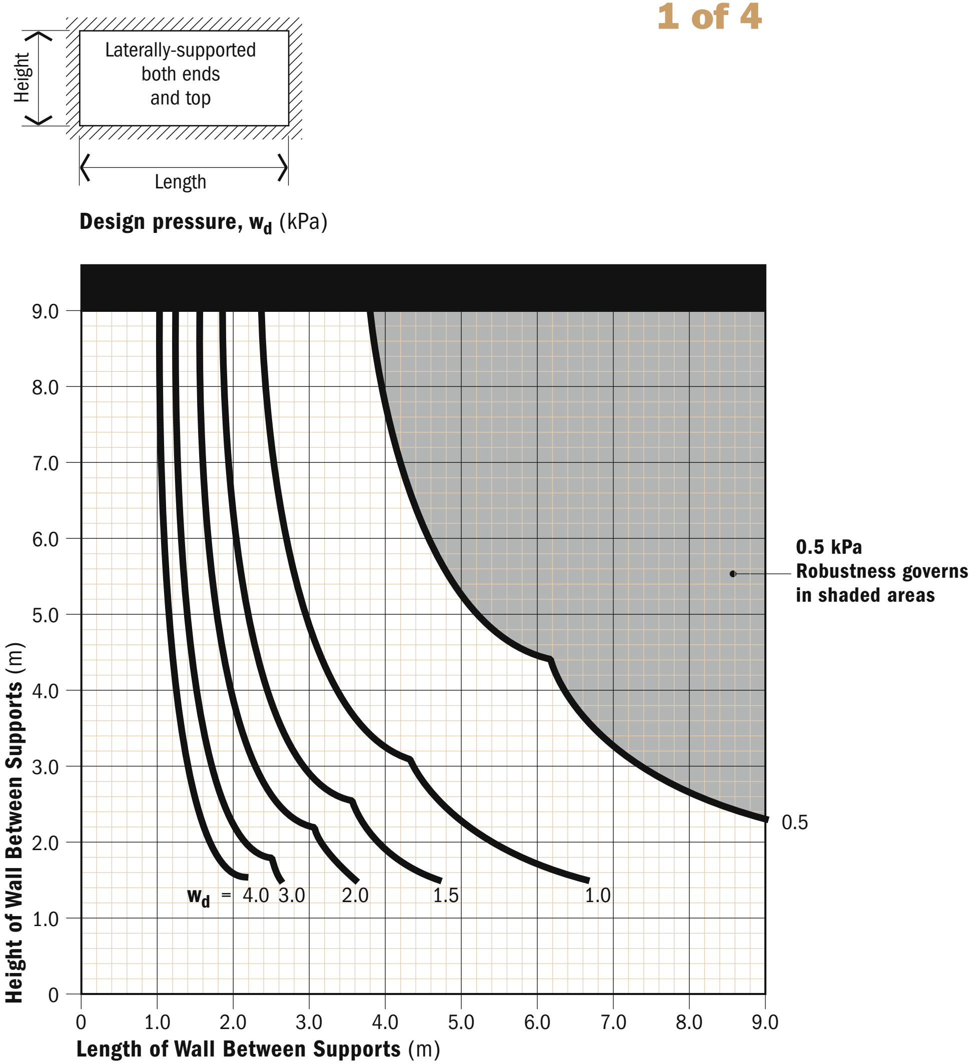

NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties



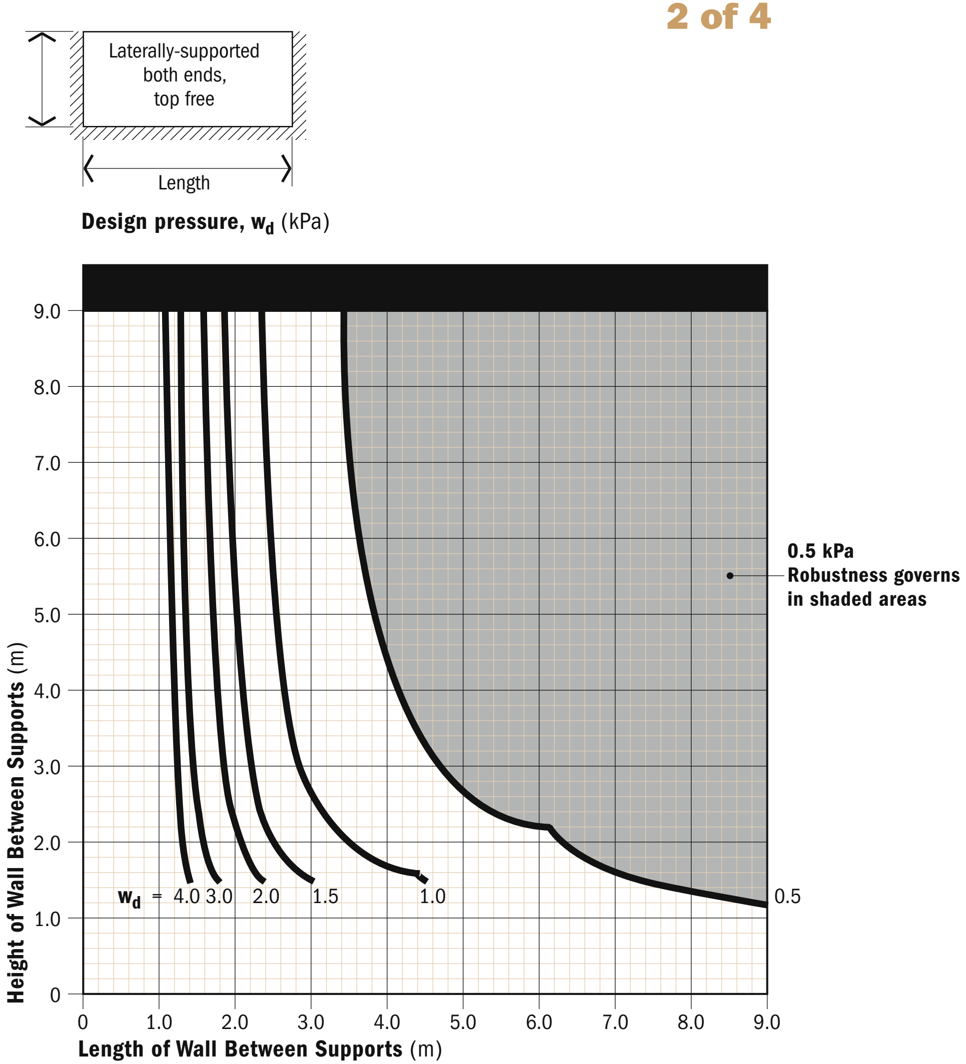


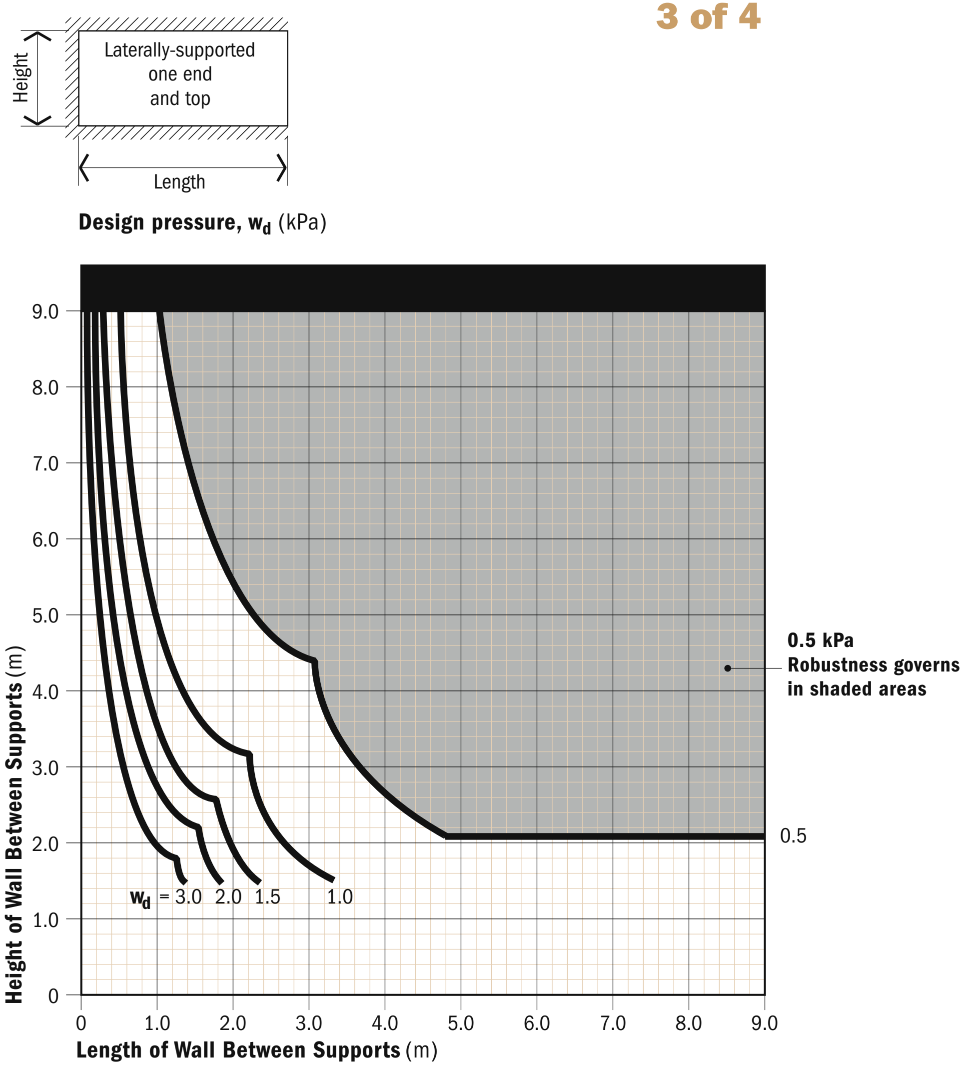

NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties



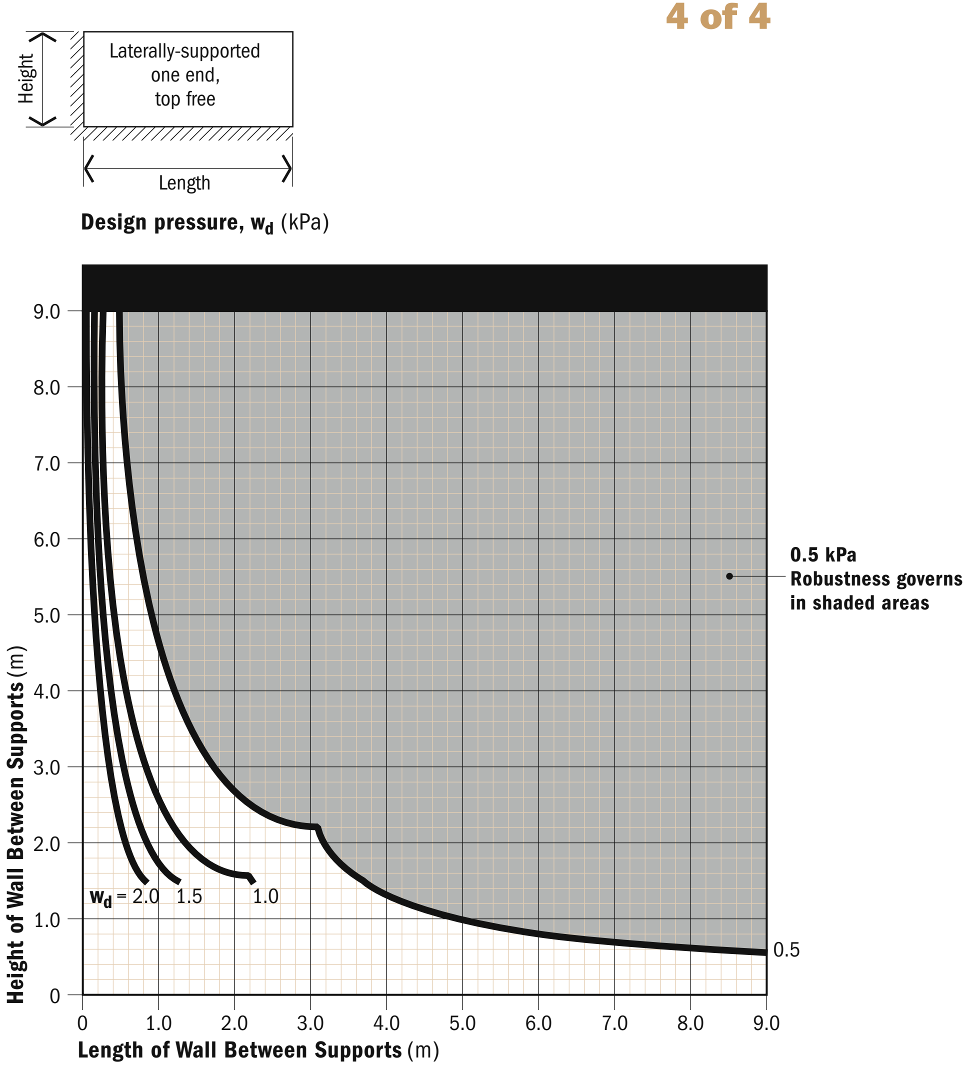


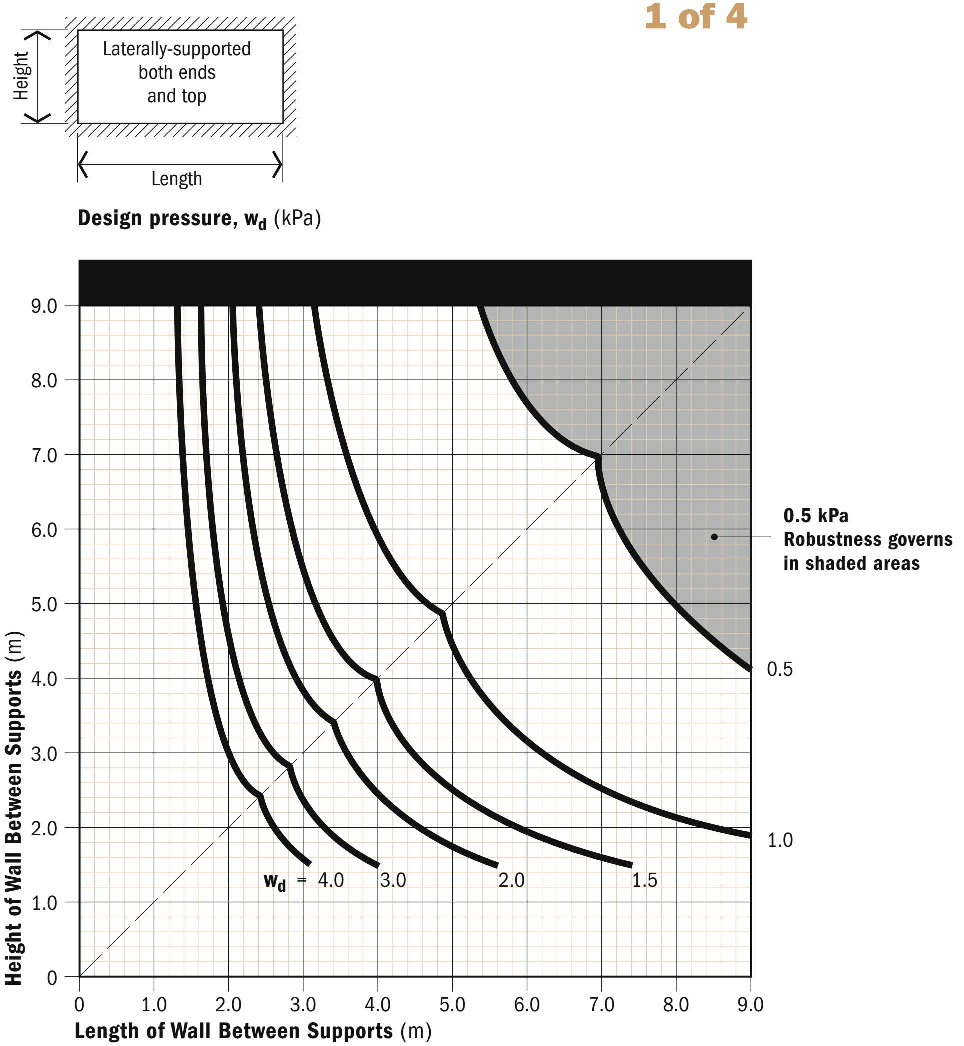

NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties



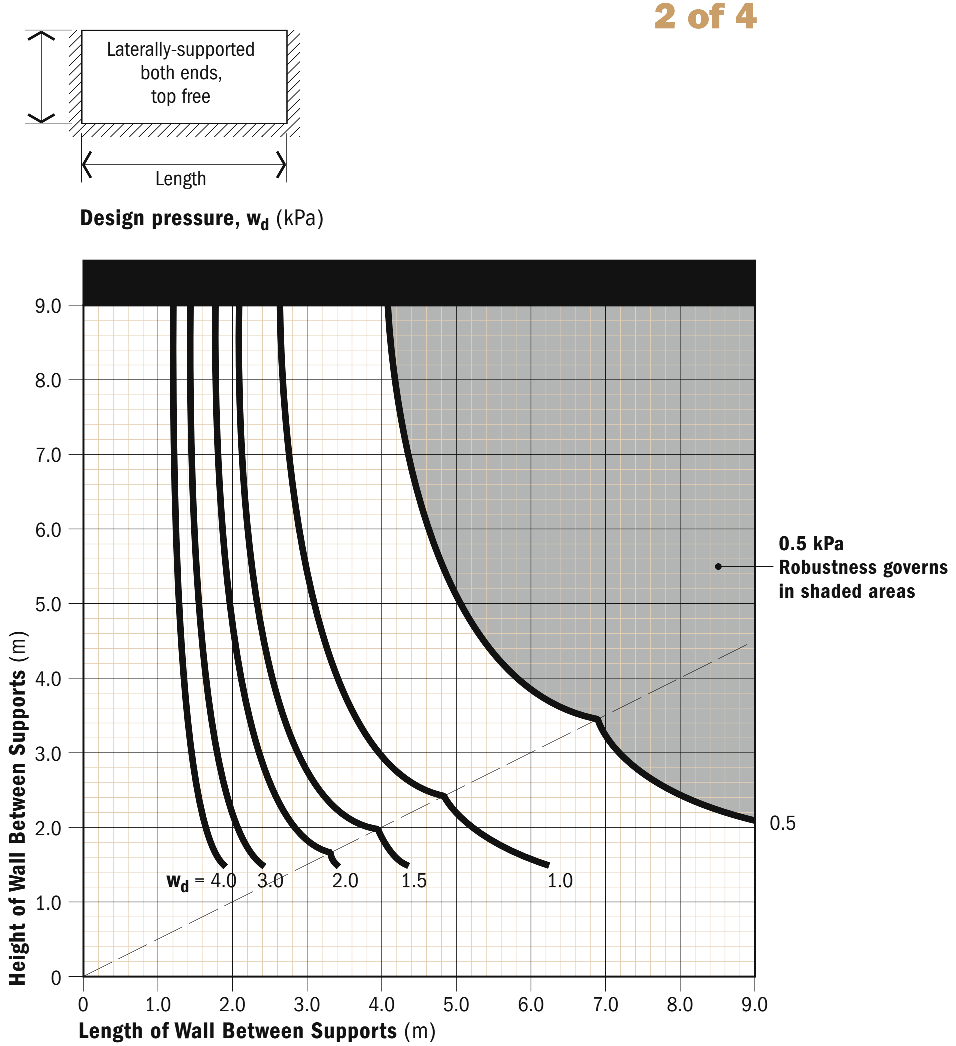


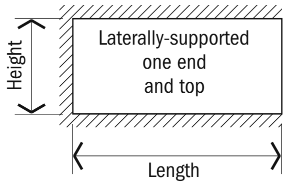

NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties



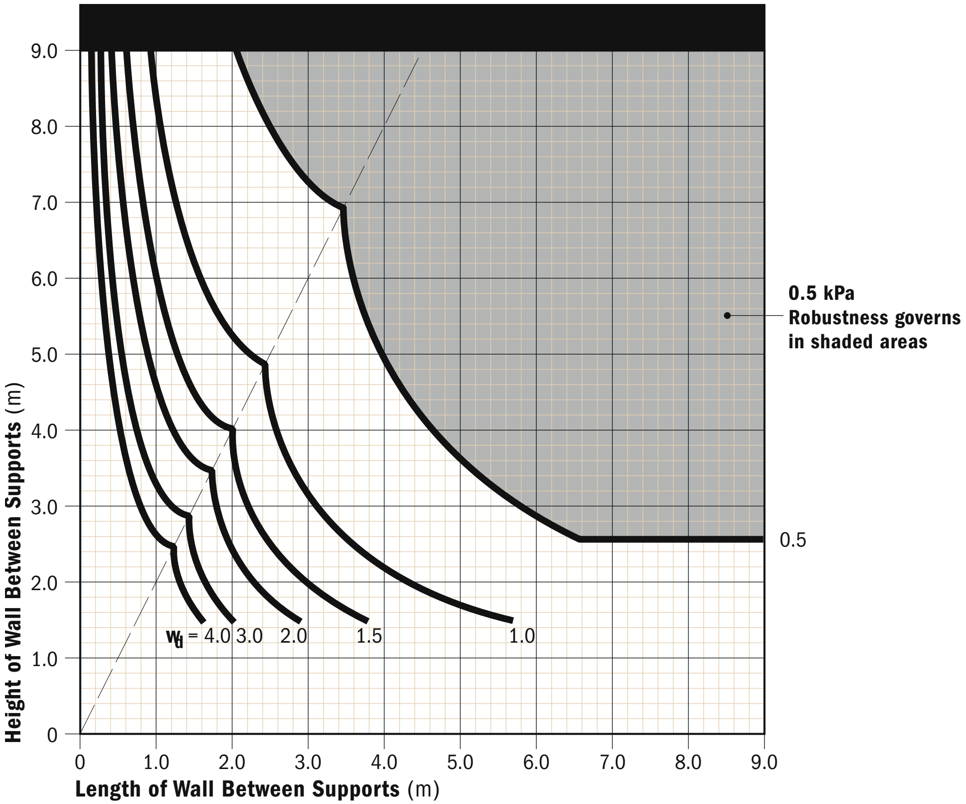


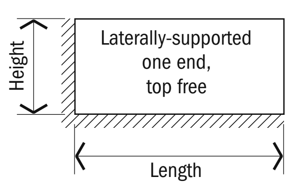

NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties




**Design pressure, $w_d$** (kPa)



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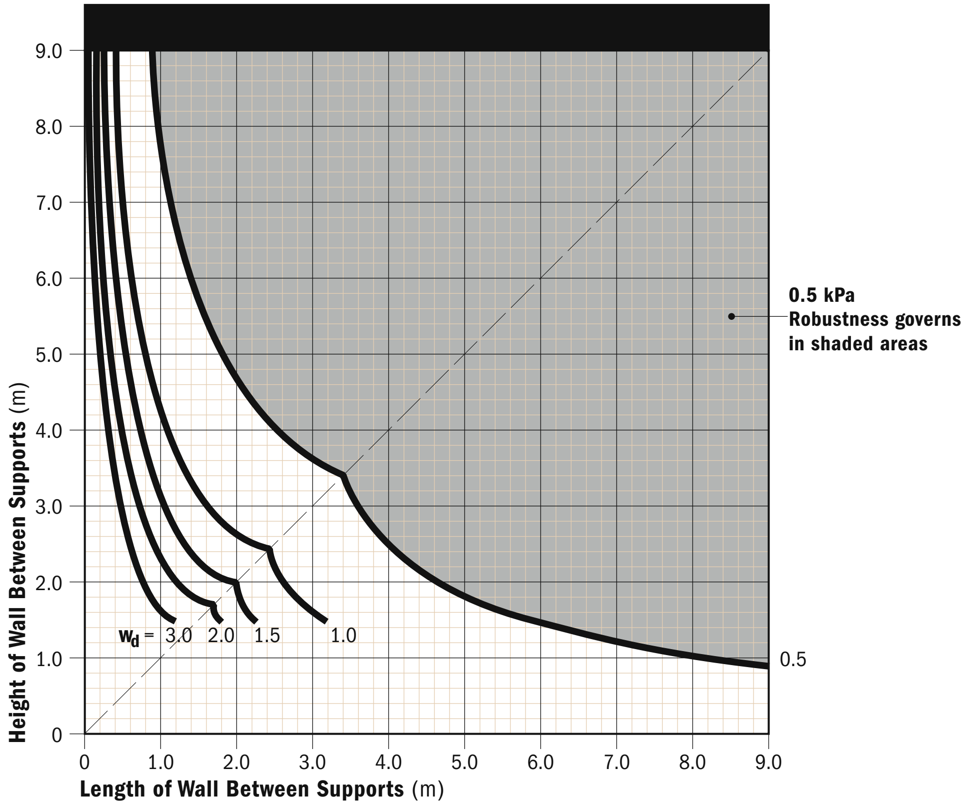
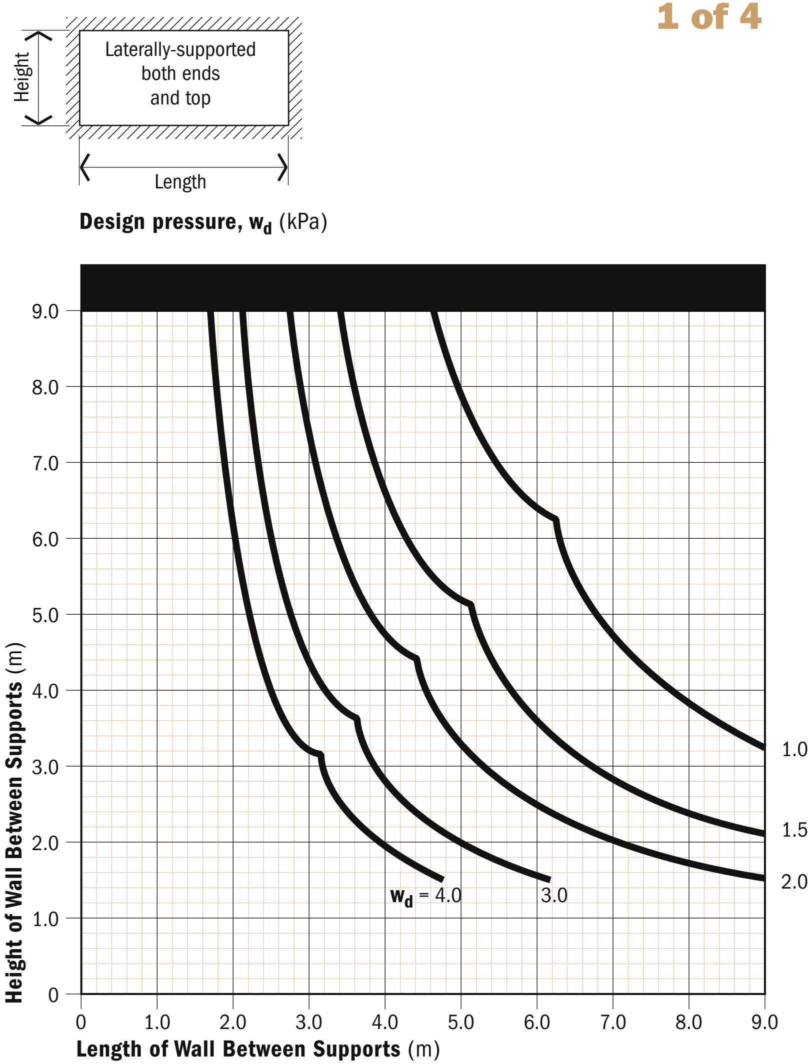
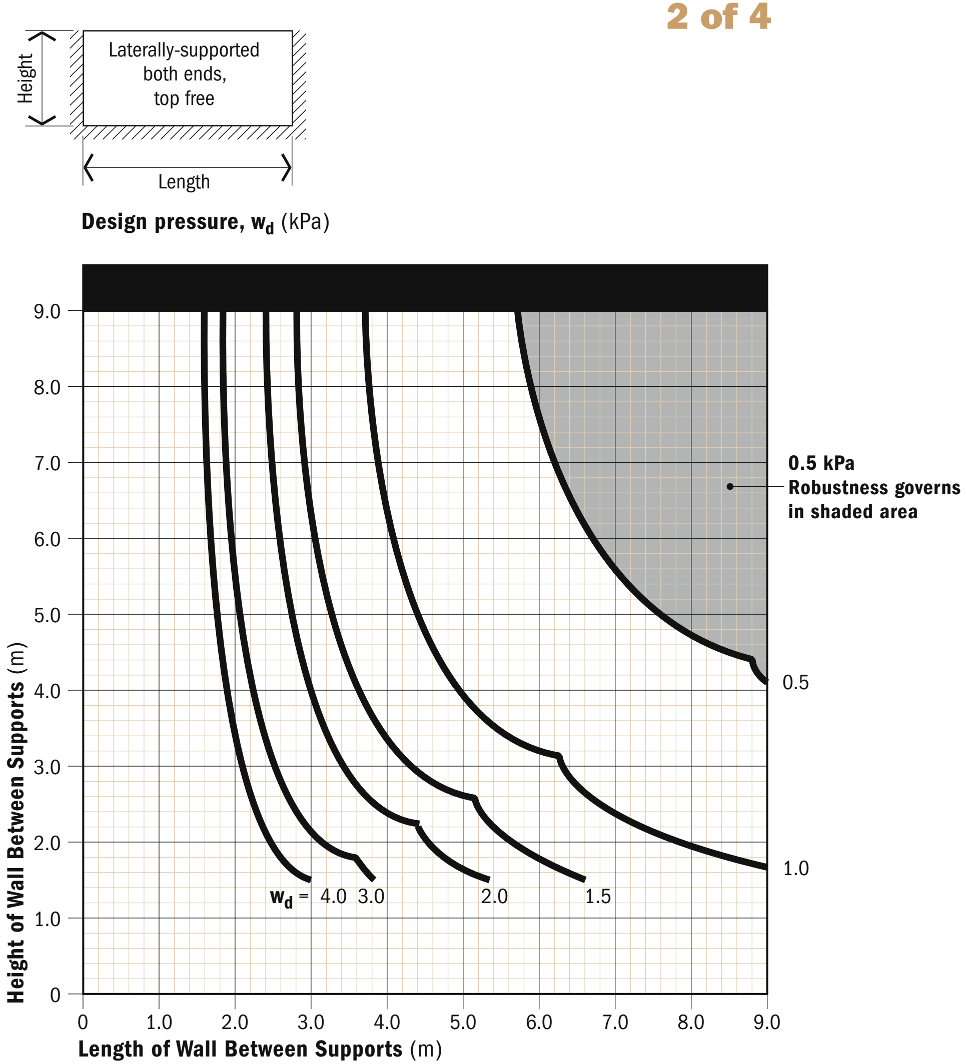


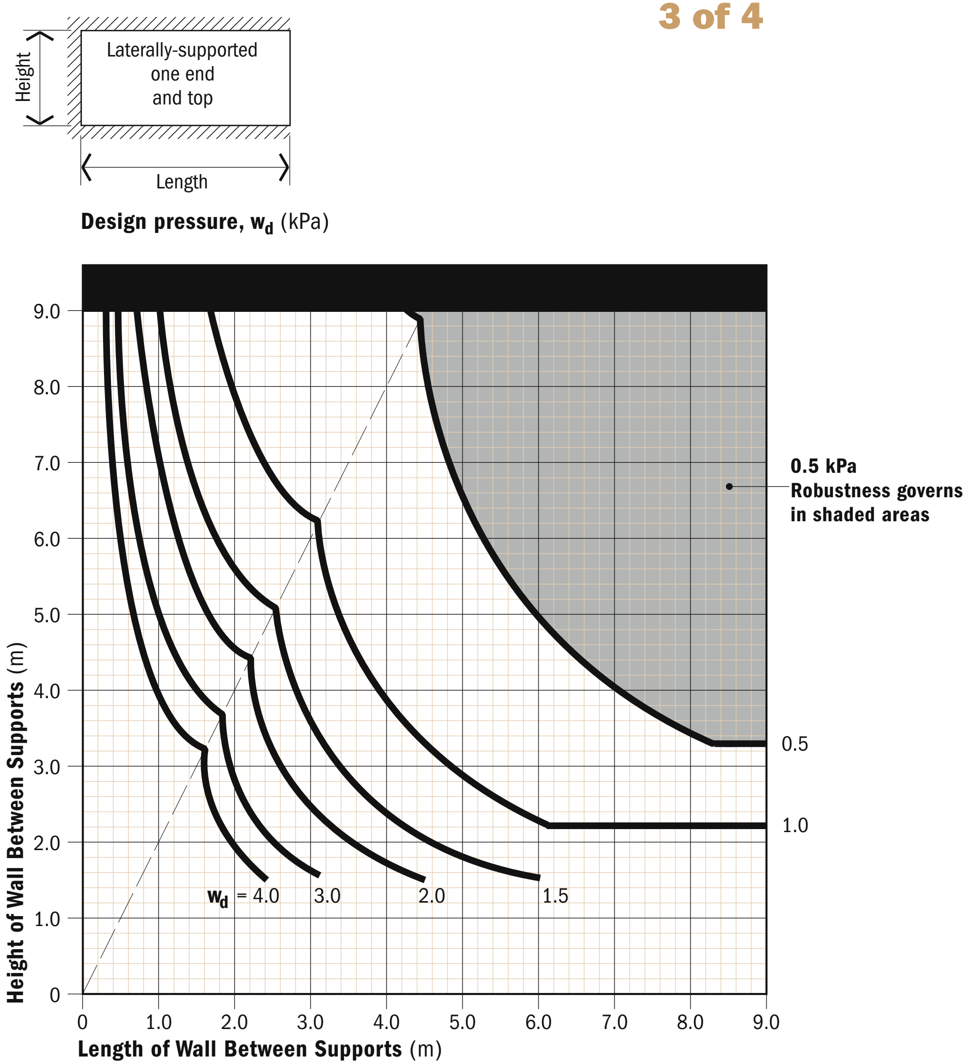

NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties



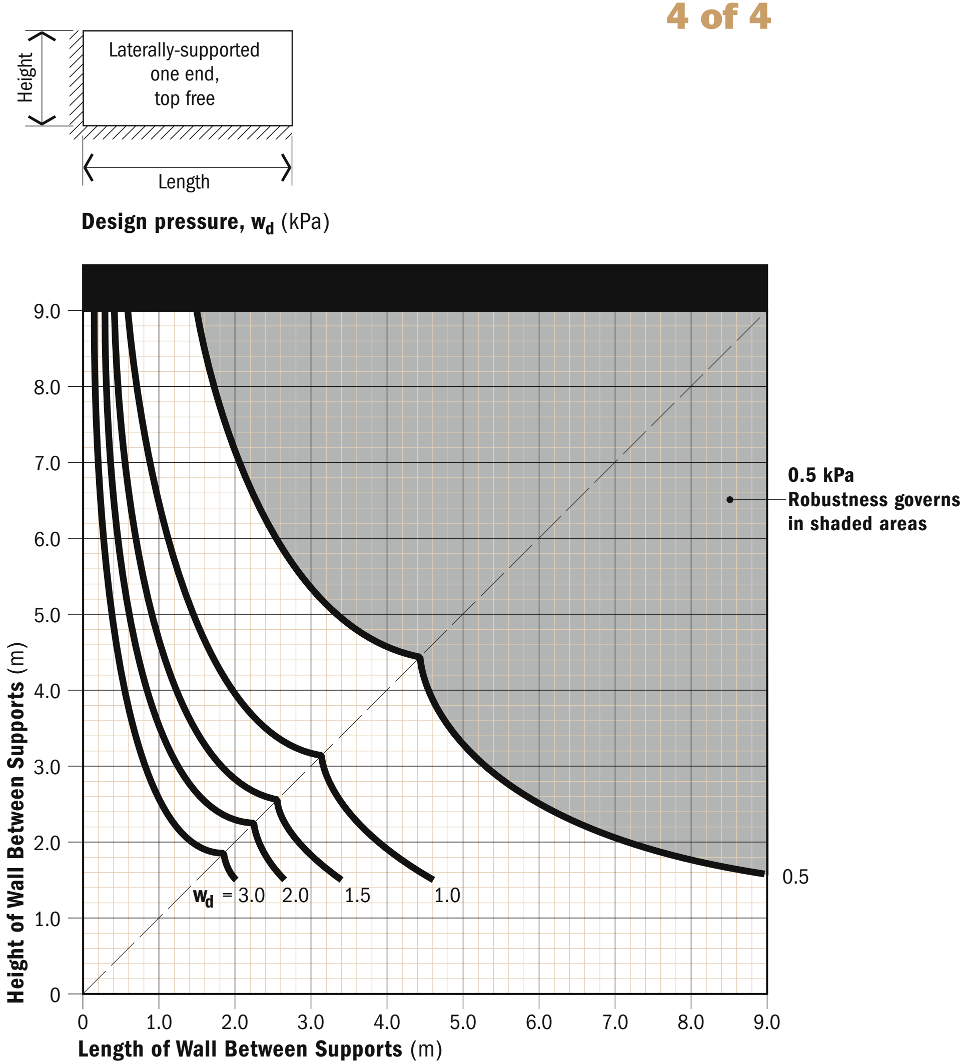



NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties



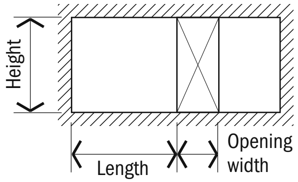


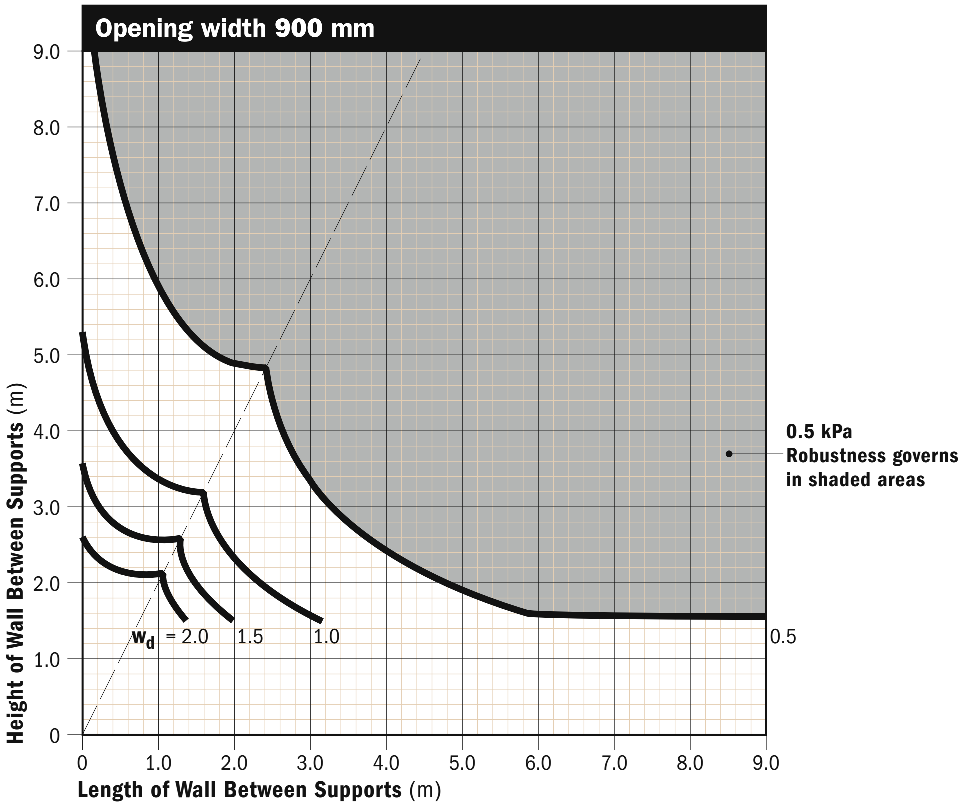

NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties

Laterally-supported both ends and top



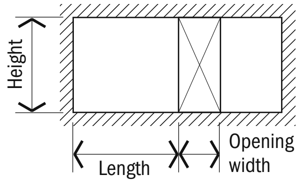



**Design pressure, $w_d$** (kPa)

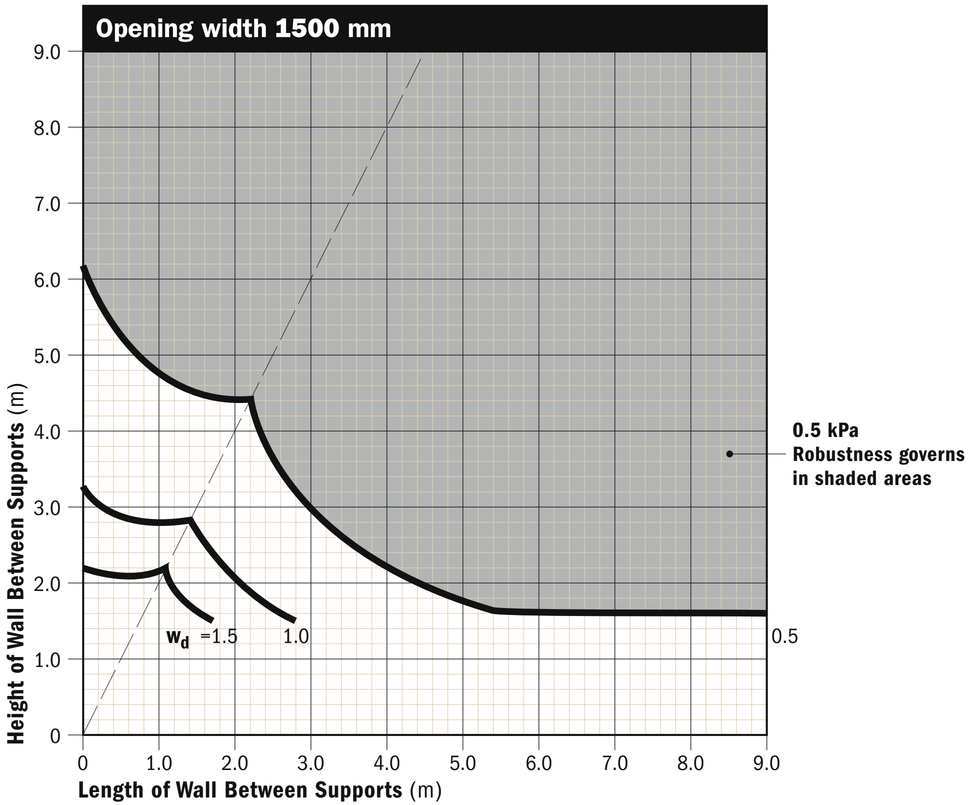



Laterally-supported both ends and top





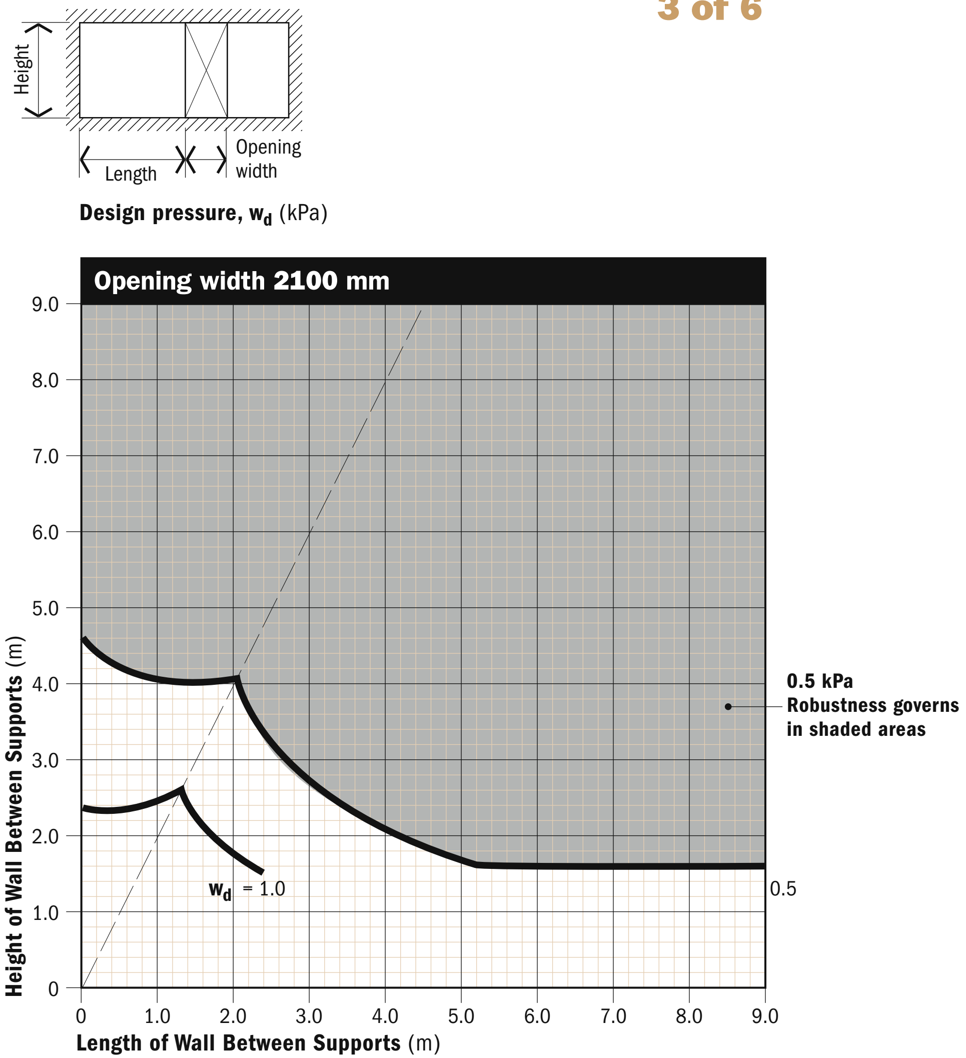

**Design pressure, $w_d$** (kPa)




NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties

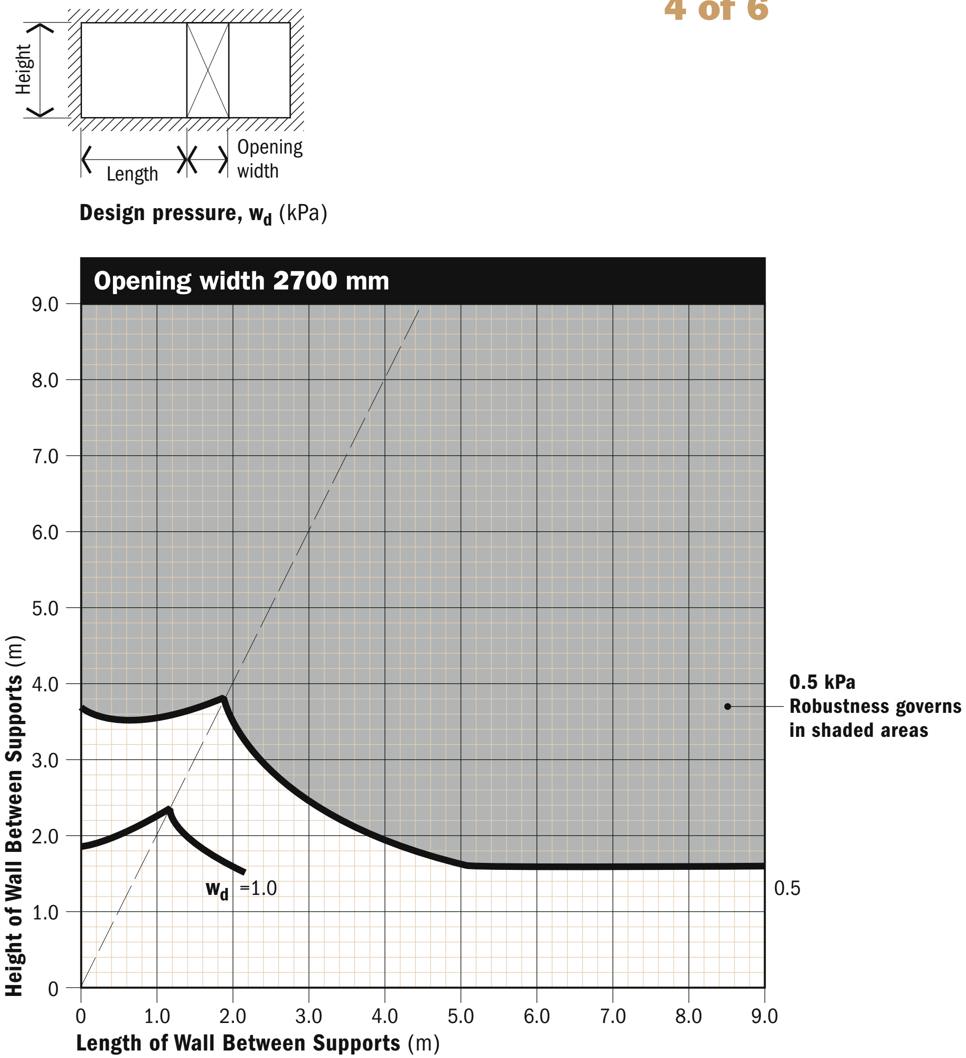
Laterally-supported both ends and top






Laterally-supported both ends and top

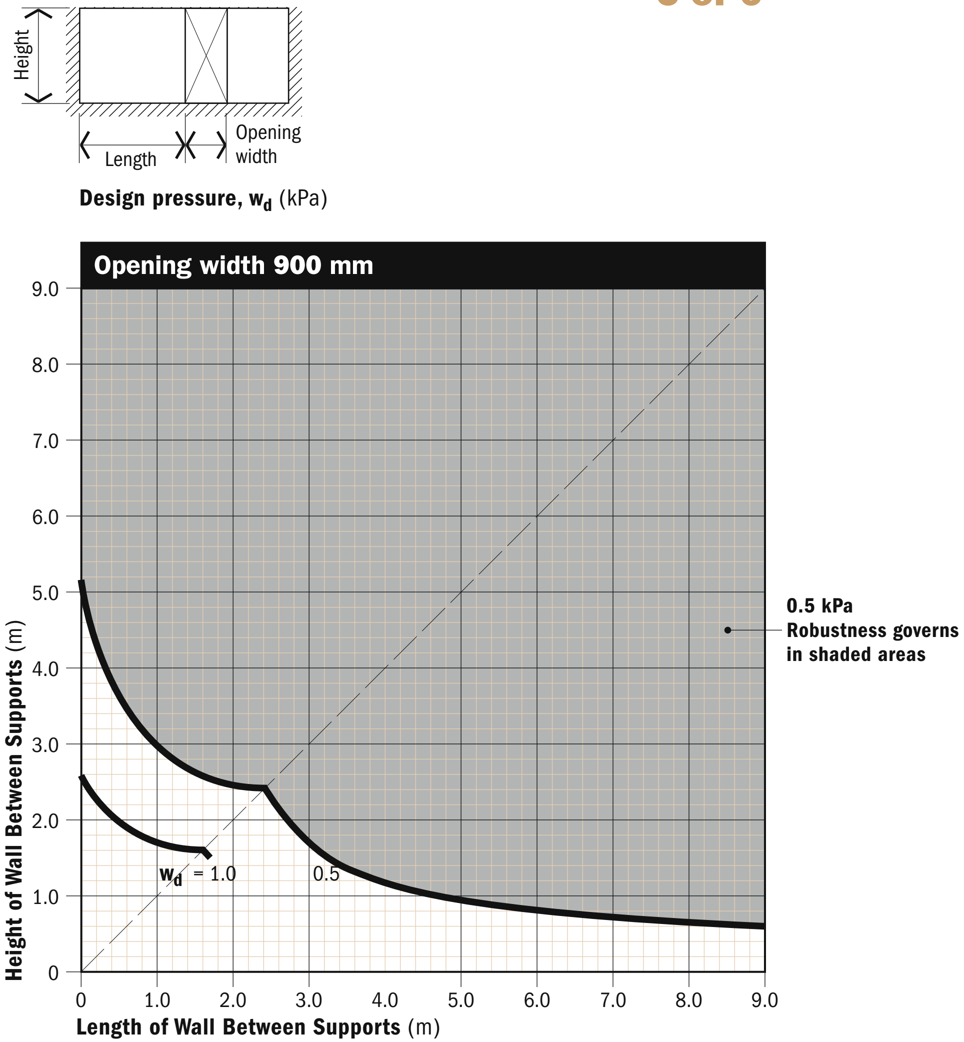





NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties

Laterally-supported both ends, top free

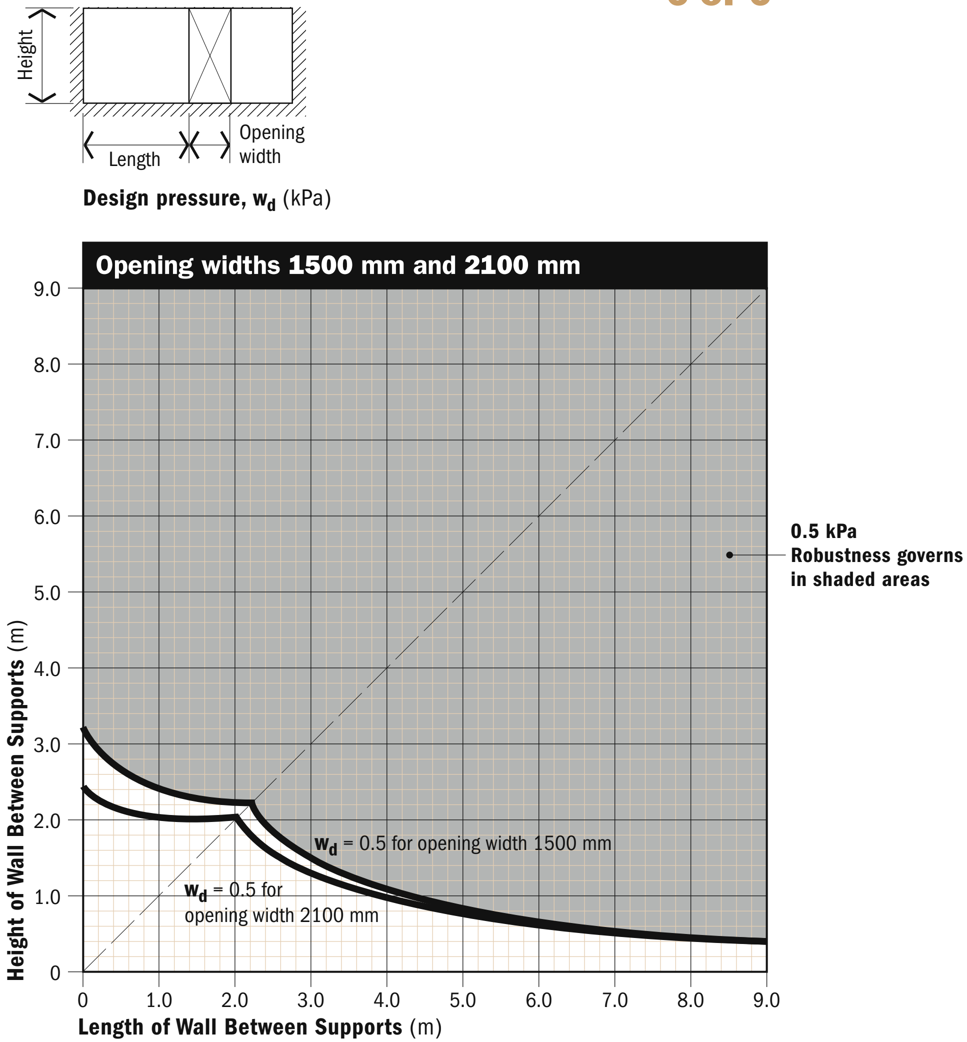
**Design pressure, $w_d$** (kPa)






Laterally-supported both ends, top free

**Design pressure, $w_d$** (kPa)

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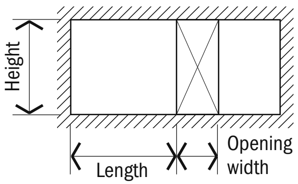
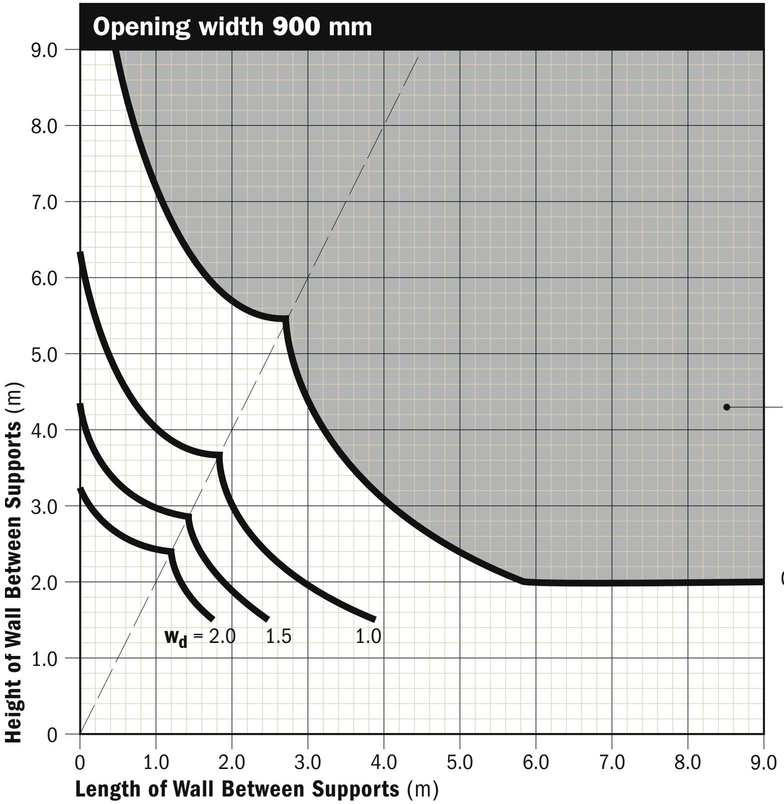
Laterally-supported both ends and top



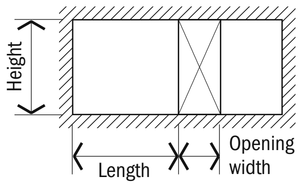



**Design pressure, $w_d$** (kPa)

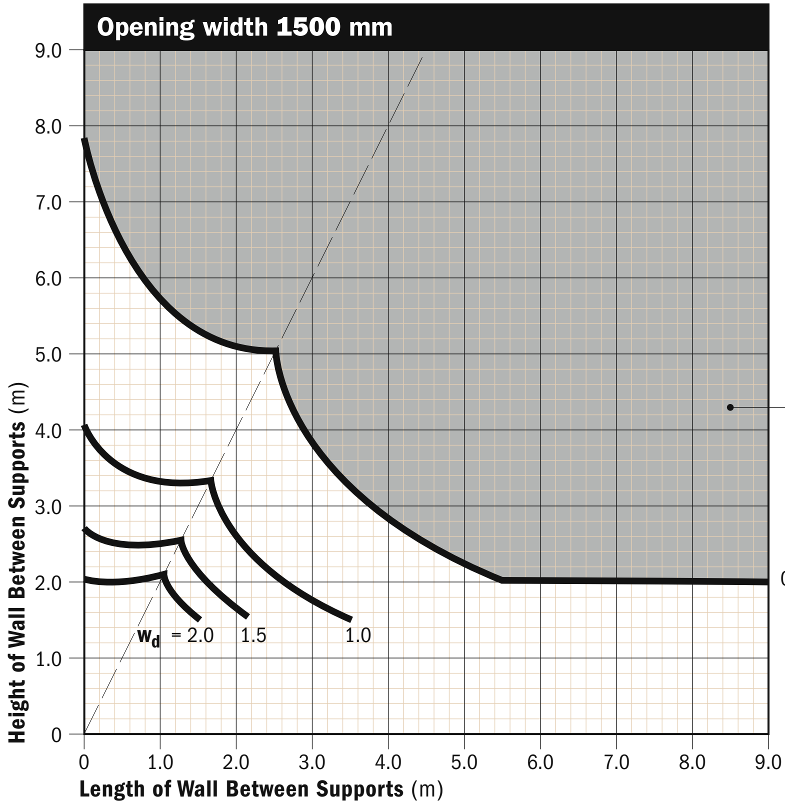



Laterally-supported both ends and top





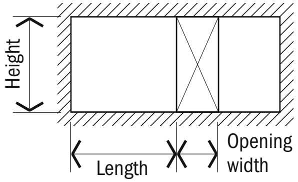

**Design pressure, $w_d$** (kPa)



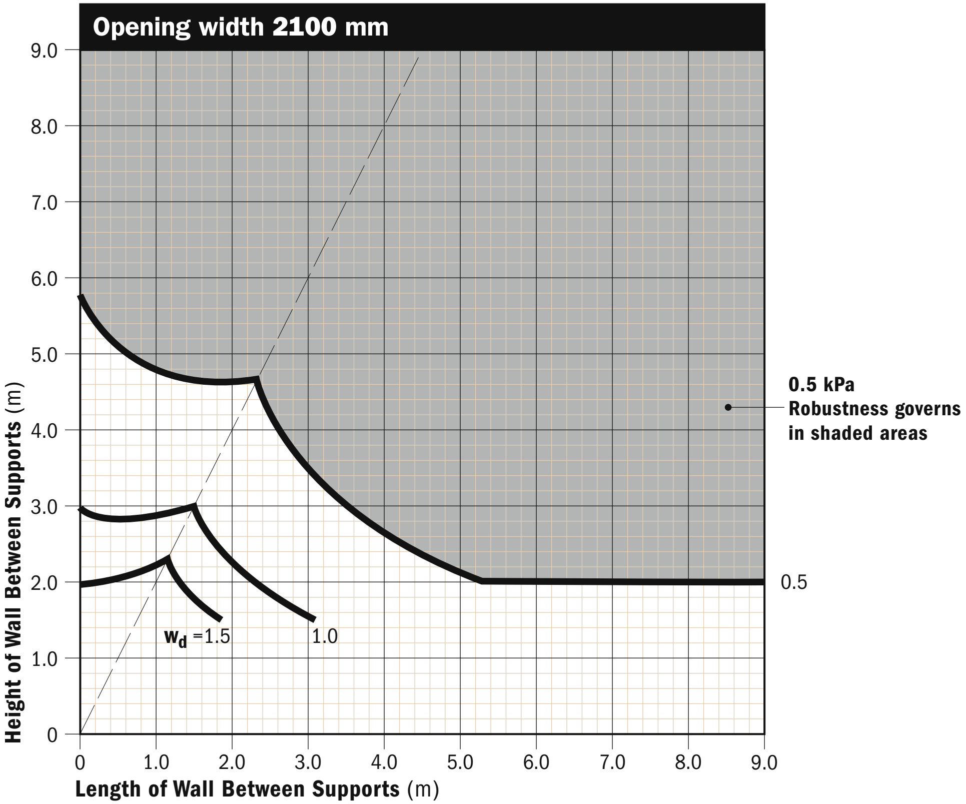

NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties

Laterally-supported both ends and top



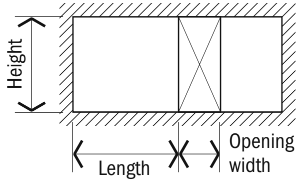



**Design pressure, $w_d$** (kPa)

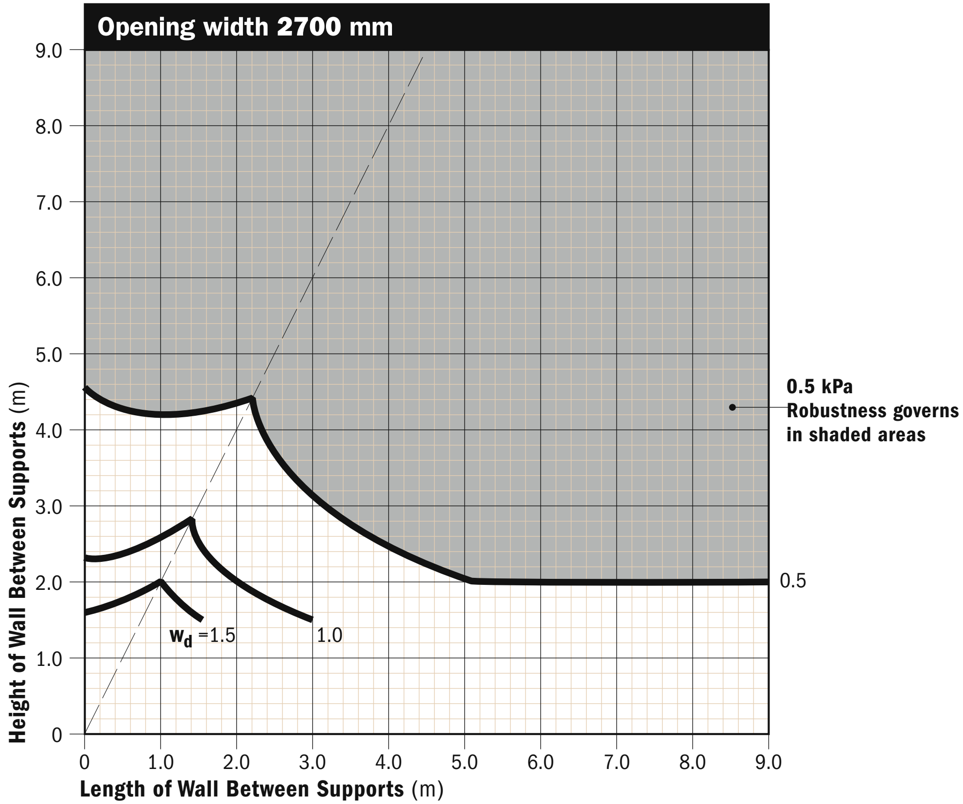



Laterally-supported both ends and top






**Design pressure, $w_d$** (kPa)

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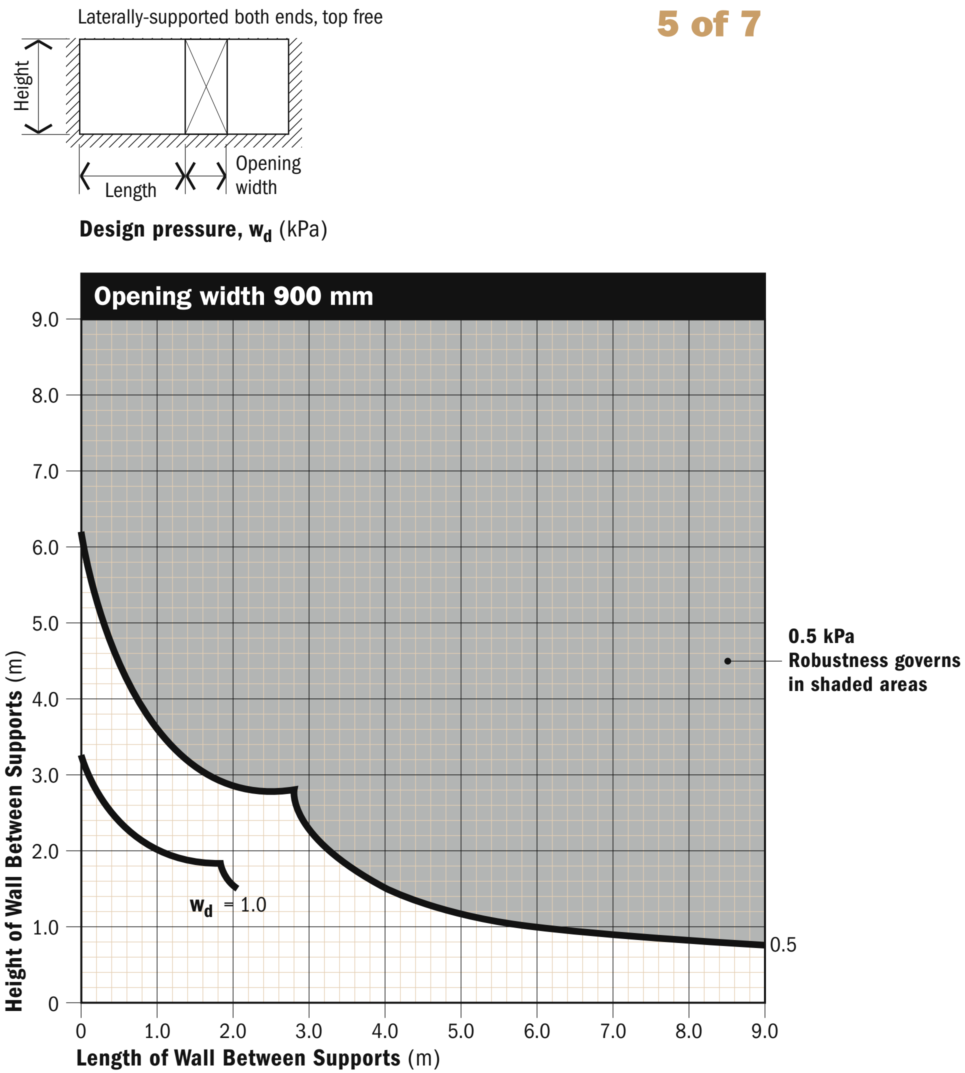
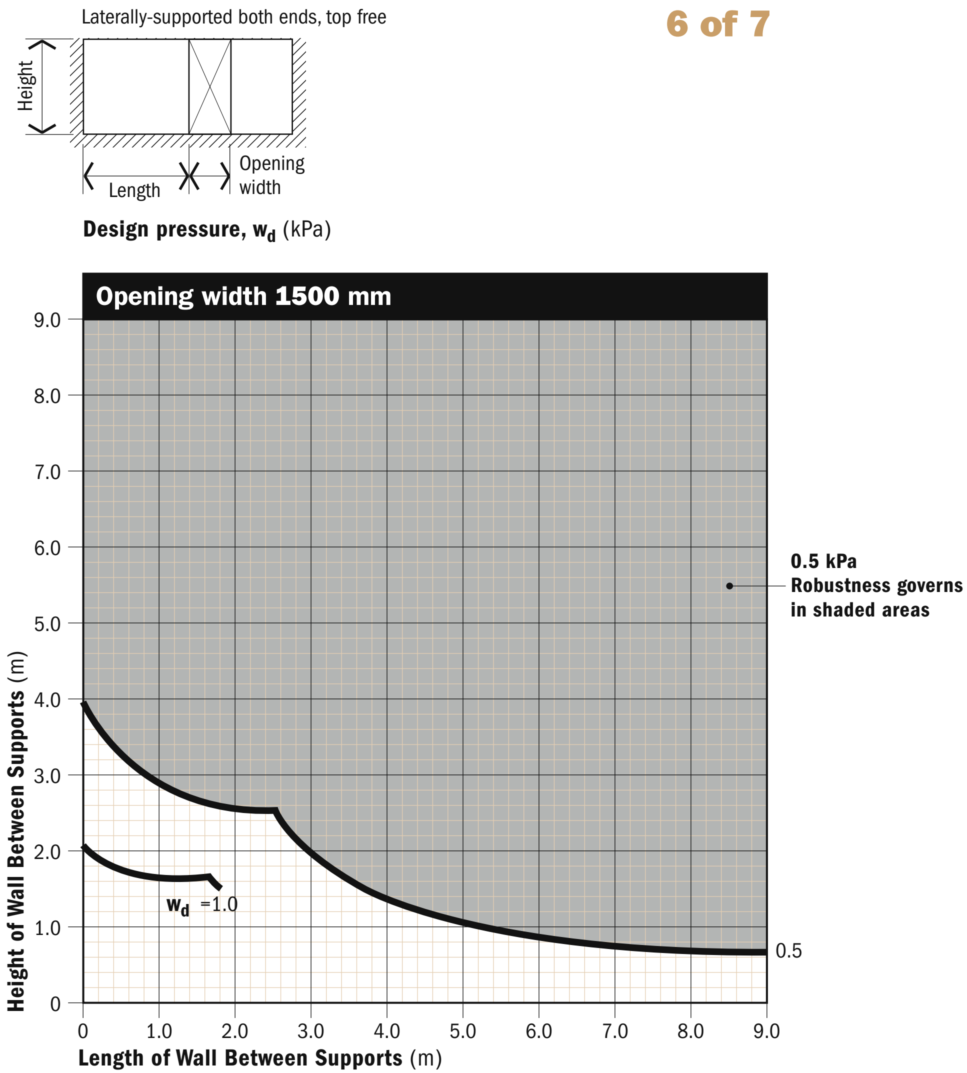


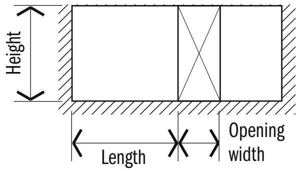


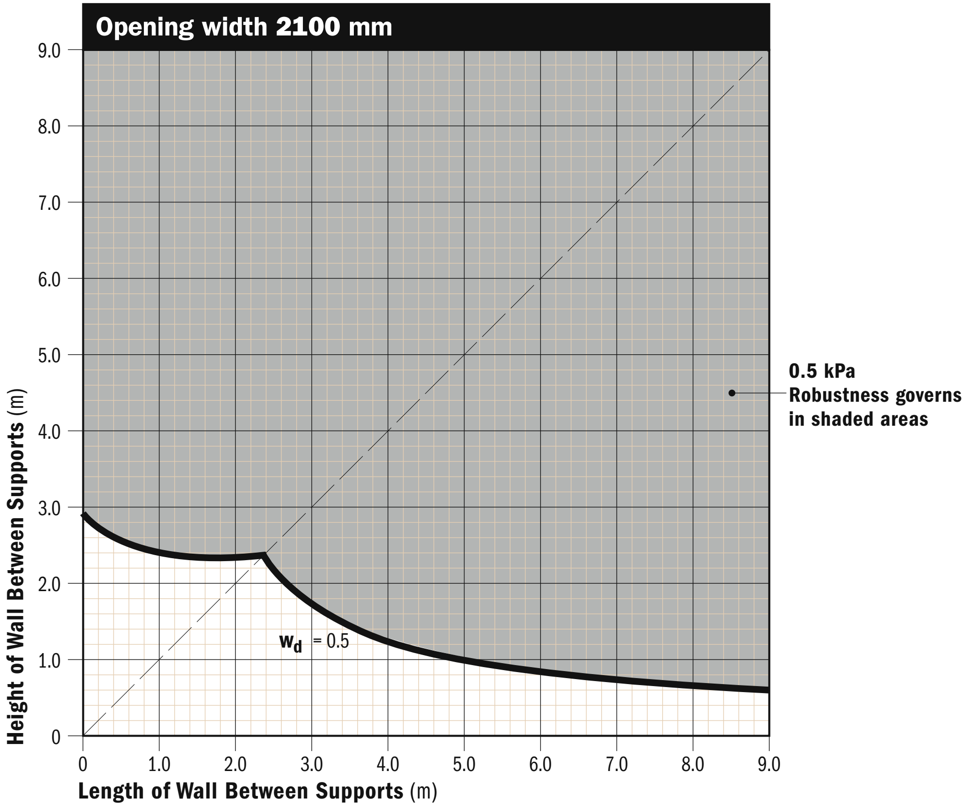

NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties

Laterally-supported both ends, top free





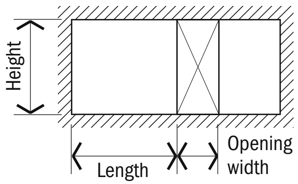

**Design pressure, $w_d$** (kPa)




NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties

Laterally-supported both ends and top



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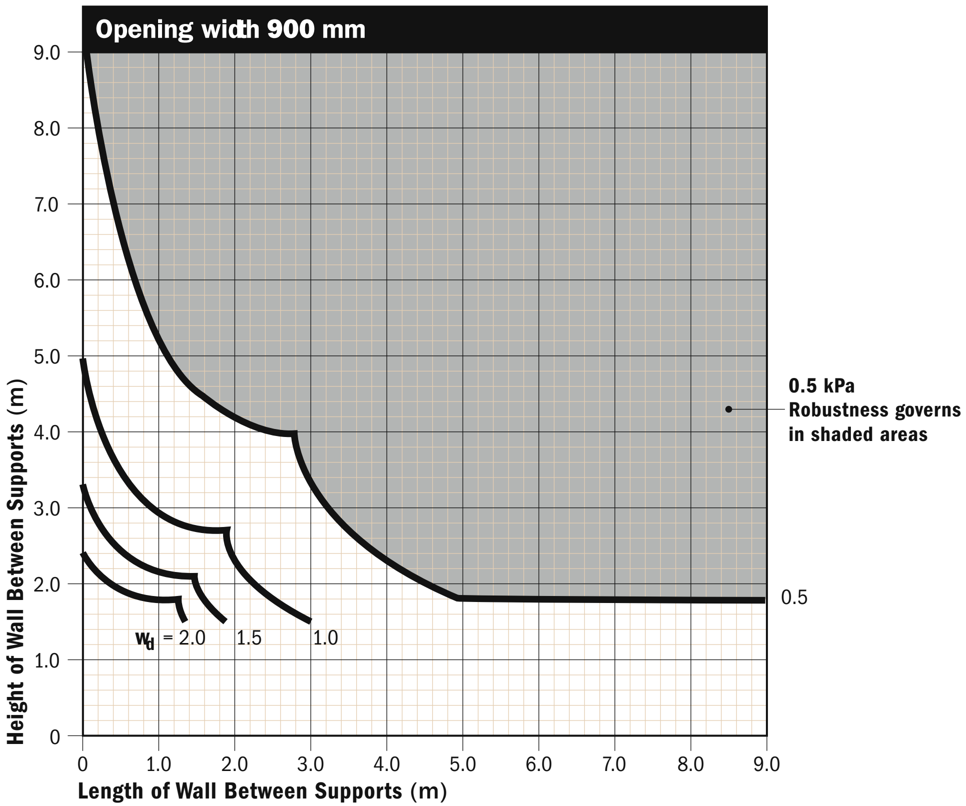
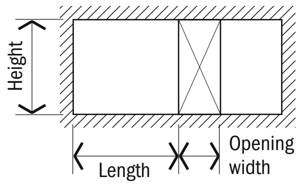
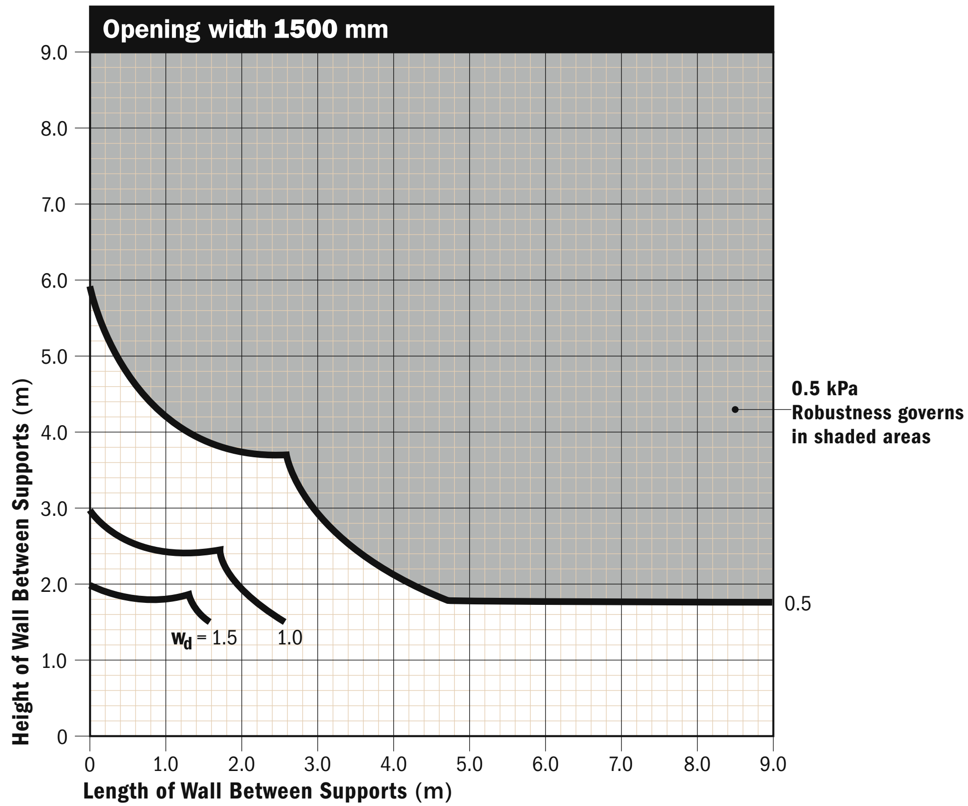



Laterally-supported both ends and top



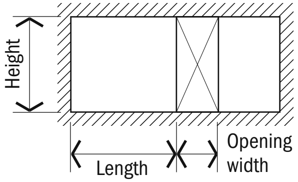



**Design pressure, $w_d$** (kPa)

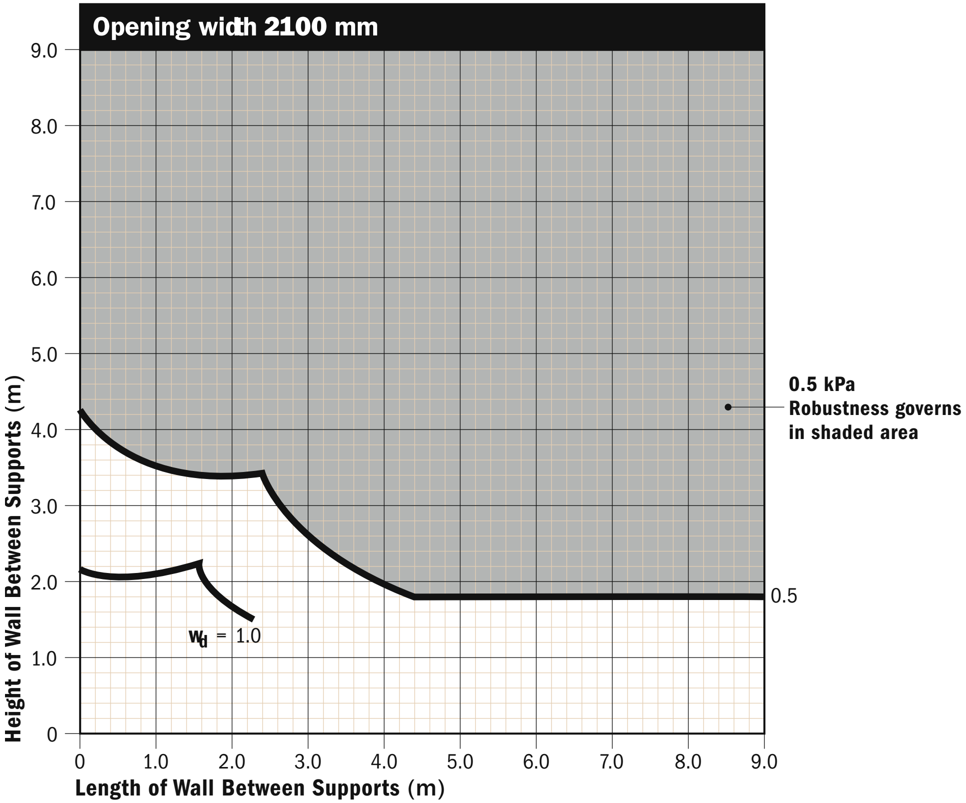



NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties

Laterally-supported both ends and top

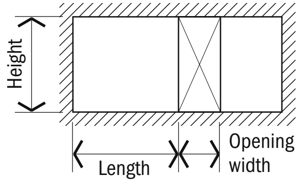



Design pressure, $w_d$ (kPa)

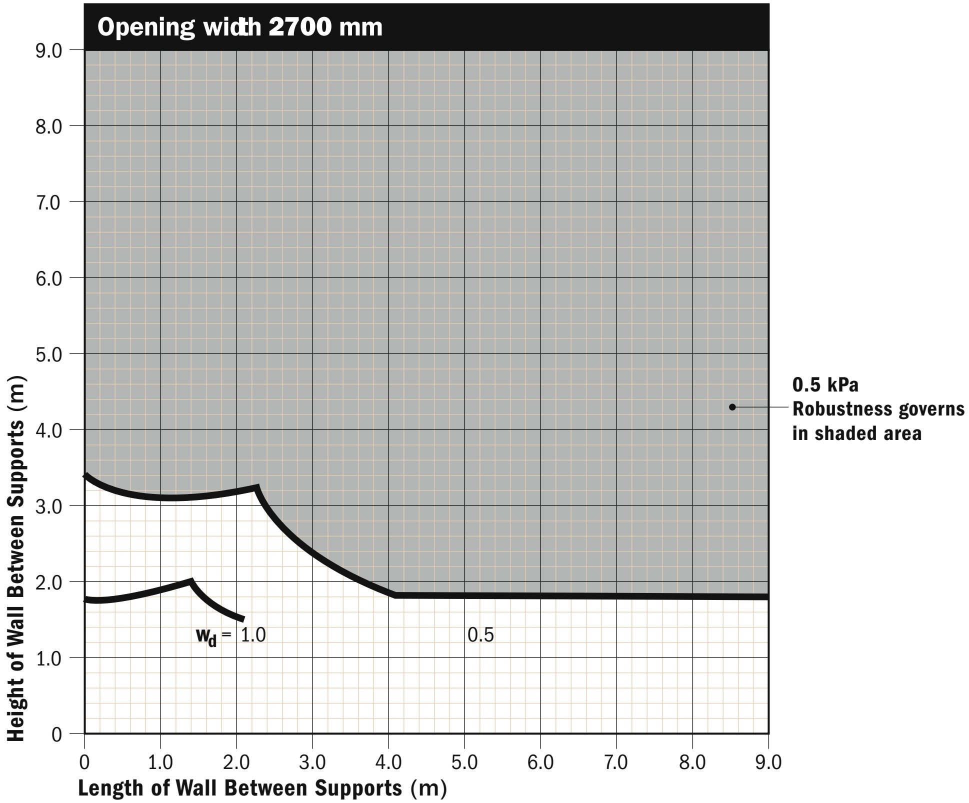





Laterally-supported both ends and top



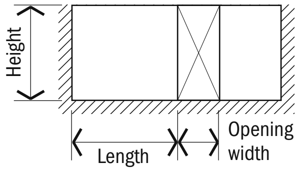

Design pressure, $w_d$ (kPa)



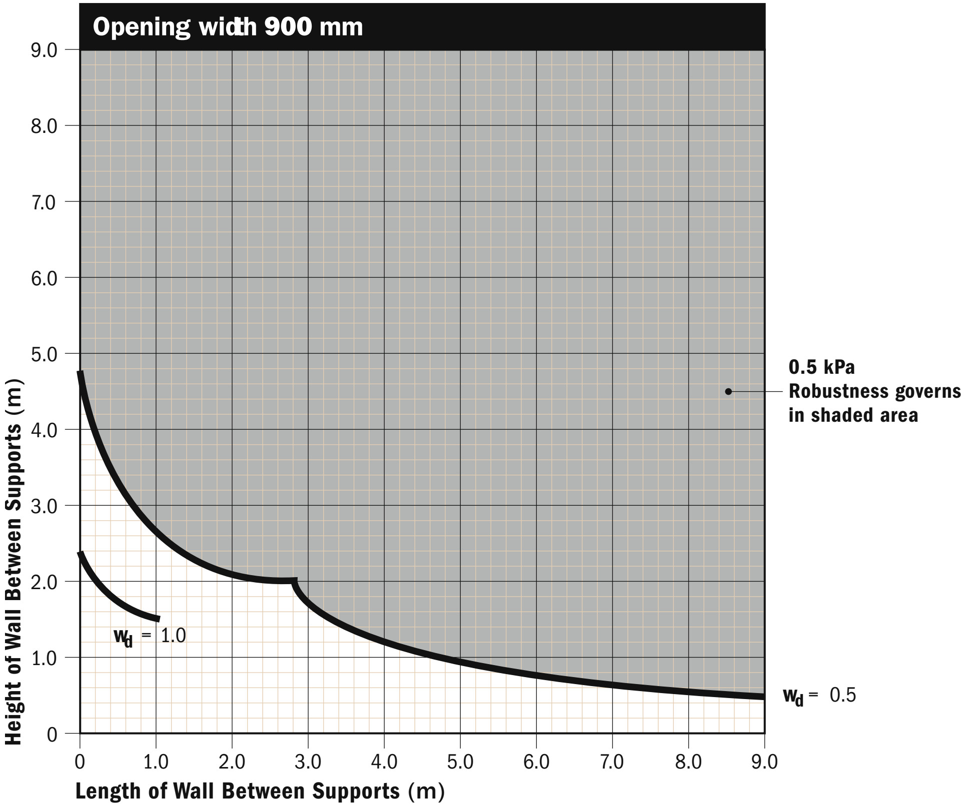

NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties

Laterally-supported both ends, top free



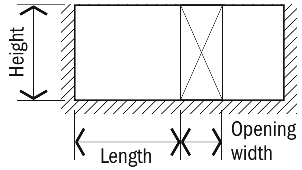



**Design pressure, $w_d$** (kPa)

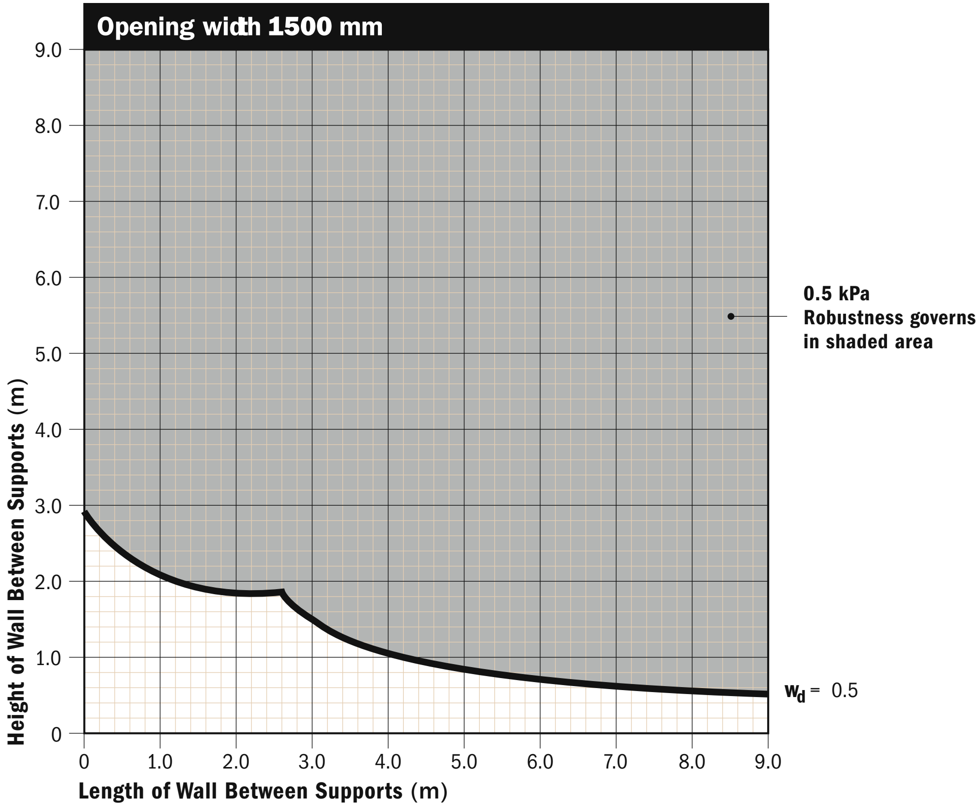



Laterally-supported both ends, top free





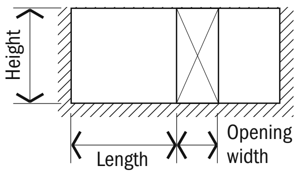

**Design pressure, $w_d$** (kPa)



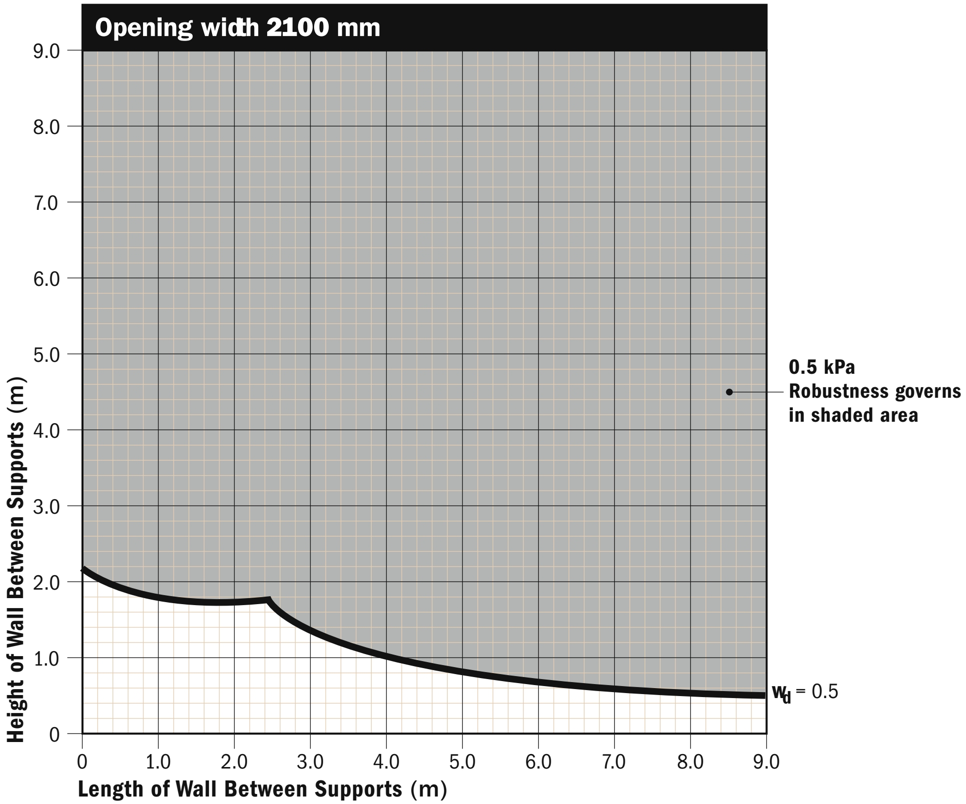

NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties

Laterally-supported both ends, top free



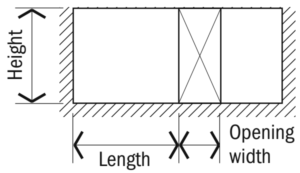



**Design pressure, $w_d$** (kPa)

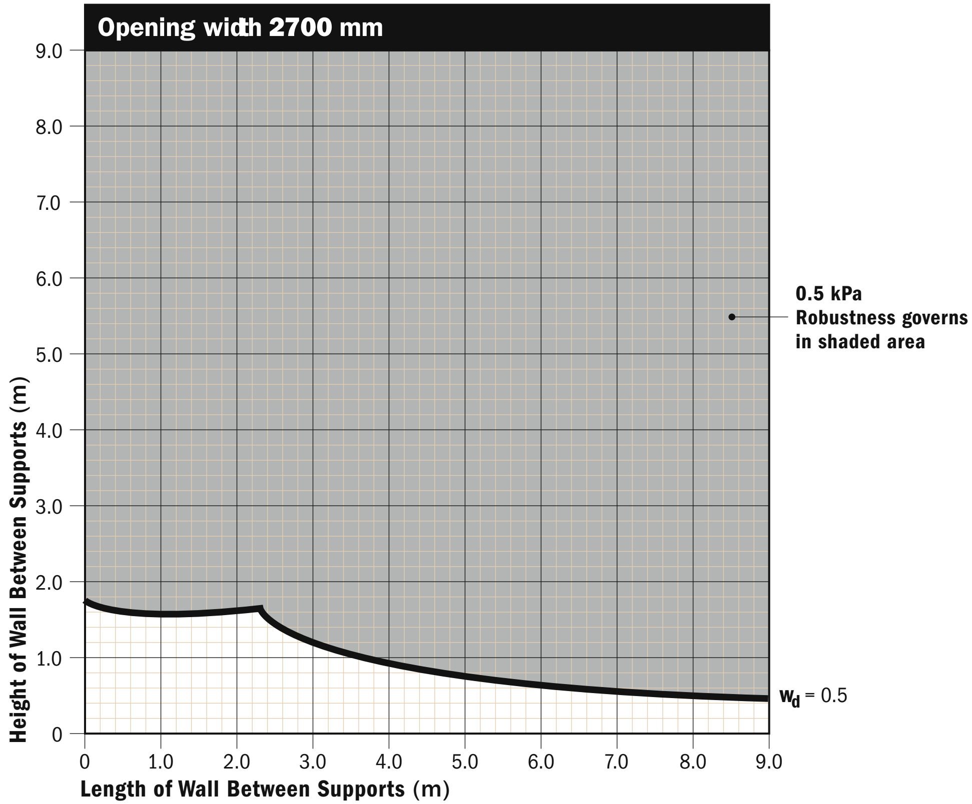



Laterally-supported both ends, top free

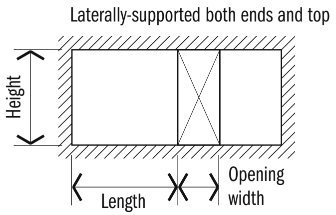




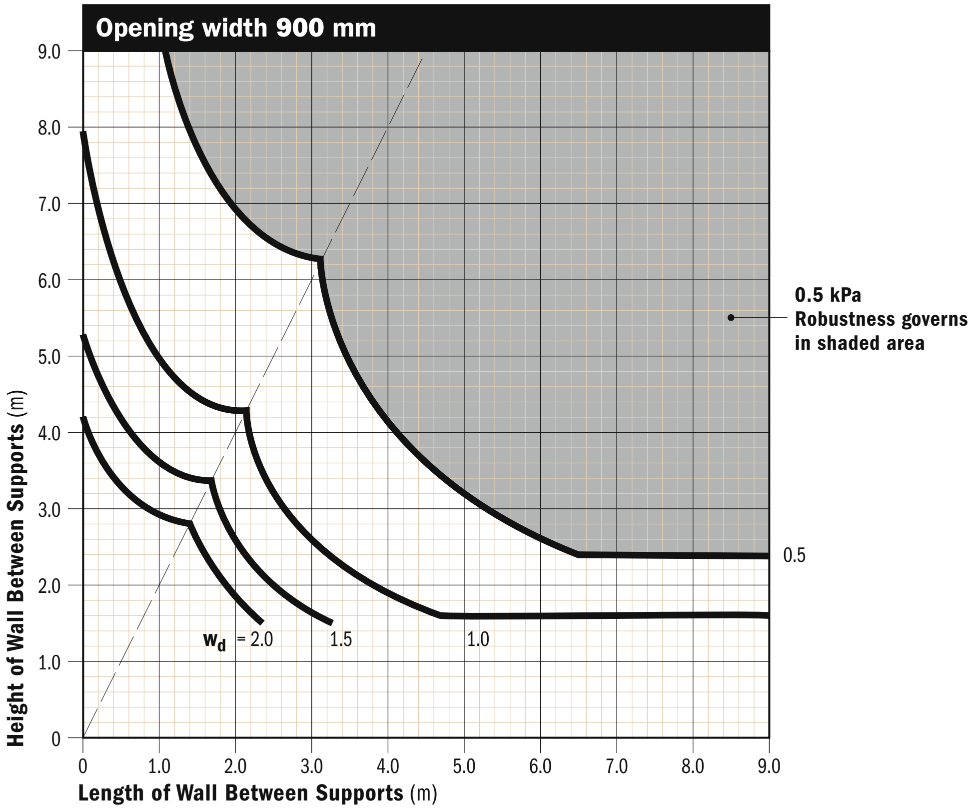

**Design pressure, $w_d$** (kPa)



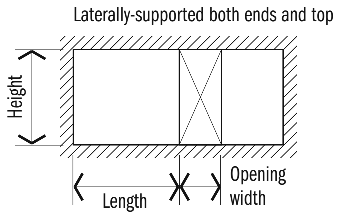

NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties



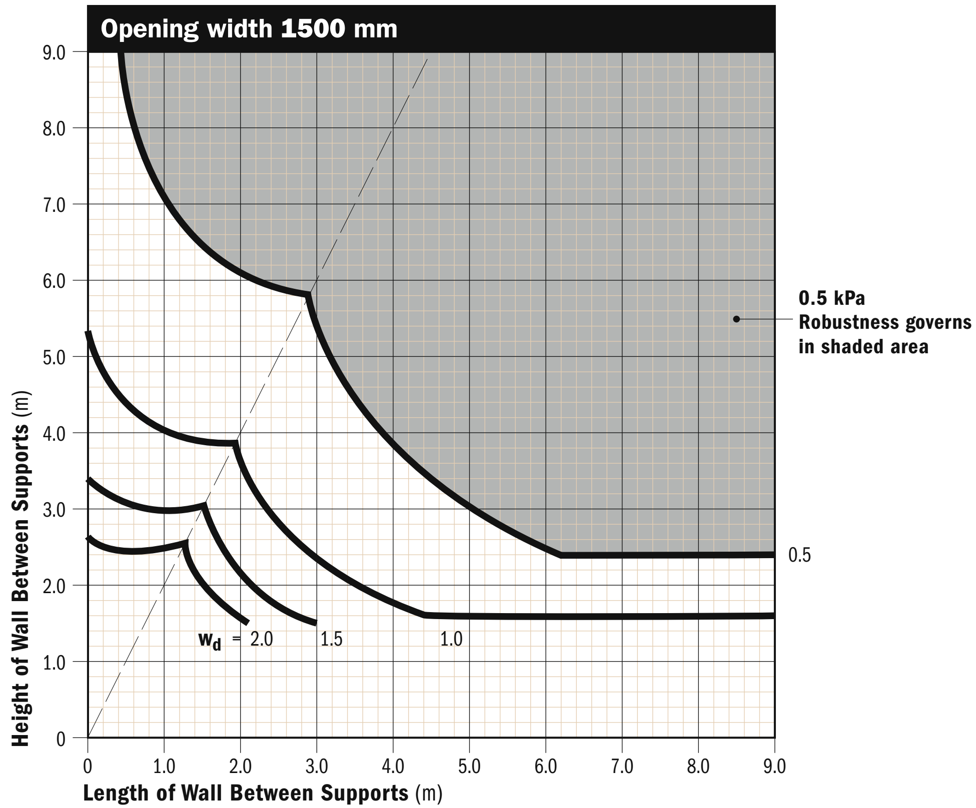

**Design pressure, $w_d$** (kPa)





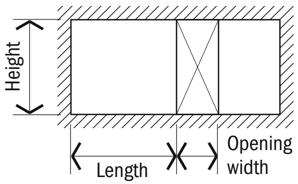




**Design pressure, $w_d$** (kPa)

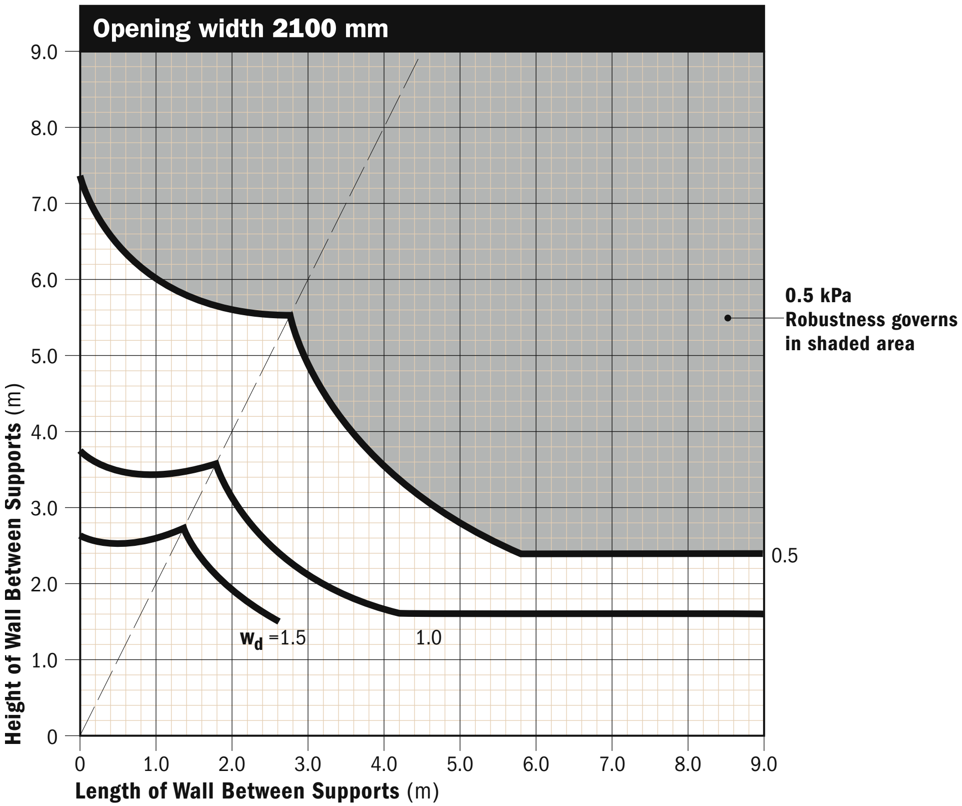



NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties

Laterally-supported both ends and top

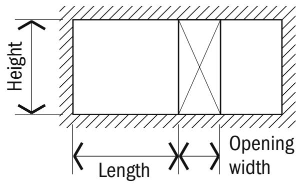



**Design pressure, $w_d$** (kPa)

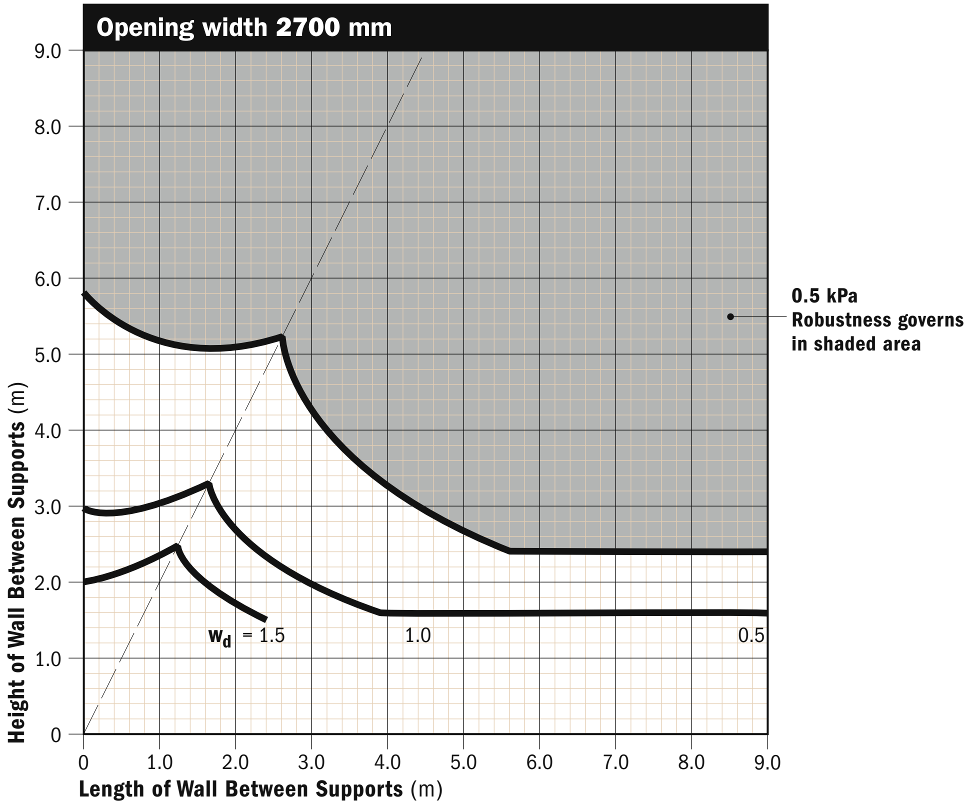





Laterally-supported both ends and top



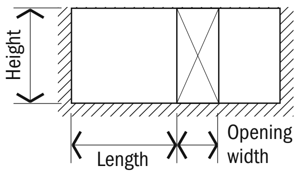

**Design pressure, $w_d$** (kPa)



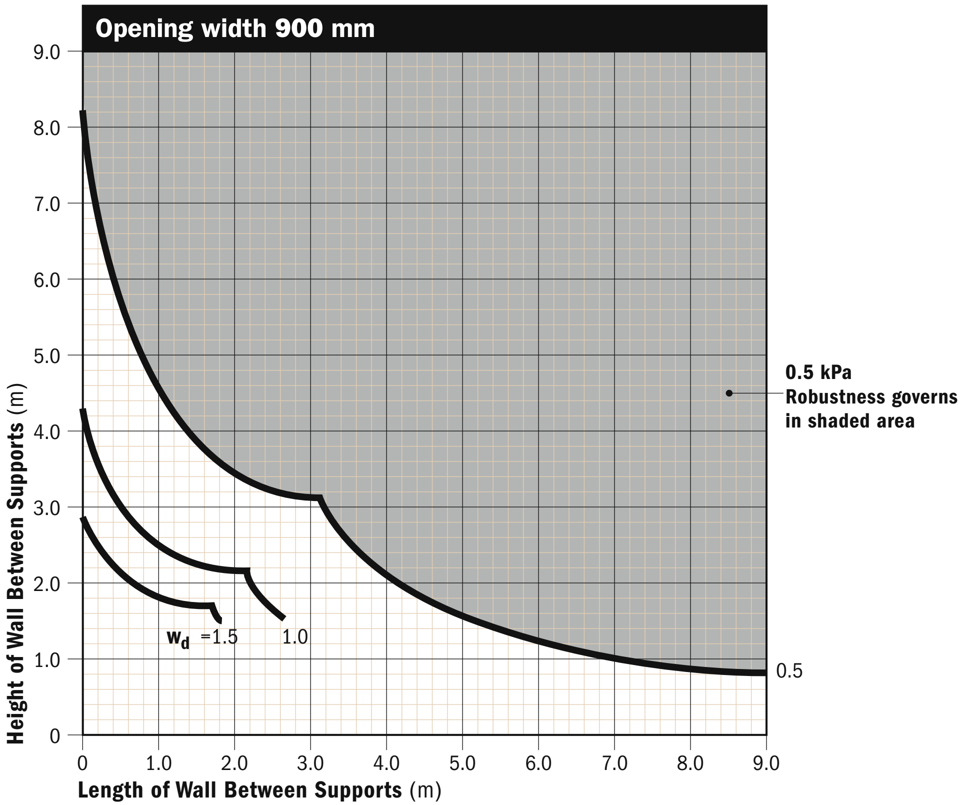

NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties

Laterally-supported both ends, top free



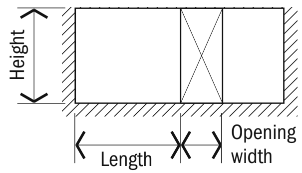



**Design pressure, $w_d$** (kPa)

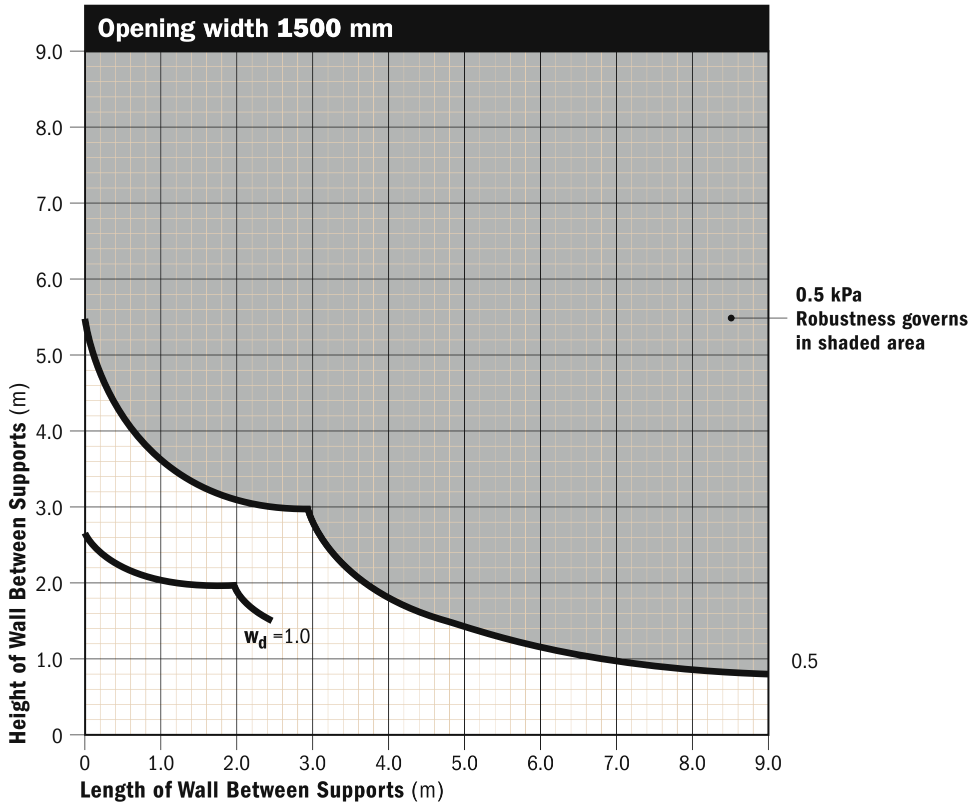



Laterally-supported both ends, top free



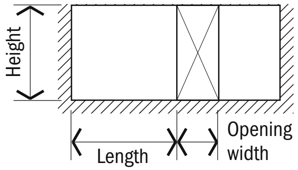



**Design pressure, $w_d$** (kPa)

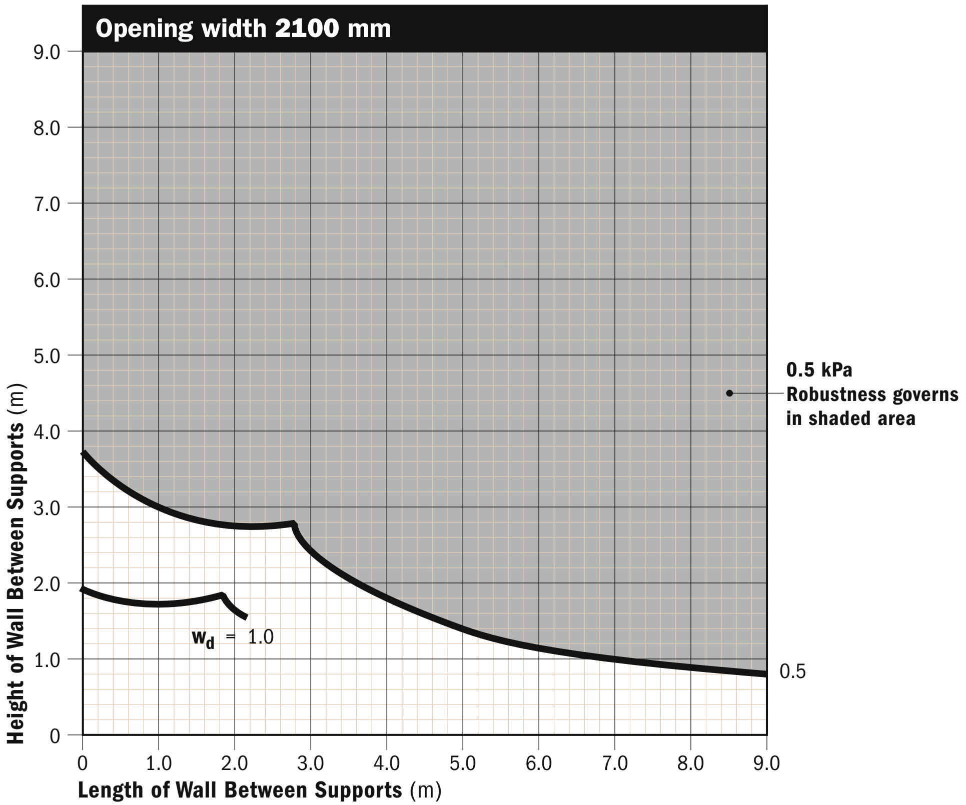



NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties

Laterally-supported both ends, top free

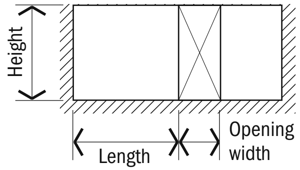



**Design pressure, $w_d$** (kPa)

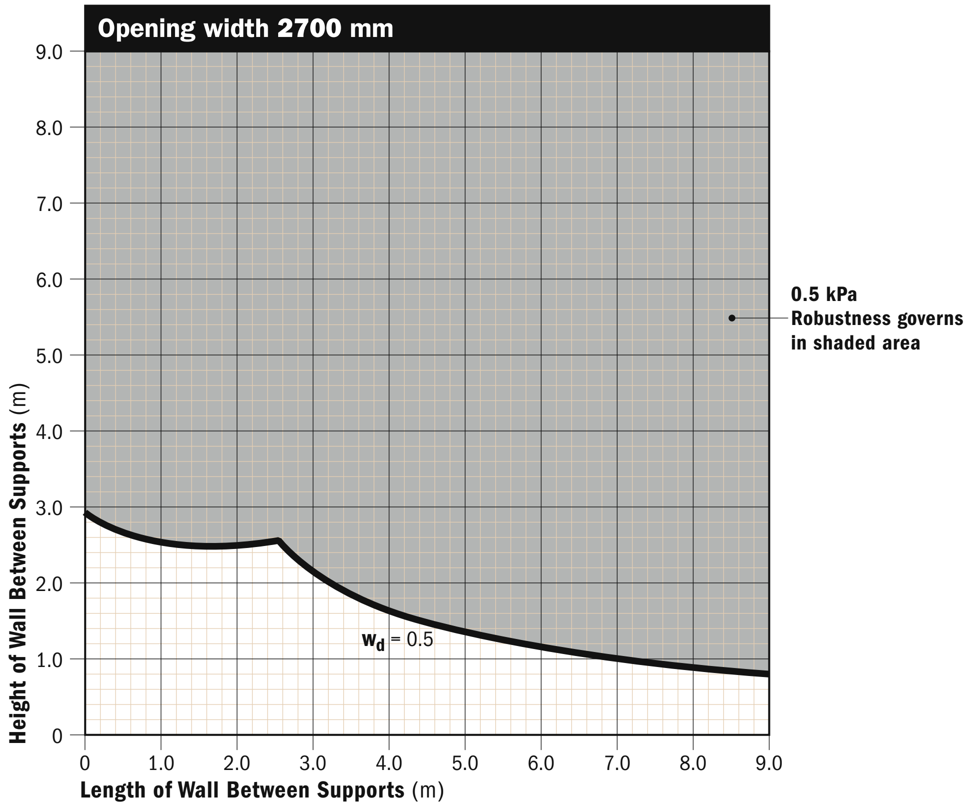





Laterally-supported both ends, top free




**Design pressure, $w_d$** (kPa)



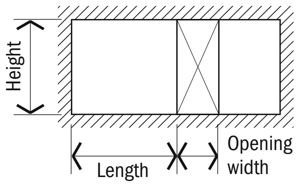

NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties

# UNREINFORCED MASONRY – with openings

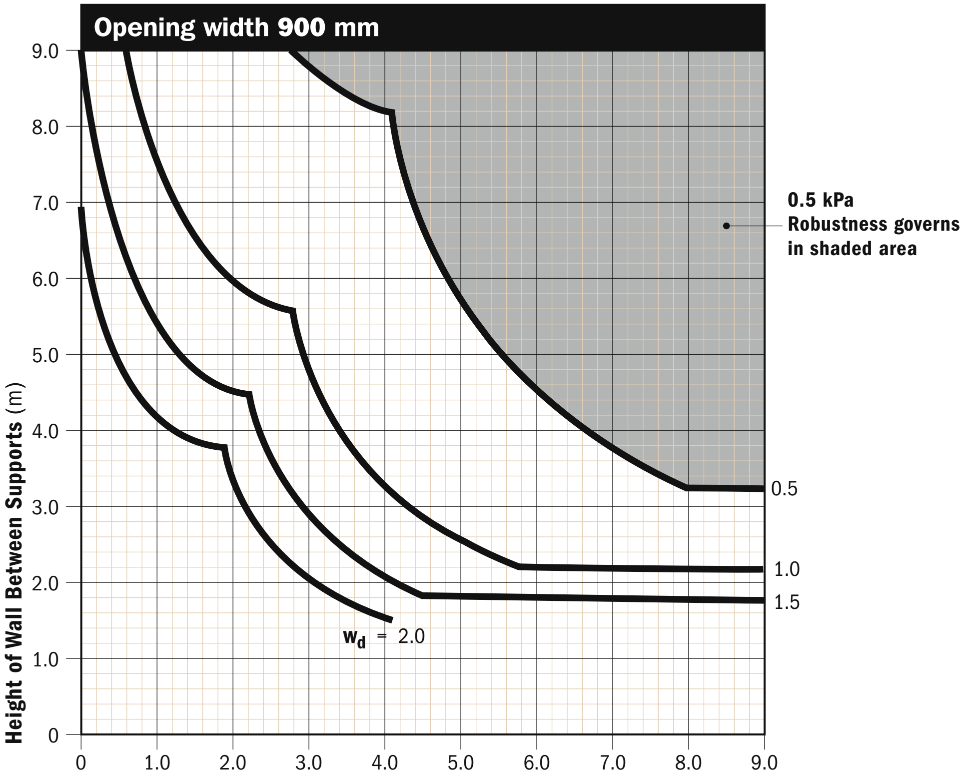
## 190-mm leaf (390 x 190 units 30 mm face-shell bedded)

Laterally-supported both ends and top



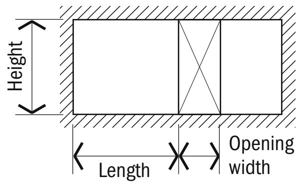



**Design pressure, $w_d$** (kPa)

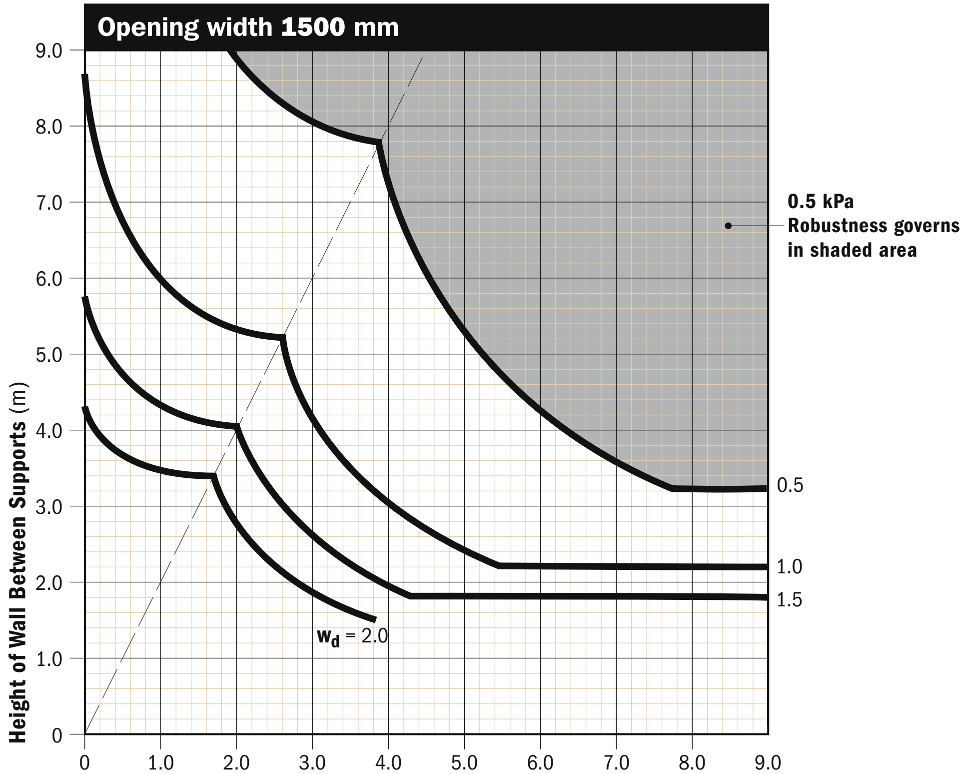



Laterally-supported both ends and top



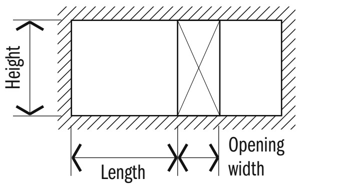



**Design pressure, $w_d$** (kPa)

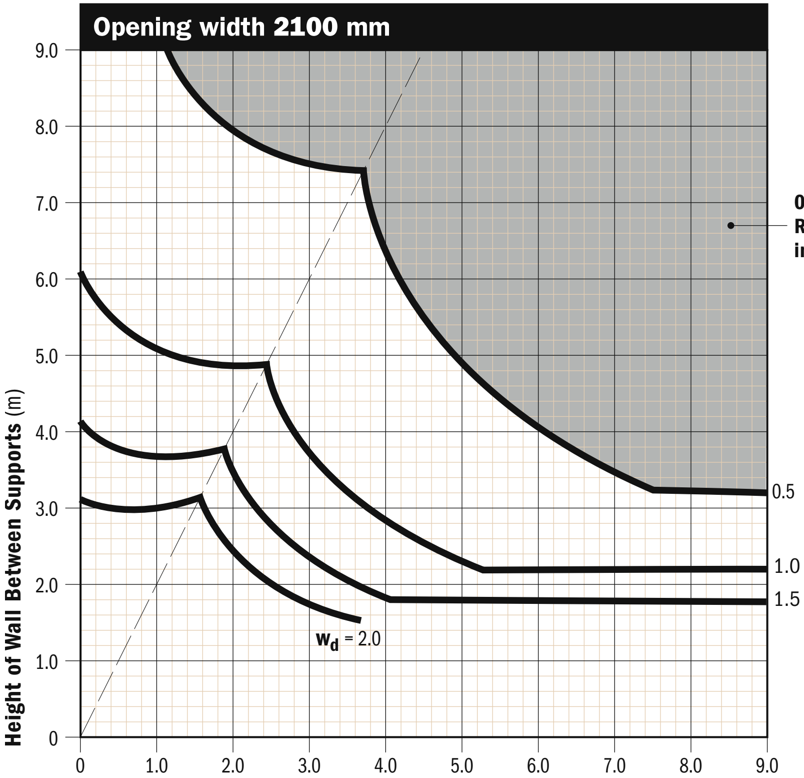



NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties

Laterally-supported both ends and top

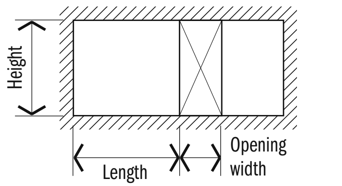



**Design pressure, $w_d$** (kPa)

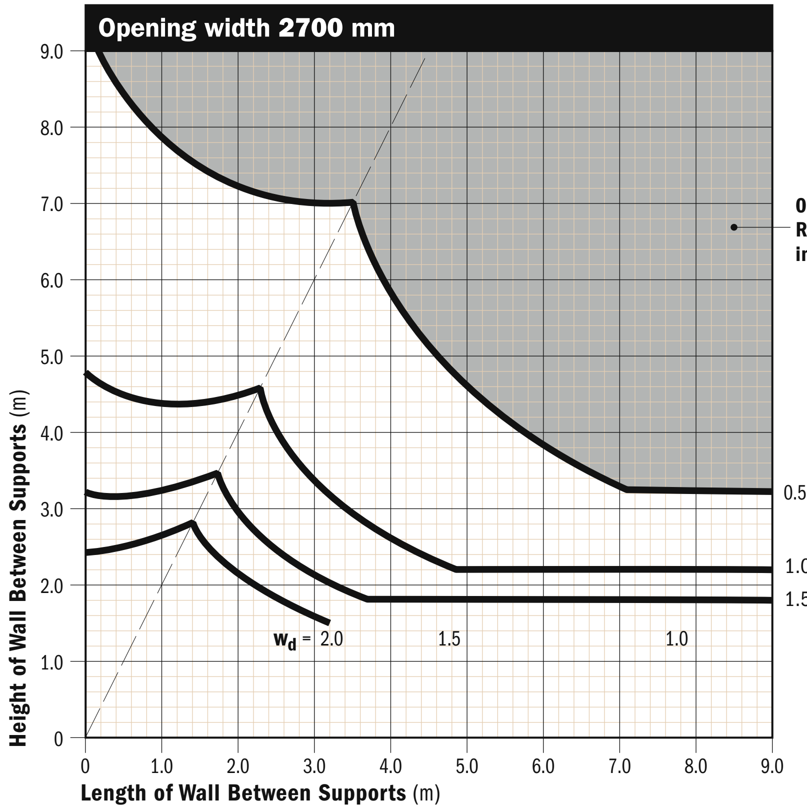





Laterally-supported both ends and top



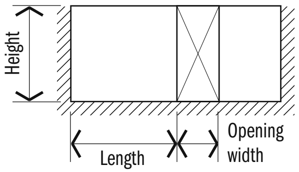

**Design pressure, $w_d$** (kPa)



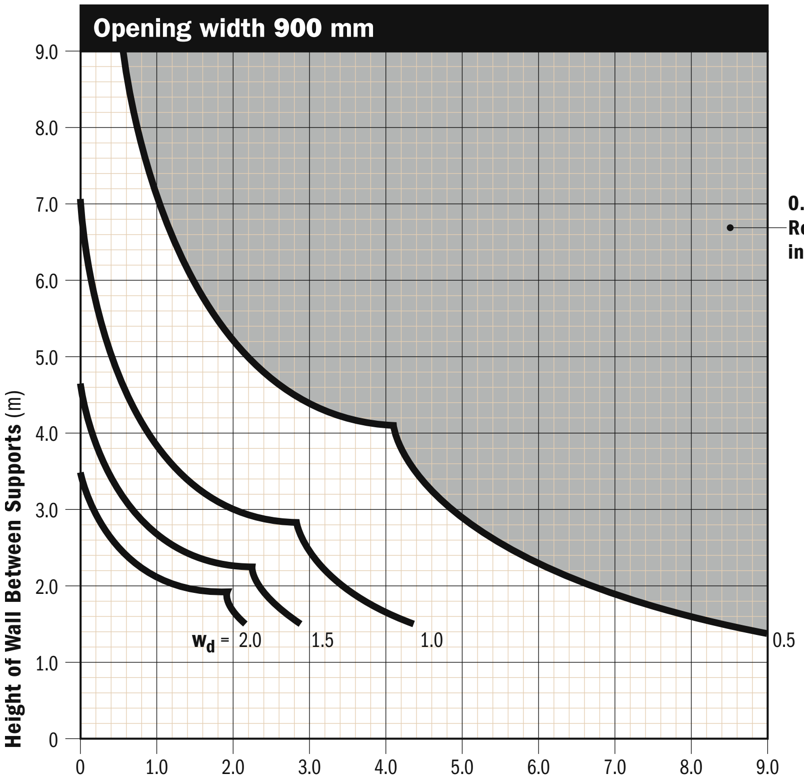

NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties

Laterally-supported both ends, top free



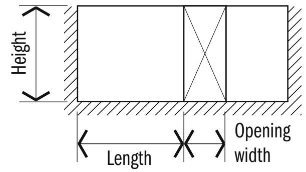



**Design pressure, $w_d$** (kPa)

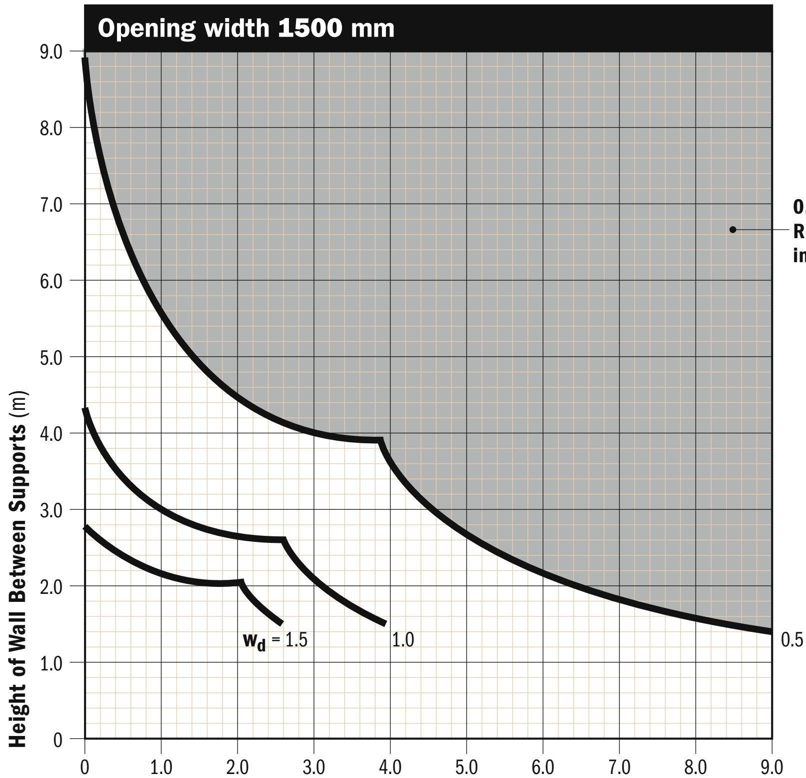



Laterally-supported both ends, top free





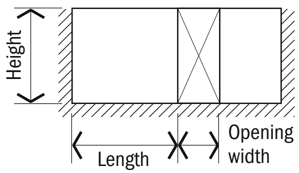

**Design pressure, $w_d$** (kPa)



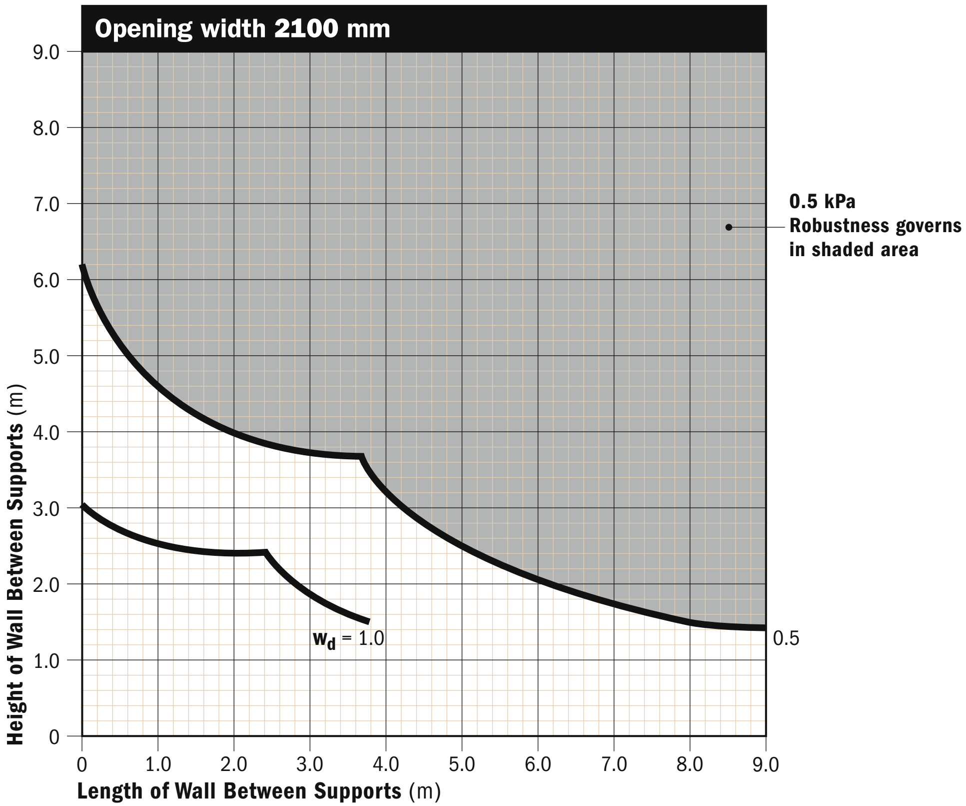

NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties

Laterally-supported both ends, top free



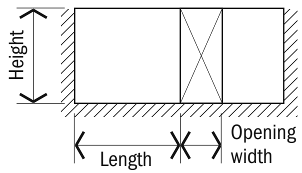



**Design pressure, $w_d$** (kPa)

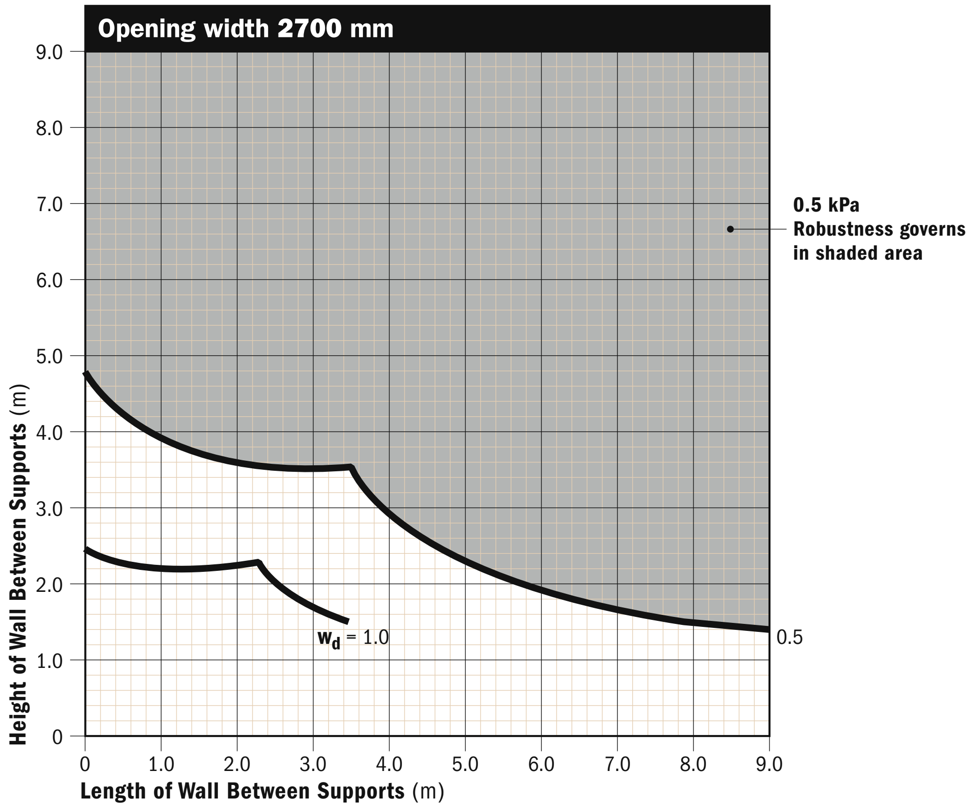



Laterally-supported both ends, top free

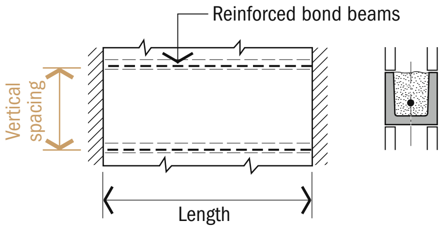




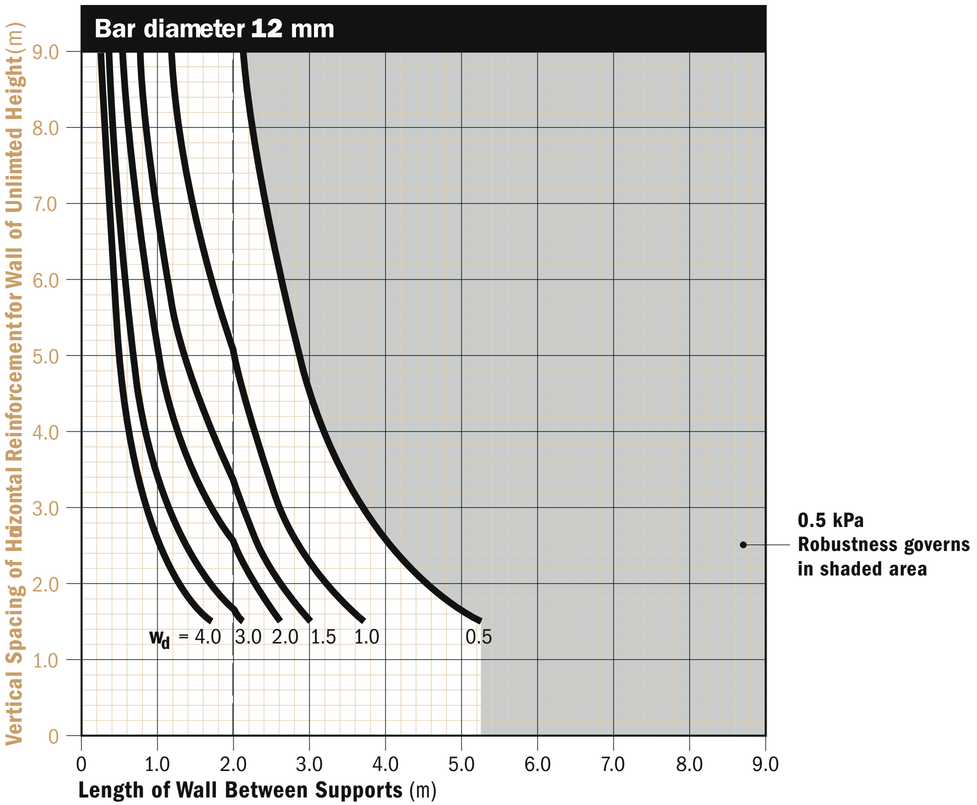

**Design pressure, $w_d$** (kPa)



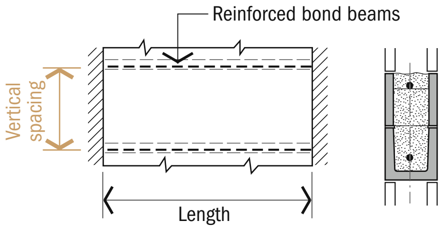

NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties



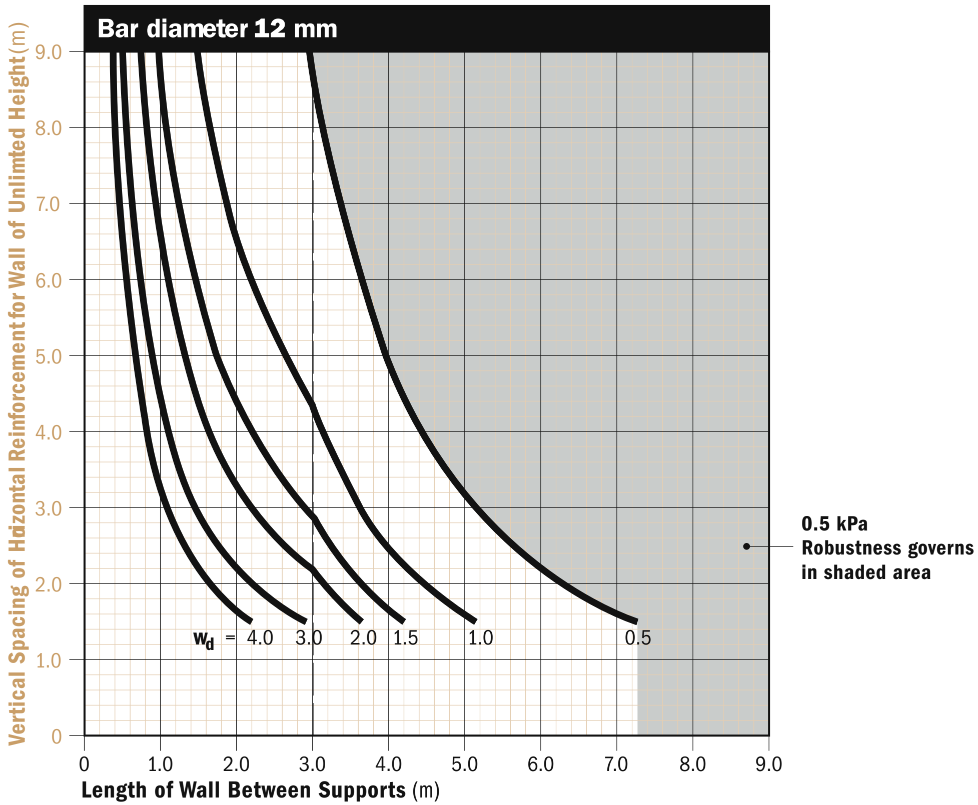

**Design pressure, $w_d$** (kPa)



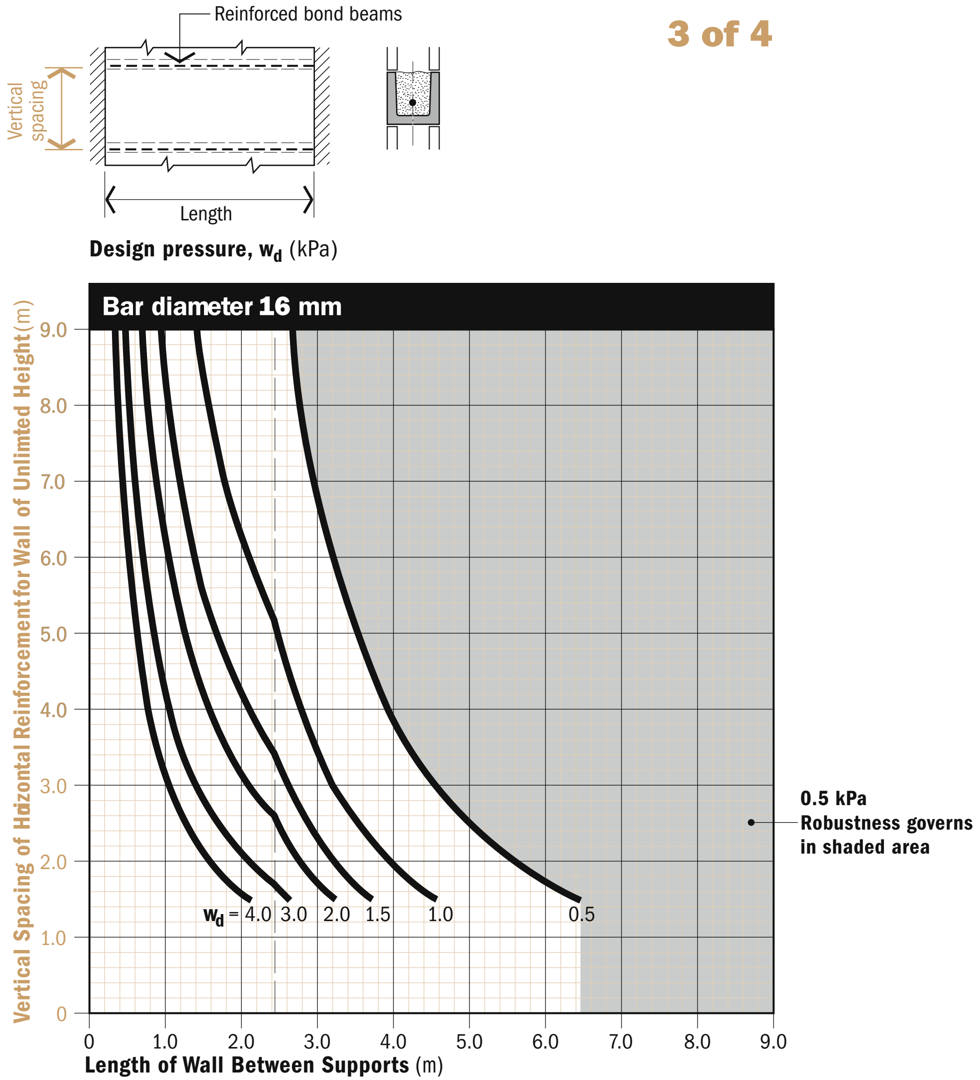


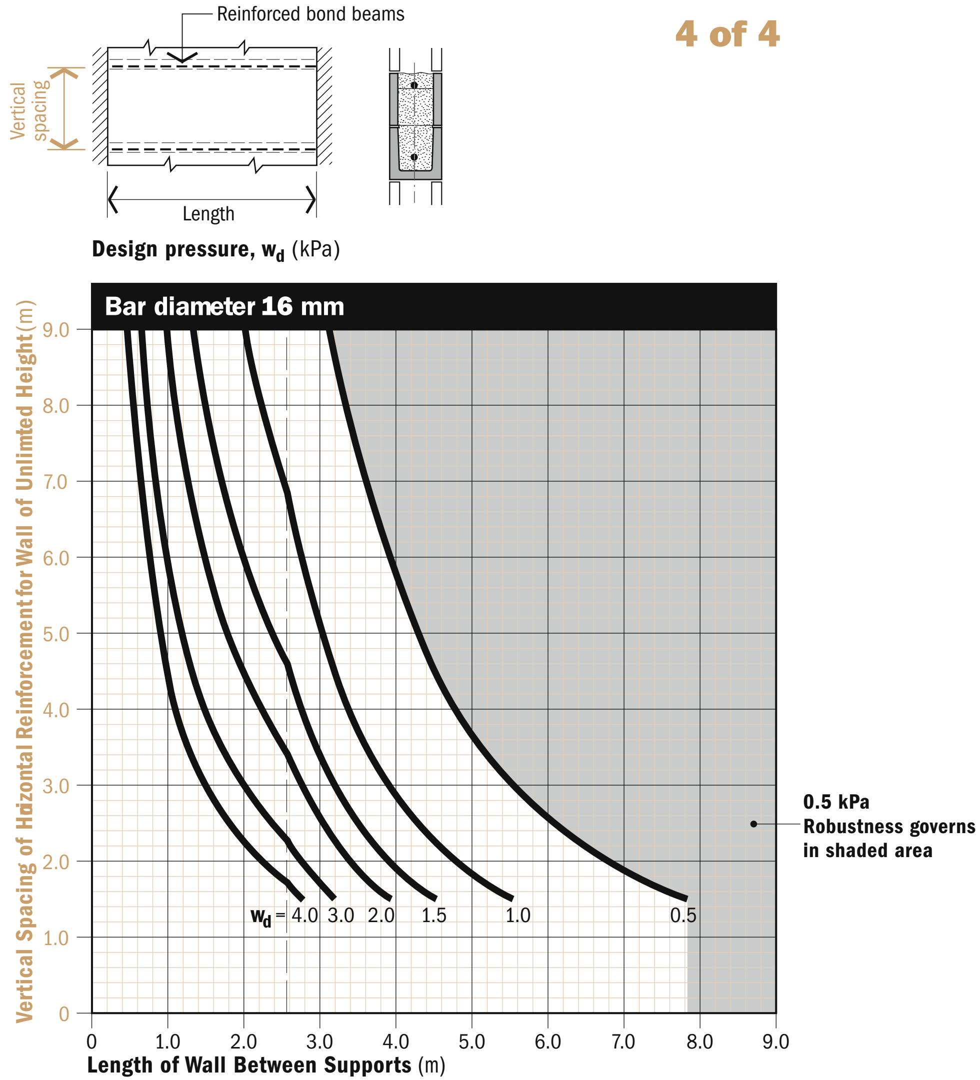

**Design pressure, $w_d$** (kPa)



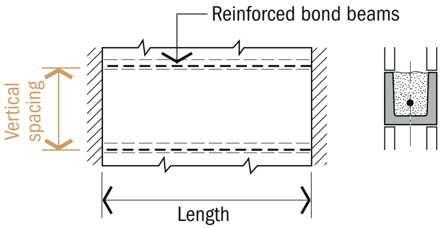

NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties



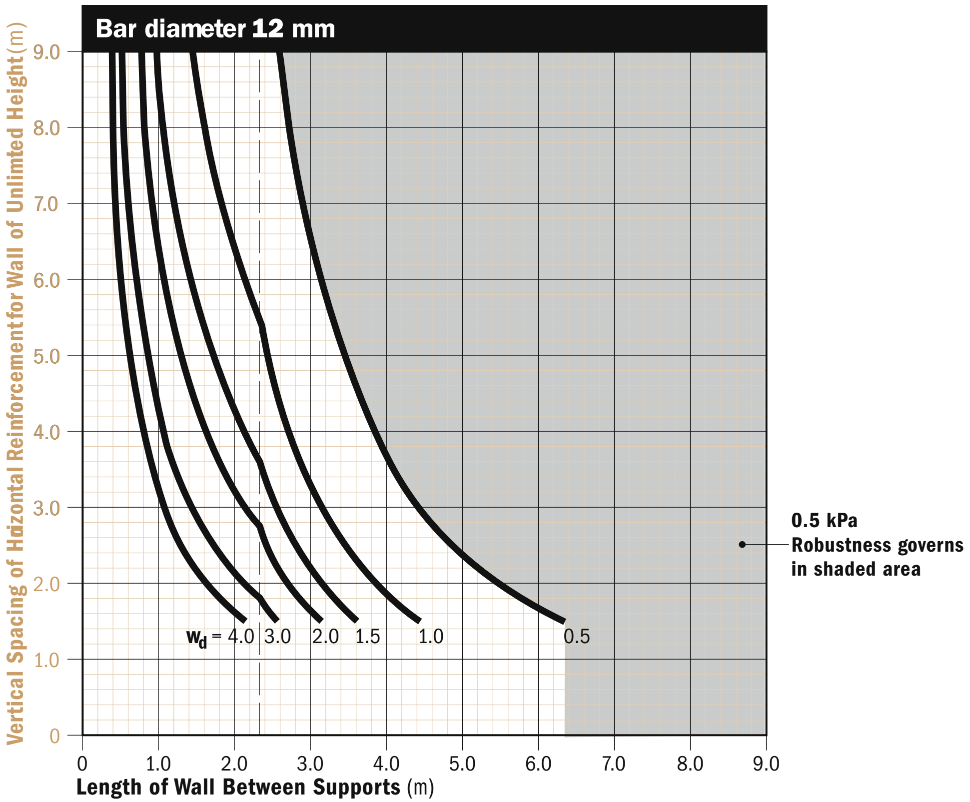


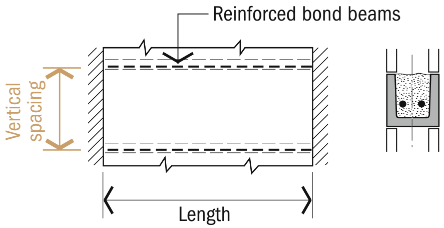

NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties



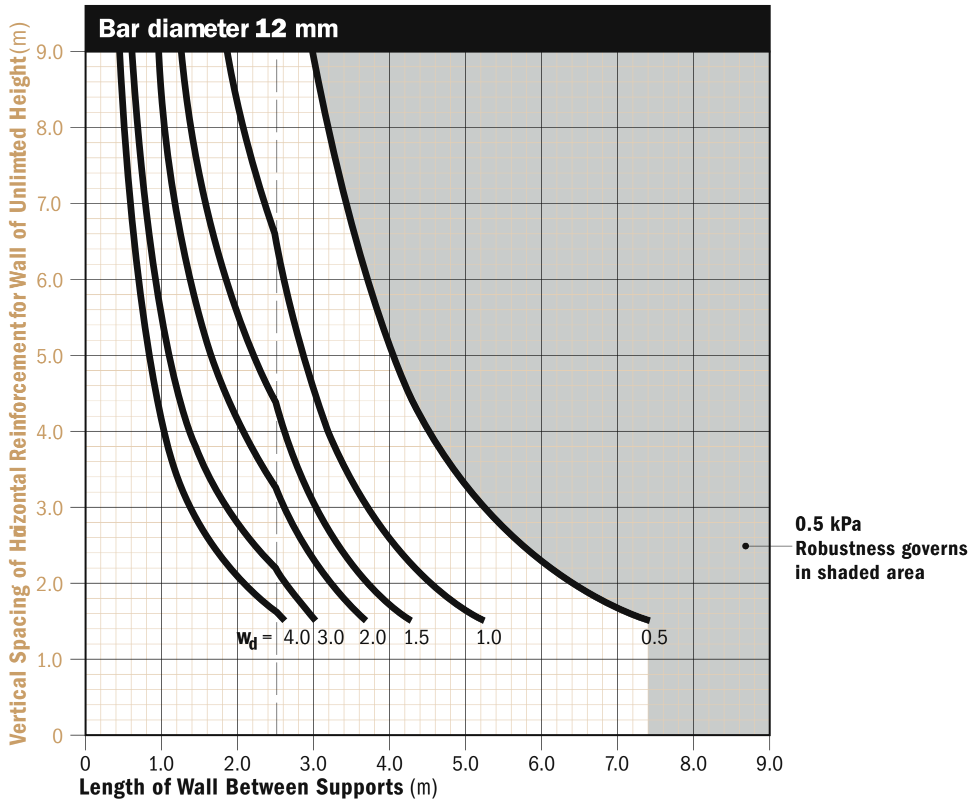

Design pressure, $w_d$ (kPa)



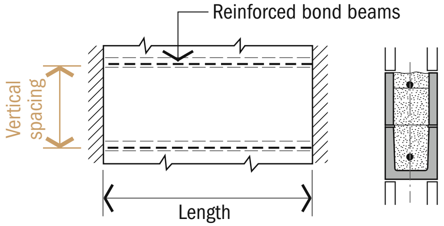


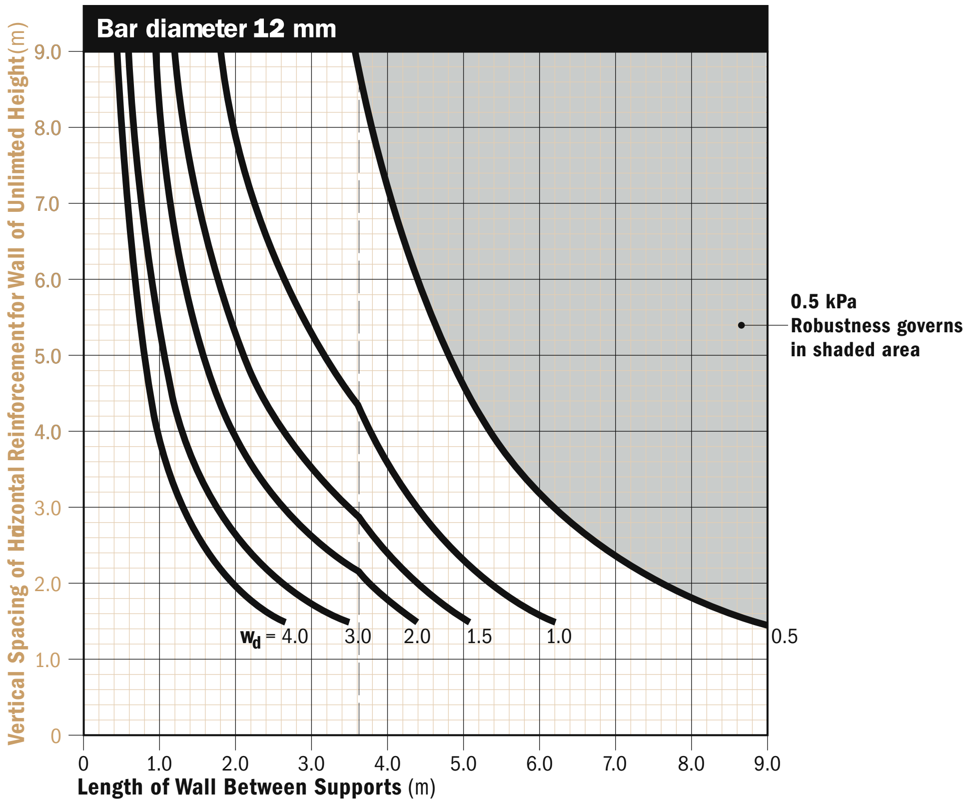

Design pressure, $w_d$ (kPa)



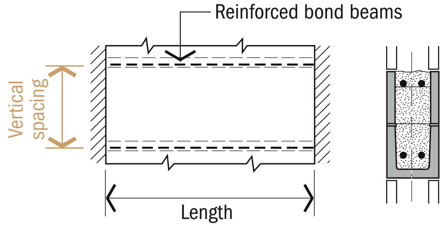

NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties



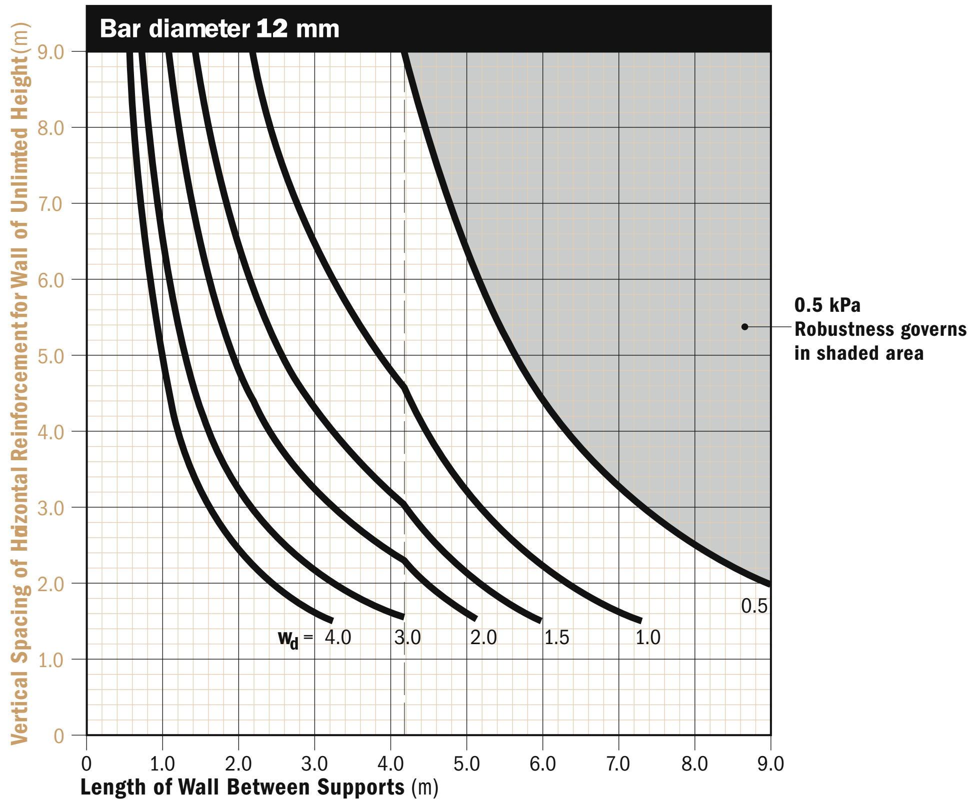

Design pressure, $w_d$ (kPa)



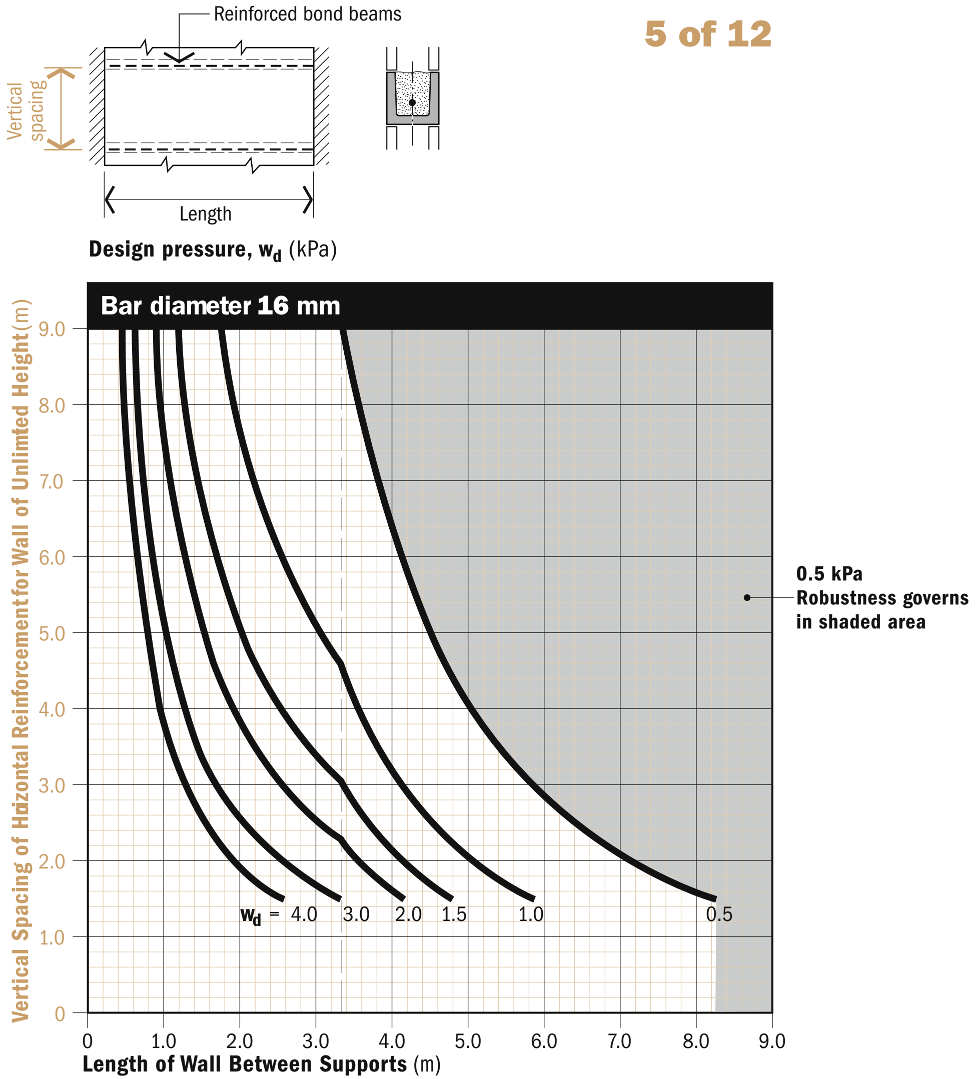


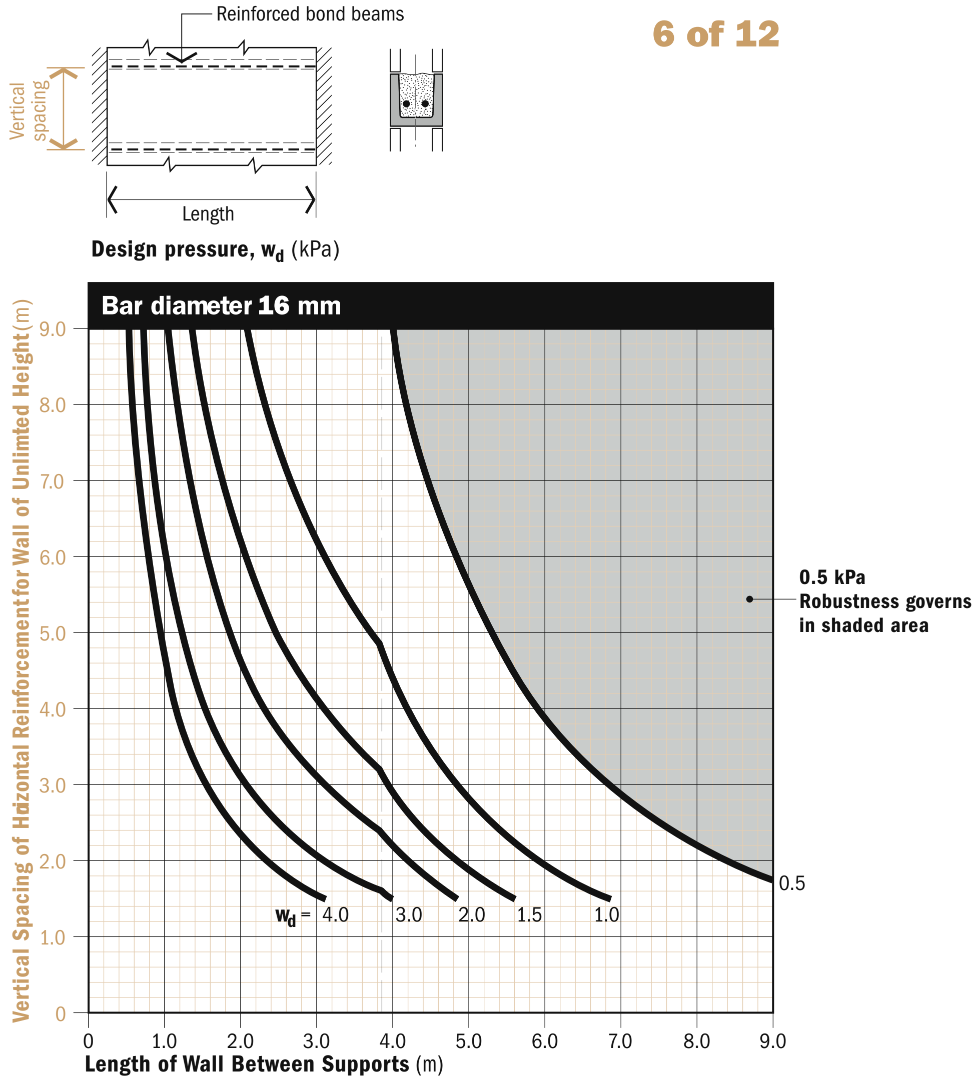

Design pressure, $w_d$ (kPa)



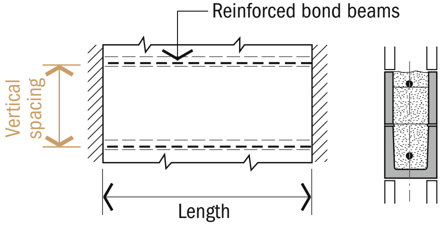

NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties

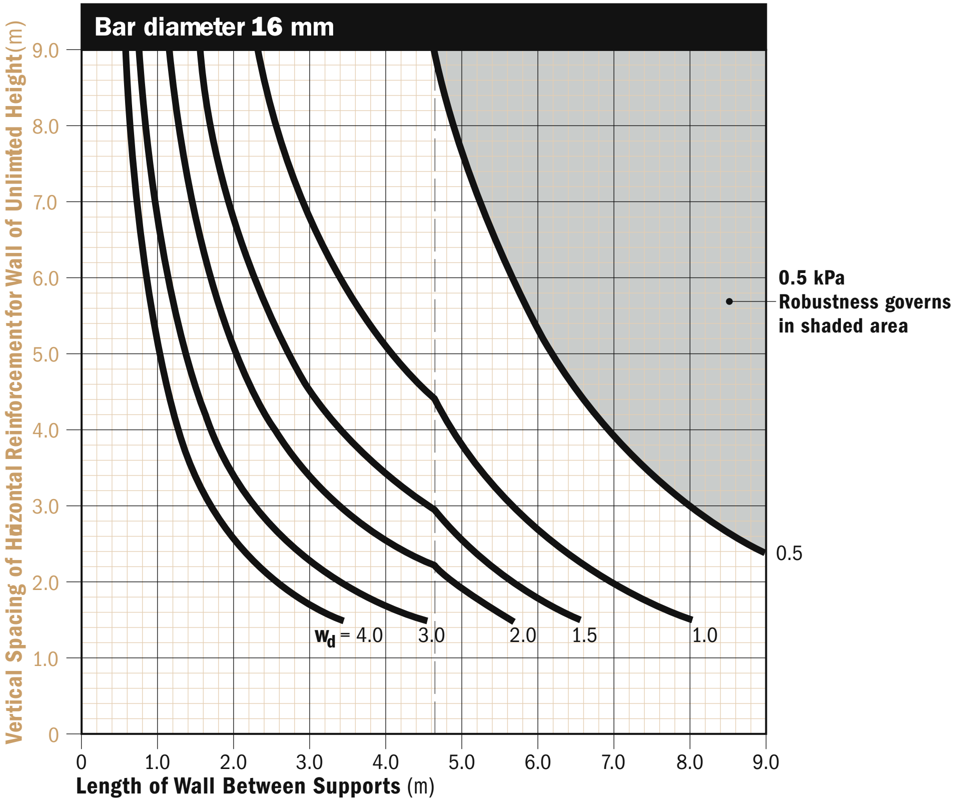




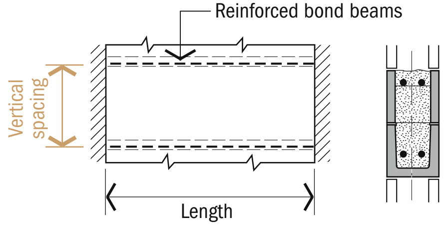




NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties

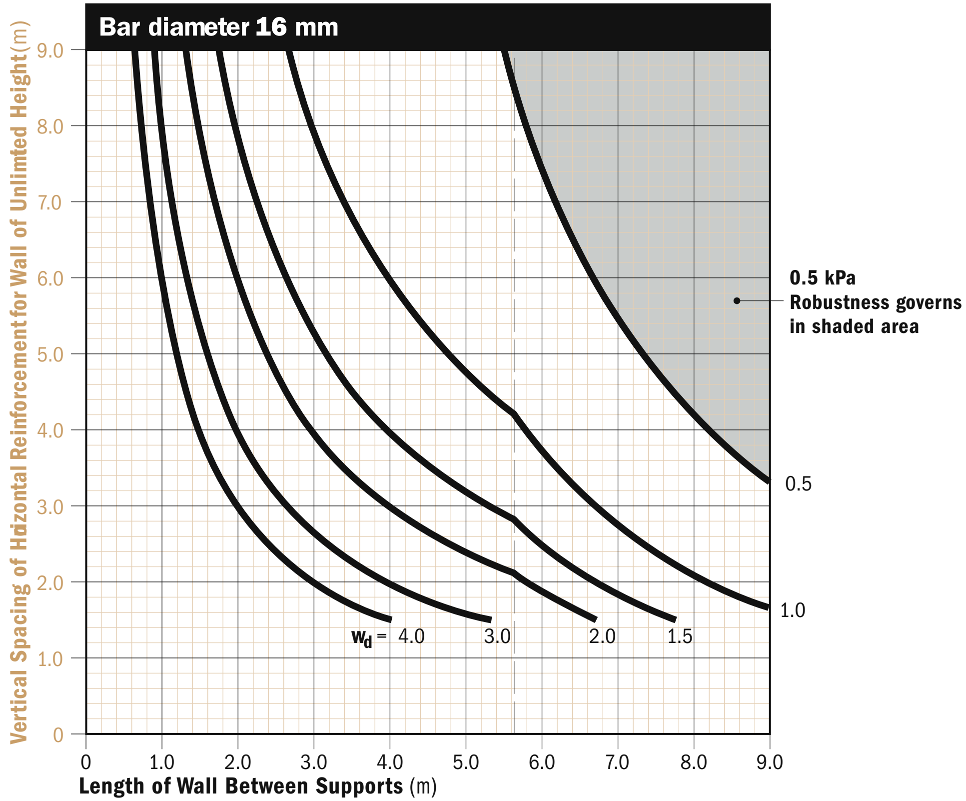




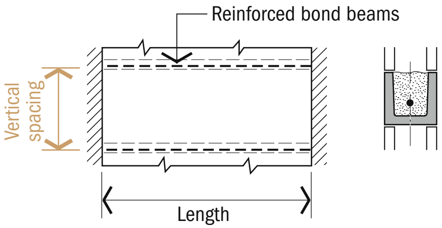

**Design pressure, $w_d$** (kPa)



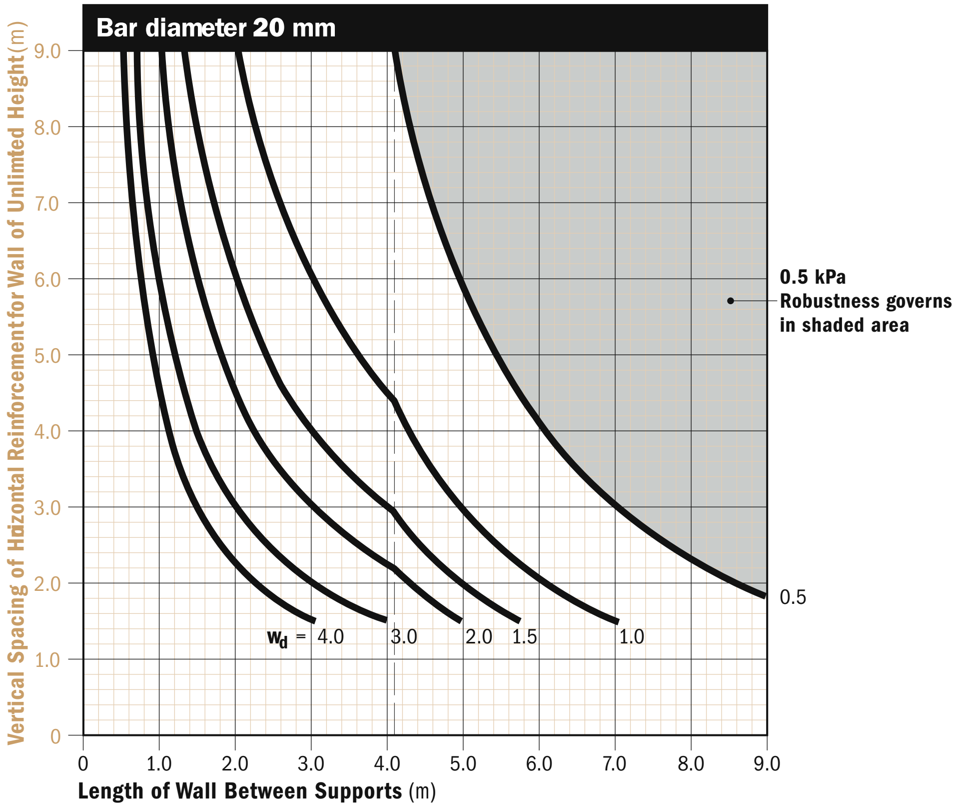

NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties

## REINFORCED AND MIXED CONSTRUCTION – Horizontally-reinforced bond beams 190-mm leaf (390 x 190 units 30 mm face-shell bedded)

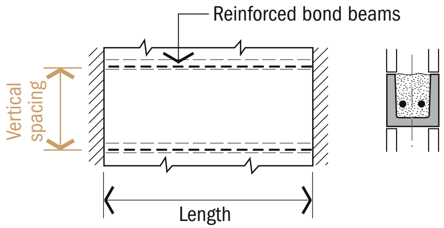




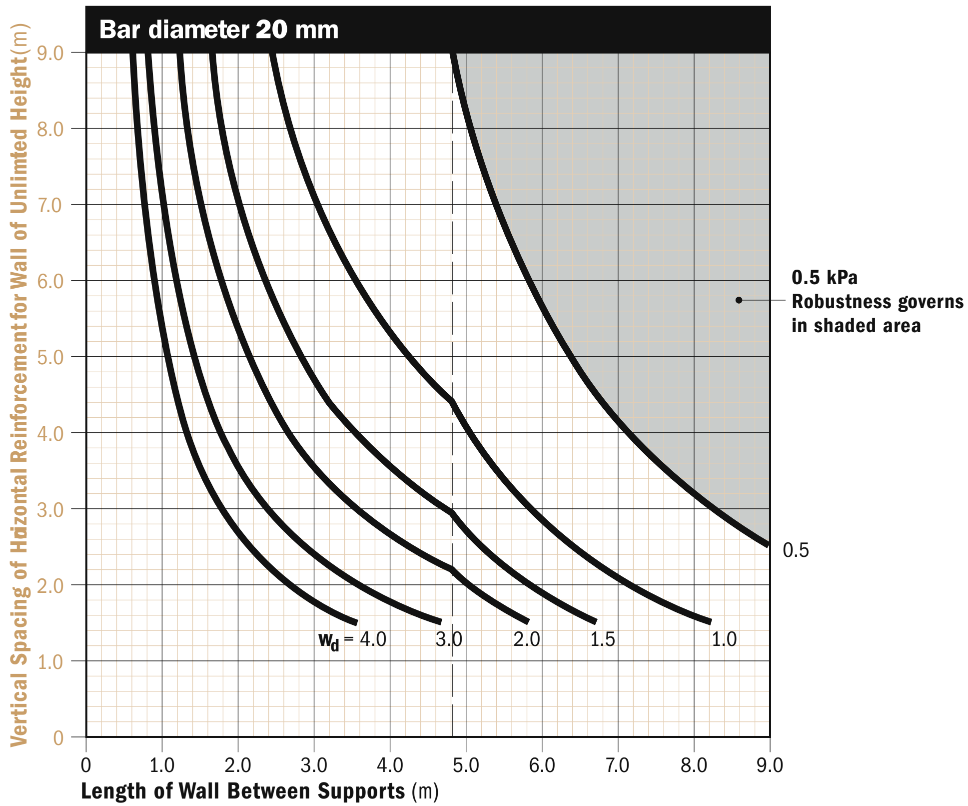

**Design pressure, $w_d$** (kPa)



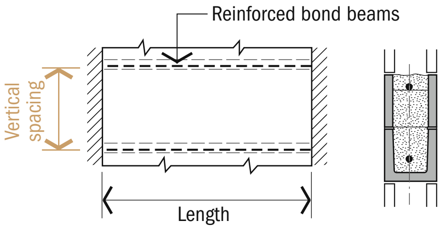


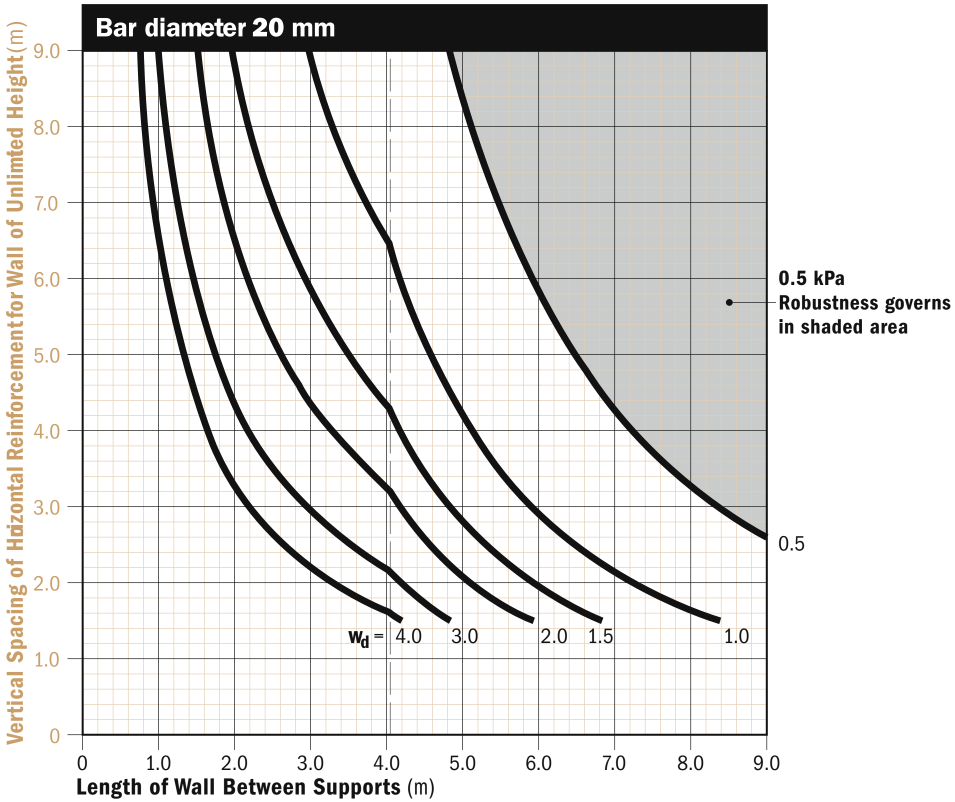




**Design pressure, $w_d$** (kPa)

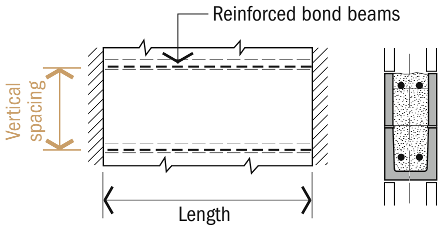



NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties

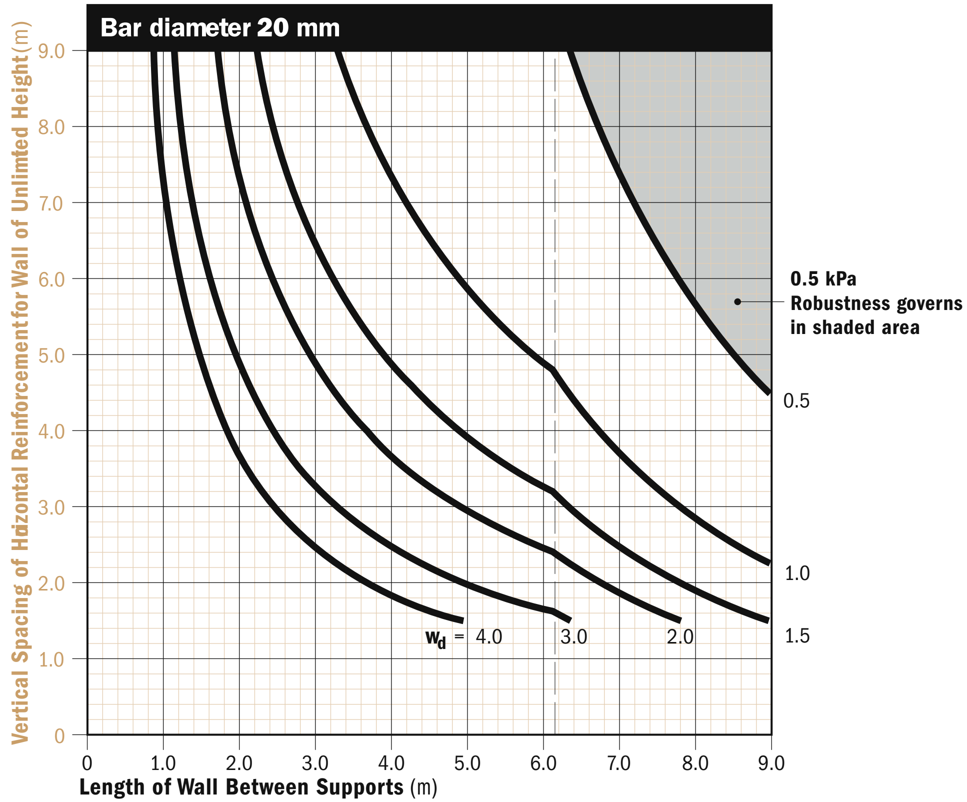


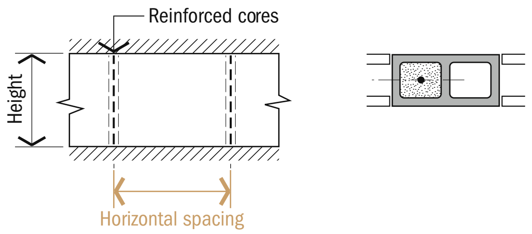


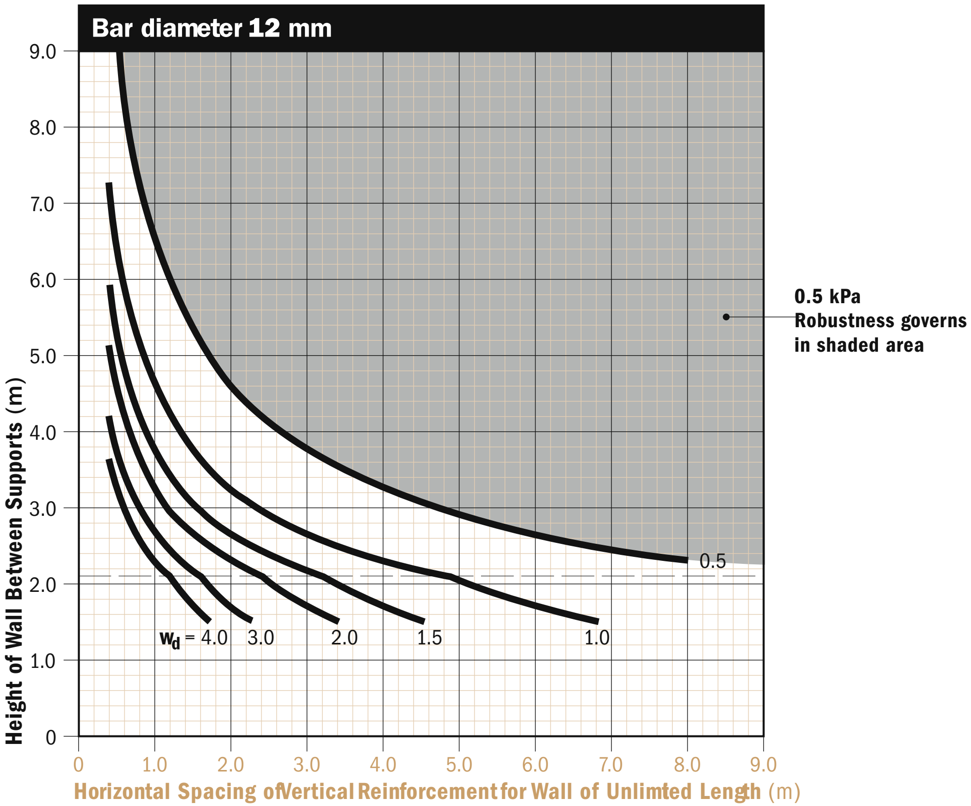

**Design pressure, $w_d$** (kPa)



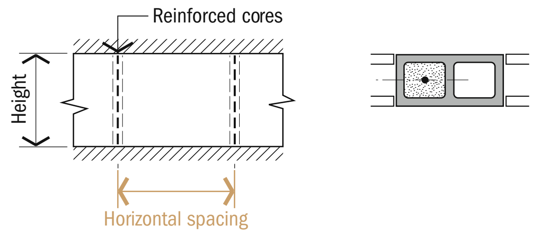

NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties



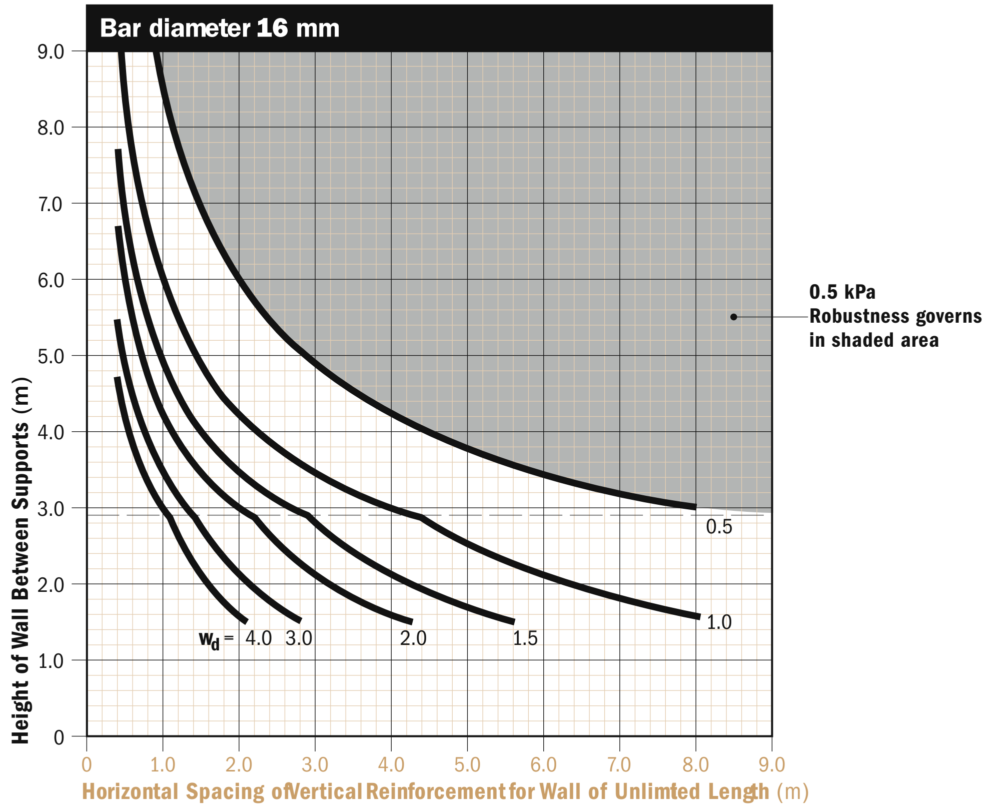

Design pressure, $w_d$ (kPa)



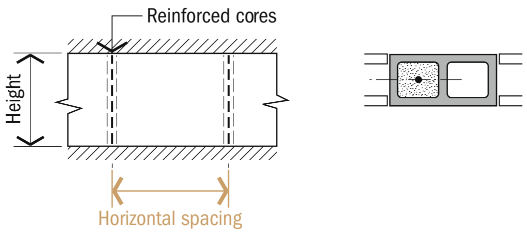



Design pressure, $w_d$ (kPa)

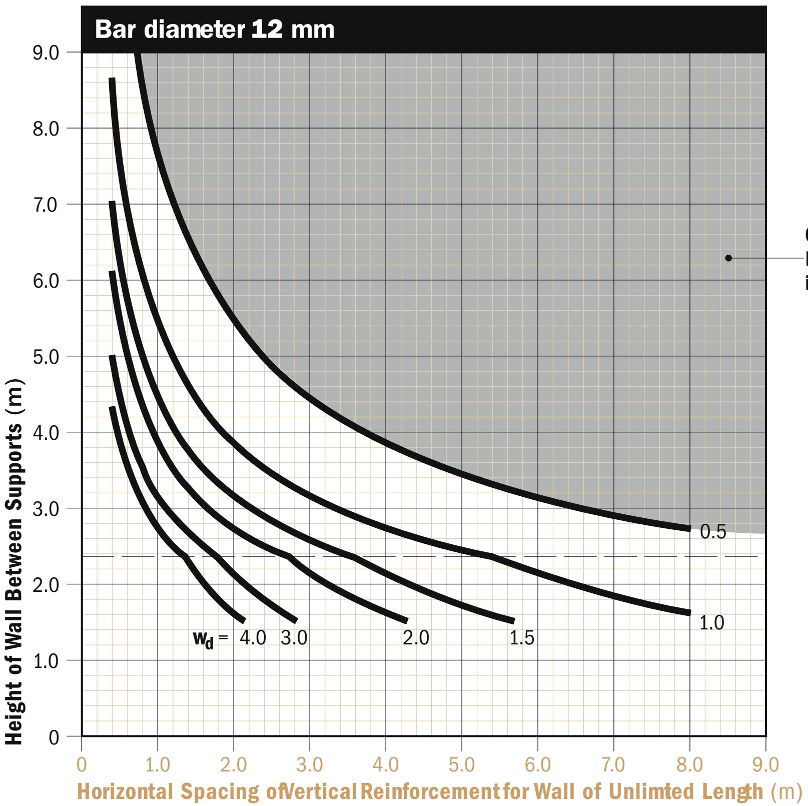


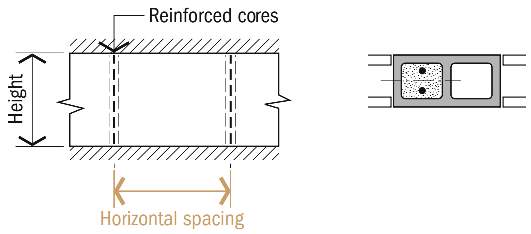

NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties





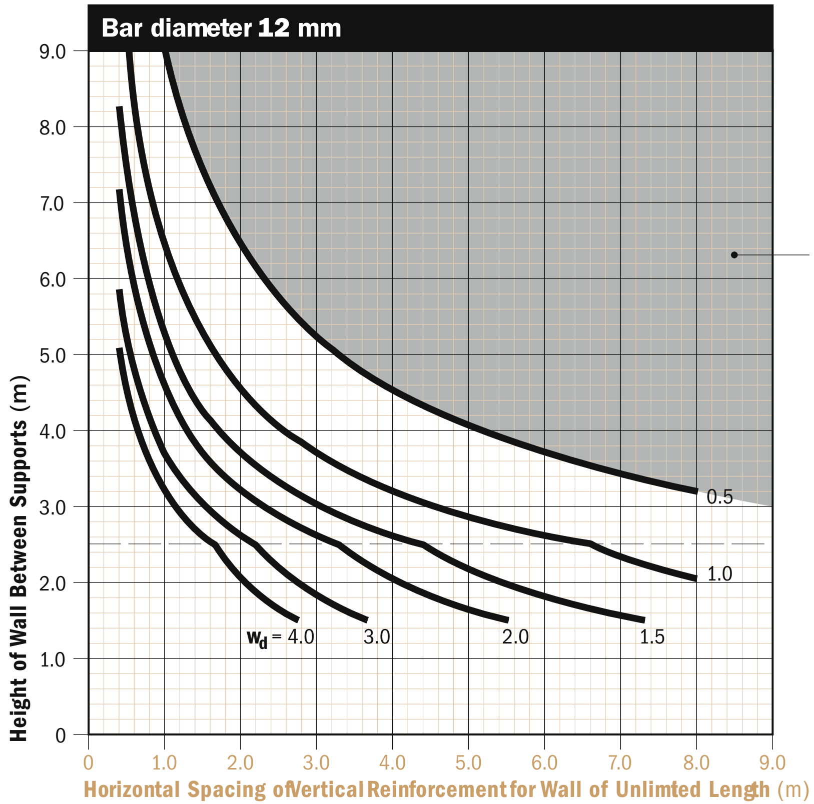

**Design pressure, $w_d$** (kPa)



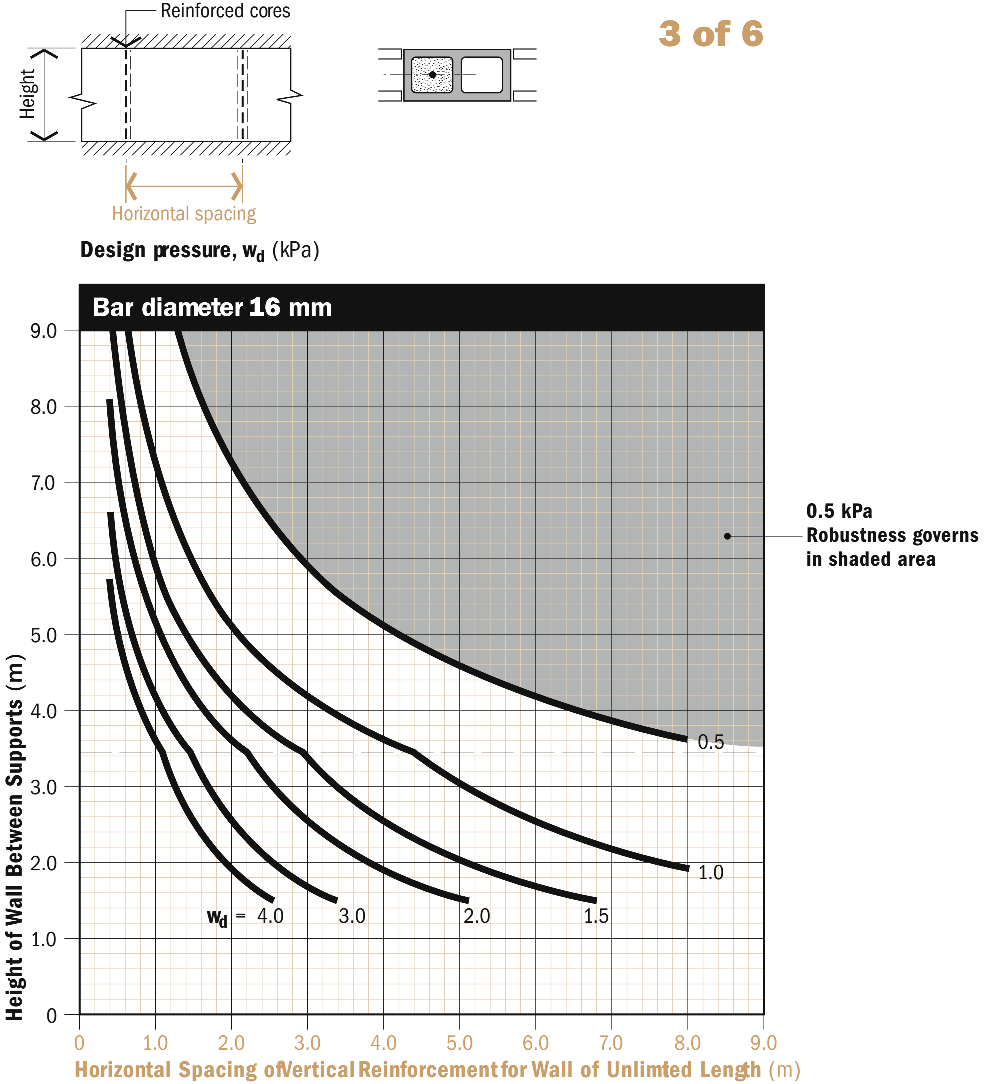





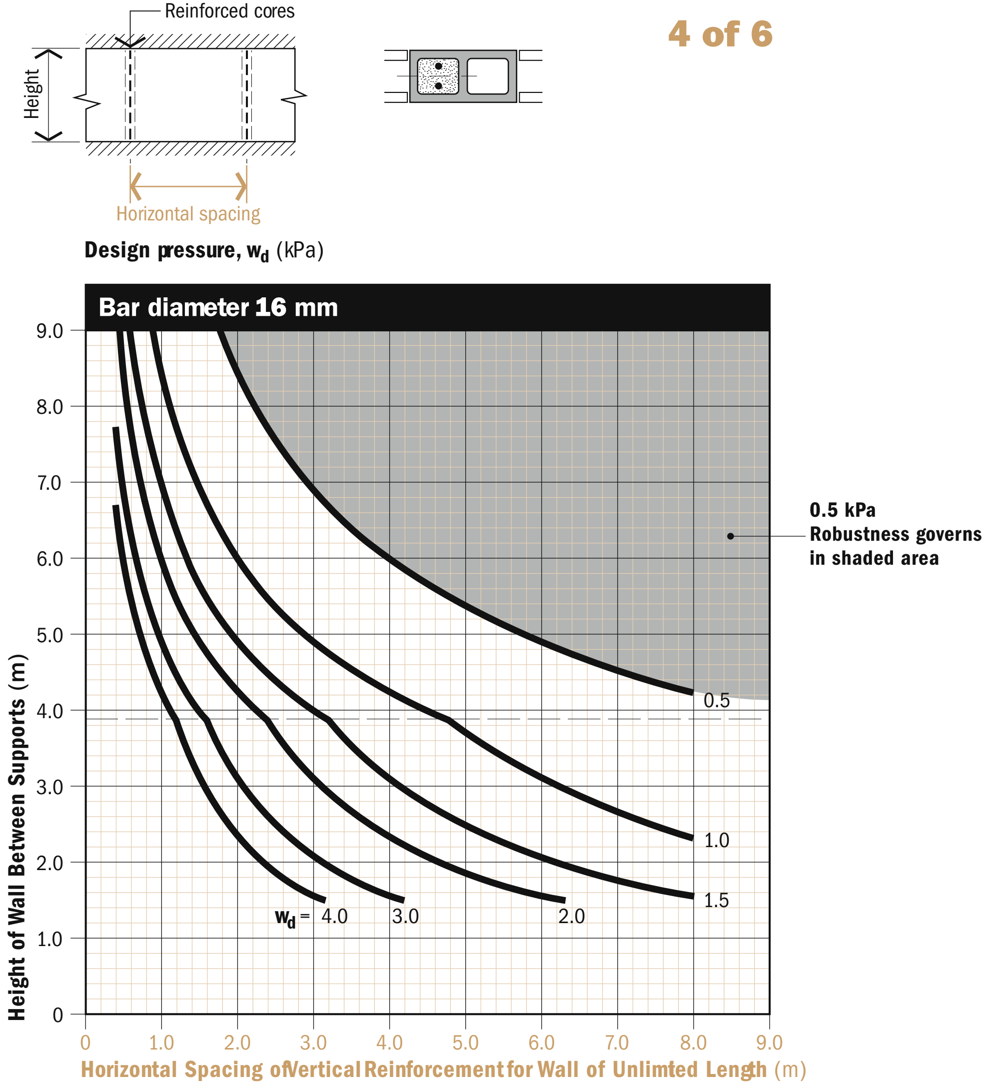

**Design pressure, $w_d$** (kPa)



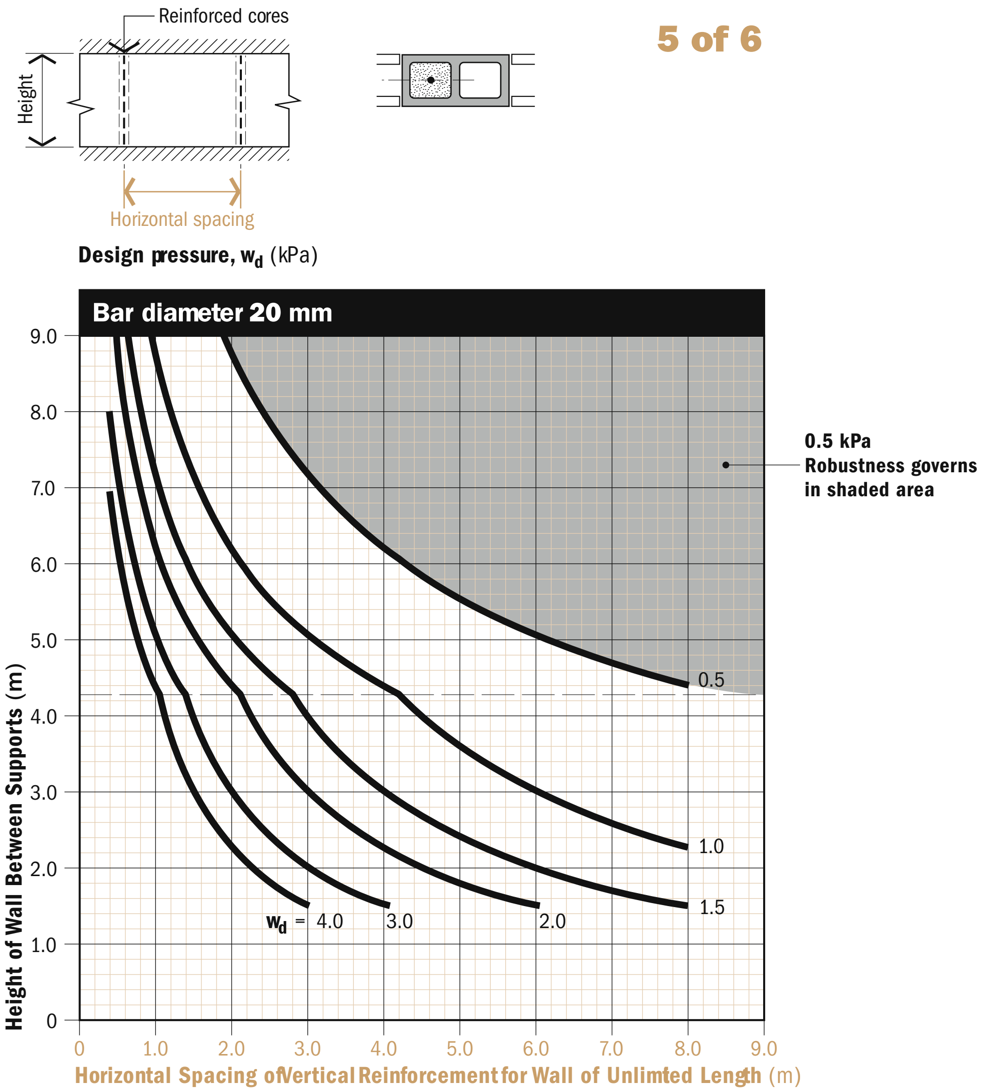

NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties



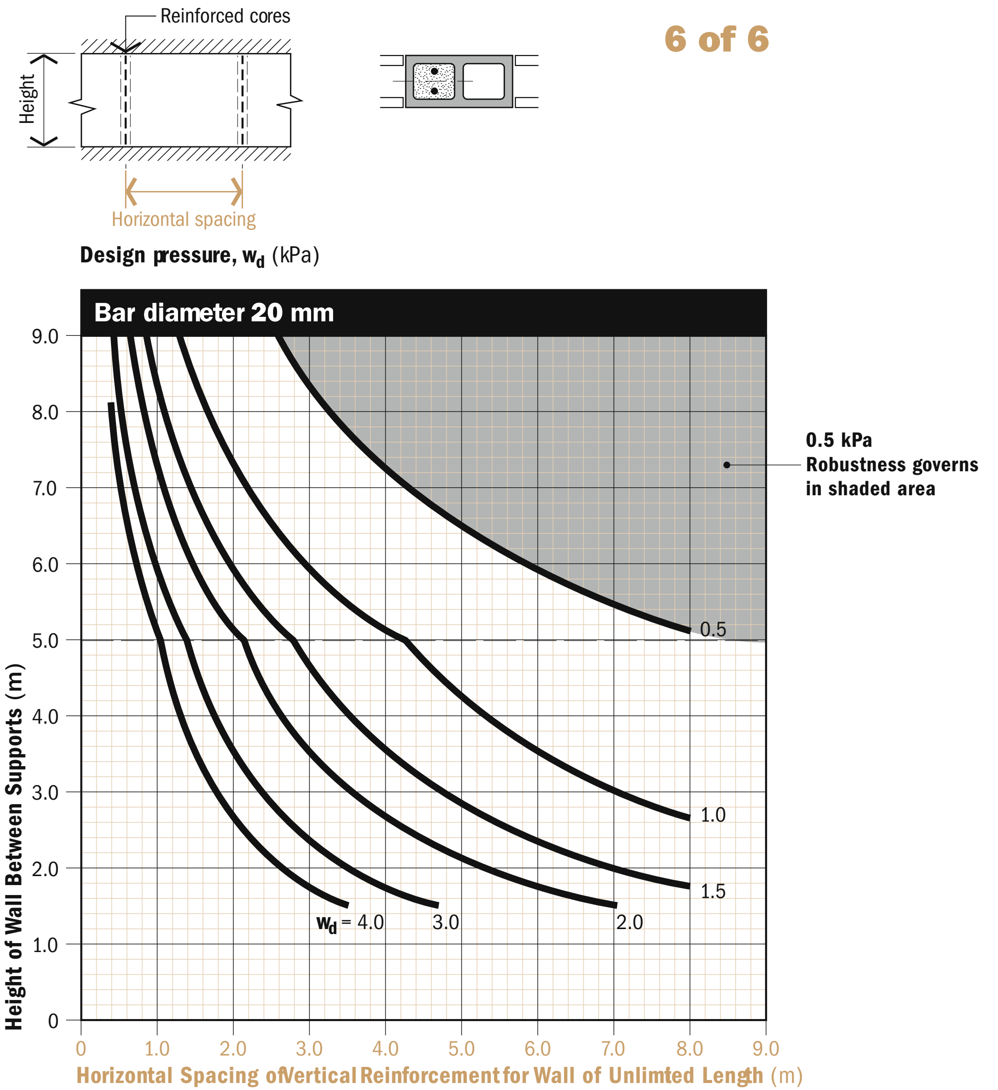



NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties






NOTE: It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties

# 6.4 WORKED EXAMPLE

## 6.4.1 GENERAL

### Purpose of the worked examples

The purpose of the following worked examples is to demonstrate the steps to be followed when performing manual calculations or when preparing computer software for the analysis and design of masonry. The worked examples also serve the purpose of demonstrating the origin of the Standard Designs which are based on similar masonry capacity considerations. Although comprehensive in its treatment of AS 3700, the worked examples are not intended to analyze or design all parts of the particular structure. They deal only with enough to demonstrate the design method.

### Design and detailing

All design and detailing shall comply with the requirements of AS 3700 and, where appropriate, AS/NZS 1170.

It is the designer's responsibility to allow for the effects of control joints, chases, openings, strength and stiffness of ties and connectors, and strength and stiffness of supports, in addition to normal considerations of loads and masonry properties. Control joints and openings must be treated as free ends as specified by AS 3700.

### Masonry properties

The worked examples in this chapter are based on masonry properties complying with the General Specification set out in **Part C:Chapter 2**, modified as noted in the calculations and as noted below.

**Hollow concrete blocks**
Width 90 mm, 110 mm, 140 mm and 190 mm

Height 190 mm

Length 390 mm

Face-shell bedded

Minimum face-shell thickness,
$t_s$ = 25 mm for 90 mm, 110 mm and 140 mm units
$t_s$ = 30 mm for 190 mm units

Minimum characteristic compressive strength,
$f'_{uc}$ =15 MPa

Minimum characteristic lateral modulus of rupture,
$f'_{ut}$ = 0.8 MPa

**Solid or cored concrete bricks**
Width 110 mm

Height 76 mm

Length 230 mm

Fully bedded

Minimum characteristic compressive strength,
$f'_{uc}$ =10 MPa

Minimum characteristic lateral modulus of rupture,
$f'_{ut}$ = 0.8 MPa

**Mortar joints**
Mortar type M3 (or M4)

Joint thickness 10 mm

**Concrete grout**
Minimum characteristic compressive strength,
$f'_c$ = 20 MPa

Minimum cement content 300 kg/m$^3$

**Steel reinforcement**
N12, N16 or N20 as noted.

## 6.4.2 INDEX TO WORKED EXAMPLES

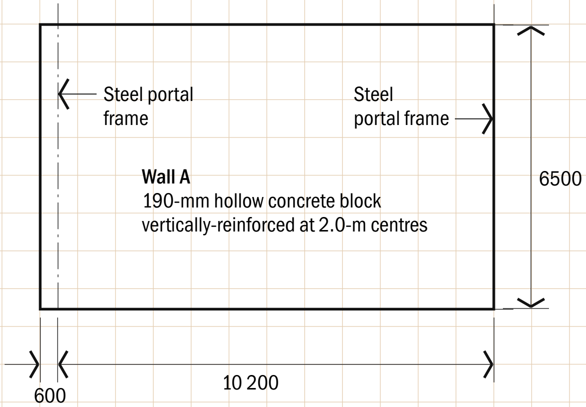
Two examples are provided.

**Example 1:** Design for wind and earthquake loading, reinforced and unreinforced walls of a low-rise industrial building.

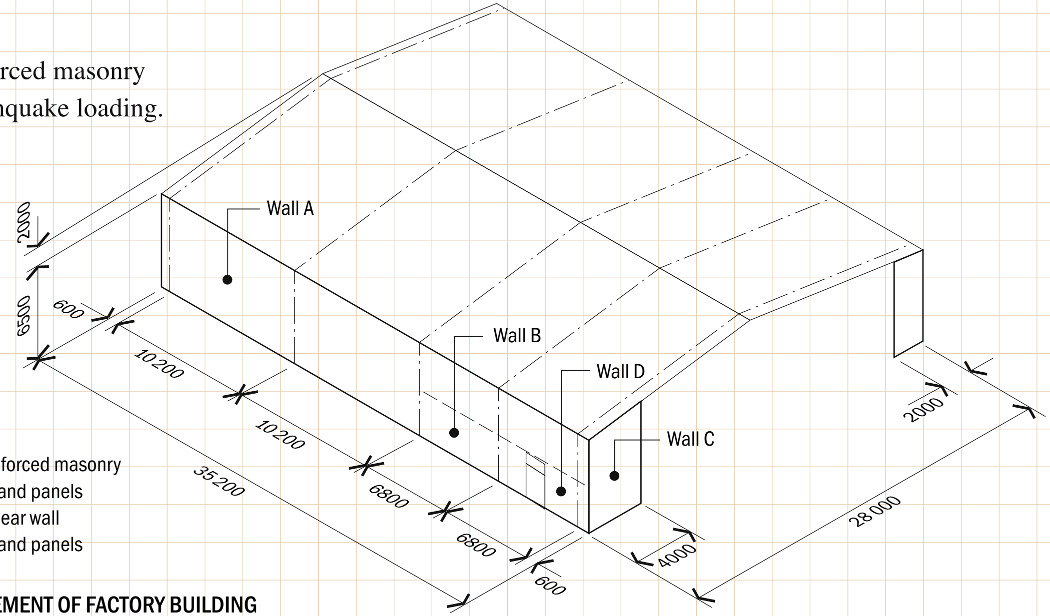
**Example 2:** Design for earthquake loading, reinforced and unreinforced walls in a medium-rise residential building.

## DESIGN BRIEF

For low-rise industrial building in Sydney on less than 30 m of hard clay, design unreinforced masonry (Wall 'D') and reinforced masonry (Wall 'A' and Wall 'C') for wind load of 1 kPa and earthquake loading.

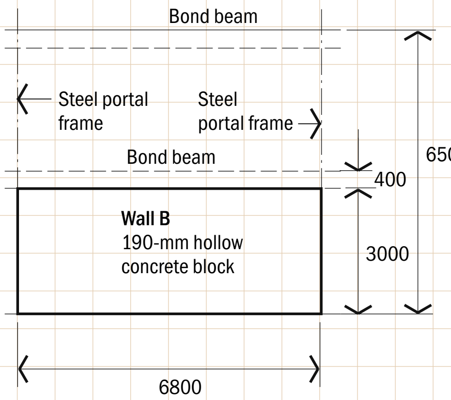



WALL 'A' ARRANGEMENT

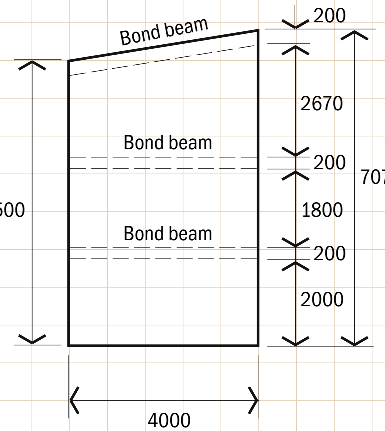



Wall A: Vertically-reinforced masonry
Wall B: Bond beams and panels
Wall C: Reinforced shear wall
Wall D: Bond beams and panels

GENERAL ARRANGEMENT OF FACTORY BUILDING

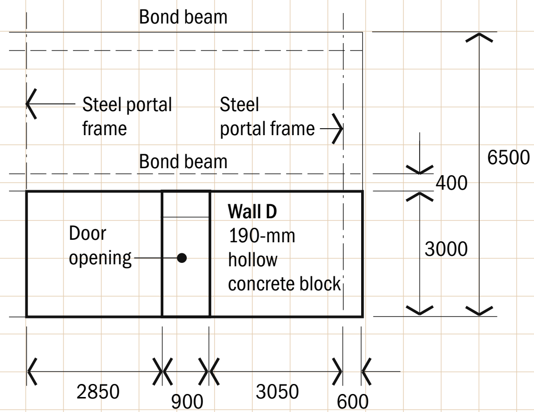



WALL 'B' ARRANGEMENT




**Wall C**
190-mm hollow concrete block with bond beams and vertical reinforcement

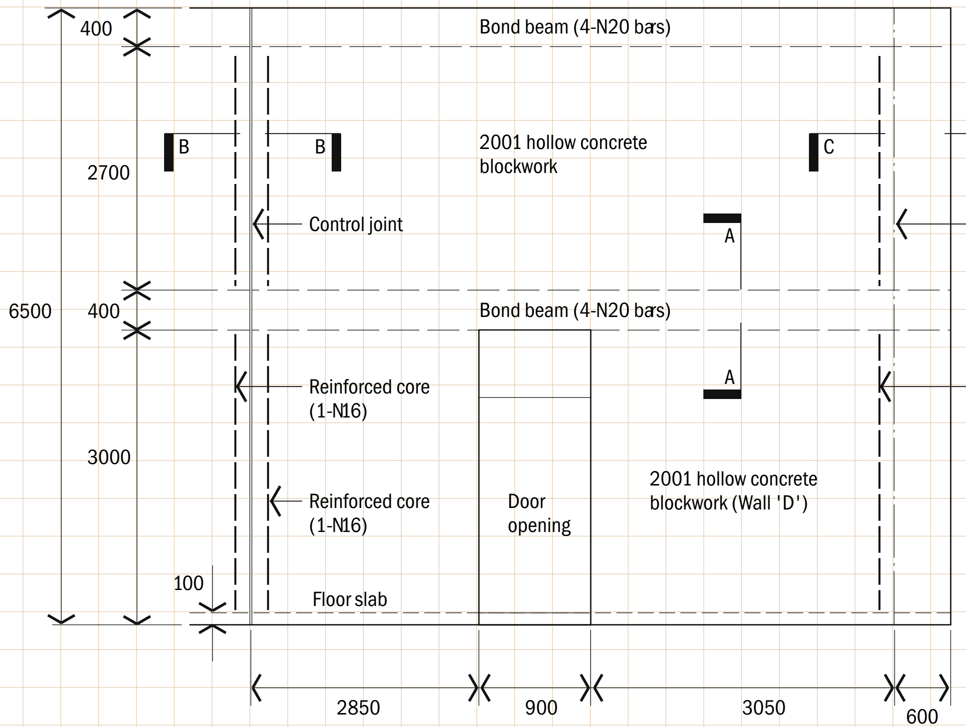
WALL 'C' ARRANGEMENT



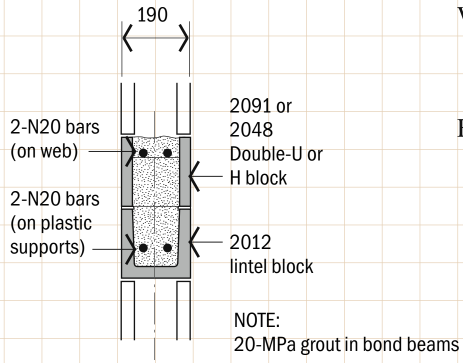

WALL 'D' ARRANGEMENT

## UNREINFORCED MASONRY

**Masonry Properties**




WALL 'D' DETAILS




SECTION A-A

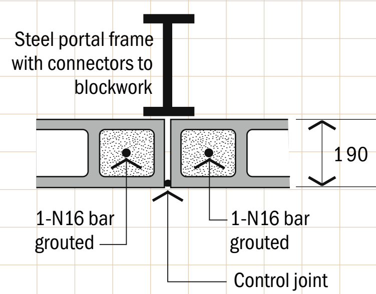
Width of masonry unit

$t_u = 190$ mm

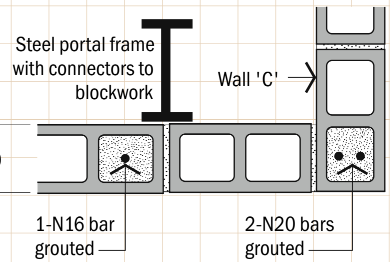
Face-shell thickness

$t_s = 30$ mm

NOTE:
20-MPa grout in reinforced cores




SECTION B-B




SECTION C-C

Characteristic flexural strength

$f'_{mt} = 0.2$ MPa *3.3.3*

Capacity reduction factor

$ø = 0.6$ *Table 4.1*

Shear strength at mortar joints

$f'_{ms} = 1.25\ f'_{mt}$ *3.3.4*

$= 1.25 \times 0.2$

$= 0.25$ MPa

$< 0.35$ MPa OK

$> 0.15$ MPa OK

Shear strength at dpc

$f'_{ms} = 0$

Load factor on resisting loads

$\gamma_R = 0.8$ *AS/NZS 1170.1*

*NOTE: All references are to AS 3700 unless noted otherwise*

Density of wall material

$D_{ens} = 21.8$ kN/m$^3$

Material thickness of wall

$t_m = 96$ mm

Height of wall acting at base

$H_e = 6.5$ m

**Shear Capacity of Unreinforced Masonry for Loads other than Earthquake**

Design compressive force at base of wall

$F_d = \gamma_R\ D_{ens}\ t_m\ H_e$

$= 0.8 \times 21.8 \times 0.096 \times 6.5$

$= 10.9$ kN/m

Area resisting shear

$A_{dw} = 2\ t_s$

$= 2 \times 30 \times 1000$

$= 60{,}000$ mm$^2$/m

Design compressive stress at base

$f_d = \dfrac{F_d}{A_d}$ Where $A_d = A_{dw}$

$= \dfrac{10.9 \times 1000}{60{,}000}$

$= 0.18$ MPa

$< 2.0$ MPa OK *7.5.4.1*

Shear factor for embossed plastic dpc

$k_v = 0.3$ *3.3.5 Table 3.3*

Shear bond strength at mortar joint

$V_o = ø\ f'_{ms}\ A_{dw}$ *7.5.4(1)*

$= \dfrac{0.6 \times 0.25 \times 60{,}000}{1{,}000}$

$= 9.0$ kN/m

Shear friction strength

$V_1 = k_v\ f_d\ A_{dw}$ *7.5.4(1)*

$= \dfrac{0.3 \times 0.18 \times 60{,}000}{1{,}000}$

$= 3.2$ kN/m

Shear capacity at mortar joint

$V_{cap} = V_o + V_1$ *7.5.4(1)*

$= 9.0 + 3.2$

$= 12.2$ kN/m

Shear capacity at dpc

$V_{cap} = V_o + V_1$ *7.5.4(1)*

$= 0 + 3.2$

$= 3.2$ kN/m

$>$ wind load

$V_d = \dfrac{wH}{2}$ For 1 kPa wind

$= \dfrac{1.0 \times 3.0}{2}$

$= 1.5$ kN/m OK

**Shear Capacity of Unreinforced Masonry for Earthquake Loads**

Design compressive force at base of wall

$G_g = \gamma_R\ (G + \psi_c Q)$

$= F_d$

$= 10.9$ kN/m

Design compressive stress at base

$f_d = \dfrac{F_d}{A_d}$ Where $A_d = A_{dw}$

$= \dfrac{10.9 \times 1000}{60{,}000}$

$= 0.18$ MPa

Shear factor for embossed plastic dpc

$k_v = 0.3$ *3.3.5 Table 3.3*

Shear bond strength at mortar joint

$V_o = ø\ f'_{ms}\ A_{dw}$

$= \dfrac{0.3 \times 0.25 \times 60{,}000}{1{,}000}$

$= 9.0$ kN/m

Shear friction strength

$V_1 = 0.9\ k_v\ f_{de}\ A_{dw}$

$= \dfrac{0.9 \times 0.3 \times 0.18 \times 60{,}000}{1{,}000}$

$= 2.9$ kN/m

Shear capacity at mortar joint

$V_{cap} = V_o + V_1$
$= 9.0 + 2.9$
$= 11.9$ kN/m

Shear capacity at dpc

$V_{cap} = V_o + V_1$
$= 0 + 2.9$
$= 2.9$ kN/m
> earthquake load
$V = 0.10\ G_{gi}$
$= 0.10 \times 10.9/_2$
$= 0.55$ kN/m OK

**Moment Capacity of Unreinforced Masonry Subject to Transient Loads (eg wind or earthquake)**

Section moduli $Z_d = Z_u = Z_p$

$$Z_d = \frac{1000\ t_u^2}{6}\left[6\left(\frac{t_s}{t_u}\right) - 12\left(\frac{t_s}{t_u}\right)^2 + 8\left(\frac{t_s}{t_u}\right)^3\right]$$

$$= \frac{1000 \times 190^2}{6}\left[6\left(\frac{30}{190}\right) - 12\left(\frac{30}{190}\right)^2 + 8\left(\frac{30}{190}\right)^3\right]$$

$= 4.09 \times 10^6$ mm$^3$/m

Mortar joint thickness

$t_j = 10$ mm

Length of masonry unit

$l_u = 390$ mm

Height of masonry unit

$h_u = 190$ mm

Lateral load parameters

$$G = \frac{2\ (h_u + t_j)}{l_u + t_j} \qquad 7.4.4.2$$

$$= \frac{2\ (190 + 10)}{390 + 10}$$

$= 1.0$

$$A = (l_u + t_j)\sqrt{1 + G^2} \qquad 7.4.4.3\ *$$

$= (390 + 10)\sqrt{1 + 1^2}$

$= 566$

$$B = \frac{h_u + t_j}{\sqrt{1 + G^2}} \qquad 7.4.4.3$$

$$= \frac{190 + 10}{\sqrt{1 + 1^2}}$$

$= 141$

$$C = \left[\frac{B\ t_s}{1.5\ B + 0.9\ t_s} + t_u - t_s\right] \qquad 7.4.4.3\ *$$

$$= \left[\frac{141 \times 30}{(1.5 \times 141 + (0.9 \times 30)} + 190 - 30\right]$$

$= 178$

* *Note: Terms A and C are not used in AS 3700, but are included here for simplicity*

Equivalent torsional section modulus

$$Z_t = \frac{2\ B\ t_s\ C}{A} \qquad 7.4.4.3$$

$$= \frac{2 \times 141 \times 30 \times 178}{566}$$

$= 2666$ mm$^3$/mm

Equivalent torsional strength

$f'_t = 2.25\sqrt{f'_{mt}}$ 7.4.4.3
$= 2.25\sqrt{0.2}$
$= 1.01$ MPa

Characteristic lateral modulus of rupture

$f'_{ut} = 0.8$ MPa 3.2

Load factor on resisting loads

$\gamma_R = 0.8$

Density of wall material

$D_{ens} = 21.8$ kN/m$^3$

Material thickness of wall

$t_m = 96$ mm

Height of wall acting at mid-height of the panel being designed

$H_e = 6.5 - 1.5$
$= 5.0$ m

Assuming that bond beams do not distribute vertical loads to the ends of the panels

Design compressive force at mid-height of wall

$F_d = \gamma_R \; D_{ens} \; t_m \; H_e$

$= 0.8 \times 21.8 \times 0.096 \times 5.0$

$= 8.4$ kN/m

Design compressive stress at mid-height

$f_d = \dfrac{F_d}{A_d}$

$= \dfrac{8.4 \times 1000}{60{,}000}$

$= 0.14$ MPa

Perpend spacing factor

$k_p = \min.\left(\dfrac{S_p}{t_w}, \dfrac{S_p}{h_u}, 1.0\right)$ *7.4.3.4*

$= \min.\left(\dfrac{190}{190}, \dfrac{190}{190}, 1.0\right) = 1.0$

Vertical moment capacity * *7.4.2*

$M_{cv} = \phi f'_{mt} Z_d + f_d Z_d$

$= (0.6 \times 0.2 \times 4.09) + (0.14 \times 4.09)$

$= 1.06$ kNm/m

$< 3.0 \, \phi \, f'_{mt} Z_d$

$= 3.0 \times 0.6 \times 0.2 \times 4.09$

$= 1.47$ kNm/m

* *Note: In this example, $M_{cv}$, is never required*

Horizontal moment capacity *7.4.3*

$M_{ch} = \phi \, (0.44 f'_{ut} Z_u + 0.56 f'_{mt} Z_p)$

$= 0.6[(0.44 \times 0.8 \times 4.09) + (0.56 \times 0.2 \times 4.09)]$

$= 1.14$ kNm/m

$< 2.0 \, \phi \, k_p \sqrt{f'_{mt}} \left(1 + \dfrac{f_d}{f'_{mt}}\right) Z_d$

$= 2.0 \times 0.6 \times 1.0 \times \sqrt{0.2} \left(1 + \dfrac{0.14}{0.2}\right) 4.09$

$= 3.73$ kNm/m OK

$< 4.0 \, \phi \, k_p \sqrt{f'_{mt}} \; Z_d$

$= 4.0 \times 0.6 \times 1.0 \times \sqrt{0.2} \times 4.09$

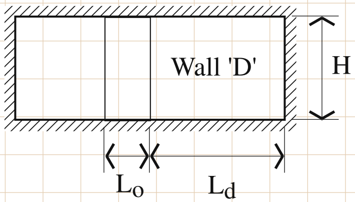
$= 4.39$ kNm/m OK

Diagonal moment capacity

$M_{cd} = \phi \, f'_t Z_t$ *7.4.4.3(1)*

$= 0.6 \times 1.01 \times \dfrac{2666}{1000}$

$= 1.61$ kNm/m




$L_o = 0.9$ m

$L_d = 3.05$ m

$H_d = \dfrac{H}{2}$

$= 1.5$ m

Slope factor

$\alpha = \dfrac{G \, L_d}{H_d}$ *7.4.4.2*

$= \dfrac{1.0 \times 3.05}{1.5}$

$= 2.03$

$> 1.0$

Aspect factor

$a_f = \dfrac{\alpha}{1 - \dfrac{1}{3\alpha} + \dfrac{L_o}{2L_d}}$ *Table 7.5*

$= \dfrac{2.03}{1 - \dfrac{1}{3 \times 2.03} + \dfrac{0.9}{2 \times 3.05}}$

$= 2.066$

Restraint factor for continuous past first supported edge

$R_{f_1} = 1$

$k_1 = R_{f_1}$ *Table 7.5*

$= 1$

$k_2 = 1 + \dfrac{1}{G^2}$ *Table 7.5*

$= 1 + \dfrac{1}{1.0^2}$

$= 2.0$

Lateral load capacity

$$w_{cap} = \frac{2\, a_f}{L_d^2}(k_1\, M_{ch} + k_2\, M_{cd}) \qquad \textit{7.4.4.2(2)}$$

$$= \frac{2 \times 2.066}{3.05^2}[(1 \times 1.14) + (2 \times 1.61)]$$

$= 1.94$ kPa

> Windload

$w_d = 1.0$ kPa OK

> earthquake load

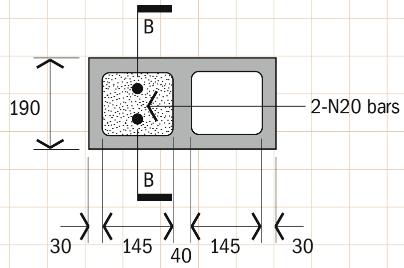
$w_d = 0.10\, G_{gi}$

$= 0.10\,(21.8 \times 0.096)$

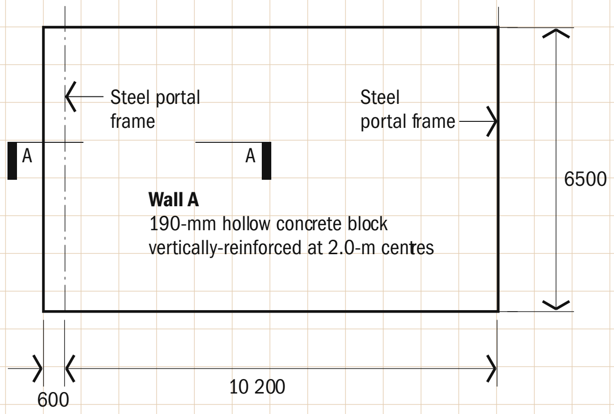
$= 0.21$ OK

**REINFORCED MASONRY**

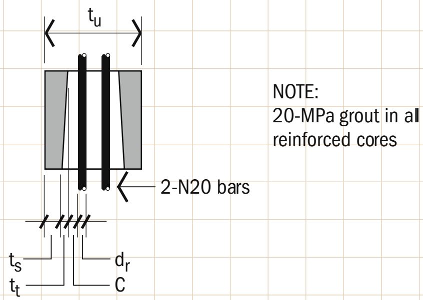
**Masonry Properties**



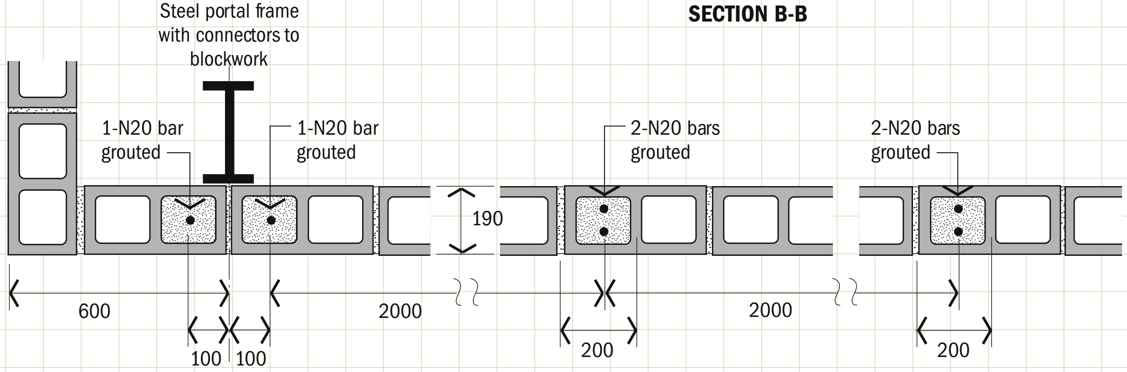

**TYPICAL REINFORCED CORE**




**WALL 'A' ARRANGEMENT**




**SECTION B-B**




$p_1 = \frac{2000 - 200}{2000} = 0.9$ $p_2 = 1.0 - p_1 = 0.1$

**SECTION A-A**

Width of masonry unit
$t_u = 190$ mm

Face-shell thickness
$t_s = 30$ mm

NOTE:
$t_s$ and $t_t$ may vary depending on manufacturer

Taper in face-shell
$t_t = 5$ mm

Bedded area of ungrouted masonry
$A_b = 2\, t_s\, l\, p_1$ *4.5.4*
$= 2 \times 30 \times 1000 \times 0.9$
$= 54{,}000\ mm^2/m$

Design cross-sectional area of member
$A_d = 2\, t_s\, l\, p_1 + t_u\, l\, p_2$ *4.5.6*
$= (2 \times 30 \times 1000 \times 0.9) + (190 \times 1000 \times 0.1)$
$= 73{,}000\ mm^2/m$

Design cross-sectional area of grout
$A_c = A_d - A_b$ *4.5.7*
$= 73{,}000 - 54{,}000$
$= 19{,}000\ mm^2/m$

Block height
$h = 190$ mm

Mortar joint thickness
$t_j = 10$ mm

Steel reinforcement to AS 1302
1-Y20 bar in each face of grouted cores

Diameter of bars
$d_r = 20$ mm

Yield strength of reinforcing bars to AS 1302
$f_{sy} = 500$ MPa *Table 3.7*

Reinforced masonry is below the dpc in contact with non-aggressive soils
C = 20 mm
> 15 mm *Table 5.1*
20 mm aggregate *11.7.2.5*

Effective depth

$$d = tu - ts - tt - \frac{d_r}{2} - C$$

$$= 190 - 30 - 5 - \frac{20}{2} - 20$$

$= 125$ mm

Area of main reinforcement
$A_s = 310\ mm^2$

Spacing of main reinforcement
$S_m = 2000$ mm
2000 mm OK *8.5*

Height ratio

$$\frac{h_u}{t_j} = \frac{190}{10}$$

$= 19.0$

Compressive strength factor
$k_h = 1.3$ *Table 3.2*

Masonry factor for face-shell-bedded concrete units
$k_m = 1.6$ *Table 3.1*

Mortar type
M3 (1:5 + water thickener)

Characteristic unconfined unit strength
$f'_{uc} = 15$ MPa

Characteristic masonry strength for 76 mm height units
$f'_{mb} = k_m \sqrt{f'_{uc}}$ *3.3.2(a)(i)*
$= 1.6 \sqrt{15}$
$= 6.2$ MPa

Characteristic unconfined masonry strength
$f'_m = k_h\, f'_{mb}$ *3.3.2(a)(i)*
$= 1.3 \times 6.2$
$= 8.06$ MPa

Characteristic grout cylinder strength

$f'_c = 20$ MPa

$> 12$ MPa [Cement, 300 kg/$m^3$ for durability *5.8*; Aggregate, 20 mm *11.7.3*]

Design characteristic grout strength

$f'_{cg} = 1.3\ f'_{uc}$ *3.5*

$= 1.3 \times 15$

$= 19.5$ MPa $< 20$ MPa OK

Capacity reduction factor

$\phi = 0.75$ *Table 4.1*

**Out-of-plane Shear Capacity for Reinforced Masonry**

Characteristic shear strength

$f'_{ms} = 0.35$ MPa (at interface) *3.3.4(c)*

$f'_{vm} = 0.35$ MPa *8.8*

Width of web

$b_w = 200$ mm/core

Design shear strength

$f_{vs} = 17.5$ MPa *8.8*

Cross-sectional area of main reinforcement

$A_{st} = 310\ mm^2$

$< 0.02\ b_w\ d$

$= 0.02 \times 200 \times 125$

$= 500\ mm^2$ OK

Cross-sectional area and spacing of shear reinforcement

$A_{sv} = 0$ (no stirrups) S = NA

Out-of-plane shear capacity *8.8*

$$V_{cap} = \phi\,(f'_{vm}\, b_w\, d + f_{vs}\, A_{st} + f_{vs}\, \frac{A_{sv}\, d}{S})$$

$$= 0.75\left[(0.35 \times 200 \times 125) + (17.5 \times 310) + 0\right]$$

$= 10.6$ kN/core

$>$ Wind load

$$V_d = \frac{w_{dw}\, B\, H}{2}$$

$$= \frac{1.0 \times 2.0 \times 6.4}{2}$$

$= 6.4$ kN/core OK

$>$ Earthquake load

$$V_d = \frac{w_{de}\, B\, H}{2}$$

$$= \frac{0.6 \times 2.0 \times 6.4}{2}$$

$= 3.8$ kN/core OK

**Moment Capacity for Reinforced Masonry**

Width of compression face

$b = 2\ t_u \times 2$ *8.6*

$= 2 \times 190 \times 2$

$= 760$ mm

$< 400 \times 2$

$= 800$ OK

$<$ Distance to structural end $+ 2\ t_u$ OK

Design area of reinforcement

$A_{sd} = A_s$

$= 310\ mm^2$

$$< \frac{(0.29)\ 1.3 f'_m\, b\, d}{f_{sy}}$$ *8.6*

$$= \frac{0.29 \times 1.3 \times 8.06 \times 760 \times 125}{500}$$

$= 577\ mm^2$ OK

$> 0.0013\ b\ d$ *8.6*

$= 0.0013 \times 760 \times 125$

$= 124\ mm^2$ OK

Moment capacity *8.6*

$$M_{cap} = \phi\, f_{sy}\, A_{sd}\, d\left[1 - \frac{0.6\, f_{sy}\, A_{sd}}{1.3 f'_m\, b\, d}\right]$$

$$= 0.75 \times 500 \times 310 \times 125\left[1 - \frac{0.6 \times 500 \times 310}{1.3 \times 8.06 \times 760 \times 125}\right]$$

$= 13.2$ kN.m/core

$>$ Wind load

$$M_d = \frac{w_{dw}\, B\, H^2}{8}$$

$$= \frac{1.0 \times 2.0 \times 6.4^2}{8}$$

$= 10.2$ kN.m/core OK

$>$ Earthquake load

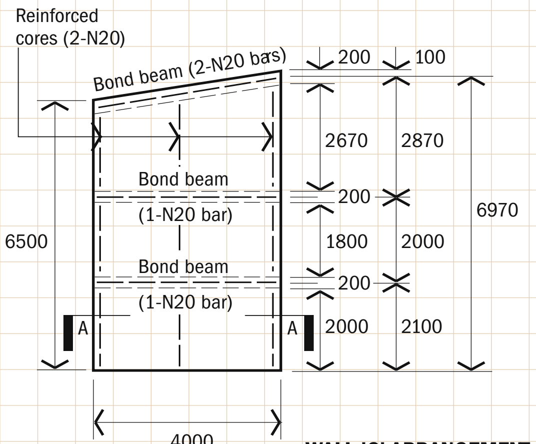
$$M_d = \frac{w_{de}\, B\, H^2}{8}$$

$$= \frac{0.6 \times 2.0 \times 6.4^2}{8}$$

$= 6.1$ kN.m/core OK

**In-plane Shear Capacity for Reinforced Masonry**

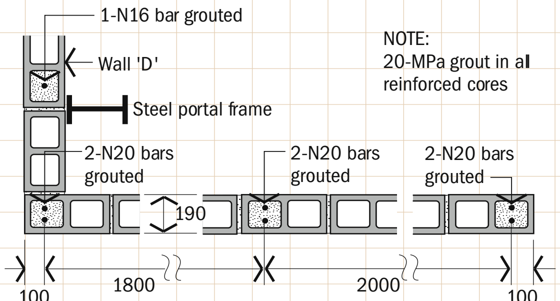
In-plane shear capacity is affected by the geometry of the wall being considered (Wall 'C')




**WALL 'C' ARRANGEMENT**

Vertical: $p_1 = \dfrac{4000 - (3 \times 200)}{4000} = 0.85$ $\quad p_2 = 1.0 - p_1 = 0.15$

Horizontal: $p_1 = \dfrac{6970 - (3 \times 200)}{6970} = 0.92$ $\quad p_2 = 1.0 - p_1 = 0.08$




**SECTION A-A**

Vertical reinforcement (2-N20, or equivalent, in each core)

$A_{s_v} = 2 \times 310 \times 3$ cores

$= 620\ mm^2$/core x 3 cores

$= 1860\ mm^2$

$A_{d_v} = 2\, t_s\, l\, p_1 + t_u\, l\, p_2$

$= (2 \times 30 \times 4000 \times 0.85) + (190 \times 4000 \times 0.15)$

$= 318{,}000\ mm^2$

$$\frac{A_{s_v}}{A_{d_v}} = \frac{1860}{318000}$$

$= 0.00585$

$> 0.0013$ OK

Horizontal reinforcement (2-N20 in top bond beam, 1-N20 in intermediate bond beams)

$A_{s_h} = (2 \times 310) + (1 \times 310) + (1 \times 310)$

$= 1240\ mm^2$

$A_{d_h} = 2\, t_s\, l\, p_1 + t_u\, l\, p_2$

$= (2 \times 30 \times 6970 \times 0.92) + (190 \times 6970 \times 0.08)$

$= 490{,}700\ mm^2$

$$\frac{A_{s_h}}{A_{d_h}} = \frac{1240}{490700}$$

$= 0.0025$ $\quad > 0.0007$ OK

Height of shear wall (to load application)

$H = 6.970$ m

Length of shear wall

$L = 4.000$ m

H to L ratio

$$\frac{H}{L} = \frac{6.97}{4.00}$$

$= 1.74$

$< 2.3$

$\therefore$ Design as shear wall *8.7.2*

Shear stress value

$$f_{vr} = 1.5 - 0.5\frac{H}{L}$$ *8.7.2*

$$= 1.5 - \frac{0.5 \times 6.97}{4.00}$$

$= 0.63$ MPa

For $^{H}/_{L} > 1.0$

Area of horizontal reinforcement crossing potential crack

$$A_s = \frac{A_{sh}\, L}{H}$$

$$= \frac{310 \times 4000}{6970}$$

$= 534\ mm^2$

In-plane shear capacity (based on stress)

$$V_{cap} = \phi\,(f_{vr} A_{d_v} + 0.8\, f_{sy} A_s) \qquad 8.7.2$$

$$= \frac{0.75}{1000}\left[(0.63 \times 318000) + (0.8 \times 500 \times 534)\right]$$

$$= 310 \text{ kN}$$

***Check anchorage against overturning***

Applied uniform vertical load (self weight)

$$P_v = 4.0 \times \left(\frac{6.5 + 7.07}{2}\right) \times \frac{2180 \times 0.096 \times 9.81}{1000}$$

$$= 55.4 \text{ kN}$$

$$< f'_m A_b \qquad 8.7.4$$

$$= \frac{8.06 \times 318000}{1000}$$

$$= 2563 \text{ kN} \quad \text{OK}$$

Reduction factor

$$k_{sw} = 1 - \frac{p_v}{A_b f'_m} \qquad 8.7.4$$

$$= 1 - \frac{55.4}{2563}$$

$$= 0.978$$

Anchorage steel is 2-N20 bars at end of wall

Area of anchorage steel

$$A_{sv} = 2 \times 310$$

$$= 620 \text{ mm}^2$$

Centroid distance

$l' = 100$ mm

$= 0.1$ m

In-plane shear capacity

(based on anchorage) 8.7.2

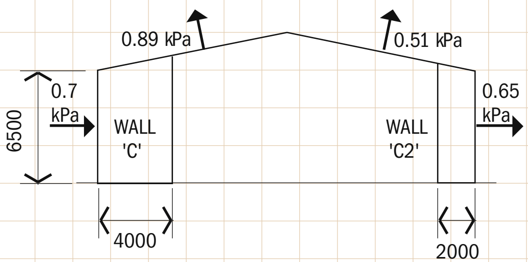
$$V_{cap} = \frac{\phi}{H}\left[k_{sw} P_v \frac{L}{2} + f_{sy} A_{sv} (L - 2\,l')\right]$$

$$= \frac{0.75}{6.97}\left[0.978 \times 55.4 \times \frac{4.0}{2} + \frac{500 \times 620}{1000}(4.0 - 2 \times 0.1)\right]$$

$$= 138 \text{ kN}$$

$\therefore$ this over-rides stress-based capacity

**In-Plane Shear Load on Wall 'C'**




WALL 'C' ARRANGEMENT

Earthquake loads from AS 1170.4

< wind loads

$\therefore$ Design shear wall for wind loads

Wind loads from AS 1170.2

Internal pressure on opposing walls does not contribute to total shear load

Assume all shear load is resisted by end walls. Half of load goes directly to floor slab, half of the residual goes to each end

Total shear load at one end

$$V_t = (0.7 + 0.65)\,\frac{6.5 \times 35.2}{4}$$

$$= 77 \text{ kN}$$

Shear on wall 'C'

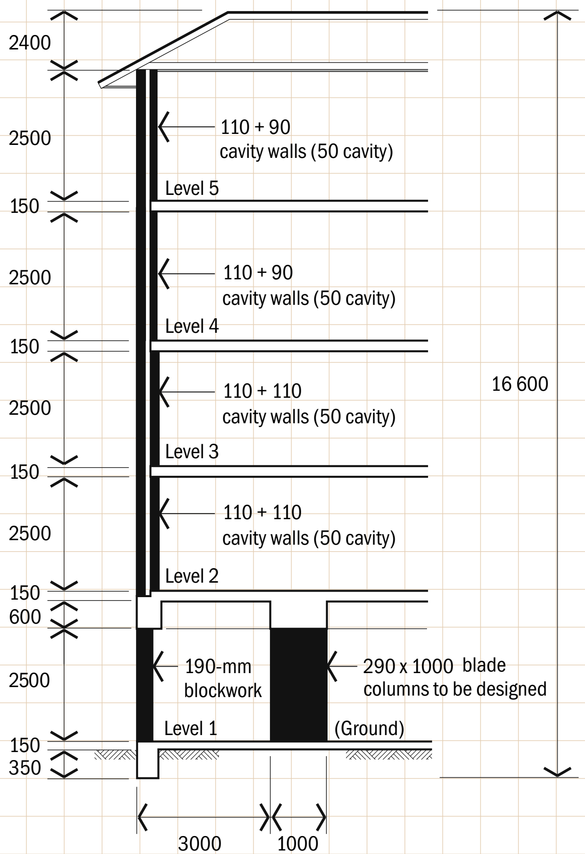
$$V_d = \frac{77 \times 4.0^3}{(4.0^3 + 2.0^3)}$$

$$= 68 \text{ kN}$$

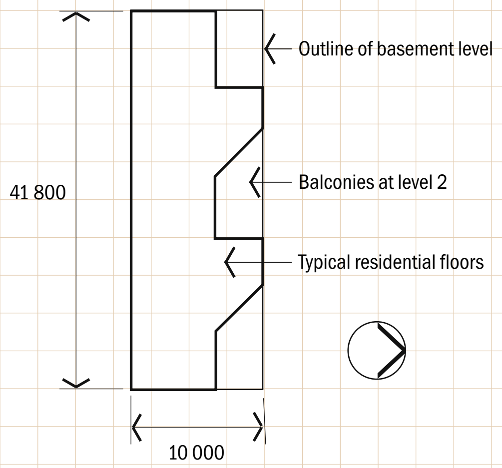
$$< 138 \text{ kN} \qquad \text{OK}$$

## DESIGN BRIEF

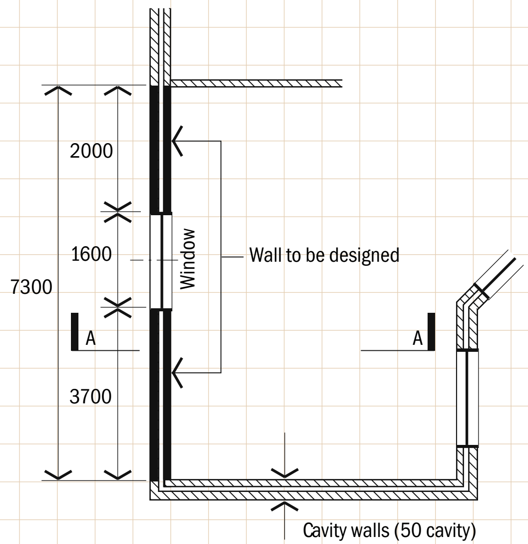
Design the masonry components of the following home unit building building for earthquake loading in the following situation.




SECTION A-A




BUILDING PLAN




TYPICAL PART PLAN

## BUILDING AND SITE PARAMETERS

Location: Sydney

Subsoil Conditions:

| Type | Undrained shear strength, $C_u$ or SPT N Value | Actual Depth, D |
|---|---|---|
| Soft clay | $12.5 < C_u < 25$ | 3.0 |
| Firm clay | $25 < C_u < 50$ | 5.0 |
| Soft clay | $12.5 < C_u < 25$ | 1.0 |
| Stiff clay | $50 < C_u < 100$ | 12.0 |
| Total | | 21.0 |

| Level | Wall number | Thickness of loadbearing masonry (mm) |
|---|---|---|
| 5 | W5 | 90 |
| 4 | W4 | 90 |
| 3 | W3 | 110 |
| 2 | W2 | 110 |
| 1 (Ground) | W1 | 190 |

Notes:

1. External walls are cavity walls, with 110 mm outer leaf.
2. It is assumed that vertical loads will dictate the thickness of walls note above.

Length of building

$L_b = 41.8$ m

Width of building

$B_b = 10.0$ m

Eaves

$E = 0.45$ m

No of floors

$I = 5$

Wall height

$H = 2.50$ m

Floor thickness

$T = 0.15$ m

Average floor to floor height

$H_{av} = H + T$

$= 2.50 + 0.15$

$= 2.65$ m

Depth of first floor beams (below slab soffit)

$H_{trans\ b} = 0.60$m

Sub-floor height above ground

$H_{sub} = 0.35$ m

Roof height above ceiling

$H_{roof} = 2.40$ m

Height of roof seismic weight above ceiling

$H_{s\ roof} = H_{roof} / 3$

$= 0.80$ m

Total height of highest seismic weight

$h_n = I.H_{av} + H_{trans\ b} + H_{sub} + H_{s\ roof}$

$= (5 \times 2.65) + 0.6 + 0.35 + 0.80$

$= 15.0$ m

Height of seismic weight of top storey walls

$h_i = (I-1)\ H_{av} + H_{trans\ b} + H_{sub} + T + 0.5\ H$

$= (5-1) \times 2.65 + 0.6 + 0.35 + 0.15 + (0.5 \times 2.50)$

$= 12.95$ m

Proportion of total height

$h_i / h_n = 12.95/15.0$

$= 0.863$

The building consists of reasonably similar floors, without soft storeys.

## DESIGN METHODOLOGY

To determine the loads on the masonry structure, the loads are calculated for the nominated “reference period” and “annual probability of exceedance” in accordance with AS 1170.4.

The resistance to base shear should be checked in the walls and columns at the ground floor level. It is assumed that these will consist of 190 mm reinforced concrete blockwork, designed in accordance with AS 3700 Section 8.

The resistance to shear should also be checked in the walls at the first floor level (transfer slab), in accordance with AS 3700 Section 7.

The strength and fixing of the masonry walls within the structure, when subjected to the out-of-plane horizontal loads, should also be checked.

## REFERENCE PERIOD

Reference period (design life)

Ref = 50 years

Equivalent annual probability of exceedance

1 in… = 500

Return period factor for reference intensity earthquake

$k_p = 1.0$

## SUBSOIL PROFILE

Soil is not Class A (Strong rock), Class B (Rock) or Class E (very soft soil)

Low amplitude natural site period (Evaluated for a layered sub-soil)

| Type | Undrained shear strength, $C_u$ or SPT N Value | Maximum Depth, $D_{max}$ | Actual Depth, D | Low amplitude natural period, $T_{soil}$ = sum(0.6 D/$D_{max}$) |
|---|---|---|---|---|
| Soft clay | $12.5 < C_u < 25$ | 20 | 3.0 | 0.09 |
| Firm clay | $25 < C_u < 50$ | 25 | 5.0 | 0.12 |
| Soft clay | $12.5 < C_u < 25$ | 20 | 1.0 | 0.03 |
| Stiff clay | $50 < C_u < 100$ | 40 | 12.0 | 0.18 |
| Total | | | 21.0 | **0.42 < 0.6** |
| | | | | Therefore Class C site |

## WEIGHT OF BUILDING

**Roof**

Roof area acting

$A_R = (L_b + 2E) . (B_b + 2E)$

$= (41.8 + 0.45 + 0.45) \times (10.0 + 0.45 + 0.45)$

$= 465.4\ m^2$

Uniform permanent load

$g_R = 1.0$ kPa

Permanent load

$G_R = A_R\, g_R$

$= 465.4 \times 1.0$

$= 465.4$ kN

Permanent load factor

$\gamma_G = 1.00$

Factored unit permanent load

$G^*_R = \gamma_G\, G_R$

$= 1.0 \times 465.4$

$= 465.4$ kN

Uniform imposed load

$q_R = 0.25$ kPa

Imposed load

$Q_R = A_R\, q_R$

$= 465.4 \times 0.25$

$= 116.3$ kN

Imposed load factor

$\gamma_Q = 0.3$

Factored imposed load

$Q^*_R = \gamma_G\, G_R$

$= 0.3 \times 116.3$

$= 34.9$ kN

Factored loads

$F^*_R = G^*_R + Q^*_R$

$= 465.4 + 34.9$

$= 500.3$ kN/m

**Wall 5**

Height of walls

$H_{w5} = 2.5$ m

Length of walls (Including lengths of all leaves)

$L_{w5} = 311$ m

Wall area acting

$A_{w5} = H_{w5}\, L_{w5}$

| | | | |
|---|---|---|---|
| | $= 2.5 \times 69.6$ | $2.5 \times 70.2$ | $2.5 \times 170.9$ |
| | $= 174\ m^2$ | $175\ m^2$ | $427\ m^2$ |
| Wall leaf thickness | | | |
| $t_{w5}$ | = 90 mm | 110 mm | 90 mm |
| Percentage solid lead | | | |
| $p_{w5}$ | = 70% | 70% | 70% |
| Thickness of plaster | | | |
| $t_{p5}$ | = 10 mm | 0 mm | 10 mm |
| No of surfaces plastered | | | |
| $N_{p5}$ | = 1 | 0 | 2 |
| Density of wall material | | | |
| $\gamma_5$ | = 2,180 $kN/m^3$ | 2,180 $kN/m^3$ | 2,180 $kN/m^3$ |

Surface density of wall

$g_5 = t_{w5}5\ p_{w5}\ \gamma_5\ 9.81/1{,}000 + t_{p5}\ N_{p5}\ 800 \times 9.81/1{,}000$

$= (90 \times 0.7 \times 2{,}180 \times 9.81/1{,}000{,}000) +$

$(10 \times 1 \times 800 \times 9.81/1{,}000{,}000)$

$= 1.43\ kN/m^2$   $1.65\ kN/m^2$   $1.50\ kN/m^2$

Weight of wall

$G_5 = g_5 \; A_{w5}$

| | | |
|---|---|---|
| $= 1.43 \times 174$ | $1.65 \times 175$ | $1.50 \times 472$ |
| $= 248$ kN | 289 kN | 643 kN |

$= 1{,}180$ kN

Permanent load factor

$\gamma_G \; = 1.00$

Factored load

$F^*_{w5} = \gamma_G \; G5$

$= 1.0 \times 1{,}180$

$= 1{,}180$ kN

**Concrete Slab 4**

Floor slab acting

$A_{S4} \; = L_b \; B_b$

$= 41.8 \times 10.0$

$= 418.0 \; m^2$

Thickness of slab

$t_{S4} = 150$ mm

Density of concrete

$\gamma_{S4} = 25 \; kn/m^3$

Uniform permanent load

$g_{S4} = t_{S4} \; \gamma_{S4}$

$= 150 \times 25/1{,}000$

$= 3.75$ kPa

Permanent load

$G_{S4} = A_R \; g_R$

$= 418 \times 3.75$

$= 1{,}567$ kN

Permanent load factor

$\gamma_G = 1.00$

Factored unit permanent load

$G^*_{S4} \; = \gamma_G \; G_R$

$= 1.0 \times 1{,}567$

$= 1{,}567$ kN

Uniform imposed load

$q_{S4} \; = 2.0$ kPa

Imposed load

$Q_{S4} = A_R \; q_R$

$= 418 \times 2.0$

$= 836$ kN

Imposed load factor

$\gamma_Q = 0.3$

Factored imposed load

$Q^*_{S4} \; = \gamma_G \; G_R$

$= 0.3 \times 836$

$= 251$ kN

Factored load

$F^*_{S4} \; = G^*R + Q^*R$

$= 1{,}567 + 251$

$= 1{,}818$ kN/m

**Wall 4**

Same as Wall 5

Factored load

$F^*_{W4} = 1{,}180$ kN

**Slab 3**

Same as Slab 4

Factored load

$F^*_{S3} \; = 1{,}818$ kN

**Wall 3**

Same as Wall 5 except 110 mm masonry units

Factored load

$F^*_{W3} = 1{,}360$ kN

**Slab 2**

Same as Slab 4

Factored load

$F^*_{S2} = 1{,}818$ kN

**Wall 2**

Same as Wall 5 except 110 mm masonry units

Factored load

$F^*_{W2} = 1{,}360$ kN

**Slab 1**

Same as Slab 4 plus 600 x 450 transfer beams at 3.0 m centres

Factored load

$F^*_{S1} = 2{,}242$ kN

**Wall 1**

Same as Wall 5 except 190 mm masonry units

Factored load

$F^*_{W1} = 1{,}556$ kN

**Total loads**

Total load on slab 1 (transfer slab)

$F^*_2 = F^*_{W2} + F^*_{S2} + F^*_{W3} + F^*_{S3} + F^*_{W4} + F^*_{S4} + F^*_{W5} + F^*_R$

$= 1{,}360 + 1{,}818 + 1{,}360 + 1{,}818 + 1180 + 1{,}818 + 1{,}180 + 500$

$= 11{,}034$ kN

Total load at base

$F^*_1 = F^*_{W1} + F^*_{S1} + F^*_{W2} + F^*_{S2} + F^*_{W3} + F^*_{S3} + F^*_{W4} + F^*_{S4} + F^*_{W5} + F^*R$

$= 1{,}556 + 2{,}242 + 1{,}360 + 1{,}818 + 1{,}360 + 1{,}818 + 1{,}180 + 1{,}818 + 1{,}180 + 500$

$= 14{,}832$ kN

**VIBRATION OF BUILDING UNDER EARTHQUAKE ACTION**

Empirical method factor for structures that are not moment-resisting steel frames, moment-resisting concrete frames, or eccentrically-braced steel frames

$k_t = 0.05$

Period of vibration for ultimate limit state (calculated value)

$T_1 = 1.25$ kt hn 0.75

$= 1.25 \times 0.05 \times 12.950.75$

$= 0.48$ s

$< 0.7$ s

For Subsoil Class C, $T_1 > 0.4$ s OK

Spectral shape factor

$C_h(T_1) = 2.62$ Interpolated

Structural performance factor

$S_p = 0.77$

Ductility of part

$\mu = 1.25$

Location

Sydney

Hazard factor

$Z = 0.08$

Ordinate of the elastic site spectrum (acceleration of the site) for design earthquake

$C(T_1) = k_P\, Z\, C_h(T_1)$

$= 1.0 \times 0.08 \times 2.62$

$= 0.210$ g

Horizontal design action coefficient (acceleration of the structure for the site) for design earthquake

$C_d(T_1) = C(T_1)\, S_P/\mu$

$= 0.210 \times 0.77/1.25$

$= 0.129$g

Base shear for design earthquake at bottom storey

$V = F^*_1 \,.\, C_d(T_1)$

$= 14{,}832 \times 0.129$

$= 1{,}918$ kN

Exponent dependent on structure period

$k = 1.0$

$T_1 = 0.48$

For $T_1 < 0.5$, $k = 1.0$ OK

For $T_1 > 2.5$ , $k = 2.5$

Force at any floor

$$F_i = \frac{W_i\, h_i^{\,k}}{\Sigma_{i=1}^{n}(W_i\, h_i^{\,k})} V$$

Example: Top Floor

Horizontal acceleration factor at top floor

$R_{floor5/5} = (5^{1.0})/(1^{1.0} + 2^{1.0} + 3^{1.0} + 4^{1.0} + 5^{1.0}) \times 5$

$= 1.67$

Horizontal acceleration at top floor for design earthquake

$a_{floor5/5} = R_{floor5/5}\ C_d(T_1)$

$= 1.67 \times 0.129$

$= 0.216$ g

Note: The same result may be obtained using AS 1170.4 Table 5.4 and Equation 5.4.

$a_{floor5/5} = K_s\ (k_p\ Z\ S_p\ /\mu)$

$= 4.4 \times 1.0 \times 0.08 \times 0.77/1.25$

$= 0.216$ g

Shear applied at floor

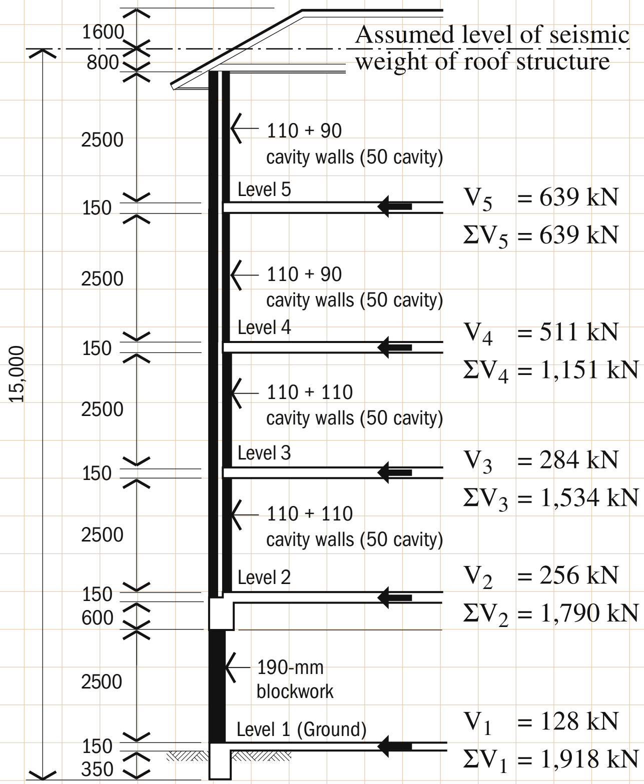
$V_{i=5} = a_{floor5/5}\ \ F_1^*/I$

$= 0.216 \times 14{,}832/5$

$= 639$ kN

Other floors may be calculated in a similar manner. AS 1170.4 Table 5.4 tabulates the relevant factors, although there are small discrepancies involved in using the rounded values of the table.

| Level | Wall Number | Tabulated $K_S$ | Calculated $K_S$ | Design $a_{floor}$ | Design Incremental Shear, $F_{di}$ (kN) | Design Cumulative Shear, $F_d$ (kN) |
|---|---|---|---|---|---|---|
| 5 | W5 | 4.4 | 4.37 | 0.216 | 639 | 639 |
| 4 | W4 | 3.5 | 3.50 | 0.172 | 511 | 1,151 |
| 3 | W3 | 2.6 | 2.62 | 0.129 | 384 | 1,534 |
| 2 | W2 | 1.7 | 1.75 | 0.086 | 256 | 1,790 |
| 1 (Ground) | W1 | 0.9 | 0.87 | 0.043 | 128 | **1,918** |



## DESIGN OF BLADE COLUMNS IN BASEMENT

The following analysis of the strength limit state of the reinforced masonry blade columns in thebasement is carried out in the fundamental (E-W) direction only. The four, 3.0-m-long shear walls should also be designed by the same method. Consideration should also be given to the strength limit state in the orthogonal direction.
A separate analysis should be carried out for stability.

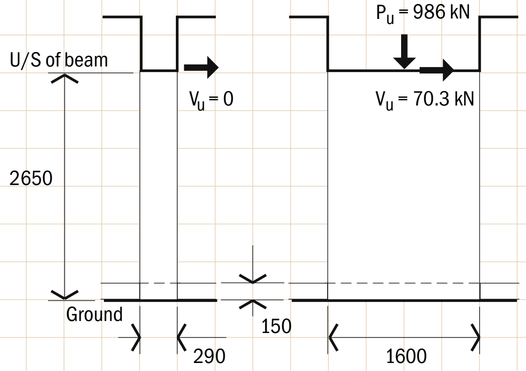
**Masonry blade column specification**

Each column is 290 x 1600 x 2500 mm high with 4-N12 vertical bars and 4-N12 horizontal bars, evenly-spaced up the column.

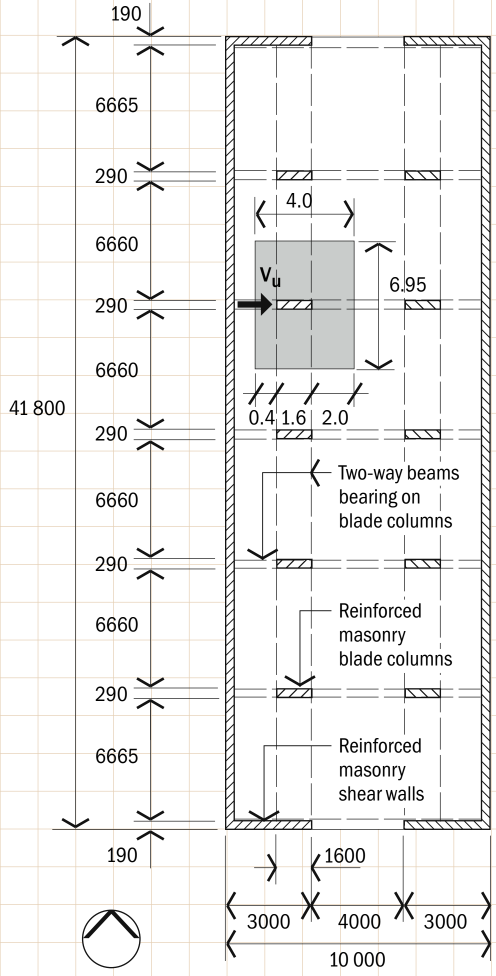
Beams are very stiff.
No bending moment from beams
ie effectively concentric loads

$M_x = 0$

$M_y = 0$




BLADE COLUMN ELEVATIONS




PLAN AT BASEMENT LEVEL

Suspended concrete floor acts as a diaphragm.
All piers and shear walls are 2.500 m clear height plus 0.150-m concrete floor = 2.650 m from ground to under side of beams

10 columns (290 x 1600 cross section)

$$I = \frac{290 \times 1600^3}{12}$$

$$= 99{,}000 \times 10^6 \text{ mm}^4$$

4 shear walls (190 x 3000 cross section)

$$I = \frac{190 \times 3000^3}{12}$$

$$= 427{,}500 \times 10^6 \text{ mm}^4$$

Shear on each blade column

$$V_u = \frac{99{,}000}{(10 \times 99{,}000) + (4 \times 427{,}500)} \text{ V}$$

$$= 0.0367 \times 1{,}918$$

$$= 70.3 \text{ kN}$$

Contributory area

$$A = 4.0 \times 6.95$$

$$= 27.8 \text{ m}^2$$

Vertical load for strength limit state

$$P_u = \frac{14{,}832 \times 27.8}{418}$$

$$= 986 \text{ kN}$$

**Blockwork Strength**

Characteristic strength of units

$f'_{uc} = 15$ MPa

Mortar type: M3

Prism factor

$k_m = 1.6$

Masonry characteristic strength

$f'_{mb} = k_m \, f'_{uc}$

$= 1.6 \times 15$

$= 6.2$ MPa

Block height

$h_1 = 190$ mm

Mortar thickness

$h_2 = 10$ mm

Height ratio

$$\frac{h_1}{h_2} = 1.9$$

Compressive strength factor

$k_h = 1.3$

Blockwork characteristic strength

$f'_m = k_h \, f'_{mb}$

$= 1.3 \times 6.2$

$= 8.06$ MPa

**Grout Strength**

Specified strength

$f'_{csp} = 20$ MPa

Design strength

$f'_c = 1.3 \, f'_{uc}$

$= 1.3 \times 15$

$= 19.5$ MPa

Block density

$D_{ens} = 2200$ kg/m$^3$

$> 2000$ kg/m$^3$

$k_c = 1.4$

**Design for Compression**

Steel reinforcement can NOT be tied in two directions. Use the refined calculation method for unreinforced masonry supporting a concrete slab at the top and laterally supported top and bottom.

$a_v = 0.85$ *AS 3700 7.3.4.3*

$H = 2200$ mm

No stiffening returns

$k_t = 1.0$ *AS 3700 Table 7.2*

Slenderness

$$S_r = \frac{a_v \, H}{k_t \, t}$$

$$= \frac{0.85 \times 2650}{1.0 \times 290}$$

$= 7.77$

Concentric stiff beams in two directions apply the vertical and shear loads

Eccentricity ratio

$$\frac{e_1}{t_w} \quad 0.05$$

Slenderness and eccentricity factor

$k_s = 1.18 - 0.03 \, S_r$ *AS 3700 8.4*

$= 1.18 - (0.03 \times 7.77)$

$= 0.947$

Capacity reduction factor (unreinforced)

$\phi = 0.45$ for compression

Bedded area *

$A_b = (30 + 30) \times 1600$

$= 96{,}000$ mm$^2$/m

* *NOTE: It is both conservative and consistent with AS 3700 to assume the face shells extend along the long sides only of the piers, and not across the ends*

All cores and web areas grouted *

$A_c = (290 \times 1600) - 96{,}000$

$= 368{,}000 \text{ mm}^2$

* *NOTE: It is both conservative and consistent with AS 3700 to assume the face shells extend along the long sides only of the piers, and not across the ends*

Basic compressive strength

$$F_o = \phi\, k_s \left[ f'_m A_b + k_c \sqrt{\frac{f'_{cg}}{1.3}}\, A_c + f_{sy} A_s \right] \qquad \textit{AS 3700 8.5}$$

$$= 0.45 + 0.947 \times \left[ (8.06 \times 96{,}000) + 1.4 \sqrt{\frac{19.5}{1.3}}\, 368{,}000 + (400 \times 0) \right]$$

$= 1180$ kN

For combined bending and compression, the compressive capacity would be factored down

$F_{ucom} = 0.85\, F_u$ *AS 3700 8.11*

$= 0.85 \times 1180$

$= 1003$ kN

$> 986$ kN OK

**Design for In-plane Shear**

Height

$H = 2650$ mm

Length

$L = 1600$ mm

$$\frac{H}{L} = \frac{2650}{1600}$$

$= 1.656$

$< 2.3$ ∴ Design as shear wall *AS 3700 8.7.2*

$> 1.0$ ∴ Horizontal reinforcement is basis of $A_s$

Area of horizontal reinforcement crossing potential crack

$A_s = 1 \times 110$ (1 of 4-N12 bars)

$= 110 \text{ mm}^2$

Note: This is more conservative than AS 3700 requirements

Design cross-sectional area

$A_d = 290 \times 1600$

$= 464{,}000 \text{ mm}^2$

$$f_{vr} = \left(1.50 - 0.5\, \frac{H}{L}\right)$$

$= 1.50 - (0.5 \times 1.656)$

$= 0.672$ MPa

$f_{sy} = 500$ MPa

Capacity reduction factor (reinforced masonry)

$\phi = 0.75$ *AS 3700 Table 4.1*

In-plane shear capacity

$V_u = \phi\, (f_{vr} A_d + 0.8\, f_{sy} A_s)$ *AS 3700 8.7.2*

$$= 0.75 \left[ (0.672 \times 464{,}000) + (0.8 \times 500 \times 110) \right] \frac{1}{10^3}$$

$= 267$ kN

$> 70.3$ kN OK

Check columns for local overturning

$P_v > f'_m A_b$

$$k_{sw} = 1 - \frac{P_v}{A_b f'_m}$$

$$= 1 - \frac{986}{464{,}000 \times 8.06 \times 10^{-3}}$$

$= 0.736$

$l' = 100$ mm

Resistance to overturning (based on one end bar)

$$V_u = \frac{\phi}{H} \left\{ k_{sw} \frac{P_v L}{2} + f_{sy} A_{sv} (L - 2l') \right\} \qquad \textit{AS 3700 8.7.4}$$

$$= \frac{0.75}{2.65} \left\{ \frac{0.736 \times 986 \times 1.6}{2} + \frac{500 \times 110}{1000} [1.6 - (2 \times 0.1)] \right\}$$

$= 186$ kN

$> 70.3$ kN OK

Carry out similar analysis for shear walls in basement and shear walls in each floor

**SECTION PROPERTIES OF WALL 2**
**[AT LEVEL 2, SUPPORTED ON SLAB 1 (TRANSFER SLAB)]**

The masonry walls above the Transfer Slab consist of 110 mm inside loadbearing masonry leaf.

Notes:
A 110 mm internal loadbearing masonry leaf is required in preference to a 90 mm loadbearing masonry leaf, which is inadequate for the support of three concrete slabs and a roof, for both vertical gravity loads and horizontal earthquake loads. The external leaf is connected only by flexible ties and does not play any significant role in resisting horizontal seismic forces.

Total second moment of area

$I = 529.0 \times 10^{12}\ mm^4$

Total bedded and grout area of section

$A = 6.29 \times 10^{6}\ mm^2$

Shear walls (parallel to the direction of load) are stiff in this direction and will resist in-plane shear. Transverse walls (perpendicular to the direction of load) are flexible in this direction and will not resist in-plane shear, except that they may contribute as the flanges of T or L sections. AS 3700 Clause 4.5.2 (e) (i) restricts the flange width to 0.08 times the wall height for T sections and AS 3700 Clause 4.5.2 (e) (ii) restricts the flange width to 0.06 times the wall height for L sections.

**Overall dimensions:**

| | |
|---|---|
| Depth North-South | 41,800 mm |
| Width East-West | 10,000 mm |
| Number of walls East-West | 8 |
| Number of walls North-South | 2 |
| Proportion openings | 50 % |
| Proportion solid | 70 % |
| Thickness | 110 mm |
| Effective thickness | 38.5 mm |

| **Section** | **Depth**, D | **Width**, b | **Area**, A | **Lever Arm**, d | **First Moment**, Ad | **Centroid depth**, $d_e$ |
|---|---|---|---|---|---|---|
| North wall (Runs E-W) | 39 | 10,000 | 385,000 | 20,900 | 8,039,000,000 | 20,900 |
| Centre walls (Runs E-W) | 231 | 10,000 | 2,310,000 | 0 | 0 | 0 |
| South wall (Runs E-W) | 39 | 10,000 | 385,000 | -20,900 | -8,039,000,000 | -20,900 |
| West wall (Runs N-S) | 41,723 | 39 | 1,606,000 | 0 | 0 | 0 |
| Centre walls (Runs N-S) | 41,723 | 0 | 0 | 0 | 0 | 0 |
| South wall (Runs N-S) | 41,723 | 39 | 1,606,000 | 0 | 0 | 0 |
| Totals | | | 6,292,000 | | 0 | |

| **Section** | **Second Moment**, $A\ d_e^2$ | **Second Moment**, $I_o$ |
|---|---|---|
| North wall (Runs E-W) | 31,446,000,000,000 | 47,000,000 |
| Centre walls (Runs E-W) | 0 | 10,271,000,000 |
| South wall (Runs E-W) | 31,446,000,000,000 | 47,000,000 |
| West wall (Runs N-S) | 0 | 233,026,000,000,000 |
| Centre walls (Runs N-S) | 0 | 0 |
| South wall (Runs N-S) | 0 | 233,026,000,000,000 |
| Totals | 62,892,000,000,000 | 466,064,000,000,000 |

Effective shear wall outstand / Wall height

$k_{T \text{ or } L} = 0.06$ *AS 3700 Clause 4.5.2 (e) (ii)*

Effective shear wall outstand

$B_{T \text{ or } L} = k_{T \text{ or } L} H$

$= 0.06 \times 2{,}500$

$= 150$ mm

Average wall length between returns

$L_{av} = 4{,}000$ mm

Proportion of total walls that are in the direction of load (i.e. length of shear walls / total length of wall)

$p_{shear\ w} = 0.50$

Proportion of total walls that act as shear walls to resist racking forces and proportion of total weight that acts on these shear walls

$p_{shear\ A} = p_{shear\ w}\ (2\ B_{T \text{ or } L} + L_{av}\ \ )\ L_{av}$

$= 0.50\ \{1 + [(2 \times 150 + 4000)] / 4000\}$

$= 0.538$

This assumes there is one return at each end of the shear wall.

It has been assumed that the frictional resistance to racking forces is provided by the shear factor, $K_v$, multiplied by the weight of the building normally supported by the shear walls. In other words, there is no contribution to the resistance racking forces by the weight of the building above walls perpendicular to the shear walls, except as provided for the in the flanges of T or L sections.

Total area of section resisting shear

$A_{dw} = A\ \ p_{shear\ A}$

$= 0.538 \times 6.29 \times 10^6$

$= 3.38 \times 10^6$ mm$^2$

**EFFECTIVE SECTION PROPERTIES OF STRUCTURE ABOVE THE SUPPORT**

The bottom storey serves as a carpark, and has different section properties from the rest of the building. Above the transfer floor, all storeys have similar arrangements and made from similar masonry units. They are assumed to have similar section properties. In determining the behaviour of the whole structure, the analysis must account for the difference in section properties of each storey including any “soft” storeys. It is a condition of this design that there are no “soft” storeys.

Total bedded and grout area of section

$A_{dw} = 3.38 \times 10^6$ mm$^2$

Total second moment of area

$I = 529.0 \times 10^{12}$ mm$^4$

**SHEAR DEFORMATION ANALYSIS**

The loadbearing masonry shall consist of 110-mm thick masonry units set in M3 (1:1:6) mortar.

The notes with AS 3700 Figures 10.3 and 10.11 require that slip material have a shear factor, $k_v$, (similar to the coefficient of static friction) in the range 0.15 minimum to 0.30. Because the shear factor, $k_v$, of mortar joints is taken as 0.3, it is assumed that slip on the slip material is more probable that slip on cracked and uncracked mortar joints.

In the following design, slip material with a mean value of shear factor, $k_v$ of 0.225 ( i.e. midway between these two limits) shall be positioned at the bottom and at the top of all load bearing walls.

Slip material characteristic shear factor (coefficient of friction)

$k_v = 0.15$

Characteristic bond strength at the base and top of the wall

$f_v = 0$ MPa

Capacity reduction factors for design earthquake

$\phi = 0.6$ *AS 3700 Clause 4.4 Table 4.1*

Limiting horizontal force from design earthquake before the initiation of slip at the slip joints

$V_{lim} = \phi f_v A_{dw} + P_b k_v$

$= [(0.6 \times 0 \times 6.29 \times 10^6) + (11{,}034 \times 0.15)]/1000$

$= 1{,}655$ kN

$< \phi f_v A_{dw} + 2.0 A_{dw} k_v$

$= [(0.6 \times 0 \times 6.29) + (2.0 \times 6.29 \times 10^6 \times 0.30)]/1000$

$= 3{,}774$ kN/m OK

< Cumulative shear at the base of wall (transfer floor level)

$= 1{,}790$ kN

Therefore the wall will slip under the earthquake action.

Uncracked modulus of elasticity subject to design earthquake

$E_{m\ uncr} = 1{,}000 f'_m$ *Australian Masonry Manual Table 2.2*

$= 1{,}000 \times 5.42$

$= 5{,}420$ MPa

Initial modulus of elasticity reduction factor, to allow for cracking in the masonry prior to the earthquake

$k_{crack} = 0.75$

Design initial modulus of elasticity

$E_m = k_{crack} E_{m\ uncr}$

$= 0.90 \times 5{,}420$

$= 4{,}880$ MPa

Poisson's ratio

$\upsilon = 0.2$ *Australian Masonry Manual Table 2.3.6*

Modulus of rigidity

$G = E_m/2\ (1+\upsilon)$

$= E_m/2\ (1+0.2)$

$= 0.42 E_m$

Use $G = 0.4 E_m$ *Australian Masonry Manual Table 2.3.6*

Design initial modulus of rigidity

$G = 0.4 E_m$

$= 0.4 \times 4{,}880$

$= 1{,}950$ MPa

Shear deformation coefficient

$\alpha = 1.0$

For rectangular sections $\alpha = 1.2$

For flanged sections $\alpha = 1.0$

*Australian Masonry Manual Table 5.2.5*

Bending coefficient

$k_b = 3$ Based on cantilever action of the building

Height of structure above the transfer floor

$h_i = h_n - H_{av} - H_{trans\ b} - H_{sub}$

$= 15.0 - 2.65 - 0.6 - 0.35$

$= 11.4$ m

Nominated limit on horizontal deformation of the building above the transfer floor

$D_{lim} = 0.05\ t_w\ (I - I_i)$

$= 0.05 \times 110 \times (5-1)$

$= 22.0$ mm

This limit of 5% of wall thickness, multiplied by the number of floors, has been selected so that the limit on interstorey drift is approximately 5% of the wall thickness. This is considered to be a reasonable limit to ensure that there is only a small increase in eccentricity of vertical load on the walls during the earthquake.

Design initial shear stiffness

$k_i = G A_{dw}/(\alpha h_i)$

$= 1{,}950 \times 3.38 \times 106 / (1.0 \times 11.4 \times 106)$

$= 579$ kN/mm

Design initial deflection at limiting acceleration before slip occurs

$\Delta_i = p_{shear\ A} k_v F_d / k_i$

$= 0.538 \times 0.225 \times 11{,}034 / 579$

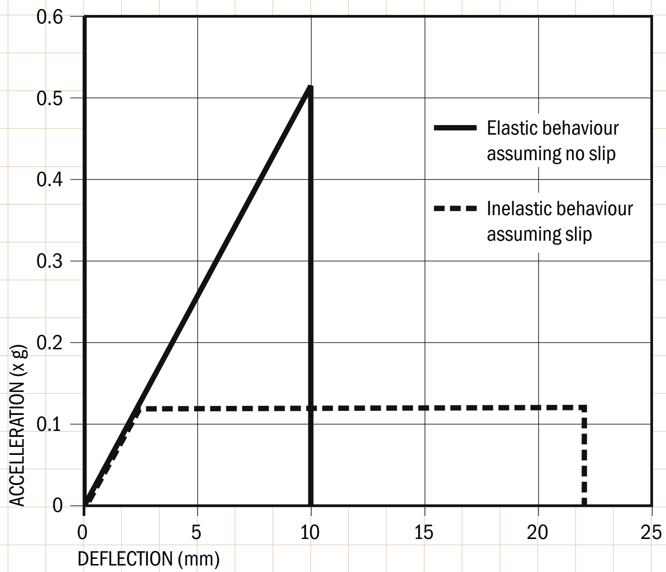
$= 2.30$ mm

Design limiting horizontal acceleration before slip occurs

$ai = p_{shear\ A} k_v$

$= 0.538 \times 0.225$

$= 0.121$ g




Area under the design a - Δ curve

$A_1$ = (2.30 x 0.121 / 2) + (22.0 – 2.30) 0.121

= 2.52 mm.g

Design initial shear stiffness

$K_i = a_i / \Delta_i$

= 0.121 / 1.85

= 0.0525 g/mm

Design effective limiting acceleration

$a_{lim\ eff} = (2\ K_i\ A_1)^{0.5}$

$= (2 \times 2.52 \times 0.0525)^{0.5}$

= 0.514g

Design limiting shear capacity

$F_{lim} = F_d\ a_{lim\ eff} / a_i$

= 1,790 x 0.578 / 0.121

= 7,615 kN

$> F_d$ = 1,790 kN OK

**DEFLECTIONS ABOVE TRANSFER FLOOR**

Deflection due to shear deformation during design earthquake

$$\Delta_v = \frac{\alpha\ F_H\ h_i}{G_m\ A}$$

$$= \frac{1.0 \times 1{,}790 \times 10^3 \times 11.4 \times 10^3}{1{,}950 \times 3.38 \times 10^6}$$

= 3.09 mm

Deflection due to bending deformation during design earthquake

$$\Delta_b = \frac{F_H\ h_i^3}{k_b\ E_m\ I}$$

$$= \frac{1{,}790 \times 103 \times 11.43 \times 109}{3 \times 4{,}880 \times 529 \times 106}$$

= 0.34 mm

Total deflection at design earthquake load during design earthquake

$\Delta_d = \Delta_v + \Delta_b$

= 3.09 + 0.34

= 3.43 mm

Average interstorey drift at design earthquake

$\delta = \Delta / (I - I_i)$

= 3.43 / (5 – 1)

= 0.86 mm

**VERTICAL LOAD**

It is assumed throughout this design, that vertical loads will dictate the following thickness of walls.

| Level | Wall number | Thickness of loadbearing masonry (mm) |
|---|---|---|
| 5 | W5 | 90 |
| 4 | W4 | 90 |
| 3 | W3 | 110 |
| 2 | W2 | 110 |
| 1 (Ground) | W1 | 190 |

External wall will be cavity walls, with 110 mm outer leaf.

The vertical load capacity of all walls should be checked with the interstorey drift applied as an eccentricity. Two-way action could be taken into account, although if there is vertical cracking, walls could become isolated from the supporting returns or columns. Therefore, the check will be carried out assuming one-way buckling, with supports at top and bottom only.

The highest vertical load occurs in the bottom storey, but at this point it would be normal to use 190 mm reinforced concrete blockwork. Therefore, the capacity check will be carried out on the 110 mm loadbearing walls at first floor.

Vertical load on first storey walls for design earthquake

$F^*_{2\,net} = F^*_2 - F_{W2}$

$= 11{,}034 - 1{,}360$

$= 9{,}674$ kN

Length of loadbearing wall

$L_2 = 241$ m

Allowance for load concentrations

$k_c = 1.2$ This is based on the assumption that stiff slabs distribute the load relatively uniformly to the loadbearing walls,with a concentration of 20%.

Line load applied to wall due to permanent and imposed loads

$F^{*'}_2 = k_c F^*_2 / L_2$

$= 1.2 \times 11{,}034 / 241$

$= 55.0$ kN/m

VERTICAL LOAD CAPACITY

Height of wall

$H = 2{,}500$ mm

Wall leaf thickness

$t_w = 110$ mm

Engaged pier thickness coefficient

$k_t = 1.00$

Masonry unit characteristic unconfined compressive strength

$f'_{uc} = 15.0$ MPa

Units are cored and ungrouted

Block type factor

$k_m = 1.4$

Equivalent brickwork strength

$f'_{mb} = k_m (f'_{uc})^{0.5}$

$= 1.4\,(15.0)^{0.5}$

$= 5.42$ MPa

Mortar joint height

$h_j = 10$ mm

Masonry unit height

$h_b = 76$ mm

Ratio of block to joint thickness

$h_b/h_j = 76/10$

$= 7.6$

Block height factor

$k_h = 1.0$

Characteristic masonry strength

$f'_m = k_h f'_{mb}$

$= 1.0 \times 5.42$

$= 5.42$ MPa

Bedded area

$A_b = 110{,}000$ mm$^2$/m

Vertical slenderness coefficient

$a_v = 0.75$

Slenderness

$S_r = a_v H/t_w$

$= 0.75 \times 2{,}500/110$

$= 17.0$

Small eccentricity

$e_2 = 0$ mm

***At design earthquake***

Capacity reduction factor

$\phi = 0.50$ *AS 3700 Table 4.1*

Basic compressive capacity

$\phi F_o = \phi f'_m A_b$ *AS 3700 7.3.2(1)*

$= 0.5 \times 5.42 \times 110{,}000$

$= 298$ kN/m

Large eccentricity

$e_1$ = Interstorey drift

$= 0.86$ mm

Large eccentricity ratio

$e_1 / t_w = 0.86/110$

$= 0.0078$

$< 0.05$

Therefore the interstorey drift is too small to adversely affect the loadbearing capacity.

Slenderness and eccentricity factor *AS 3700 7.3.4.5(1)*

$$k = 0.5\,(1 + \frac{e_2}{e_1})\left[(1 - 2.083\,\frac{e_1}{t_w}) - (0.025 - 0.037\,\frac{e_1}{t_w})\,(1.33S_r - 8)\right] + 0.5\,(1 - 0.6\,\frac{e_1}{t_w})\,(1 - \frac{e_2}{e_1})\,(1.18 - 0.03S_r)$$

$$= 0.5\,(1 + \frac{0.0}{0.86})\left[(1 - 2.083\,\frac{0.86}{0.0}) - (0.025 - 0.037\,\frac{0.86}{110})\,(1.33 \text{ x } 17.0 - 8)\right] + 0.5\,(1 - 0.6\,\frac{0.86}{110})\,(1 - \frac{0.0}{0.86})\,(1.18 - 0.03 \text{ x } 17.0)$$

$= 0.653$

Vertical load capacity

$\phi F = \phi F_o k$

$= 298 \text{ x } 0.653$

$= 195$ kN/m

$> 55.0$ kN/m OK

**HORIZONTAL FACE LOAD DUE TO EARTHQUAKE**

Check the face load capacity of all walls.

The worst case will be in the top storey.

Self weight of two leaves of masonry

110 mm cored external leaf + 90 mm hollow internal leaf

$W_c = 1.62 + 2.12$

$= 3.74$ kN/m$^2$

Component Importance factor

$I_c = 1.0$

Attachment amplification factor

$a_c = 1.0$ Connections other than spring-type mountings

Component ductility factor

$R_c = 1.0$ Rigid components with non-ductile or brittle materials or connections

Spectral shape factor at zero period

$C_h(0) = 1.3$ *AS 1170.4 Table 6.4*

Total height of the structure above the structural base

$h_n = 15.0$ m

$> 12.0$ m

Structural base to centre of roof weight

Height at which the component is attached above the structural base

$h_x = 14.2$ m

Top connection of the wall to the roof structure

Height factor

$k_c = 2/h_n$ for $h_n$ 12.0 or 0.17 for $h_n < 12.0$

$= 2/15.0$

$= 0.133$

Height amplification factor

$a_x = 1 + (k_c\ h_x)$

$= 1 + (0.133 \text{ x } 14.2)$

$= 2.893$

Horizontal face load from reference earthquake

$F_{ph\ mean} = a_{floor}\ [I_c\ a_c/R_c]\ W_c$ *AS 1170.4 Clause 8.2 Design Accelerations*

$= 0.216 \text{ x } 1.0 \text{ x } 1.0/1.0\ W_p$

$= 0.216 \text{ x } 3.74$

$= 0.808$ kPa

$F_{ph\ mean} = [k_p\ Z\ C_h(0)]a_x\ [I_c\ a_c/R_c]\ W_c$ *AS 1170.4 Clause 8.3 Simple Method*

$= 1.0 \text{ x } 0.08 \text{ x } 1.3 \text{ x } 2.893 \text{ x } 1.0 \text{ x } 1.0 / 1.0\ Wp$

$= 0.301\ W_p \quad > 0.05\ W_c$ OK

$= 0.301 \text{ x } 3.74$

$=1.125$ kPa

Use $F_{ph\ mean} = 0.808$ kPa

**HORIZONTAL FACE LOAD CAPACITY OF EXTERNALWALLS ABOVE TOP FLOOR (LEVEL 5)**

Use AS 3700 Clause 7.4.4 to check the out-of-plane bending capacity of the wall

Capacity reduction factor

$\phi = 0.6$ *AS 3700 Table4.1*

Characteristic flexural tensile strength of masonry

$f'_{mt} = 0.2$ MPa *AS 3700 3.3.3*

Section Moduli

Hollow units (ie 90-mm units)

Section moduli $Z_{d90} = Z_{u90} = Z_{p90}$

$$Z_{d90} = \frac{1000\ t_u^2}{6}\left[6\left(\frac{t_s}{t_u}\right) - 12\left(\frac{t_s}{t_u}\right)^2 + 8\left(\frac{t_s}{t_u}\right)^3\right]$$

$$= \frac{1000 \times 90^2}{6}\left[6\left(\frac{25}{190}\right) - 12\left(\frac{25}{190}\right)^2 + 8\left(\frac{25}{190}\right)^3\right]$$

$$= 1.23 \times 10^6\ \text{mm}^3/\text{m}$$

Solid units (ie 110-mm units)

Section moduli $Z_{d110} = Z_{u110} = Z_{p110}$

$$Z_{d110} = \frac{1000\ t^2}{6}$$

$$= \frac{1000 \times 110^2}{6}$$

$$= 2.02 \times 10^6\ \text{mm}^3/\text{m}$$

Mortar joint thickness

$t_j = 10$ mm

Length of masonry unit

$l_{u90} = 390$ mm

$l_{u110} = 230$ mm

Height of masonry unit

$h_{u90} = 190$ mm

$h_{u110} = 76$ mm

Lateral load parameters

$$G = \frac{2\,(h_u + t_j)}{l_u + t_j}$$ *AS 3700 7.4.4.2*

$$G_{90} = \frac{2\,(190 + 10)}{390 + 10}$$

$$= 1.0$$

$$G_{110} = \frac{2\,(76 + 10)}{230 + 10}$$

$$= 0.717$$

$$A = (l_u + t_j)\sqrt{1 + G^2}$$ *AS 3700 7.4.4.3**

$$A_{90} = (390 + 10)\sqrt{1 + 1^2}$$

$$= 566$$

$$A_{110} = (230 + 10)\sqrt{1 + 0.717^2}$$

$$= 295$$

* Note: Terms A and C are not used in AS 3700 but are included here for simplicity

$$B = \frac{h_u + t_j}{\sqrt{1 + G^2}}$$ *AS 3700 7.4.4.3*

$$B_{90} = \frac{190 + 10}{\sqrt{1 + 1^2}}$$

$$= 141$$

$$B_{110} = \frac{76 + 10}{\sqrt{1 + 0.717^2}}$$

$$= 70$$

$< t_u$

$= 110$ mm

$\therefore$ Use Equation 7.4.4.2(4) for solid units

For hollow units (ie 90-mm units)

$$C_{90} = \left[\frac{B\ t_s}{1.5\ B + 0.9\ t_s} + t_u - t_s\right]$$ *AS 3700 7.4.4.3**

$$= \left[\frac{141 \times 25}{(1.5 \times 141 + (0.9 \times 25)} + 90 - 25\right]$$

$$= 80$$

Equivalent torsional section modulus

$$Z_{t90} = \frac{2\ B\ t_s\ C}{A}$$ *AS 3700 7.4.4.3*

$$= \frac{2 \times 141 \times 25 \times 80}{566}$$

$$= 1001\ \text{mm}^3$$

For solid units (ie 110-mm units)

$$Z_{t110} = \left[\frac{2\,B^2\,t_u^2}{3\,t_u + 1.8\,B}\right]\frac{1}{A}$$

$$= \left[\frac{2 \text{ x } 70^2 \text{ x } 110^2}{(3 \text{ x } 110) + (1.8 \text{ x } 70)}\right]\frac{1}{295}$$

$$= 879 \text{ mm}^3$$

Equivalent torsional strength

$$f'_t = 2.25\,\sqrt{f'_{mt}} + 0.15\,f_d$$ *AS 3700 7.4.4.3*

$$= 2.25\,\sqrt{0.2} + 0.15 \text{ x } 0$$

$$= 1.01 \text{ MPa}$$

(Compressive stress negligible)

Characteristic lateral modulus of rupture

$f'_{ut} = 0.8$ MPa *AS 3700 3.2*

Perpend spacing factor

$$k_p = \text{min.}\left(\frac{S_p}{t_u},\ \frac{S_p}{h_u},\ 1.0\right)$$ *AS 3700 7.4.3.4*

$$k_{p90} = \text{min.}\left(\frac{190}{90},\ \frac{190}{190},\ 1.0\right)$$

$$= 1.0$$

$$k_{p110} = \text{min.}\left(\frac{110}{110},\ \frac{110}{76},\ 1.0\right)$$

$$= 1.0$$

Horizontal moment capacity *AS 3700 7.4.3.2*

$$M_{ch} = \phi\,(0.44\,f'_{ut}\,Z_u + 0.56\,f'_{mt}\,Z_p)$$ *AS 3700 7.4.3.2(4)*

$$M_{ch90} = 0.6[(0.44 \text{ x } 0.8 \text{ x } 1.23) + (0.56 \text{ x } 0.2 \text{ x } 1.23)]$$

$$= 0.342 \text{ kNm/m}$$

$$M_{ch110} = 0.6[(0.44 \text{ x } 0.8 \text{ x } 2.02) + (0.56 \text{ x } 0.2 \text{ x } 2.02)]$$

$$= 0.562 \text{ kNm/m}$$

$$< 2.0\,\phi\,k_p\,\sqrt{f'_{mt}}\,\left(1 + \frac{f_d}{f'_{mt}}\right) Z_d$$ *AS 3700 7.4.3.2(2)*

$$< 4.0\,\phi\,k_p\,\sqrt{f'_{mt}}\;Z_d$$ *AS 3700 7.4.3.2(3)*

$$= 4.0 \text{ x } 0.6 \text{ x } 1.0 \text{ x } \sqrt{0.2} \text{ x } 2.02$$

$$= 2.17 \text{ kNm/m}$$ OK

Diagonal moment capacity

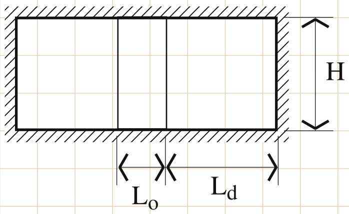
$$M_{cd} = \phi\,f'_t\,Z_t$$ *AS 3700 7.4.4.3(1)*

$$M_{cd90} = 0.6 \text{ x } 1.01 \text{ x } \frac{1001}{1000}$$

$$= 0.607 \text{ kNm/m}$$

$$M_{cd110} = 0.6 \text{ x } 1.01 \text{ x } \frac{879}{1000}$$

$$= 0.532 \text{ kNm/m}$$




$L_o = 1.6$ m

$L_d = 3.70$ m

$$H_d = \frac{H}{2} = \frac{2.50}{2} = 1.25 \text{ m}$$

Slope factor

*AS 3700 7.4.4.2*

$$\alpha = \frac{G\,L_d}{H_d}$$

$$\alpha_{90} = \frac{1.0 \times 3.70}{1.25} = 2.96 > 1.0$$

$$\alpha_{110} = \frac{0.717 \times 3.70}{1.25} = 2.12 > 1.0$$

Aspect factor

*AS 3700 Table 7.5*

$$a_f = \frac{\alpha}{1 - \frac{1}{3\alpha} + \frac{L_o}{2L_d}}$$

$$a_{f_{90}} = \frac{2.96}{1 - \frac{1}{3 \times 2.96} + \frac{1.6}{2 \times 3.70}} = 2.68$$

$$a_{f_{110}} = \frac{2.12}{1 - \frac{1}{3 \times 2.12} + \frac{1.6}{2 \times 3.70}} = 2.00$$

Restraint factors

$R_{f_1} = 1$

$R_{f_2} = 0$

*AS 3700 Table 7.5*

$$k_1 = R_{f_1} = 1$$

*AS 3700 Table 7.5*

$$k_2 = 1 + \frac{1}{G^2}$$

$$k_{2_{90}} = 1 + \frac{1}{1.0^2} = 2.0$$

$$k_{2_{110}} = 1 + \frac{1}{0.717^2} = 2.95$$

Lateral load capacity

*AS 3700 7.4.4.2(2)*

$$w_{cap} = \frac{2\,a_f}{L_d^2}(k_1 M_{ch} + k_2 M_{cd})$$

$$w_{cap_{90}} = \frac{2 \times 2.68}{3.7^2}[(1 \times 0.342) + (2 \times 0.607)] = 0.608 \text{ kPa}$$

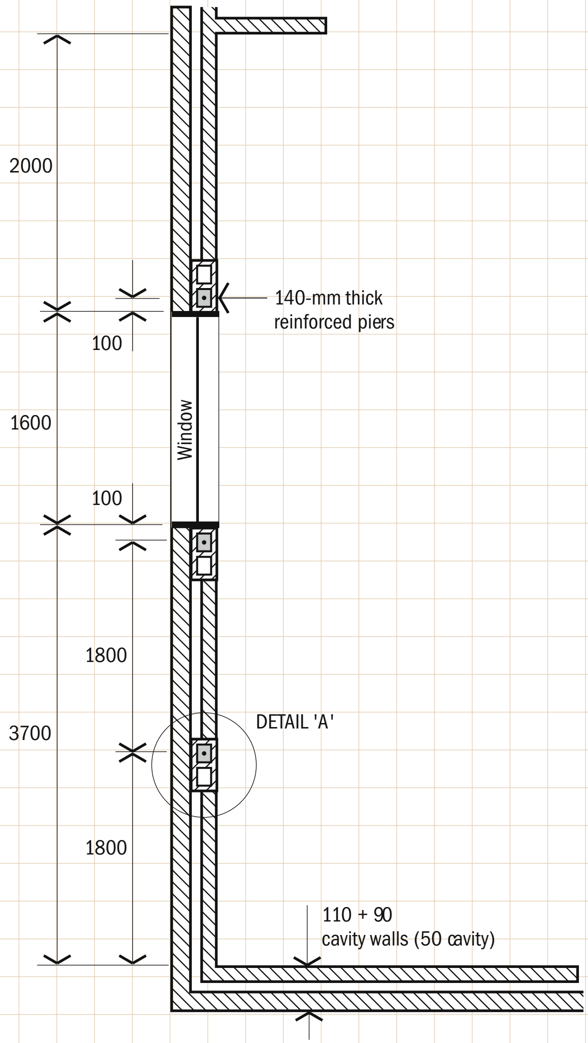
$$w_{cap_{110}} = \frac{2 \times 2.00}{3.7^2}[(1 \times 0.562) + (2.95 \times 0.532)] = 0.622 \text{ kPa}$$

$$w_{cap} = w_{cap_{90}} + w_{cap_{110}} = 0.608 + 0.622 = 1.23 \text{ kPa} > 0.808 \quad \text{OK}$$

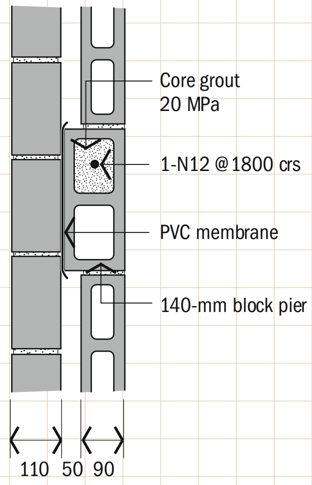
If these calculations had indicated that the external unreinforced cavity masonry walls did not have sufficient strength to resist the out-of-plane lateral loads caused by earthquakes, the wall would need to be strengthened.

One solution would be the provision of 140-mm reinforced piers, built within the cavity, as shown in the following example.

## INCREASING OUT-OF-PLANE LATERAL CAPACITY OF WALL




LAYOUT PLAN OF INTEGRAL PIERS




DETAIL A

*Design Chart this Manual*

**Reinforced masonry pier**

$M_{cap} = 2.7$ kNm/pier

$S = 1.8$ m

$$w_{cap} = \frac{8\ M_{cap}}{S\ H^2}$$

$$= \frac{8 \times 2.7}{1.8 \times 2.5^2}$$

$= 1.92$ kPa

$> 0.808$ kPa OK

**Shear in connections at top of wall**

Design shear for connection

$$V_{con} = 1.25 \left[\frac{F\ H}{2}\right]$$

$$= \frac{1.25 \times 0.808 \times 2.5}{2}$$

$= 1.26$ kN/m

Provide head ties with shear capacity and spacing

$$\frac{F}{S} = \frac{V}{\phi}$$

$$= \frac{1.26}{0.75}$$

$= 1.68$ kN/m

**Shear at base of wall**

Wall is a loadbearing external wall

Design shear at base of wall

$$V = \frac{F\ H}{2}$$

$$= \frac{0.808 \times 2.5}{2}$$

$= 1.01$ kN/m

This shear is to be resisted by mortar/concrete joint of internal leaf only

Design compressive force at base of wall

$G_g = G + \psi_c Q$

$= G_{90wall} + 0.8\ G_{roof}$

$= \left[\frac{2180 \times 9.81}{1000} \times \frac{90 \times 0.8 \times 2.50}{1000}\right] + \left[\frac{0.8 \times 1.0 \times 10.0}{2}\right]$

$= 7.85$ kN/m

Area resisting shear

$A_{dw} = 2\ t_s$

$= 2 \times 25 \times 1000$

$= 50{,}000$ mm$^2$/m

Design compressive stress at base

$f_{dc} = \frac{G_g}{A_{dw}}$

$= \frac{7.85 \times 1000}{50{,}000}$

$= 0.156$ MPa

Masonry bond strength (at concrete interface)

$f'_{ms} = 0$

Shear bond capacity

$V_o = \phi\ f'_{ms}\ A_{dw}$

$= 0.60 \times 0 \times \frac{50{,}000}{1000}$

$= 0$

Shear factor for concrete interface

$k_v = 0.3$

Shear friction capacity

$V_{1e} = 0.9\ k_v\ f_{de}\ A_{dw}$ *AS 3700 7.5.4.1*

$= 0.9 \times 0.3 \times 0.156 \times \frac{50{,}000}{1000}$

$= 2.1$ kN/m

Total shear capacity

$V_{cap} = V_o + V_{1e}$

$= 0 + 2.1$

$= 2.1$ kN/m

$< 1.01$ kN/m OK

ROOF ANCHORAGE

Roof permanent load

$g_c = 1.2 \times 1.01$

$= 1.2$ kN/m$^2$

Span

$S = 10.0$ m

Total roof weight

$G_c = 1.2 \times 10$

$= 12.0$ kN/m length along the line of the wall

Required connection strength

$P = 0.05\ G_c$

$= 0.05 \times 12.0$

$= 0.6$ kN/m length along the line of the wall for trusses to top plate and top plate to wall

NOTE: This same connection will be required to resist force resulting from the out-of-plane earthquake forces on the wall, considered below.

Provide cross-bracing in the roof system to allow diaphragm action to transmit roof loads to shear walls.