



Manual 4  
Design of  
Clay Masonry  
for Wind and  
Earthquake

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Joint Winner - Horbury Hunt Commercial Award 2012  
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Builder: Construction Engineering  
Masonry Contractor: Byrne Construction Systems  
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This manual provides guidance for the design of unreinforced clay masonry to resist wind and earthquake forces. It follows the procedures set out in the Masonry Structures Code (AS 3700)<sup>1</sup>. For any aspects not covered here, reference should be made to AS 3700. Useful guidance on the interpretation of AS 3700 can also be found in its commentary<sup>2</sup>. The standard Masonry in Small Buildings – Part 1: Design (AS 4773.1)<sup>3</sup> contains simplified rules compatible with AS 3700 that can be applied to the design of small buildings.

Most masonry construction in Australia is unreinforced and non-loadbearing. The common definition of a non-loadbearing wall is one that does not support any significant vertical loads other than its self-weight. Nevertheless, these walls are subjected to loading from wind and earthquake, as well as overall requirements for robustness. Even internal partition walls are subjected to earthquake loading. Walls with a moderate level of vertical loading derive additional stability against face loads and the most critical case for out-of-plane lateral loading is therefore a wall with no superimposed vertical load.

Wind loading in parts of Australia can be severe, and the magnitude of load from wind increases significantly in the upper stories of multistorey buildings.

Unreinforced masonry construction generally has low seismic resistance, because it is a heavy, brittle material with low tensile strength and exhibits little ductility. It is therefore unsuitable for areas of high seismicity. However, the level of earthquake forces experienced in Australia is moderate by world standards and unreinforced masonry can be used in most instances, provided the structure is designed and detailed for the appropriate earthquake forces and built to the required standard.

This manual applies to all structural forms using masonry-veneer walls, cavity walls or single-leaf walls, including single-occupancy housing, multiple-occupancy units and townhouses, industrial and commercial buildings and multistorey, framed construction with masonry infill. It covers material properties for clay masonry, general arrangement of structures, specific design procedures for out-of-plane lateral loading and in-plane shear loading, and design detailing of ties, connections and joints. It does not cover design for vertical loading (see Manual 6).

Worked examples for various cases and design charts for lateral loading (using equivalent static loads for both wind and earthquake) are included.



## 2. Types of Construction

### 2.1 Housing

The most common form of domestic construction in Australia is the single-occupancy house. The vast majority of these are clad with clay masonry and brick-veneer is the most popular form in the eastern states. Full-brick cavity construction is popular in Western Australia and single-leaf construction using hollow units is popular in North Queensland. Because the walls of houses generally support only a light roof load or no load at all, the critical design load is usually lateral load from wind or earthquake.

In a veneer-wall house, the frame (timber or steel) is relied upon to resist the main forces, including vertical forces from the roof and lateral in-plane shear. In a cavity-wall house and single-leaf construction, the masonry walls must provide the resistance to all lateral forces, including in-plane shear. The latter can be the governing action where earthquake forces are high.

### 2.2 Multiple-occupancy Domestic Units

Multiple-occupancy domestic units of loadbearing masonry (commonly called three or four-storey walk-ups) are common in Australia and two-storey semi-detached townhouses are becoming increasingly popular. In these buildings, the masonry walls usually support concrete floor slabs and the roof structure and their sizes are determined accordingly. However, especially in the upper storeys and in townhouses, wall designs can be governed by resistance to out-of-plane forces.

In these structures, the masonry walls must also provide the resistance to lateral in-plane (shear) forces, with the floor and roof acting as diaphragms to distribute forces to the walls.

### 2.3 Low-rise Commercial and Industrial Buildings

Where masonry panels are used as cladding for commercial and industrial buildings their structural design is usually governed by resistance to wind and earthquake forces. Economy in design is vital for these walls. In these buildings, the frame of concrete or steel provides the overall resistance to lateral forces and the walls must have sufficient flexural resistance to span between frame members and other supports. Deflection compatibility between frames and walls is an important consideration.

### 2.4 Multi-storey Framed Structures

Masonry cladding is popular for multi-storey structures where the frame is commonly made of reinforced concrete or steel. In these cases the walls provide the envelope to protect the interior against the weather and are only required to resist lateral out-of-plane wind and earthquake forces. Often, the inner leaf of the external walls is an infill wall tied to the frame. This may then act compositely with the frame, however design for composite action between frames and infill walls is beyond the scope of this manual (for additional discussion, see Section 3.3). Where composite action is not designed for, isolation of infill walls from frame movement is essential under heavy earthquake loads. The external leaf is usually a veneer, supported by angles or nibs on the floor slabs.

The external walls in the upper storeys of multi-storey buildings can be subjected to high wind loads because of their height above the ground and this will usually govern their design.

Internal walls are not subjected to wind loading but are still required to resist out-of-plane earthquake forces, which will also be highest in the upper storeys.

## 3. Masonry Elements

Various types of masonry elements are used to make up a typical masonry structure. These include walls (which might be of veneer, cavity, solid or diaphragm construction), piers and freestanding elements such as parapets and chimneys. These various types of elements behave in different ways and their design must take into account their particular characteristics. Detailed design of diaphragm walls is beyond the scope of this manual.

### 3.1 Veneer Walls

Unreinforced masonry is widely used as a veneer in residential, light commercial and multi-storey framed construction. Clay brick is by far the most common choice of masonry for these applications. Veneers are non-structural elements and rely on the supporting backup frame or wall and the accompanying tying system for stability. Although they are non-structural, veneers are nevertheless subject to wind and earthquake loading. In particular, the seismic performance of veneers is important because of their widespread use and the high cost of repair if their performance proves to be inadequate.

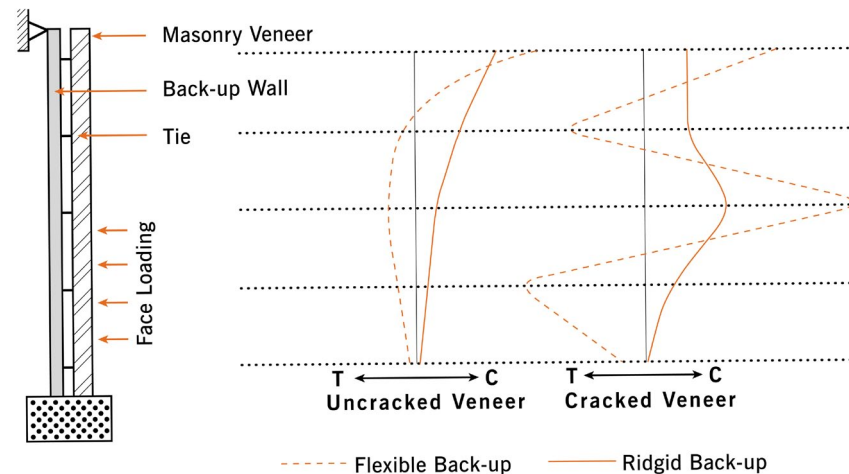
The behaviour of a veneer subjected to face loading is quite complex because it depends upon the relative flexibility of the veneer and the backup system, as well as the stiffness and location of the wall ties. These factors affect the degree of load sharing between the veneer and backup and the amount of redistribution of load that can occur. There is also a substantial difference in behaviour when the veneer is cracked rather than uncracked, because, in its uncracked state, the veneer is usually much stiffer than its backup. The masonry veneer itself does not usually need to be designed. For design purposes, it is sufficient to know the wall tie forces and the corresponding loads on the backup frame or wall. Typical distributions of tie force derived from an elastic analysis<sup>4</sup> are shown in Figure 1, where T indicates a tensile force in the ties and C indicates a compressive force. In

that study, the veneer system comprised an external 110 mm brickwork skin connected by medium-duty ties to either a flexible backup system (typically timber or steel stud wall) or a rigid backup system (typically an internal masonry leaf). Both uncracked and cracked (at mid-height) conditions were examined. The marked difference in tie forces for the cracked and uncracked states is shown in Figure 1.

Under wind loading, the force in each tie is directly influenced by the stiffness of the backup. The forces are tensile in some locations for a flexible frame. Before the wall cracks, the top ties adjacent to the backup attract a much greater proportion of the load than would be expected from their tributary area. This explains the logic of deemed-to-comply rules, which require the number of ties to be doubled in these locations if the backup is flexible. If the veneer cracks longitudinally at mid-height along a bed joint there is a dramatic redistribution of load in the ties, with the ties near the mid-height of the wall becoming heavily loaded. This is particularly the case when the backup is a flexible frame. It is clear that the ties play a crucial role in this interaction and their strength and stiffness are both important.

A veneer wall relies on flashing and damp-proof courses, in conjunction with weep-holes, to act as an effective barrier to moisture entering the building. The presence of flashing and a damp-proof course will influence behaviour under lateral load.

**Figure 1.** Variation of Tie Forces for Flexible and Rigid Backups



### 3.3 Masonry Infill

Unreinforced masonry infill panels have the potential to add considerably to the strength and rigidity of a framed structure if they are designed and detailed for composite action. Interaction between infill and frame depends on the contact area at the interface of the two components. The extent of composite action will depend on the level of lateral load, the degree of bond or anchorage at the interfaces, and geometric and stiffness characteristics of the frame and infill masonry. The possibility of the mobilisation of the infill, especially to

resist seismic loads, should be considered at the design stage. However, in Australia it is good practice to leave gaps at the vertical edges and top of infill panels to allow for long-term moisture expansion of clay bricks. The infill panels are secured to the frame by ties, which permit the desired relative movements, and flexible sealant fills the gaps. In these cases, composite action will not occur until large frame deflections have taken place. Consideration of composite action between masonry infill and frames is beyond the scope of this manual.

Design of infill panels that are isolated from the frame are usually governed by flexural action to resist lateral out-of-plane forces. If there is the possibility of a shallow arch developing within the thickness of the wall as it deflects, that should also be considered. However, this arching action is unlikely if expansion gaps are left between the wall and the frame and design for this action is difficult because of the uncertainty of its extent. Consequently, AS 3700 does not give design rules for this arching. Infill wall panels are usually designed as one-way or two-way spanning plates of masonry with simple supports provided by the framing members.

### 3.2 Cavity Walls

Cavity walls are constructed of two leaves of masonry separated by a cavity, typically 50 mm in width, intended primarily to prevent water penetration into the building. This form of construction has been popular in Australia and other parts of the world since the early twentieth century because it provides a wall having good thermal and strength properties, without the need to maintain an external coating.

In resisting applied loads normal to the face, cavity walls rely on the interaction between the two leaves through the ties. Behaviour of the whole system is complex and a detailed structural analysis would be required in order to predict accurately the forces in individual components. This is usually impractical and simplified rules are employed to design the masonry leaves and the ties.

Proper detailing of flashings, damp-proof courses and weep-holes is essential to ensure that a cavity wall remains an effective waterproof barrier. As in the case of veneer walls, the presence of flashing and a damp-proof course will affect behaviour under lateral load.

### 3.4 Freestanding Elements

Parapets and other freestanding elements are commonly used in unreinforced masonry structures. Because of the low flexural strength of the masonry, these elements have little resistance to lateral load and must rely on gravity for stability. The presence of a flashing or damp-proof course at the base exacerbates the situation. In addition, these elements are usually located at or near the top of the structure where the wind loading is highest and the effects of seismic ground motion are magnified by the dynamic response of the building.

It is desirable to avoid the use of freestanding elements, or, if they must be used, for them to be supported or locally reinforced to provide flexural strength (see Section 6.3.3).

### 3.5 Diaphragm Walls

Diaphragm walls are a form of construction that offers high resistance to lateral out-of-plane load. They are formed of two leaves, separated by a substantial distance and joined by webs of masonry that are fully bonded into the leaves. These webs transfer shear between the two leaves, allowing the wall to act in flexure as a fully composite member rather than two separate leaves. The lateral resistance of a diaphragm wall depends primarily on the separation between the two leaves and the webs joining the leaves must be specifically designed to resist the forces imposed on them.

The detailed design of diaphragm walls is beyond the scope of this manual. Design rules are given in AS 3700 and further guidance is available in the Australian Masonry Manual<sup>5</sup>.

# 4. Material Properties

Fired clay bricks have been in use for at least 5,000 years. Clay masonry is particularly noted for its attractive appearance, long life and good loadbearing qualities. When properly constructed and detailed it provides one of the most functional walling systems ever developed.

## 4.1 Masonry Units

The applicable Australian standard governing the manufacture of masonry units is AS/NZS 4455.1<sup>6</sup>. Units for use in masonry construction are required by AS 3700 to satisfy that standard. Test methods are specified in a companion standard AS/NZS 4456<sup>7</sup>. Not all the tests described in this standard are required to be specified; AS 3700 clearly sets out which tests and properties are required in each particular case.

While durability classification, dimensions and aesthetic requirements must always be considered, the important properties of masonry units for walls designed to resist wind and earthquake loads are:

- Absorption characteristics compatible with the mortar to be used, so that the required flexural tensile bond strength is achieved.
- Lateral modulus of rupture sufficient for the required flexural strength.

## 4.2 Mortar

Mortar is an important ingredient in masonry construction because its characteristics have a strong influence on both the strength and durability of the masonry assemblage. It is also the component most susceptible to site problems related to mixing and batching. Mortar must be workable when wet and have sufficient strength and be adequately bonded to the masonry units when set. The tensile bond strength of masonry can vary from zero to more than 1.0 MPa depending on the correct match of mortar and unit properties, in particular the match between mortar consistency and unit suction.

Selection of sand, cement, mix composition and admixtures such as air entrainer (when appropriate) is of vital importance for the achievement of the required

tensile bond strength. Mortar mix proportions are specified as cement:lime:sand by volume. Further information on mortar composition and admixtures is given in Manual 10, Construction Guidelines for Clay Masonry<sup>8</sup>.

It is essential for the job specification to refer to either the class of mortar or the mix proportions (and ideally both). AS 3700 no longer requires different mix proportions for GP (Portland) and GB (blended) cements. For example, AS 3700 classifies the following three mortars as M3:

- 1:1:6 using GB or GP cement
- 1:0:5 using GB or GP cement
- 1:0:4 using masonry cement.

## 4.3 Masonry Properties

By definition, masonry is a composite material consisting of masonry units set in mortar. Because the units and mortar have different characteristics, masonry exhibits distinct directional properties with potential planes of weakness being created by the low tensile strength at each unit/mortar interface. For resistance to wind and earthquake forces it is this bond strength at the interface that is important, both in flexure and in shear.

Flexural tensile strength ( $f'_{mt}$ ) is required by AS 3700 to be at least 0.2 MPa for all masonry. This is a 95% characteristic value, which means that 95% of all masonry in the building should be stronger than this design value and only 5% of the masonry is weaker. This bias is taken into account in setting the required safety factors. Design values as high as 1 MPa can be taken, but only if site control testing is carried out as part of the construction to verify that the required strengths are being achieved. Any masonry where strengths higher than 0.2 MPa are used and the specification calls for quality control testing is classified as Special Masonry. The designer should be quite sure about the materials specified and the potential strength before using a design strength higher than the minimum value.

Shear strength on horizontal planes in clay masonry ( $f'_{ms}$ ) is defined by AS 3700 as  $1.25 f'_{mt}$  but not greater than 0.35 MPa nor less than 0.15 MPa. This property is therefore also related to the basic bonding between masonry units and mortar. Additional shear resistance is provided by the friction effect of vertical load, which is accounted for by a shear factor prescribed in AS 3700 (see Section 8.2).

## 4.4 Ties and Connectors

Masonry wall ties are a structural component of the wall, not an optional accessory. It is most important that ties should be appropriately designed and specified, should have the necessary durability and should be properly installed. The standard covering wall ties is AS 2699 Part 1<sup>9</sup> and for other connectors and accessories AS 2699 Part 2 governs<sup>10</sup>. Rarely, if ever, should the designer need to refer to these standards, as they are intended to control the manufacture of the ties and accessories, not their use. AS 3700 gives everything necessary for the specification and use of these components. Design capacities of connectors should be obtained from the manufacturer.

Ties are classified based on strength and stiffness as light duty, medium duty and heavy duty. This rating is determined by tension and compression tests on small tie/masonry assemblages, with the test results reflecting both the behaviour of the tie itself and its attachment to the masonry and the frame. Designers should use the procedure in AS 3700 to determine the grade required for each particular loading situation (see Section 9.2). This required grade should then be clearly specified on the documents.

Ties and connectors are commonly made from steel with a protective coating. Where a high level of durability is required, stainless steel or polymer ties can be used. AS 3700 gives the requirements for durability in terms of a rating from R1 to R5 (see AS 3700 Table 5.1). The means of satisfying these rating requirements are given in AS 2699.1. The designer should specify the durability requirement for ties and connectors on the documents.

## 4.5 Damp-proof Courses, Joints and Other Accessories

Damp-proof courses (DPC) must comply with the Australian standard AS/NZS 2904<sup>11</sup>. Most loadbearing masonry structures subjected to earthquake forces rely to some extent on the transfer of shear across the DPC to develop the necessary resistance. AS 3700 provides friction shear factors for the common DPC materials to allow these calculations to be made (see Section 9.4). If shear resistance on a damp-proof course or other joint is a critical design factor, the documents should clearly indicate the type of material that is required to satisfy the design assumptions.



# 5. Loading Conditions

Wind and earthquake produce horizontal lateral forces on a structure, which generate in-plane shear loads and out-of-plane face loads on individual members. While both wind and earthquake generate horizontal forces, they are different in nature. Wind loads are applied directly to the surfaces of building elements, whereas earthquake loads arise due to the inertia inherent in the building when the ground moves. Consequently, the relative forces induced in various building elements are different under the two types of loading.

## 5.1 Wind Loading

Wind load is often the most important load acting on a structure. Levels of wind loading vary greatly throughout Australia, from moderate in the interior to very high in the cyclonic regions of the north. There are also local factors to consider, such as topography, surrounding shelter and height above ground.

Basic wind speeds and the numeric multipliers for dealing with factors such as topography are given by the wind loading standard (AS 1170.2)<sup>12</sup>. This standard is designed for use by structural engineers and detailed illustration of its use is beyond the scope of this guide.

For housing structures, wind loads are determined from a classification system given in the relevant standard AS 4055<sup>13</sup>. This system uses a wind classification based on regions, terrain categories, topographic classes and shielding classes. Provided the structure is within certain restrictions on height, shape and slope of roof, it is classed as N1 to N6 for non-cyclonic regions and C1 to C4 for cyclonic regions. This wind class then determines the design wind speeds for the serviceability and ultimate limit states. For the ultimate limit state these range from 34 metres/second to 86 metres/second. These wind speeds are used to derive forces on the structure by considering pressure coefficients based on the shape and size of the structure and the presence of openings. AS 4055 Section 3 tabulates the resulting pressures for ultimate and serviceability conditions.

Wind acting on a structure causes three main effects that must be accounted for in design. These are out-of-plane

bending, in-plane shear and uplift causing direct tension. Design for the first two is covered in Section 7 and Section 8. The effect of the third should be considered for individual members when they are designed for flexure and shear. Net direct tension on the cross-section of a masonry member must be avoided for design in accordance with AS 3700, as the tensile strength of the material is considered to be zero in such circumstances.

## 5.2 Earthquake Loading

Earthquake loading is the force generated by horizontal and vertical ground movements caused by earthquake. These movements induce inertial forces in the structure, related to the distributions of mass and rigidity, and the overall forces produce bending, shear and axial effects in the structural members. Earthquake loads are different in nature to wind loads and can produce different effects in some cases. Earthquake loading is governed by AS 1170.4<sup>14</sup>.

The 1989 Newcastle earthquake highlighted the need for seismic design in Australia and, with the incorporation of AS 1170.4 into the Building Code of Australia, consideration of seismic effects is mandatory for all structures except Importance Level 1 (minor structures) as defined in AS 1170.0<sup>15</sup> Appendix F.

Because of the low levels of Australian seismicity, it is feasible to use properly detailed and constructed unreinforced masonry in most areas. For domestic structures within certain limits, the lateral racking forces from earthquake loading are given in AS 1170.4 Appendix A (see Section 5.2.1 below).

Masonry veneer attached to a ductile frame of timber or steel is considered non-structural and requires no specific design for earthquake if it complies with AS 3700. Other unreinforced masonry (in solid or cavity walls) is classed as non-ductile and must be designed to remain essentially elastic. A non-ductile structure is required to carry a higher level of applied load than a ductile structure. Ductility factors for masonry are given in AS 3700 Section 10. As for all seismic design, clear load paths must be established and irregularities in plan

and elevation must be considered. The establishment of load paths includes the effective transmission of seismic forces across the various connections and any other discontinuities in the structure, and to this end the influence of flashings, membrane-type damp-proof courses and slip joints must be considered.

For simplicity, earthquake loading can be converted to equivalent static forces with appropriate allowance for the dynamic characteristics of the structure, foundation conditions etc. This approach is sufficient for most masonry structures, which normally have a short fundamental period of vibration and low dynamic response.

AS 1170.0 provides load combinations for seismic design against strength and stability limit states. Serviceability is not considered for seismic design, but must be considered when detailing connections for overall performance.

In the application of AS 1170.4, four factors are considered in determining an earthquake design category:

1. The structure importance level.
2. The combination of probability factor and hazard factor.
3. The site sub-soil class.
4. The height of the building.

The structure importance level is related to the consequences of failure. The probability factor is determined from the annual probability of exceedance that is being designed for and the hazard factor is determined by the geographic location of the structure. Together, these generate an equivalent acceleration coefficient. The site sub-soil class depends on the soil profile of the site, ranging from strong rock to very soft soil.

The resulting earthquake design category of I, II or III determines the design principles and the type of analysis (static or dynamic) required to determine the

forces imposed on the structure. Design and detailing requirements become more stringent as the earthquake design category increases from I to III. The design of the structure and its individual elements is covered by the relevant material standards (e.g. AS 3700 for masonry).

AS 3700 provides deemed-to-satisfy details for masonry structures up to 15m in height. These are intended to ensure that masonry elements have adequate support and tying into the structure to prevent collapse during an earthquake.

In addition, AS 3700 restricts the use of loadbearing unreinforced masonry to buildings less than certain heights limits, except for some very restrictive conditions that are intended to permit small plant rooms and the like on top of multi-storey structures with adequate framing systems to resist seismic loads. The height limits depend on the hazard factor and sub-soil class and are all less than or equal to 15m (see AS 3700 Table 10.3). All loadbearing masonry structures in excess of the height limits require the use of reinforced masonry for the structural system. The impact of these limitations on current practice is small because the majority of unreinforced masonry is used in residential construction, low-rise commercial and industrial structures. Most of these structures are typically four storeys or less and the major population centres such as Sydney and Melbourne are not located in severe earthquake zones.

### 5.2.1 Domestic structures

Earthquake resistance of domestic construction depends on good detailing more than on structural analysis. Domestic construction derives its resistance from overall system behaviour, which can only occur if all the parts of the structure are adequately connected. The intention of the structural detailing requirements is to ensure that this connection is provided, so that all forces on the structure are transferred to the foundations. In particular, the following are important:

- Horizontal resistance must be provided for connections of beams and trusses to their supports.
- External walls must be anchored to roofs and floors

## 6. General Design Aspects

for horizontal support and internal loadbearing walls must be restrained at their top and bottom.

Other measures can be taken to improve horizontal earthquake resistance, for example by incorporating sub-floor braces for discrete footings.

Attention should be paid particularly to detailing of unreinforced 'non-structural' masonry components, as they are the elements most at risk during an earthquake. Non-ductile components such as unreinforced masonry gable ends, internal non-loadbearing walls, chimneys and parapets require appropriate restraint.

AS 1170.4 makes special provision for domestic structures. Within certain geometric limits shown in AS 1170.4 Figure A1, and for a combination of probability factor and hazard factor less than or equal to 0.11 (which covers most areas of Australia) there is no specific earthquake design required for masonry structures that are otherwise designed in accordance with AS 3700. This is because the system already in place to resist lateral wind load should provide sufficient wall, floor and roof diaphragms to resist horizontal earthquake loading.

Where the combination of probability factor and hazard factor exceeds 0.11 the structure can be designed using simplified calculation of racking forces and tying of walls, chimneys, parapets and the like to resist specified anchorage forces (see AS 1170.4 Appendix A). Otherwise the structure must be designed as for other structures of Importance Level 2.

### 6.1 Structural Behaviour

#### 6.1.1 Wind Loading

Traditional masonry structures were massively proportioned to provide stability and prevent tensile stresses. In the period after 1945, traditional loadbearing construction was replaced by structures using the shear wall concept, where stability against lateral loads is achieved by aligning walls parallel to the load direction. Lateral forces are therefore transmitted to the lower levels by in-plane shear. This, combined with the use of concrete floor systems acting as diaphragms, produces robust box-like structures with thin walls and the capacity to resist lateral load. Loadbearing structures of this type offer an economical alternative to framed construction for low and medium-rise buildings, particularly for structures with repetitive floor layouts. For these structures, the walls subjected to face loading must be designed to have sufficient flexural resistance and the shear walls must have sufficient in-plane resistance.

The alternative structural form consists of a frame, usually of steel, timber or concrete, which resists the lateral forces by bending (frame action). The masonry walls are attached to this frame as a cladding and distribute the applied lateral forces into the framing members. In this system, the masonry walls are designed for local flexural action only.

#### 6.1.2 Earthquake Loading

In buildings subjected to earthquake loading the walls in the upper levels are more heavily loaded by seismic forces, because of dynamic effects, and are therefore more susceptible to damage caused by face loading. The resulting damage is consistent with that due to wind or other out-of-plane loading. Racking failures are more likely to occur in the lower storeys where shear forces are greatest and are characterised by stepped diagonal cracking. This damage does not usually result in wall collapse, but can cause considerable distress. Racking damage can also occur in structures with masonry infill when large frame deflections cause load to be transferred to the non-structural walls.

Both plan and elevation symmetry is desirable to avoid torsional and soft-storey effects during seismic activity. Compact plan shapes behave better than extended wings. If irregular shapes cannot be avoided, then more detailed earthquake analysis may be necessary. In some cases, it may be possible to separate wings by suitable isolation joints and thereby convert the structure into a series of regular shapes.

Recent research<sup>16</sup> examined the earthquake resistance of unreinforced masonry residential structures up to 15 m in height, with a view to identifying the critical actions under a range of conditions. The study considered the wall forces and associated actions arising from earthquake loads corresponding to AS 1170.4. The seismic demands under various conditions were compared with the corresponding seismic capacities given by AS 3700. A parametric study was used to examine the effects of a wide range of parameters, including number of storeys, wall geometries, support conditions and openings. The results of the parametric study indicate, for a typical office building and a typical home unit building, the range of conditions leading to earthquake failure using the current design criteria of AS 3700. The following were the main findings:

- Out-of-plane bending tends to govern as wall length, site sub-soil class, hazard factor and the number of levels increase. This applies for both office buildings and home unit buildings. This finding is based on the failure criterion of maximum flexural strength being reached.
- Out-of-plane shear governs in relatively few cases and, when it occurs, it is in conjunction with out-of-plane bending and/or in-plane shear failure. There is no difference in this respect between typical office buildings and home unit buildings.
- In-plane shear in the direction of the short plan dimension of a building is governed by the arrangement of the internal walls. For the assumed wall distributions used in this study, in-plane shear in the short direction was not critical for the office building and, for the home unit building, occurred simultaneously with in-plane shear failure in the long direction.

- In-plane shear in the long direction is the most significant mode governing structural performance when the failure criterion of onset of sliding at the base of the wall is used. The assumed layout of internal walls was found to be a significant factor influencing behaviour. However, the onset of sliding does not necessarily constitute failure under seismic action, as it often does not lead to collapse or provide a risk to life.

A follow-up study<sup>17</sup> summarises the results of a displacement-based assessment (DBA) of the seismic capacity of typical loadbearing unreinforced masonry buildings between two and five storeys in height across a range of site sub-soil classes and earthquake hazard factors, covering all of the capital cities and major regional centres in Australia. This study found that the DBA for out-of-plane bending of walls in the top storey of buildings identified far fewer cases of failure than did a traditional strength-based assessment. A similar trend was observed for the DBA for in-plane shear of walls at the ground storey of buildings. The DBA implied, for all practical purposes, that typical walls will have the in-plane shear displacement capacity to withstand the earthquake induced loads and displacements for any site soil conditions and earthquake hazard factor up to 0.12. This contrasts with the corresponding strength-based calculations, which identified significant numbers of cases where failure would occur. While current standards are based on strength-based analysis, there is scope for designers to use displacement-based methods (with due caution) and for standards to be augmented along these lines in the future.

## 6.2 Mechanism of Load Transmission

The fundamental aspect common to both wind and earthquake loading is that load imposed on the structure must be transmitted through a load path to the foundation. It is important for this load path to be identified and the respective structural elements designed for the part they play in it. The load path is different for structures that rely on masonry walls as loadbearing elements and those where the masonry forms infill or veneer applied to a structural frame.

### 6.2.1 Housing

The mechanisms of load transfer in masonry housing are different for cavity construction and for masonry veneer.

In the case of cavity construction, the inner leaf supports the vertical load of any upper floors and the roof, while the outer leaf provides the weather-resistant cladding. Lateral out-of-plane forces are shared between the two leaves in proportion to their respective stiffness. The inner leaf is usually much stiffer, because of its load supporting function, and therefore resists a larger portion of the lateral load. The extreme case is when the outer leaf is attached only by the ties and is designed as a veneer on a stiff backup. Cross walls within the building, which might either be of masonry or of framed construction, act to transfer lateral shear forces to the footings.

For masonry veneer construction, the building frame supports all the applied forces (both vertical and horizontal) while the masonry cladding protects against the weather. The face loads applied to the walls are transmitted to the backup frame through the ties. The framing members then transfer the forces to shear diaphragms such as floors, ceilings and the roof. Lateral shear resistance is provided by the frame, which must be fitted with suitable bracing to transfer forces to the footings. It is only necessary to check the masonry veneer for its ability to span in flexure between ties and the tie spacing specified in AS 3700 will usually ensure that this capacity is adequate. To limit the size of any crack in the veneer, the deflection of the structural backing is limited by AS 3700 (Clause 7.6.2(b)) to span divided by 300.

### 6.2.2 Framed Structures

In a framed structure, load is transferred from the face-loaded walls to the framing members through their connections. The framing members then act together to resist the lateral force, either by sway action or by braced-truss action, thereby transferring the force to the foundations. The important elements to be considered by the masonry designer are the masonry walls (in out-of-plane flexure) and the connections. Isolation of the masonry walls from any large frame sway movements might also be necessary.

### 6.2.3 Loadbearing Structures

The basic mechanism of lateral load transmission for a loadbearing structure is shown in Figure 2. Walls aligned in a plane normal to the load are subjected to face loads and span vertically between the floors and, in some cases, horizontally between loadbearing walls. The concrete floors then act as rigid diaphragms and transfer the load to the shear walls, which in turn transmit the forces to the ground by in-plane shear.

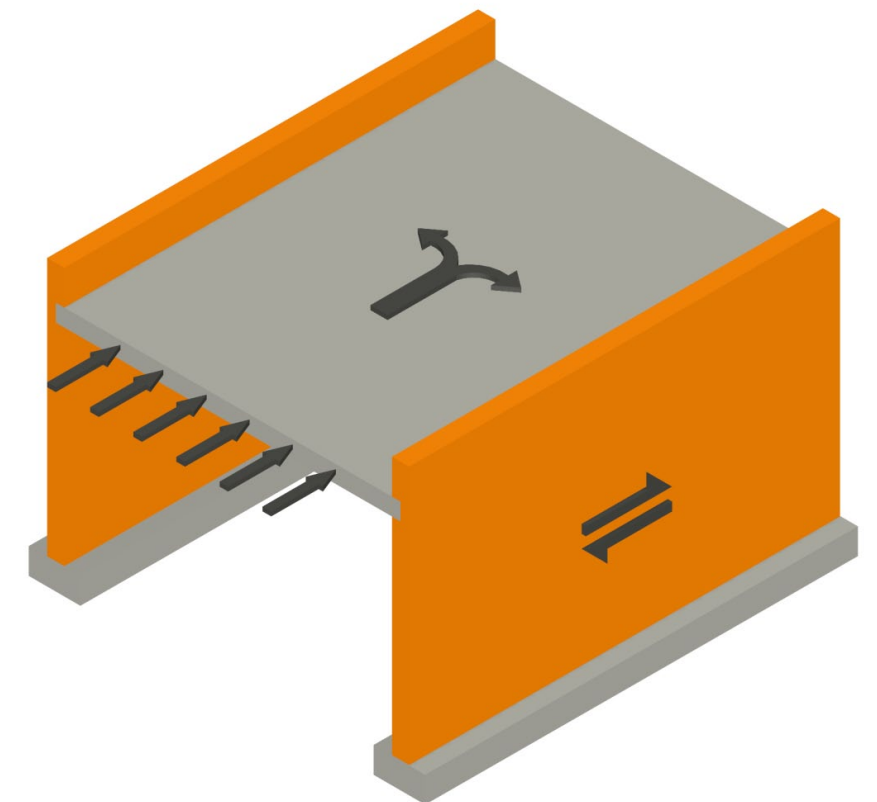
The resulting structures are usually quite robust, with relatively short-span concrete slab systems supported by numerous walls running in both principal directions. The effective performance of this system depends on the ability of the individual masonry elements to sustain their share of the load, as well as the capability of the connections between the elements to transmit the appropriate forces. The elements to be considered by the masonry designer are the out-of-plane flexural action, the in-plane shear action and the connections between structural elements.

## 6.3 Tying and support of elements

The correct performance of ties and connections between structural elements is just as important as the behaviour of the elements themselves, especially under earthquake loading. Any failure or inadequacy of the connections can lead to catastrophic collapse of the structure. Important connections include the following:

- Ties between the leaves of a cavity wall.
- Ties between a masonry veneer and its backup.
- Roof tie-downs.

**Figure 2.** Diaphragm and Shear Wall Action in a Loadbearing Structure



- Connectors between the edges of a non-loadbearing wall panel and its supports.
- Connectors attaching masonry infill to a structural frame.
- Connections providing support for non-structural components such as parapets and gables.
- Slip joints, DPC membranes and flashings.

It is important to remember that ties and connectors are usually required to provide resistance in only one direction, and they must not unduly restrain movement in other directions. These considerations lead to the test requirements for obtaining wall tie ratings, where they must be subjected to a significant lateral movement



# 7. Design of Walls for Out-of-Plane Load

prior to application of the compressive or tensile force. Similarly, in the case of slip joints and DPC membranes, movements due to thermal and moisture strains and long-term expansion or shrinkage of the masonry units must be allowed to occur without causing distress in the structure. Under earthquake loading, the behaviour of ties, connectors and joints under cyclic reversing load must also be taken into account.

### 6.3.1 Slab/Wall Connections

A slab-wall connection must be capable of transmitting the horizontal force induced in the wall to the slab. For unreinforced masonry this requirement creates potential serviceability problems, since if a positive form of attachment is adopted, the long-term movements mentioned above will be restrained, thus inducing cracking in the masonry. If a positive form of connection is not adopted, then reliance must be placed on the transfer of the seismic force by friction. Design for friction is covered in Section 9.4.

### 6.3.2 Shear Capacity of Membranes and Joints

Membrane-type damp-proof courses are widely used in Australia as a barrier at the base of walls to prevent the passage of moisture from the ground to the structure. These same membranes are also used for flashings and in

slip joints. The use of these membranes in masonry walls has significant structural implications, as both in-plane and out-of-plane forces must be transmitted across the joint containing the membrane. Design of these joints is covered in Section 9.4.

### 6.3.3 Parapets and Freestanding Elements

Freestanding elements such as parapets must be adequately supported and tied to the structure. They can be subjected to high loads, especially from seismic action, where the dynamic response of the building can magnify the force. The provisions of AS 1170.4 allow for this effect by the application of a height amplification factor, which has a maximum value of 3.0 (see Clause 8.3 of AS 1170.4). This is a simplification of what is quite a complex phenomenon.

Grouted and reinforced cavity construction or grouted reinforced hollow clay units can be used to provide sufficient resistance for this purpose. Alternatively, unreinforced parapets can be tied to the main structure or proportioned to span horizontally between returns or piers that are designed to provide overall stability. Examples of details for tying parapets and chimneys back to a roof structure are shown in AS 3826<sup>18</sup>.

## 7.1 Introduction

For lateral out-of-plane loading, whether it arises from wind or earthquake, the response of the structural element is usually calculated by considering the load as an equivalent static uniform pressure. Although there is a fundamental difference in the type of load caused by wind and earthquake, there is no difference in design procedure between the two in most cases.

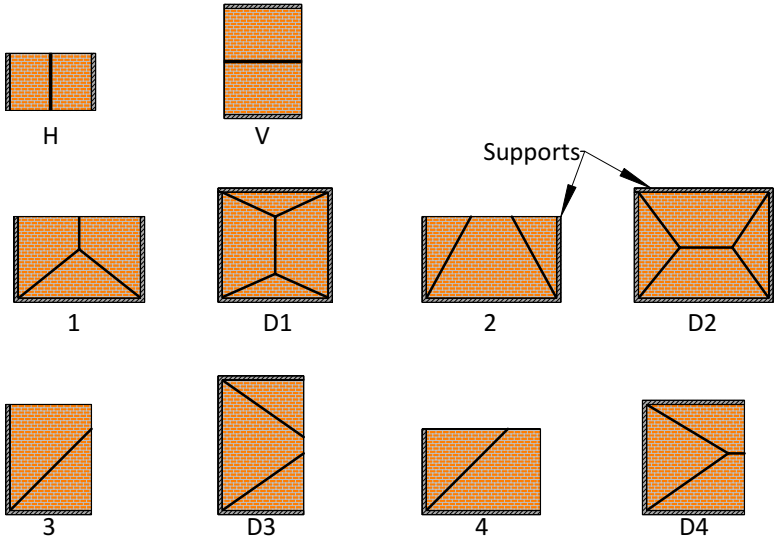
Masonry elements subjected to out-of-plane loading resist the load by flexural action. The load capacity of unreinforced masonry wall panels depends upon the dimensions and support conditions, the level of compressive stress in the wall and the tensile strength of the masonry. The presence of door and window openings

also has a strong influence on the behaviour. For veneer or lightly-loaded panels where the level of compressive stress is low, flexural tensile strength is particularly important. Determination of design loads for wind and earthquake is discussed in Section 5.

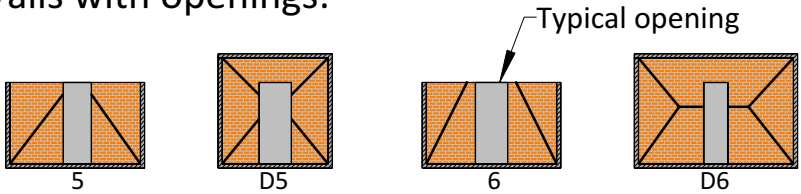
Masonry walls behave differently under simple bending in one direction (for example between top and bottom supports) and bending in two directions (such as when the wall has support on at least two adjacent edges). This is a result of the flexural action of plates under distributed loading and also the different flexural properties of masonry normal to and parallel to the bed joints. Most walls are designed to have support conditions that result in two-way bending because this action is much stronger than simple one-way bending.

Figure 3. Idealised Crack Patterns for Various Wall Configurations

### Walls without openings:



### Walls with openings:



The design charts given in this manual give design lengths and heights for walls with a range of support conditions, for various wind pressures. Masonry walls crack in particular patterns under face loading, depending on the support conditions and the dimensions. Figure 3 shows the idealised cracking patterns that occur for various wall configurations, with edge supports indicated by dark lines. The crack lines shown, together with cracks along continuous or restrained edges, allow a mechanism to form, at which point the wall is considered to have failed.

Wall edges can be free of any support or subjected to two types of edge restraint, namely lateral restraint and full rotational restraint. The latter can be referred to as a fixed edge. These two types of restraint are defined as follows:

- *Lateral restraint* occurs when the edge of the masonry wall has only restraint in the lateral (out-of-plane) direction and there is no capacity to transmit a moment across the edge. Control joints or isolation joints where the masonry is tied to a support provide such a case and the bed joint at the base of a wall is also considered to be of this type.
- *Edge fixity* occurs when the edge of the wall is restrained against both lateral movement and

rotation. This condition can occur when a wall extends continuously past a support for a sufficient distance or where a wall returns for a sufficient distance around a corner. The standard AS 3700 does not give any guidance as to what return distance around a corner to the edge of the nearest opening or end of the wall is sufficient for this purpose. In the absence of other guidance, a distance of at least ten times the wall thickness can be considered a reasonable minimum. Commonly, where there is some degree of rotational restraint but it cannot be considered as full restraint, a rotational restraint factor of 0.5 or some other value between 0 (no restraint) and 1 (full restraint) is used. Charts are provided later in this manual for partial as well as full restraint (see Section 11.4). It should be noted that it is difficult to achieve full rotational restraint by building the edge of a wall up to a supporting member and incorporating metal ties. Such a condition should usually be considered as providing lateral restraint only.

The top of a masonry wall might be considered either to have lateral restraint or to be a free edge. The base of a wall is considered to have lateral restraint only.

7.2 One-Way Vertical Bending

The simplest and most common form of one-way bending is simple vertical bending. In this mode, the wall panel acts as a simple beam between top and bottom supports, with the main flexural stresses acting across the bed joints. This is a simple mechanism and results in a brittle failure. When a bed joint crack occurs in masonry acting in this way the joint is assumed to have no further resistance to applied moment. Resistance to vertical bending is only provided by the flexural tensile strength of the masonry, although it is enhanced by any superimposed vertical load on the wall. This superimposed load, if any, is taken into account in the design procedure by considering the resulting compressive stress as acting uniformly on the bed joint and partially relieving the tensile stress due to bending. Because a wall under pure vertical bending fails in a brittle manner and the flexural tensile strength of masonry is usually quite low, there are few cases where one-way spanning walls can be justified. Wherever possible, walls should be provided with additional support along one or both the vertical edges or, if this is not practical, the use of reinforced elements should be considered.

The design procedure for vertical bending is based on the moment of resistance  $M_{cv}$ , which is calculated as follows

$M_{cv} = \phi f'_{mt} Z_d + f_d Z_d$  .....(1)

Where

- $\phi$  = Capacity reduction factor (0.6 for bending).
- $f'_{mt}$  = Characteristic flexural tensile strength of the masonry (0.2 MPa except in the case of Special Masonry).
- $Z_d$  = Section modulus of the bedded area.
- $f_d$  = Minimum design compressive stress due to superimposed vertical load.

However, the amount of vertical compression force  $f_d$  that can be used to enhance the bending resistance is limited to  $2 \phi f'_{mt}$ , which is 0.24 MPa for most masonry (see AS 3700 Equation 7.4.2(3)).

For a case where the flexural tensile strength  $f'_{mt}$  is zero (for example at a damp-proof course or slip joint) the moment of resistance is:

$M_{cv} = f_d Z_d$  ..... (2)

Where  $f_d$  is limited to 0.36 MPa.

The use of this design procedure is illustrated with a worked example in Section 10.1 and a design chart is given in Section 11.1.

7.3 One-Way Horizontal Bending

It is possible for portions of a wall to act in what is called one-way horizontal bending. In these cases, the section of masonry is supported at the two sides, but not at the top and bottom. This might occur for example in a strip of masonry over a window or door opening, where the top of the wall is not supported. Such cases can be designed using the AS 3700 provisions for horizontal bending.

The design procedure for horizontal bending is based on the moment of resistance  $M_{ch}$ , which is given by the lower of two expressions:

$M_{ch} = 2 \phi k_p \sqrt{f'_{mt}} (1 + \frac{f_d}{f'_{mt}}) Z_d$  .....(3)

$M_{ch} = \phi (0.44 f'_{ut} Z_u + 0.56 f'_{mt} Z_p)$  .....(4)

Where

- $k_p$  = A perpend spacing factor (1.0 for traditional stretcher-bonded brickwork).
- $f'_{ut}$  = The characteristic lateral modulus of rupture of the masonry units (0.8 MPa in the absence of test data).

$Z_u$	=	The lateral section modulus of the masonry units.
$Z_p$	=	The lateral section modulus based on the bedded area of the perpend joints.

The other symbols are as defined above. Note that  $Z_p$  is less than  $Z_u$  if the perpend joints are not completely filled; otherwise they will be equal. The lateral section moduli and the lateral modulus of rupture are determined about the axis of bending in the wall, that is, a vertical axis.

The first expression is derived from an empirical fit of test results; the second expression is based on a model of failure through units and perpends. A third expression given in AS 3700 (Equation 7.4.3.2(3)) has the effect of limiting the amount of vertical compression force  $f_d$ , used to enhance the bending resistance, to  $f'_{mt}$ , i.e. 0.2 MPa for most masonry.

The use of this design procedure is illustrated with a worked example in Section 10.2 and a design chart is given in Section 11.2.

## 7.4 Two-Way Bending

### 7.4.1 Introduction

Wall panels with support on at least two adjacent sides undergo a combination of vertical and horizontal bending. This action provides more opportunity for load sharing between various parts of the wall as cracks develop and leads to a less brittle behaviour. It is therefore more desirable than one-way bending.

Observations of cracking in numerous tests have led to the identification of characteristic patterns that depend on the wall support conditions<sup>9</sup>. Each of these cracking patterns divides the wall panel into a number of sub-plates, which can be considered as being joined by hinges at their edges. While the exact distribution of moment along these crack lines is unknown, the total residual moment capacity of a crack line has been shown to relate to the shape of the units and the basic bond strength  $f'_{mt}$ . The collection of sub-plates in the cracked wall forms a mechanism that allows the wall to deflect, at almost constant load, until the total deflection is sufficient to

cause collapse. This pseudo-ductile behaviour is the basis of the virtual work method of analysis adopted in the AS 3700 design provisions<sup>20</sup>. The moment capacities of these potential crack lines are derived from the vertical and horizontal moment capacities considered in Section 7.2 and Section 7.3, along with consideration of the diagonal bending capacity (see below). More recent work has extended our understanding of flexural behaviour but is not yet incorporated into the design standard<sup>21,22</sup>.

The idealised cracking patterns are shown in Figure 3. The cracking pattern is always consistent with the shape and boundary conditions of the panel. When the top of a wall is supported, the first crack to develop is usually a horizontal crack in a bed joint at approximately the mid-height of the panel. This crack is difficult to detect and will almost disappear if the wall is unloaded. When adjacent vertical and horizontal edges are supported, diagonal crack lines form and radiate out from at or near the corners. These cracks form in the bed and perpend joints and their angle is therefore governed by the length-to-height dimensions of the masonry units. These cracks extend until they reach an edge or another failure line. When a panel is high, compared to its length, a vertical crack develops at approximately the mid-length. This failure line extends to the top of the wall if unsupported or to the intersection of the diagonal failure lines at both top and bottom. When vertical edges have rotational restraint (for example because of continuity) they will crack either before, or at the same time as, the diagonal cracks develop.

The presence of vertical load applied simultaneously with the lateral load can enhance the strength of the panel and this will be reflected in a higher value of  $M_{ch}$ . When the load becomes substantial, such as where in-plane arching can develop due to the panel edges bearing against the structural frame as the wall deflects, the wall strength can be substantially increased. However, there is no reliable design method for this action.

### 7.4.2 Virtual Work Method

The lateral load design method used in AS 3700 is based on the virtual work approach. This semi-empirical method relies on the identification of a particular

cracking pattern (as outlined above) and certain assumptions about the material properties. The method was developed by examining crack patterns of a large number of test panels and relating the ultimate load capacity of the walls to the energy developed on the crack lines. This was then calibrated to give the closest fit to the results and applied to predict the behaviour of other cases. The calibration involved deriving an equivalent torsional stress, which is closely related to the basic flexural tensile strength  $f'_{mt}$ . This derivation is the only empirical step in the development of the method.

The assumed behaviour of the three types of crack lines is as follows.

- Horizontal crack line in a bed joint – assumed to have no residual moment of resistance after cracking. This type of crack has virtually no influence on the overall strength.
- Vertical crack line – results from failure through masonry units and perpends or from the stepped shearing failure alternating between the mortar joints and perpends. The moment capacity is influenced by the lateral modulus of rupture of the masonry units and the flexural tensile strength of the masonry and is expressed by  $M_{ch}$ .
- Diagonal crack line propagating from a corner – results from torsional shearing action in the mortar bed joints and perpends. The geometry of the masonry units determines the slope of the line and the tensile strength of the masonry determines the flexural strength along the crack, expressed as the diagonal bending capacity  $M_{cd}$ . This type of crack has the greatest effect of the three in determining the overall strength.

The virtual work method can be summarised as follows. A mechanism is postulated, based on the assumed crack pattern for the given wall, and this mechanism is given a unit (virtual) deflection. The incremental internal energy (work done on the hinge lines) is the sum along all crack lines of the products of moment of resistance and angle of rotation. The incremental external work done during this deflection is the sum over all panel

segments of the products of load and deflection for each segment centroid. Equating the internal energy and the external work done results in an equation that can be solved to derive the load resisted by the cracked panel at the time the mechanism is formed. This predicted load capacity depends on geometrical factors for the wall and the material properties, which are expressed as the horizontal moment capacity  $M_{ch}$  (see Equations (3) and (4)) and the diagonal moment capacity  $M_{cd}$  (see Equation (5)).

The diagonal bending moment capacity is given in AS 3700 as:

$$M_{cd} = \phi f'_t Z_t \dots\dots\dots(5)$$

Where

$f'_t$	=	The equivalent torsional strength = $2.25\sqrt{f'_{mt}}$ .
$Z_t$	=	The equivalent torsional section modulus, measured normal to the diagonal crack line, as calculated by the expressions given in AS 3700 (Clause 7.4.4.3).

Other symbols are as defined previously.

### Treatment of Openings

When a wall panel contains door and window openings these will cause variations to the crack pattern (see Figure 3). The virtual work method can still deal with these cases by considering sub-panels on each side of the opening and using the energy developed on the actual crack lines. Any influence of a frame around the opening, or any other effect of a door or window structure is ignored. However, the load applied to the door or window is taken into account as a part of the overall load on the wall. Whereas a rigorous analysis could consider the resistance along the crack lines for the expected crack pattern, the tabulated coefficients in AS 3700 are based on a simplification. For these purposes, the opening is treated as if it extended the full height of the wall. There is no account taken of contribution to the resistance



from any masonry between the edges of the opening. However, the load on the section between the edges of the opening is included for the calculation of the work done. This is equivalent to designing the panel on each side of the opening as an independent panel with a free edge having a line load applied.

Figure 4 shows a panel with an opening and indicates how it is divided into sub-panels.

**Application of the Method**

The general formula for the virtual work method is:

$$w = \frac{2a_f}{L_d^2} (k_1 M_{ch} + k_2 M_{cd}) \dots \dots \dots (6)$$

Where

w = predicted lateral load capacity of the panel under two way bending

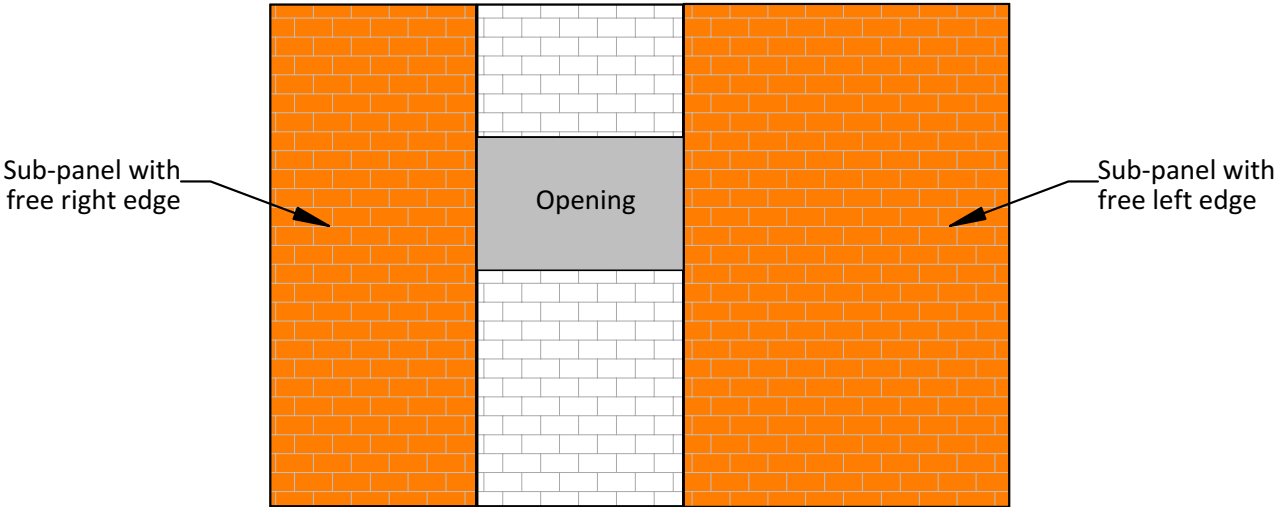
a<sub>f</sub> = aspect factor depending on the geometry  
L<sub>d</sub> = design length (see below)  
k<sub>1</sub> and k<sub>2</sub> = coefficients depending on the edge restraints and the geometry

M<sub>ch</sub> and M<sub>cd</sub> are as defined above

The design length and design height both depend on the support conditions for the wall and the presence of openings. They are found as follows:

**Design length** – When only one vertical edge of a wall is supported, the design length is the actual length of the wall. When both vertical edges are supported, the design length is half the actual length. If there is an opening in the wall, the edges of the opening are considered as if they are free edges and the design length is the distance from the edge of the wall to the edge of the opening.

**Figure 4.** Wall with Opening Divided into Two Sub-panels



**Design height** – When the top edge of a wall is not laterally supported, the design height is the actual height of the wall. When the top edge is supported, the design height is half the actual height of the wall.

Expressions for the other parameters are given in AS 3700 Clause 7.4.4. However, for design using the design charts in this manual, the designer is not required to evaluate these other factors.

**Design Charts**

The equations given are used as the basis for the design charts. Charts are given for cases with and without rotational restraint at the sides, including partial rotational restraint (factor = 0.5). In the virtual work method, it is always assumed that horizontal edges, if restrained at all, only have lateral restraint and no rotational restraint.

Each of the design charts gives limiting values of design length and design height for a range of loading and a particular set of support conditions. The following procedure is followed in using the design charts:

1. Determine the type of panel to be used (the type of masonry and its properties).
2. Determine the support conditions.
3. Calculate the face load on the panel from the wind and earthquake codes.
4. Use the appropriate chart to find an acceptable combination of design height and design length. (Note: interpolation is permitted on the charts.)

Alternatively if the panel dimensions and support conditions are known, ascertain the maximum face load from the corresponding chart.

The AS 3700 robustness requirements must also be checked (see AS 3700 Clause 4.6 and Manual 7, Design of Clay Masonry for Serviceability<sup>23</sup>).

The use of this design procedure with the aid of the design charts is illustrated with a worked example in Section 10.3. The charts are given in Sections 11.3 and 11.4.

If an opening is not centrally located within the length of a panel, each side from the opening to the edge of the panel must be checked independently.

**7.5 Veneer Walls**

In a veneer wall, the veneer itself does not require structural design but should be checked for its capacity to resist the load applied to it by spanning between the ties in flexure. For the tie spacing specified in AS 3700 this will not usually be critical.

The structural backup to a veneer wall must be designed to resist the total load applied to the wall. AS 3700 defines a flexible backup as one with stiffness less than or equal to half the stiffness of the uncracked veneer. All other backups are classed as stiff. Examples of flexible backups are steel frames and timber frames, whereas stiff backups include concrete and masonry walls. For a flexible backup and a stiff non-masonry backup, design should be in accordance with the appropriate code and is beyond the scope of this manual. When the backup is a stiff masonry wall, it should be designed in accordance with this manual (see Section 7.4). For a flexible backup, AS 3700 limits the allowable deflection under the serviceability wind load to the span divided by 300. This is intended to limit the width of any crack occurring at or about mid-height.

The primary design consideration for a veneer wall is therefore the capacity of the wall ties, and this depends on whether the backup is flexible or stiff. The strength of the wall ties is no longer covered by deemed-to-satisfy provisions of AS 3700 and must always be justified (see Section 9.2).

A typical example of design for a veneer wall on a flexible backup is shown in Section 10.4.

## 8. Design of Walls for In-Plane Load

### 7.6 Cavity Walls

In many cases, the inner leaf of a cavity wall supports the roof structure, upper floors or some other vertical load, from which it derives additional stability, whereas the outer leaf performs the function of a cladding. In these cases, the outer leaf is treated by AS 3700 as a veneer with a stiff backup and the two leaves have different design criteria.

For cases where both leaves share the load, the principal difficulty is to determine the relative distribution of load between the two leaves and the forces in the ties. AS 3700 permits a designer to assume that all loads are taken by one of the leaves and to design accordingly. Alternatively, the distribution of load can be assessed and each leaf designed individually as set out in Section 7.4.

The 2011 edition of AS 3700 introduced an approximate method of allowing for load sharing between the leaves, the basis of which originally appeared in The Australian Masonry Manual<sup>5</sup>. More precise calculations are possible, but will not often be warranted. In this approximate method, the proportion of load applied to the inner and outer leaves is not taken into account and the wall capacity is obtained by factoring down the sum of the individual leaf capacities, allowing for the stiffness of the wall ties and the amount of load they are required to transmit.

The strength of the wall ties and their ability to transmit load is no longer covered by deemed-to-satisfy provisions of AS 3700 and must always be justified (see Section 9.2).

A typical example of cavity wall design is shown in Section 10.5, including the option of distributing the load between the leaves.

### 8.1 Introduction

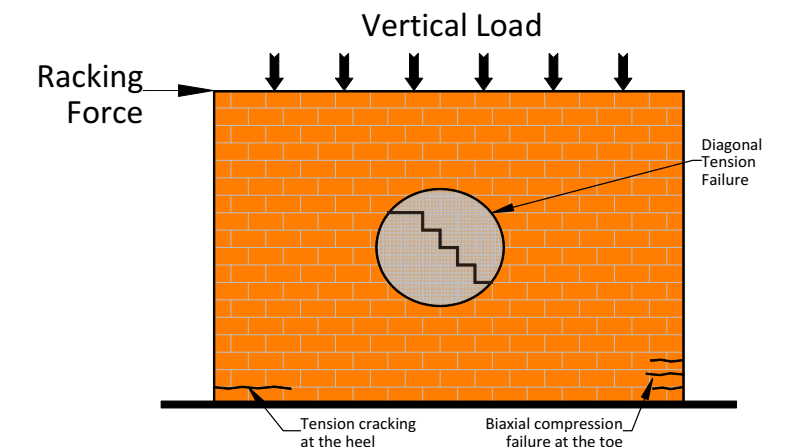
The force in a particular shear wall will depend upon its stiffness relative to the other elements resisting horizontal forces and in some cases on the flexibility of the floor diaphragms connecting the shear walls. Determination of design loads for wind and earthquake is discussed in Section 5.

The general principles of shear wall behaviour are well known. However, the stress distribution within a shear wall is complex and depends, among other things, on the geometry of the wall, the nature of the load application and the presence of openings. The strength of masonry subjected to biaxial stresses depends on the magnitude and sense of the principal stresses and the angle of inclination of these stresses to the bed and header joints. The inclination is particularly critical if tensile principal stresses are present.

As well as sliding failure, shear walls can fail locally in three ways, involving various conditions of biaxial stress. These are crushing at the toe, tensile cracking at the heel and diagonal cracking within the wall. Consideration of this local state of stress is necessary if local and progressive failures are to be predicted.

Figure 5 shows the potential regions of local cracking and failure. Toe failure will occur by crushing under biaxial compressive stress and usually causes splitting and spalling normal to the plane of the wall. Uplift at the heel occurs when vertical loads are low in relation to the racking load, resulting in the development of tensile stresses normal to the bed joint and a consequent horizontal crack. This crack might be tolerated in some circumstances, or tying might be incorporated to minimise its effect. Failure in the centre of the panel is commonly described as 'shear failure', and is typified by diagonal cracking. This failure actually occurs in the bed and header joints under a combination of principal

Figure 5. Shear Wall Behaviour



tensile and compressive stresses with subsequent sliding along the joints. The magnitude and inclination of the principal tensile stress is influenced primarily by the ratio of vertical load to horizontal racking load, with the 'shear strength' of the wall increasing significantly with an increasing level of vertical load.

Unless major openings or discontinuities are present, none of these local failures will cause collapse of the wall, although its capacity might be impaired. Walls subjected to seismic loading will progressively degrade with repeated load reversal as all or some of these failures occur in various locations depending on the direction of loading. In some cases of cyclic reversing load, a wall will rock on its base as uplift occurs alternately at each end of the wall. This may correspond to gradual shedding of bricks from the tension end or progressive local crushing in the compression region (or both simultaneously). Because of this process, and the possibility of progressive diagonal failure, unreinforced masonry shear walls have some capacity for energy absorption. For walls with major openings, significant distress and failure can also occur in the masonry piers between openings.

The presence of discontinuities in the wall, such as

# 9. Design of Wall Ties, Connectors and Joints

damp-proof courses, slip joints and interfaces with other materials such as concrete slabs, provide potential slip planes where failure can occur.

From a practical point of view, local failure may have implications for serviceability, but overall failure of the wall is the main interest. Consequently, design rules for ‘shear strength’ have been formulated from racking tests on masonry panels and the observed performance of the panels at failure. This has resulted in a simple relationship expressed in the form of a Coulomb criterion in terms of the average shear and compressive stresses in the wall.

## 8.2 Shear Wall Design

Design of shear walls in accordance with AS 3700 uses the shear capacity of the masonry, determined from a Coulomb-type equation:

$$V_d \leq \phi f'_{ms} A_d + k_v f_d A_d \dots\dots\dots(7)$$

Where

$V_d$	=	The design shear force.
$\phi$	=	Capacity reduction factor (0.6 for shear in unreinforced masonry).
$f'_{ms}$	=	Characteristic shear strength of the masonry (see Section 4.3).
$A_d$	=	Design cross-sectional area.
$k_v$	=	Shear factor (0.3 for bed joints in clay masonry).
$f_d$	=	Minimum design compressive stress on the bed joint (not greater than 2 MPa).

The first term represents the shear bond strength of the section and the second term is the shear friction strength.

The superimposed compressive stress  $f_d$  is calculated differently for earthquake and other loadings. In the case of other than earthquake loading, it represents the non-removable dead load and is usually 0.9 times the total dead load. In the case of earthquake loading, it can include a portion of the live load (see AS 1170.4). The friction component is further reduced by ten per cent to allow for upward acceleration that might be present in an earthquake. Where the vertical gravity load contributes to resistance but not the induced lateral earthquake load, the live load is ignored and 0.9 times the dead load is used.

As well as checking for shear failure on the bed joints using the above expressions, the designer should check for the possibility of local compressive failure at the toe of the wall. Crushing under compressive stresses can be checked using the appropriate parts of AS 3700, using the compressive strength of the material. If cracking occurs at the heel, this loss of section should be taken into account in calculating the compressive capacity at the toe.

At bedding planes containing damp-proof course membranes and at junctions with other materials such as a concrete slab, the characteristic shear strength is usually zero, although AS 3700 does include provision for obtaining values by test. When the strength is zero, the resistance is provided solely by friction. Values for the shear factor  $k_v$  are provided in AS 3700 for various membranes and interfaces and apply to both wind loading situations and reversing earthquake load situations. Shear capacity on these planes must be checked (see Section 9.4).

A typical example of shear wall design is shown in Section 10.7.

## 9.1 Introduction

The bulk of the design provisions in AS 3700 relate to overall structural behaviour. However, most failures are due to inadequate detailing and design of joints, wall ties and connections. The importance of correct attention to detailing for masonry structures cannot be over-emphasised.

Since masonry is a brittle material with limited tensile strength, it must be supported by suitable tying systems to keep the flexural stresses within acceptable limits. Wall ties are used to connect non-loadbearing veneer walls to a structural backup and to allow the leaves of cavity walls to share in resisting the applied loads. Various types of connectors are used to attach masonry walls to structural frames and across joints.

AS 3700 requires all connections and wall anchorages to be capable of transmitting a horizontal force of 1.25 times the induced earthquake force. In particular, roofs must be positively attached with a suitable system and many of the traditional wind hold-down details (such as strapping connections) are inadequate for this purpose.

The following sections cover design of wall ties and connectors and the design to resist seismic forces of slip joints and joints containing membranes.

## 9.2 Wall Tie Design

AS 3700 contains rational provisions for the design of wall ties and AS 4773.1 gives deemed-to-satisfy duty ratings for various wind categories applicable to veneer and cavity walls. The overall maximum spacing is 600 mm horizontally and vertically and the first row of ties is required to be within 300 mm from any edge. These requirements are primarily to ensure a satisfactory distribution of ties in the wall. They are set out in AS 3700 Clause 4.10.

As well as satisfying the overall limits on spacing, the ties must be designed to resist the applied forces. A full analysis to determine the tie forces, which considers the properties of the masonry (before and after cracking), the

structural backup and the ties themselves, is complex. AS 3700 deals with the fundamental difference in behaviour between veneer and cavity walls by providing deemed-to-satisfy design forces. For veneer walls with flexible backup, the design force for each tie is 20% of the total tributary load on a vertical line of ties. For veneer walls with stiff backup, the design force for each tie is 1.3 times the tributary load on the tie. For cavity walls, the design force is equal to the tributary load on the tie. However, if a cavity wall has only the inner leaf supported, it must be designed as a veneer wall with stiff backup, and is therefore subject to the more stringent requirements for tie forces.

In a departure from previous practice, both AS 3700 and AS 4773.1 now base the strength design of ties on the mean strength of the ties instead of the 95% characteristic strength. This is to compensate for the considerable redistribution of force that occurs in the ties supporting a masonry leaf before failure can be considered to have occurred.

An additional requirement, in consideration of the particular behaviour of veneer walls on flexible backup, is that the top of a wall should have double the number of ties required elsewhere in the wall. To satisfy this requirement, it might be necessary to spread the top row of ties across two adjacent bed joints, within 300 mm of the top of the wall. This requirement for additional ties also applies immediately above and below the plane of a horizontal floor support when the veneer is continuous past the support.

The number of ties should also be double in line with the intersection of an internal wall support, both for cavity and veneer walls. This is because the additional support stiffness will attract more of the lateral force than would otherwise be carried by the ties.

Table 1 shows the maximum wall pressures, calculated using the AS 3700 design forces, for Type A ties spaced at 600 mm in both directions. For a veneer wall with a flexible backup, the maximum pressure depends on the wall height and values are only shown for a 2.4 m high wall. Strength design of the masonry might result in lower wall capacities than those shown in the table.



**Table 1.** Maximum Wall Pressures (kPa) for Type A Ties at 600 mm Centres.

Construction	Tension			Compression		
	Light Duty	Med. Duty	Heavy Duty	Light Duty	Med. Duty	Heavy Duty
Veneer with Flexible Backup (2.4 m wall height)	0.99	1.98	4.95	1.19	2.38	5.94
Veneer with Stiff Backup	0.61	1.22	3.04	0.73	1.46	3.65
Cavity Wall (both leaves supported)	0.79	1.58	3.96	0.95	1.90	4.75

Note that for a flexible structural backing, the serviceability deflection of the backing must also be checked, using the appropriate material standard.

In all situations, it is essential that ties be properly installed. They will not perform to their full rated strength unless they are properly embedded in the mortar joints and properly attached to the frame or backup (for veneer walls). They must also be installed in the correct orientation and without a backward slope, so that they shed water properly to the outside of the wall. For face-fixed ties in masonry veneer more than 3 m above ground, AS 3700 requires screw-fixing. Typical examples of wall tie design are shown in Sections 10.4 and 10.5.

9.3 Design of Connectors

Connectors used to tie masonry walls to frames and other supporting elements must be designed to resist 125% of any calculated load normal to the plane of the wall. The minimum level of forces is specified by AS 3700 in Clauses 2.6.3 and 2.6.4. When the forces arise from earthquake action, the calculated horizontal force must similarly be increased by a factor of 1.25 (see AS 3700 Clause 10.2.5). This applies to non-structural as well as structural components. Characteristic strength values for these connectors should be provided by the manufacturer, after determination in accordance with AS 2699.2. Connectors must also comply with the durability requirements (see Section 9.5). Special connectors are available

for control joints, where they limit movement in the direction normal to the plane of the wall while allowing movement in the plane of the wall caused by shrinkage or expansion.

Where monolithic structural action is required across a vertical interface between two leaves of a solid masonry wall, or between a masonry wall and a supporting member, it must be designed in accordance with AS 3700 Clause 4.11. If ties are used between the leaves in solid masonry construction they must be rated at least medium duty, spaced at no more than 400 mm in each direction. For other interfaces, the spacing must not exceed 200 mm horizontally and 300 mm average (400 mm maximum) vertically, unless substantiated by calculation or test. The vertical joint must be filled with mortar. Other connectors of equivalent strength (characteristic tensile capacity of 0.4 kN) can be used within the same spacing limits.

A special case occurs for diaphragm walls and walls of geometric section, where shear forces are often of much larger magnitude. For these cases, connectors across a vertical interface in masonry are designed in accordance with AS 3700 Clause 7.5.6. This clause provides equations for steel connectors of rectangular and circular cross-sections, based on the dimensions and the yield stress of the steel. These properties should be provided by the manufacturer. For the shear strength of connectors between masonry and other structural members, the manufacturer should provide characteristic strengths determined by test.

9.4 Seismic Design of Slip Joints and Joints Containing Membranes

Slip joints are commonly placed between the edges of concrete slabs and the tops of the masonry walls on which they sit, to permit differential movement from sources such as shrinkage. These joints usually consist of two sheets of galvanised steel with a layer of grease between, or a double layer of damp-proof course material. In conditions of earthquake loading, these joints are likely to be required to transmit shear forces from the floor slab to the wall and their shear capacity under transient loading is therefore important.

Damp-proof courses are used at the bases of walls to prevent moisture rising. They are usually formed by embedding a layer of embossed polythene or light-gauge aluminium sheet with bitumen or polythene coating in the mortar joint. Recommended good practice is to place the damp-proof membrane within the mortar joint, rather than sitting it on the masonry units and placing the mortar on top. However, this is often not done and there is some evidence that the best performance under serviceability conditions comes from the damp-proof course material being placed directly on the masonry units. These joints are also usually required to transmit shear forces under earthquake loading and therefore must have some shear resistance.

AS 3700 Section 10.3 provides various details that are deemed to be adequate for connecting masonry to supporting elements under earthquake loading. Where no positive connectors are used the capacity of the joint to transmit the necessary forces by friction must be checked. Where positive connections are used their capacity must be checked using the relevant provisions (see Section 9.3). Typical roof connection details are shown in Manual 9 Detailing of Clay Masonry<sup>24</sup>.

The serviceability requirements for damp-proof course membranes and slip joints to prevent distress in the masonry seem to conflict with the requirement for shear transfer under earthquake load conditions. It is thus an important matter of design to ensure that the joints have sufficient freedom to accommodate the serviceability requirements while having sufficient friction capacity to

provide the shear transfer.

AS 3700 uses the Coulomb-type shear equation (see Section 8.2) with a shear factor  $k_v$ . For the joints being considered here, the shear bond strength  $f'_{ms}$  is zero and the equation for earthquake loading reduces to the simple friction equation–

$$V_d \leq k_v f_d A_d \dots\dots\dots(8)$$

Where

- $k_v$  = Shear factor
- $f_d$  = The minimum design compressive stress (see Section 8.2)
- $A_d$  = Design cross-sectional area

This equation applies to both in-plane and out-of-plane shear. Values of  $k_v$  have been investigated experimentally for membrane-type damp-proof courses under static loading<sup>25</sup> and dynamic loading<sup>26</sup>. Design values are given in AS 3700 Table 3.3 as 0.3 for bitumen-coated or embossed polyethylene and 0.15 for polyethylene-coated and bitumen-coated aluminium. For properly designed slip joints using a greased sandwich of metal sheets, the appropriate value of shear factor is zero, meaning that these joints cannot be relied upon to transmit shear forces under earthquake loading.

9.5 Durability of Ties and Connectors

As pointed out above, ties and connectors are structural components and their integrity must be maintained for the life of the structure. When ties and connectors corrode they are usually hidden in the wall and the damage does not become evident until it is advanced. The consequences then are severe. The cost of replacing corroded ties and connectors is vastly greater than the incremental cost of providing enhanced protection when a structure is first built. Designers should consider this carefully.

# 10. Worked Examples

All ties and connectors used with masonry are required to meet durability requirements set out in AS 3700 Section 5 or AS 4773.1 Section 4 (for small buildings). These requirements are expressed in terms of a classification of exposure environments, ranging from mild to severe marine, and the location within the building, comprising interior, exterior and exterior-coated. The last of these covers components that are in masonry exposed to the exterior environment but which relies for protection on a compliant weather-resistance coating.

The accessories are given a rating from R0 to R5, depending on their materials and other characteristics, and each combination of exposure environment and

location requires an appropriate durability rating. The performance requirements and deemed-to-satisfy provisions for durability of ties and connectors are given in AS 2699.1 and AS 2699.2 respectively.

There are many reports of ties deteriorating markedly after only a short time in the structure, the most notable example being after the Newcastle earthquake<sup>27</sup>. In recent years, stainless steel ties have become more common and polymer ties have been developed. The availability of an improved range of products, combined with an increased awareness by designers of the importance of tie durability, should lead to fewer problems in the future.

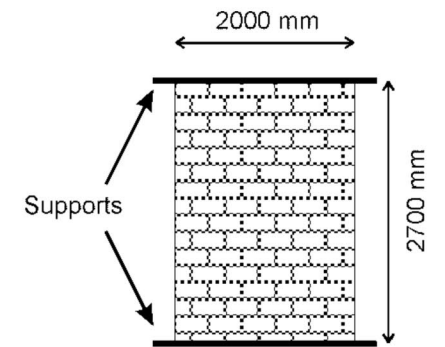
## 10.1 One-Way Vertical Bending

Design a section of wall 2 m wide, spanning between top and bottom supports at 2.7 m spacing, carrying no vertical load and an applied lateral load of 0.5 kPa.

**Procedure using AS 3700 (Clause 7.4.2) –**

Bending moment:

$$M_{dv} = \frac{wL^2}{8} = \frac{0.5 \times 2.7^2}{8} = 0.46 \text{ kN.m per metre width}$$



For a single leaf of 110 mm units, fully bedded –

$$Z_d = \frac{bt^2}{6} = \frac{1000 \times 100^2}{6} = 2.02 \times 10^6 \text{ mm}^3 \text{ per metre width}$$

Density for clay masonry = 0.19 kN/m<sup>2</sup> per 10 mm thickness (AS 1170.1).

Therefore, compressive stress at mid-height due to self-weight (based on 90% dead load) –

$$f_d = \frac{0.9 \times [(0.19 \times 11) \times 1.35] \times 1000}{110 \times 1000} = 0.023 \text{ MPa}$$

Vertical bending capacity –

$$\begin{aligned} M_{cv} &= \phi f'_{mt} Z_d + f_d Z_d \\ &= [0.6 \times 0.2 \times 2.02 \times 10^6 + 0.023 \times 2.02 \times 10^6] \times 10^{-6} \\ &= 0.29 \text{ kN.m per metre width} \end{aligned}$$

For two leaves of 110 mm masonry (assumed to be connected by heavy duty ties and therefore to share the load equally) –

Moment capacity = 0.58 kN.m per metre width, which is greater than the applied bending moment of 0.46 kN.m per metre.  $\therefore$  OK

**Procedure using the design charts (Chart 11.1) –**

Assume a leaf thickness of 110 mm.

For a height of 2.7 m, the load capacity of a single leaf is 0.28 kPa, so for two leaves the capacity is 0.56 kPa.  $\therefore$  OK

## 10.2 One-Way Horizontal Bending

Design a section of wall spanning horizontally 2.4 m between supports, for an applied lateral load of 0.7 kPa.

### Procedure using AS 3700 (Clause 7.4.3) –

Bending moment –

$$M_{dh} = \frac{wL^2}{8} = \frac{0.7 \times 2.4^2}{8} = 0.50 \text{ kN.m per metre width}$$

For a single leaf of 230 mm x 110 mm x 76 mm units, fully bedded –

$Z_d = 2.02 \times 10^6 \text{ mm}^3$  per metre width (see Example 10.1),  
and  $Z_u$ ,  $Z_p$  and equal to  $Z_d$  for full perpend and no joint raking.

Perpend spacing factor  $k_p$  is the lesser of (see AS 3700 Clause 7.4.3.4) –

$$\frac{s_p}{t_u} = \frac{110}{110} \text{ and } \frac{s_p}{h_u} = \frac{110}{76} \text{ and } 1.0$$

Ignoring the self-weight effect (which is negligible for small sections):

Horizontal bending capacity –

$$\begin{aligned} M_{ch} &= 2.0\phi k_p \sqrt{f'_{mt}} Z_d \\ &= (2.0 \times 0.6 \times 1.0 \times \sqrt{0.2} \times 2.02 \times 10^6) \times 10^{-6} \\ &= 1.08 \text{ kN.m per metre width (Equation 3)} \end{aligned}$$

OR

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

Taking  $f'_{ut}$  as the default value of 0.8MPa in the absence of test data –

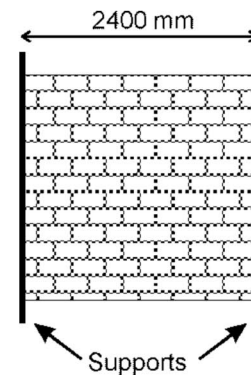
$$\begin{aligned} M_{ch} &= 0.6 \times (0.44 \times 0.8 \times 2.02 \times 10^6 + 0.56 \times 0.2 \times 2.02 \times 10^6) \times 10^{-6} \\ &= 0.56 \text{ kN.m per metre width (Equation 4)} \end{aligned}$$

Therefore, the governing value = 0.56 kN.m per metre width, which is greater than the applied bending moment of 0.50 kN.m per metre.  $\therefore$  OK

### Procedure using the design charts (Chart 11.2) –

Assume a 110 mm wall.

For a length of 2.4 m, the load capacity is 0.78 kPa, which is greater than the applied load of 0.7 kPa.  $\therefore$  OK



## 10.3 Two-way Bending (Single-leaf Wall)

Design a wall panel 4 m long and 3 m high with simple supports on all sides, for an applied lateral load of 1.0 kPa.

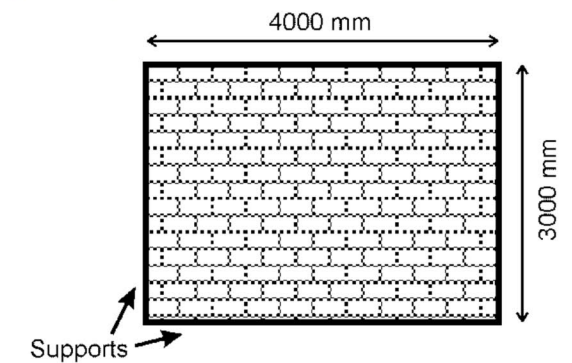
For the virtual work method –

Design length

$L_d = 2000 \text{ mm}$  (half the actual length).

Design height

$H_d = 1500 \text{ mm}$  (half the actual height).



### Procedure using AS 3700 (Clause 7.4.4) –

For 230 mm x 110 mm x 76 mm solid or cored units –

$$\text{Crack Slope } G = \frac{2(h_u + t_j)}{l_u + t_j} = \frac{2 \times 86}{240} = 0.717$$

$$\text{Slope Factor } \alpha = \frac{GL_d}{H_d} = \frac{0.717 \times 2000}{1500} = 0.96$$

Since there is no opening and both vertical edges are supported, use the first row in AS 3700 Table 7.5 –

$$\text{Aspect Ratio } a_f = \frac{1}{1 - \alpha/3} = \frac{1}{1 - 0.96/3} = 1.47$$

Sides are simply supported, so restraint factors  $R_{f1}$  and  $R_{f2}$  are both 0, therefore –

$$k_1 = 1 - \alpha = 1 - 0.96 = 0.04$$

$$k_2 = \alpha \left( 1 + \frac{1}{G^2} \right) = 0.96 \left( 1 + \frac{1}{0.717^2} \right) = 2.83$$

$$M_{ch} = 0.56 \text{ kN.m per metre width (see Example 10.2)}$$

Equivalent torsional strength (ignoring the small effect of self-weight) –

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

Height factor –

$$B = \frac{h_u + t_j}{\sqrt{1 + G^2}} = \frac{86}{\sqrt{1 + 0.717^2}} = 69.9 \text{ mm}$$

Since  $t_u = 110 \text{ mm} > B$ , the equivalent torsional section modulus –

$$Z_t = \frac{2B^2 t_u^2}{(3t_u + 1.8B)} \bigg/ \left( (l_u + t_j) \sqrt{1 + G^2} \right)$$

$$= \frac{2 \times 69.9^2 \times 110^2}{(3 \times 110 + 1.8 \times 69.9)} \bigg/ \left( (230 + 10) \sqrt{1 + 0.717^2} \right)$$

$$= 878 \text{ mm}^3 \text{ per mm crack length}$$

Diagonal moment capacity –

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

$$= (0.6 \times 1.01 \times 878 \times 1000) \times 10^{-6}$$

$$= 0.53 \text{ kN.m per metre crack length}$$

The wall load capacity is therefore –

$$w = \frac{2a_f}{l_d^2} (k_1 M_{ch} + k_2 M_{cd})$$

$$= \frac{2 \times 1.47}{2.0^2} (0.04 \times 0.56 + 2.82 \times 0.53)$$

$$= 1.11 \text{ kPa}$$

Which is greater than the applied load of 1.0 kPa.  $\therefore$  OK

**Procedure using the design charts –**

Assume a 110 mm wall, therefore use Chart 11.3.1 for four edges supported.

For a design length of 2,000 mm and design height of 1,500 mm, the load capacity is between the curves for 1.0 kPa and 1.5 kPa. Interpolate a value of 1.1 kPa.  $\therefore$  OK

## 10.4 Veneer Wall

Design a single-storey masonry veneer wall 2.7 m high supported on a timber frame. The applied lateral load is 1.5 kPa suction on the wall. Assume ties at 600 mm centres in both directions.

Using AS 3700 Section 7.6, design tie force based on 20% of the load on the tributary area for a line of ties –

$$F_{td} = 0.20 \times 2.7 \times 0.6 \times 1.5 = 0.49 \text{ kN tension}$$

Capacity reduction factor = 0.95 (AS 3700 Table 4.1).

Load capacity for medium duty ties in tension = 0.60 kN (AS 3700 Table 3.5).

$$\therefore \text{ Tie capacity} = 0.95 \times 0.6 = 0.57 \text{ kN} \therefore \underline{\text{OK}}$$

The top row of ties must be within 300 mm of the top of the veneer (AS 3700 Clause 4.10) and spaced at an average of 300 mm horizontally. This can be achieved by placing two ties at each stud (600 mm centres), either in the same bed joint or spread across two adjacent bed joints within 300 mm of the edge. Ties in the remainder of the wall are spaced at 600 mm in each direction.

Provided the tie spacing complies with AS 3700, the veneer skin itself will require no further design.



## 10.5 Cavity Wall 1 (No Load Sharing)

Design a cavity wall 3 m long and 3 m high for a multistorey building. The inner leaf is tied to a structural frame on all four edges and the outer leaf is supported on shelf angles attached to the spandrels. The applied load is 0.5 kPa internal suction and 1.0 kPa external pressure.

Only the inner leaf is supported, so design as veneer on a stiff backup (AS 3700 Clause 7.7.4).

The force to be transmitted by the ties derives from the outer leaf and the design tie force is based on 130% of the load on a tributary area for one tie (AS 3700 Clause 7.6.3).

Assuming ties are spaced at 600 mm centres in both directions –

$$F_{td} = 1.3 \times 0.6 \times 0.6 \times 1.0 = 0.47 \text{ kN compression}$$

Capacity reduction factor = 0.95 (AS 3700 Table 4.1).

Load capacity for medium duty ties in compression = 0.72 kN (AS 3700 Table 3.6).

Therefore the tie capacity =  $0.95 \times 0.72 = 0.68$  kN, which is greater than the applied force of 0.47 kN.  $\therefore$  OK

Ties throughout the wall are spaced at 600 mm in both directions, with the first row located within 300 mm of all edges, supports and around openings (AS 3700 Clause 4.10).

For the design of the wall itself, the simplest approach is to assume that the entire load is taken by the inner leaf. A leaf of 110 mm thickness, with both sides supported, a design length of 1500 mm and a design height of 1500 mm has a load capacity exceeding 1.5 kPa (Chart 11.3.1).  $\therefore$  OK

A more refined design could be carried out by assessing the load distribution between the two leaves but this approach is unnecessary in this case. For an example of load sharing between the leaves see the next example.

## 10.6 Cavity Wall 2 (With Load Sharing)

Design a cavity wall 3 m long and 3 m high for a multistorey building. The inner leaf is tied to structural columns on both sides, with the top free. The outer leaf is supported on shelf angles attached to the spandrels and is continuous past the columns at 3 m centres. The applied load is 0.5 kPa internal suction and 1.0 kPa external pressure.

The force to be transmitted by the ties is based on the difference between external pressure and internal suction i.e. 0.5 kPa. The force per tie is derived from the tributary area (AS 3700 Clause 7.7.4).

Assuming ties are spaced at 600 mm centres in both directions –

$$F_{td} = 0.6 \times 0.6 \times 0.5 = 0.18 \text{ kN compression}$$

Capacity reduction factor = 0.95 (AS 3700 Table 4.1).

Load capacity for medium duty ties in compression = 0.72 kN (AS 3700 Table 3.6).

Therefore the design tie capacity =  $0.95 \times 0.72 = 0.68$  kN, which is greater than the applied force of 0.18 kN.  $\therefore$  OK

Ties throughout the wall are spaced at 600 mm in both directions, with the first row located within 300 mm of all edges, supports and around openings (AS 3700 Clause 4.10). The ties at the vertical lateral supports (the columns) are required to resist  $2 \times F_{td} = 0.36$  kN compression (AS 3700 Clause 7.7.4).  $\therefore$  OK

For the design of the wall itself, the load is to be assessed as being shared between the two leaves (AS 3700 Clause 7.7.3). A leaf of 110 mm thickness, with both sides supported and the top free has a design length of 1500 mm and a design height of 3000 mm. The load capacity is interpolated as 0.95 kPa (Chart 11.3.1). In this case the two leaves are of equal thickness and equal load capacity, therefore the total load capacity is (AS 3700 Equation 7.7.3):

$$w = 0.9[0.95 + 0.95] = 1.7 \text{ kPa}$$

This is greater than the total applied load of 1.5 kPa.  $\therefore$  OK

Trial a design with an outer leaf thickness 90 mm and an inner leaf thickness 110 mm, using medium duty ties as above.

The 90 mm thick leaf has a load capacity interpolated as 0.75 kPa (Chart 11.3.2). Therefore the combined load capacity is (AS 3700 Equation 7.7.3):

$$w = 0.9[0.95 + \frac{90}{110} 0.75] = 1.4 \text{ kPa}$$

This is less than the total applied load of  $w_d = 1.5$  kPa, therefore the outer leaf thickness of 110 mm is required.

## 10.7 Shear Wall

Design a shear wall 4 m long and 2.7 m high, subject to a wind shear force of 40 kN. The masonry is constructed with clay units of 110 mm thickness and with an  $f'_{mt}$  of 0.2 MPa. The wall sits on a bitumen-coated aluminium damp-proof course and there is a uniform vertical load on top of the wall comprising 50 kN/metre dead load and 15 kN/metre live load.

Shear strength –

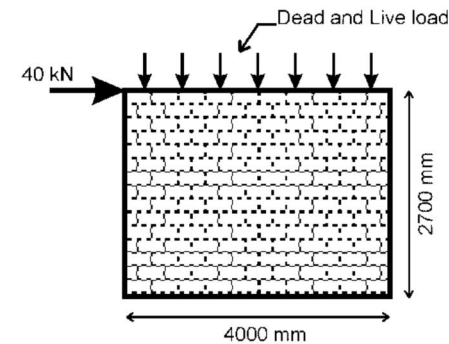
$$f'_{ms} = 1.25f'_{mt} = 0.25 \text{ MPa (AS 3700 Clause 3.3.4)}$$

Design cross-sectional area –

$$A_d = 110 \times 4000 = 0.44 \times 10^6 \text{ mm}^2$$

Therefore, shear bond capacity –

$$\begin{aligned} V_o &= \phi f'_{ms} A_d \\ &= (0.6 \times 0.25 \times 0.44 \times 10^6) \times 10^{-3} \\ &= 66.0 \text{ kN} \end{aligned}$$



Shear factor at the mortar joints and at the damp-proof course  $k_v = 0.3$  (AS 3700 Table 3.3)

Vertical stress from imposed load (using 90% of dead load) –

$$f_d = 0.9 \times \left( \frac{50 \times 10^3}{110 \times 1000} \right) = 0.41 \text{ MPa}$$

Therefore, shear friction capacity –

$$\begin{aligned} V_1 &= k_v f_d A_d \\ &= (0.3 \times 0.41 \times 0.44 \times 10^6) \times 10^{-3} \\ &= 54.1 \text{ kN} \end{aligned}$$

Total shear capacity  $V_d = V_o + V_1 = 66.0 + 54.1 = 120.1 \text{ kN}$ , which is greater than the applied shear force of 40 kN.  $\therefore$  OK

Sliding at the base of wall is OK because the shear friction capacity alone exceeds the applied shear force, the shear factor is the same (0.3) and additional frictional resistance is present at the base from self-weight.

**Check for crushing at the toe –**

$$\text{Density} = 0.19 \text{ kN/m}^2 \text{ per 10 mm thickness (AS 1170.1).}$$

Stress due to compression (using 1.2 times dead load, including self-weight, plus 40% live load) –

$$\begin{aligned} &= 1.2 \times \left( \frac{50 \times 10^3 + 2.7 \times 0.19 \times 11 \times 10^3}{110 \times 1000} \right) + 0.4 \times \left( \frac{15 \times 10^3}{110 \times 1000} \right) \\ &= 0.66 \text{ MPa} \end{aligned}$$

Overturning moment due to wind =  $40 \times 2.7 = 108 \text{ kN.m}$

$$\text{Wall section modulus} = \frac{b \times d^2}{6} = \frac{110 \times 4000^2}{6} = 293 \times 10^6 \text{ mm}^3$$

$$\text{Therefore, bending stress} = \frac{108 \times 10^6}{293 \times 10^6} = 0.37 \text{ MPa}$$

Net stress at the toe from compression + bending =  $0.66 + 0.37 = 1.03 \text{ MPa}$  compression

With a capacity reduction factor of 0.75 (AS 3700 Table 4.1) this requires  $f'_{mb}$  of 1.4 MPa, which is satisfied by units of strength 5 MPa and M2 mortar (see AS 3700 Table 3.1).  $\therefore$  OK

**Check for tension at the heel –**

Stress due to compression (using 90% dead load, including self-weight) –

$$= 0.9 \times \left( \frac{50 \times 10^3}{110 \times 1000} + 2.7 \times 0.019 \right) = 0.51 \text{ MPa}$$

Net stress at the heel from compression – bending compression (i.e. no net tension).  $\therefore$  OK

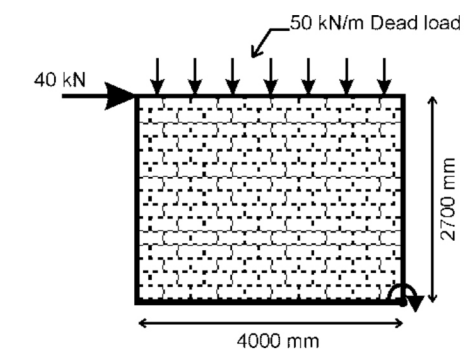
**Check for overturning about the toe –**

Overturning moment = 108 kN.m (see above).

Force resisting overturning (using 90% dead load, including self-weight) –

$$= 0.9 \times (50 \times 0.4 + 4.0 \times 2.7 \times 11 \times 0.19) = 200$$

Therefore resisting moment =  $200 \times 2.0 = 400 \text{ kN.m}$  and this is greater than the overturning moment.  $\therefore$  OK

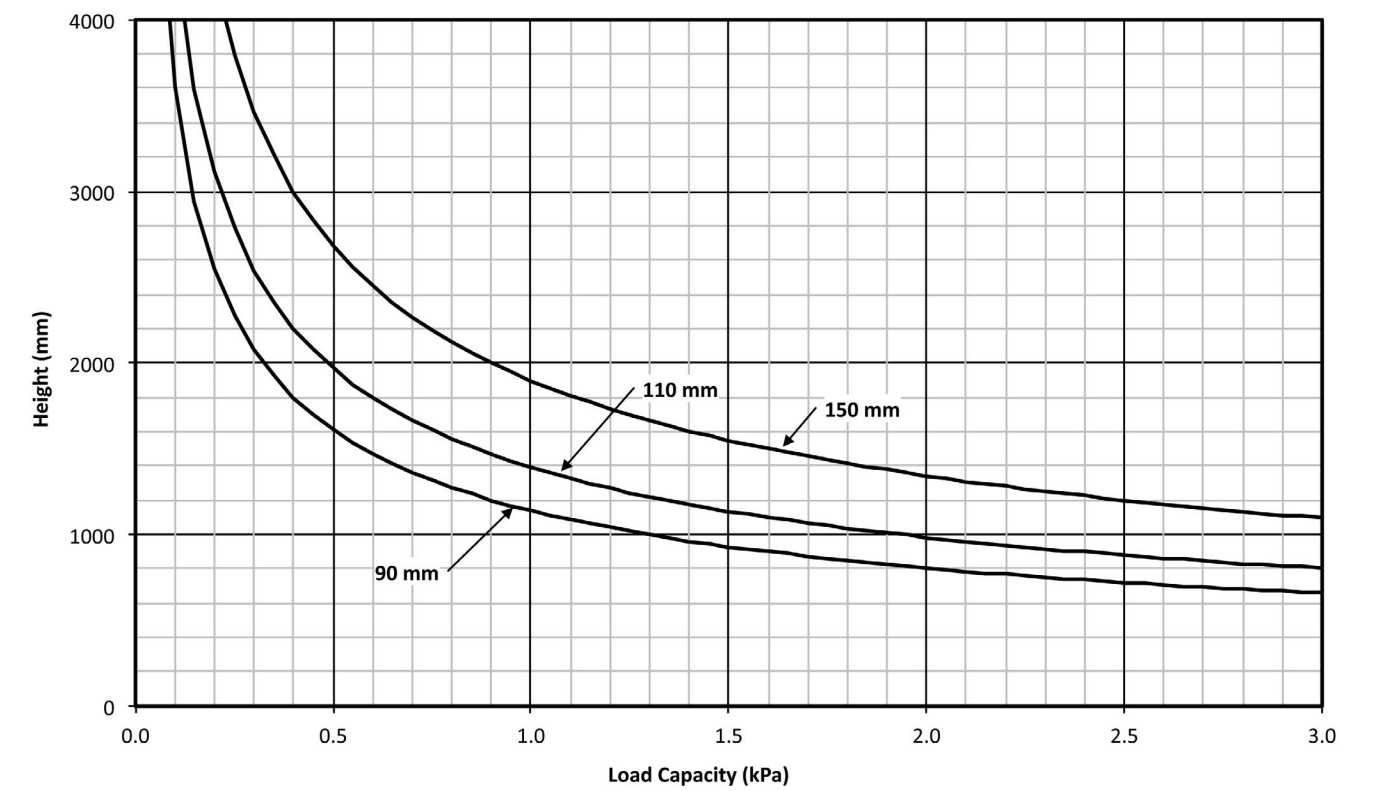


# 11. Design Charts

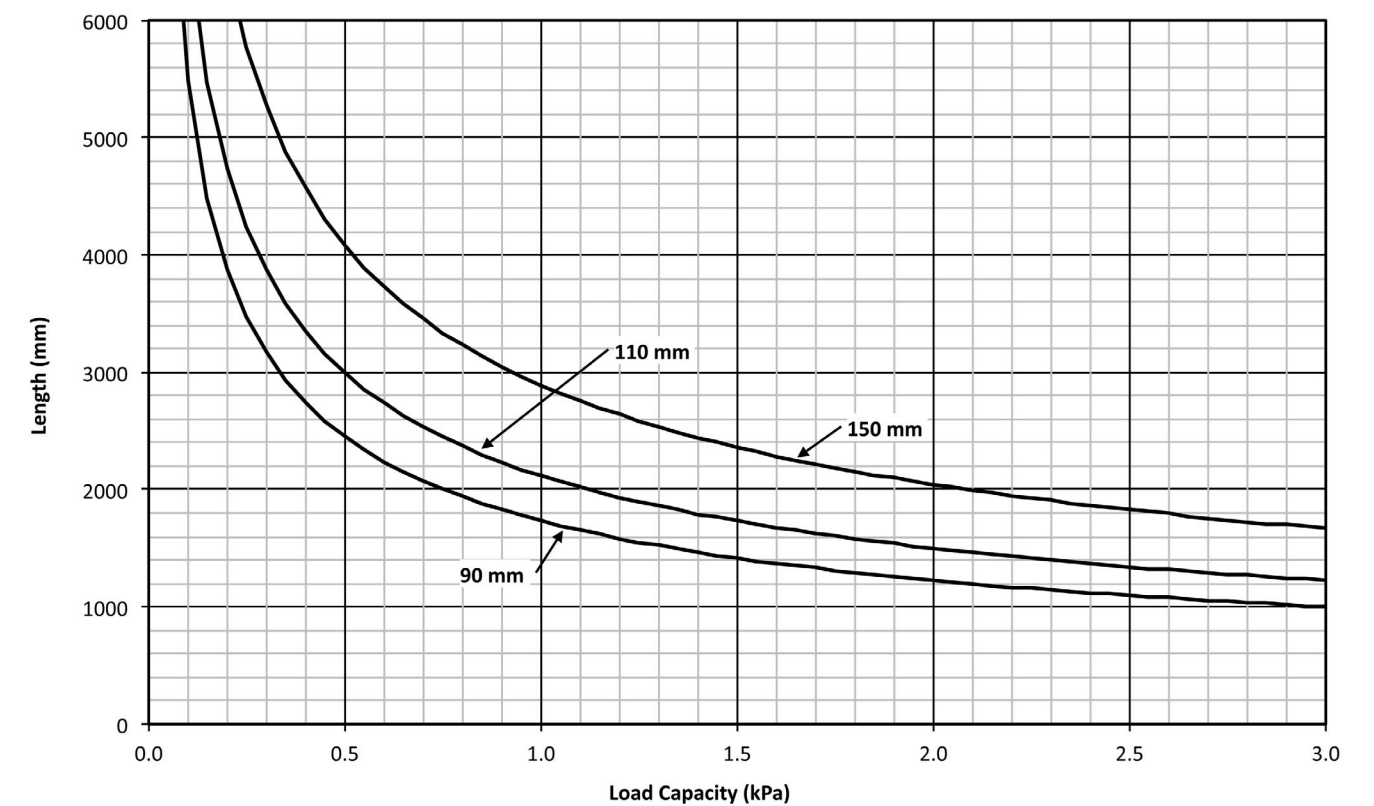
Limitations for all charts:

- They apply to single-leaf walls without engaged piers, using solid or cored masonry units.
- No superimposed compression is taken into account.
- No joint raking is assumed.
- $f'_{mt}$  is assumed to be 0.2 MPa (AS 3700 default).
- $f'_{ut}$  is assumed to be 0.8 MPa (AS 3700 default).
- Robustness should be checked separately.
- All supports are simple (without rotational restraint) unless otherwise indicated.
- The dimensions of masonry units have been taken as 230L x 76H for 110 mm units and 290L x 76H for 90 mm units. This is conservative for other common sizes of solid or cored units in horizontal and two-way bending.

## 11.1 One-Way Vertical Bending



## 11.2 One-Way Horizontal Bending

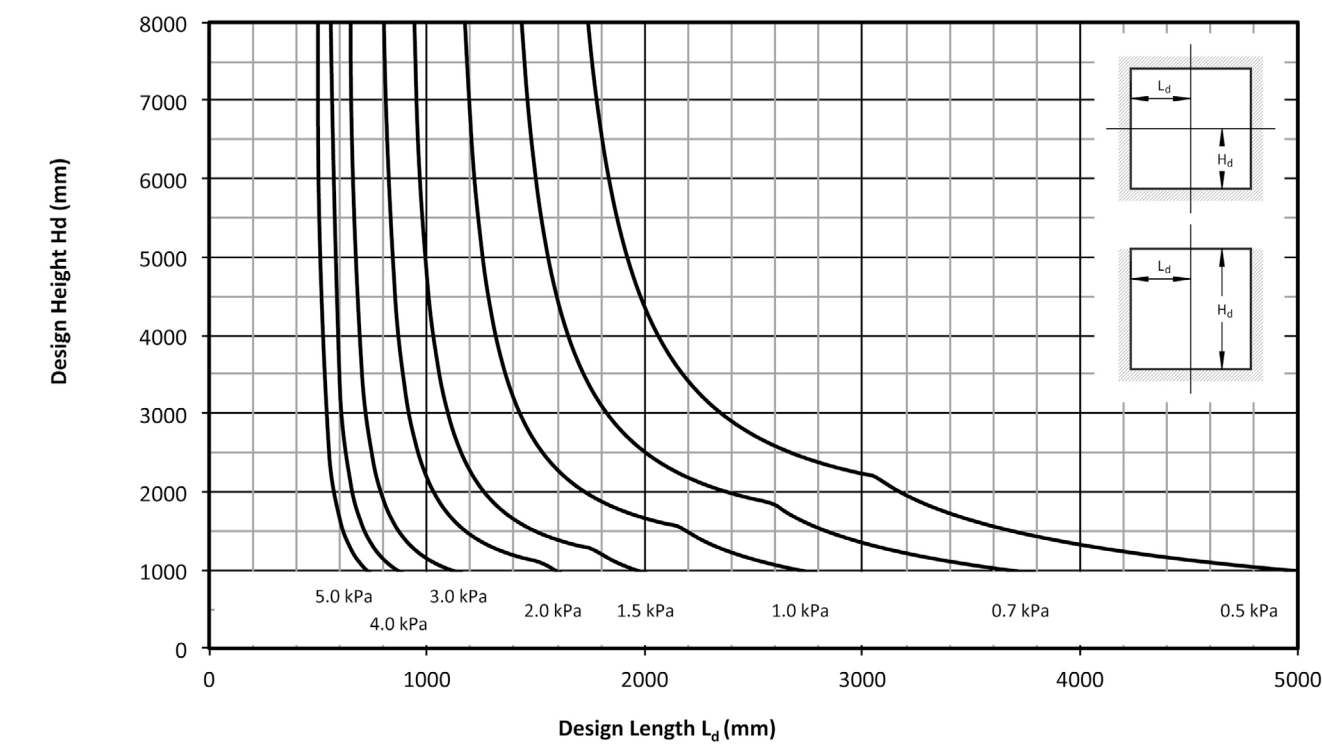




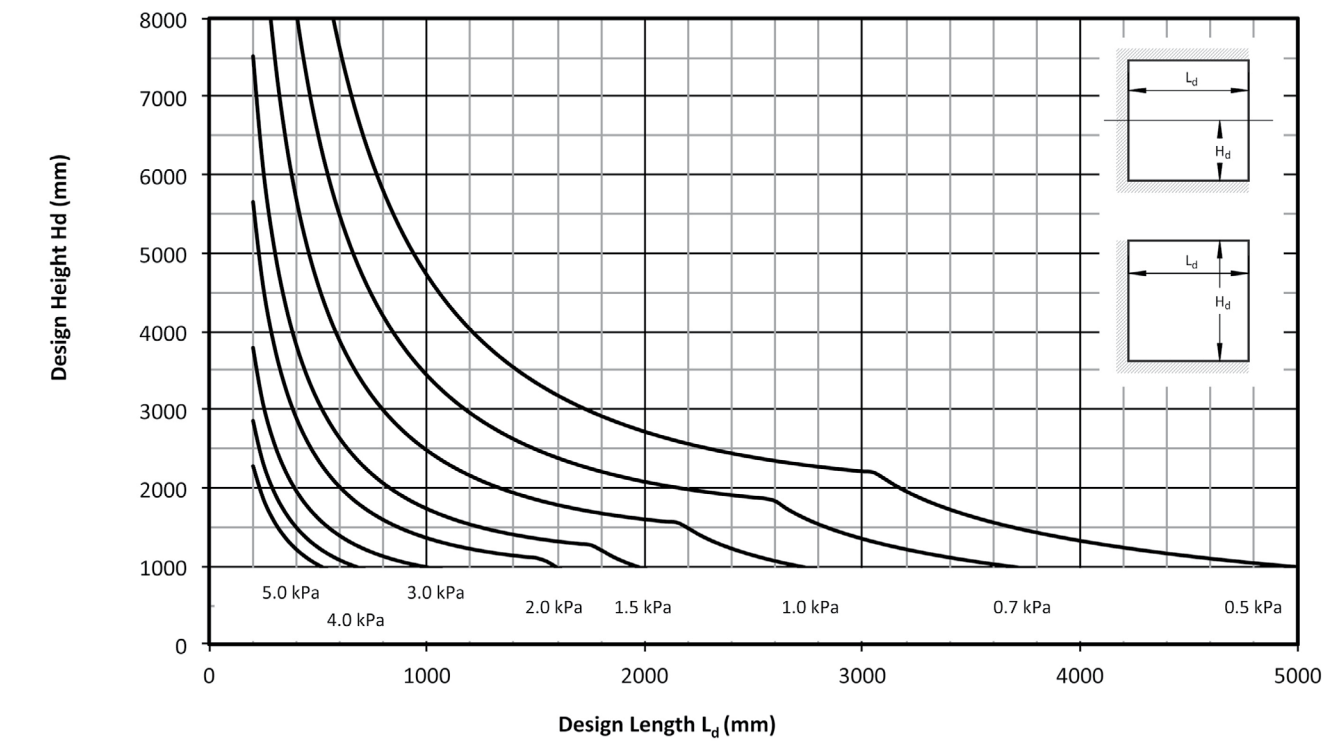
11.3 Two-way Bending Without Openings

11.3.1 110 mm without openings (no rotational restraint at the sides)

Both Sides Supported

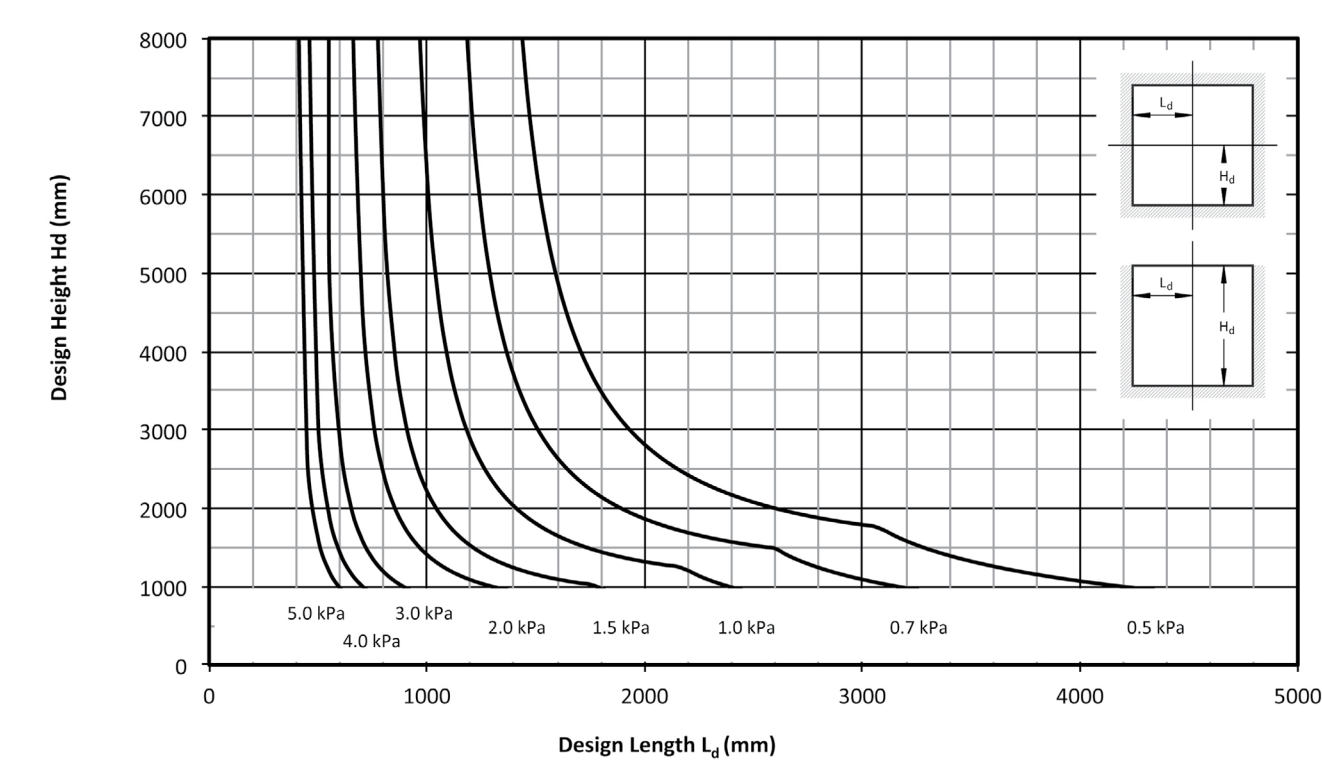


One Side Supported

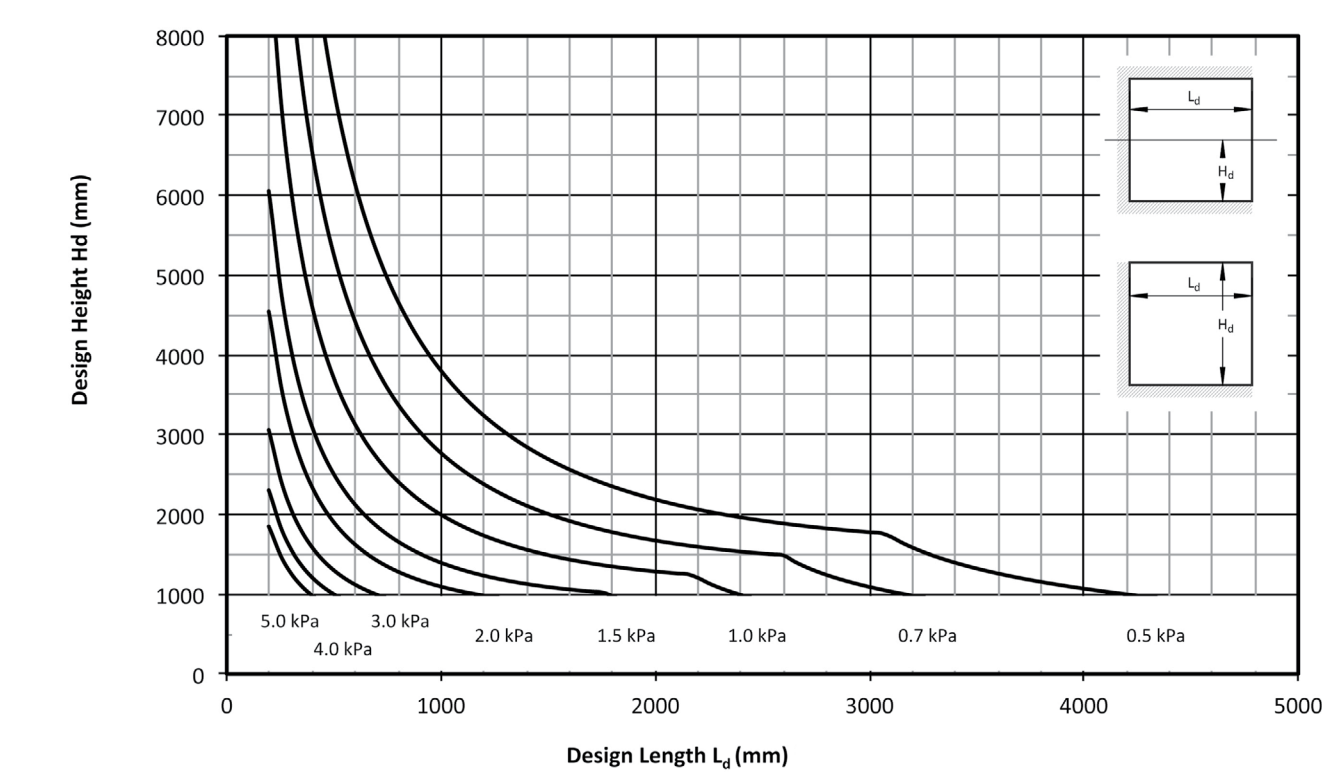


11.3.2 90 mm without openings (no rotational restraint at the sides)

Both Sides Supported



One Side Supported

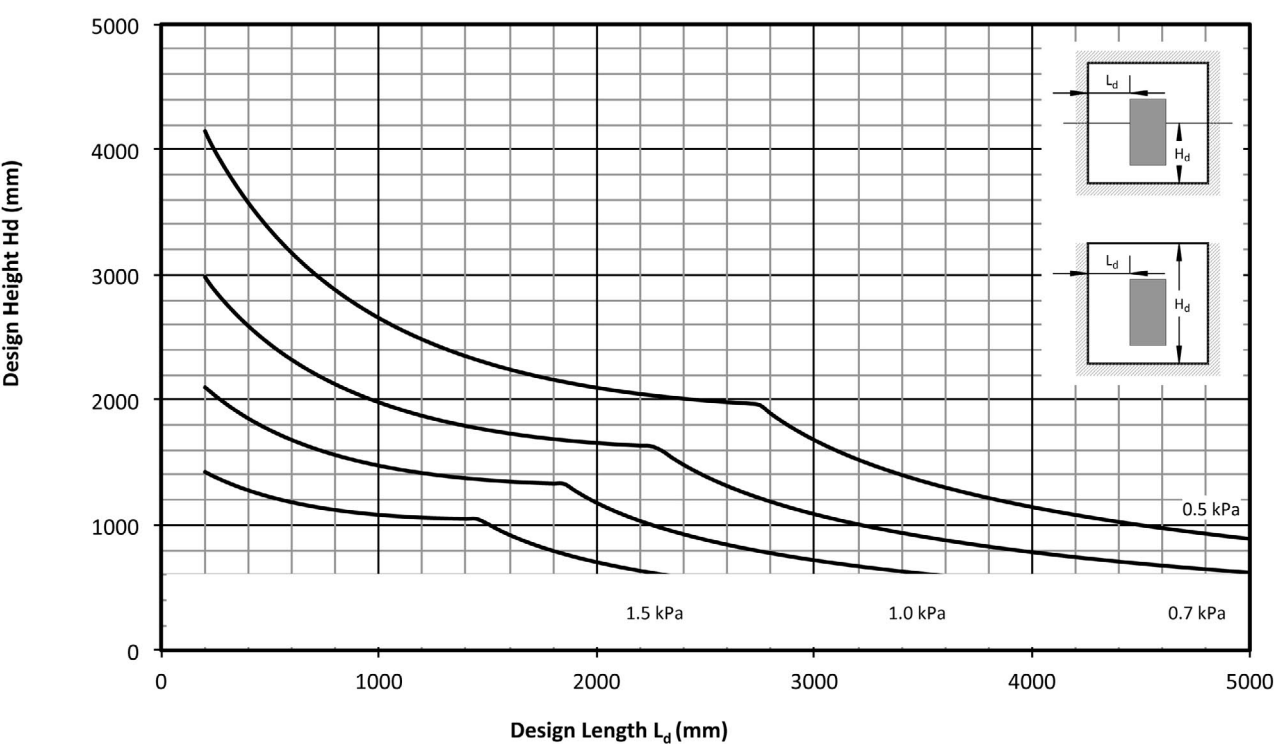




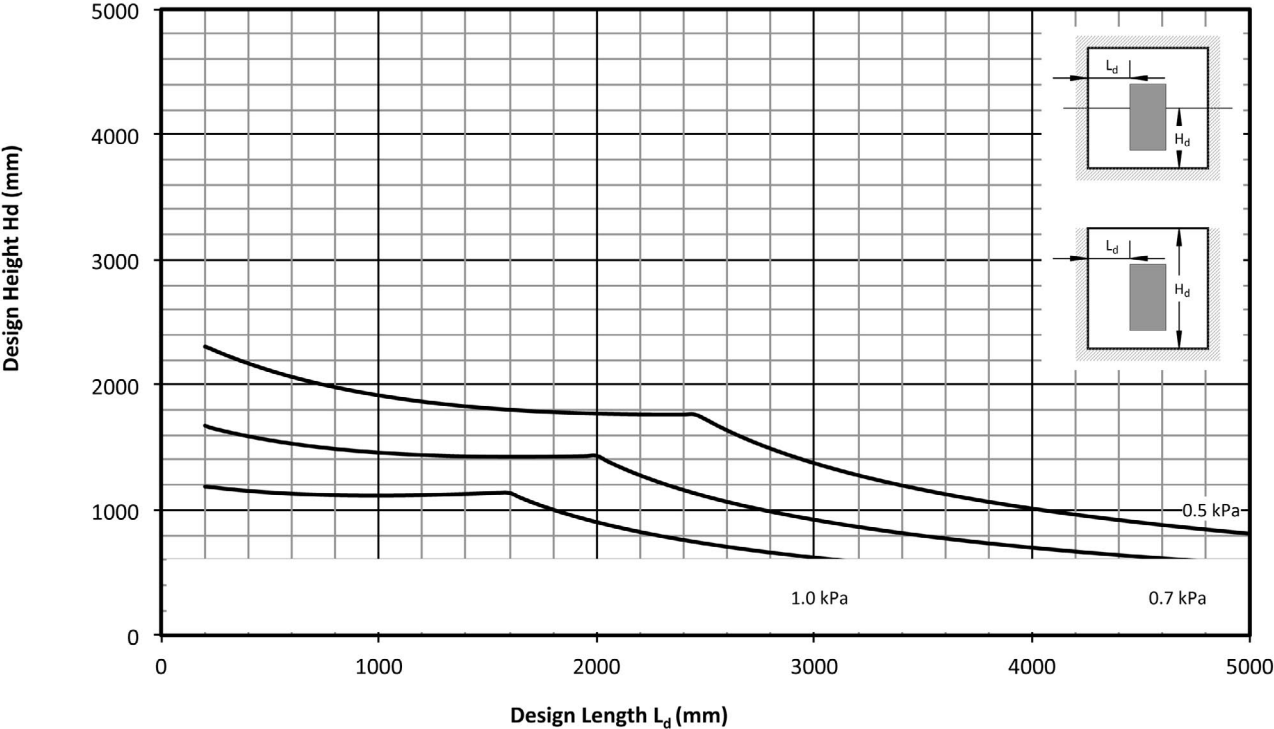
11.4 Two-way Bending With Openings

11.4.1 110 mm with openings (no rotational restraint at the sides)

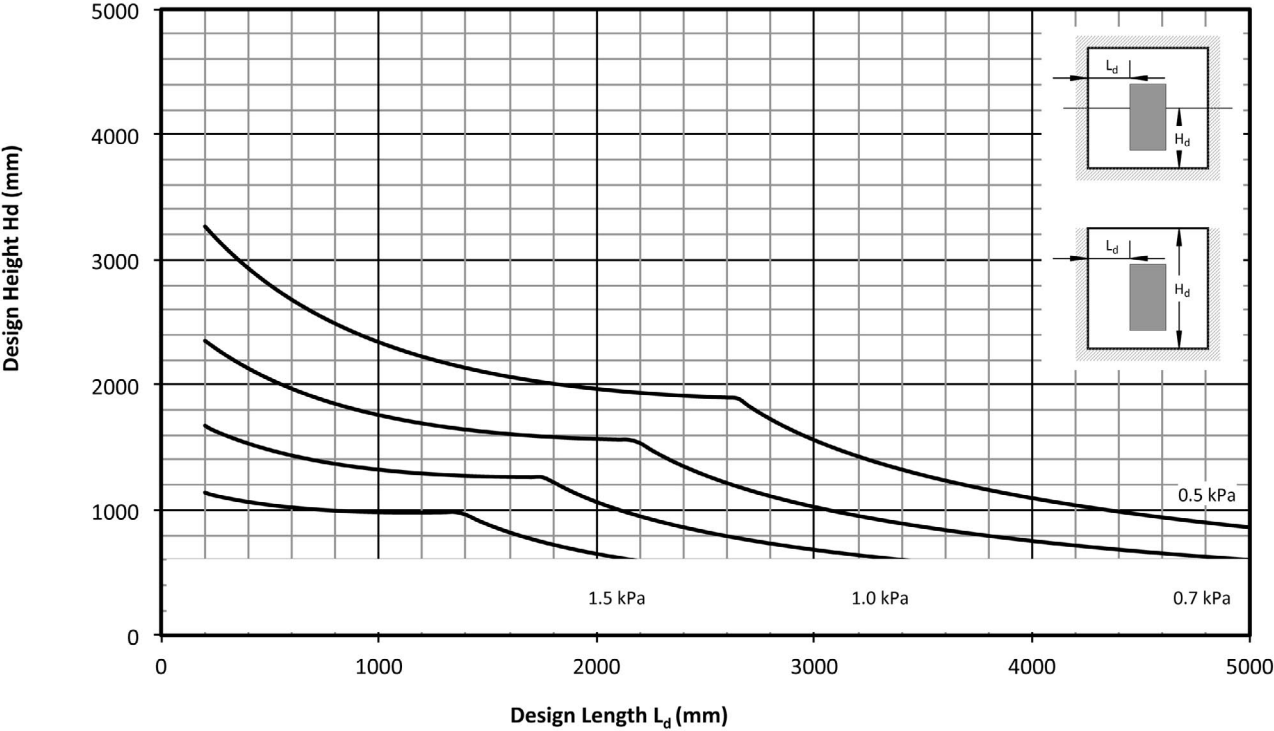
900 mm Wide Opening (Any Height), Both Sides Supported



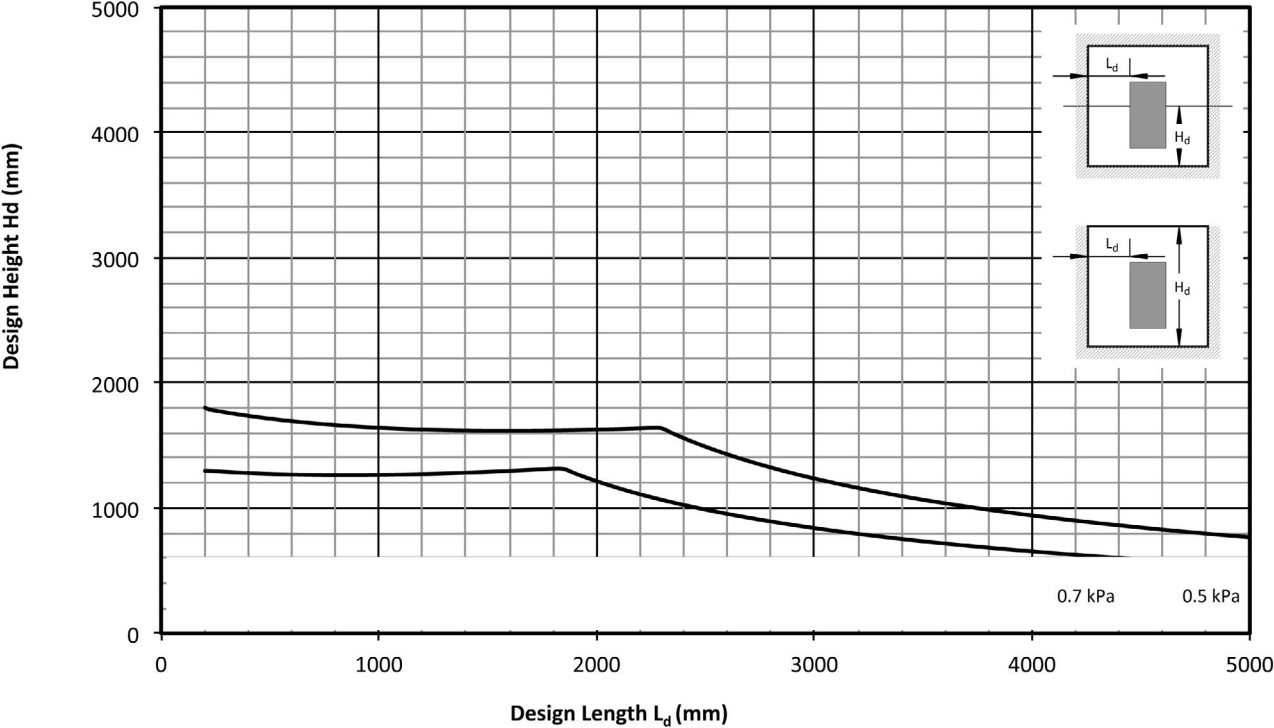
1800 mm Wide Opening (Any Height), Both Sides Supported



1200 mm Wide Opening (Any Height), Both Sides Supported

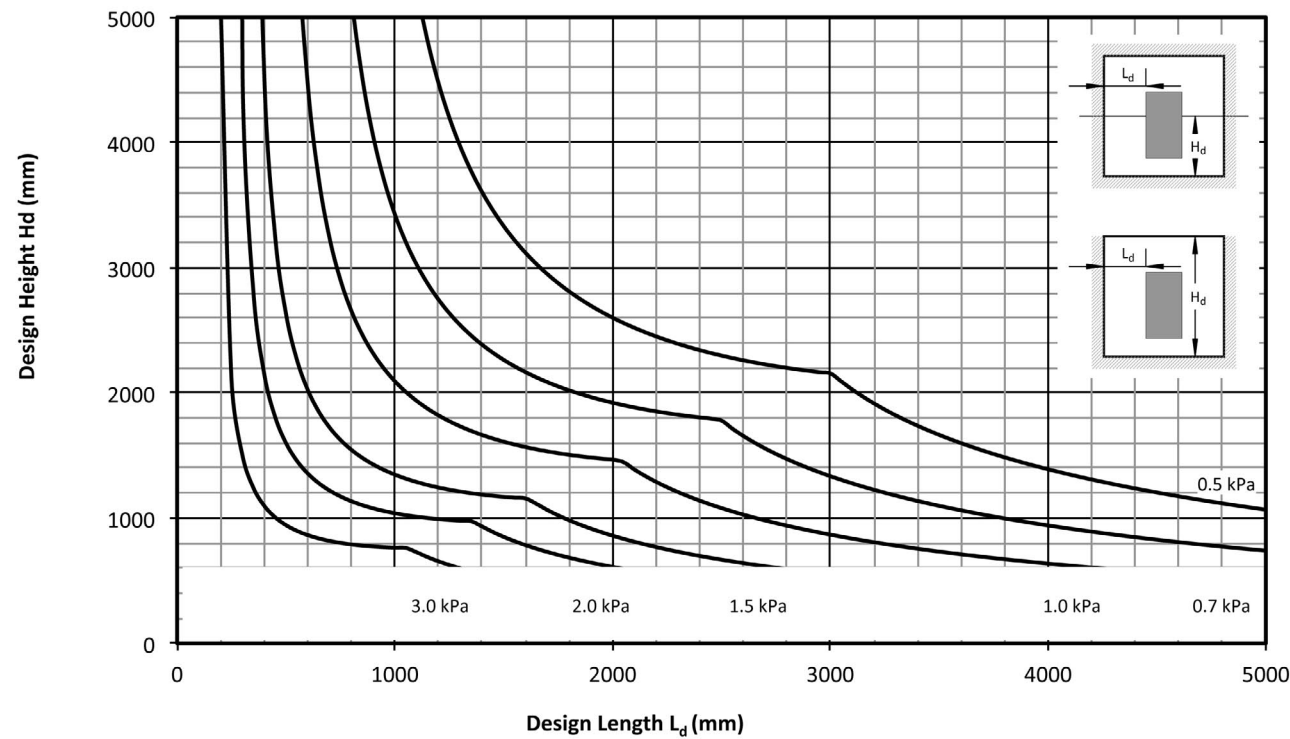


2400 mm Wide Opening (Any Height), Both Sides Supported

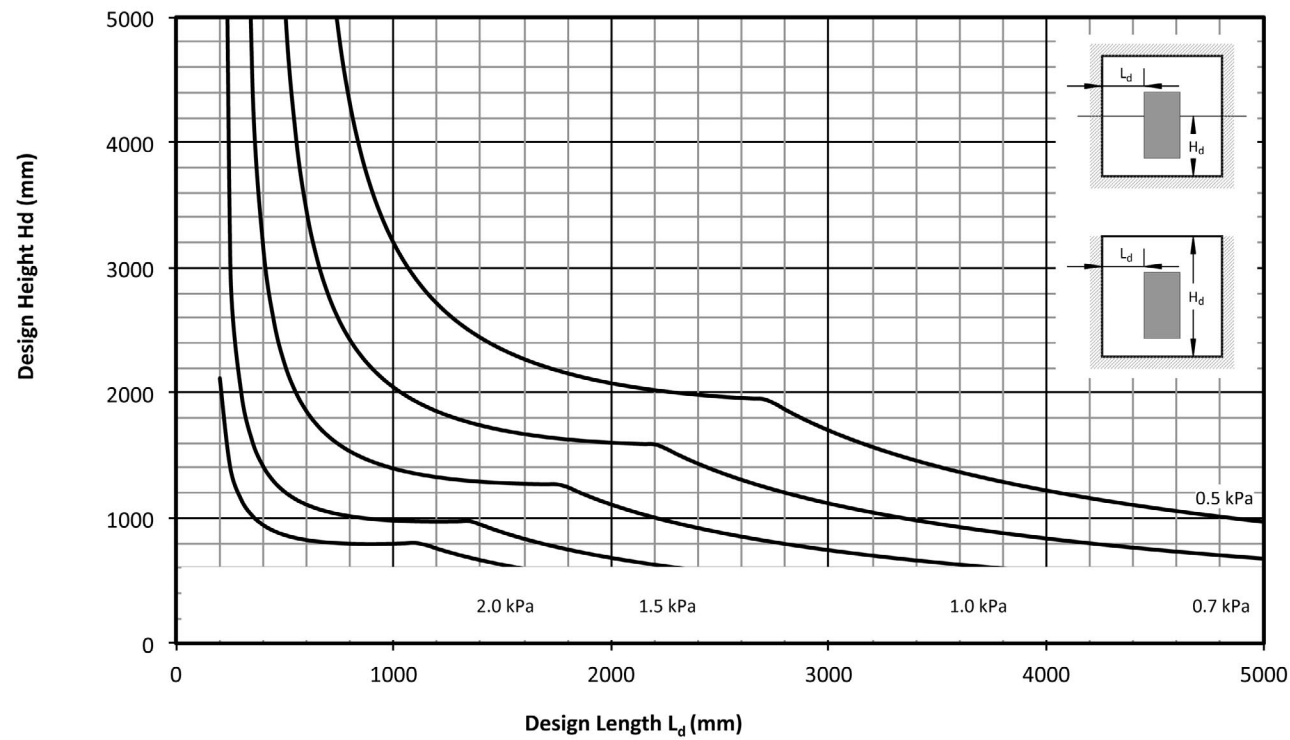


11.4.2 110 mm with openings (partial rotational restraint at the sides - factor 0.5)

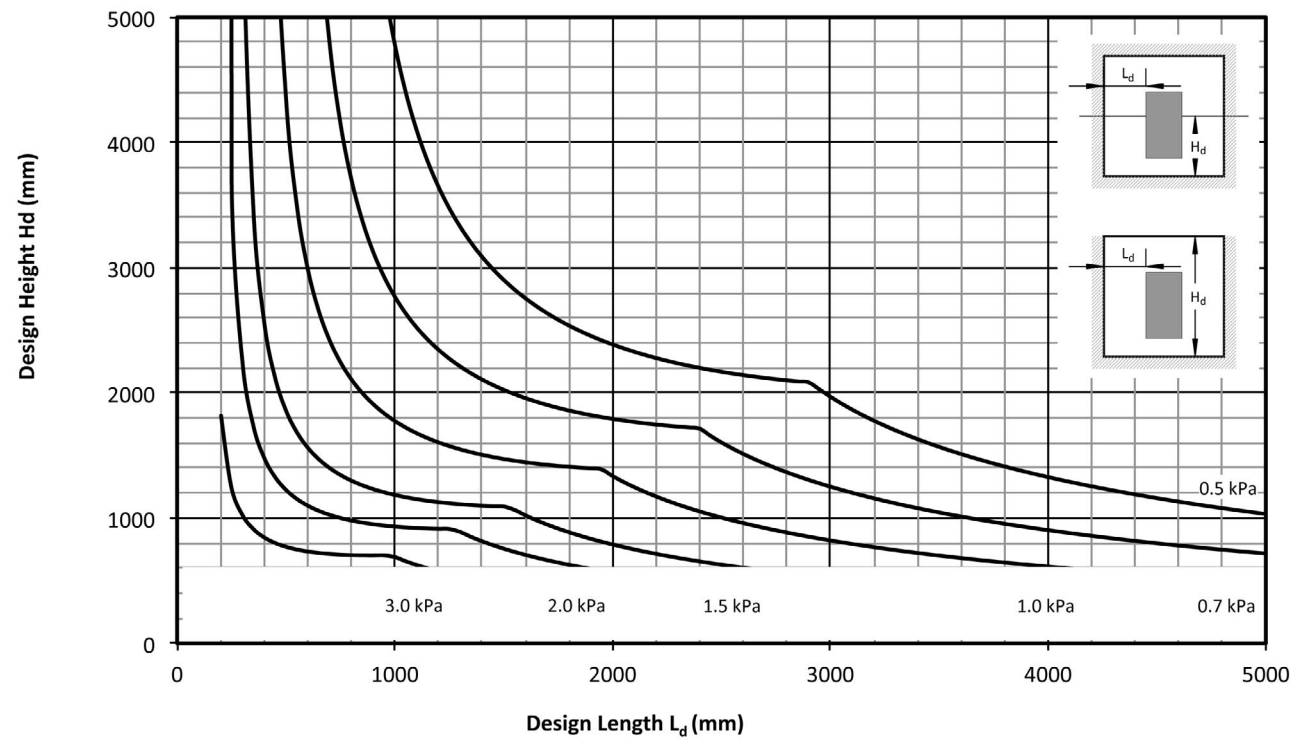
900 mm Wide Opening (Any Height), Both Sides Supported



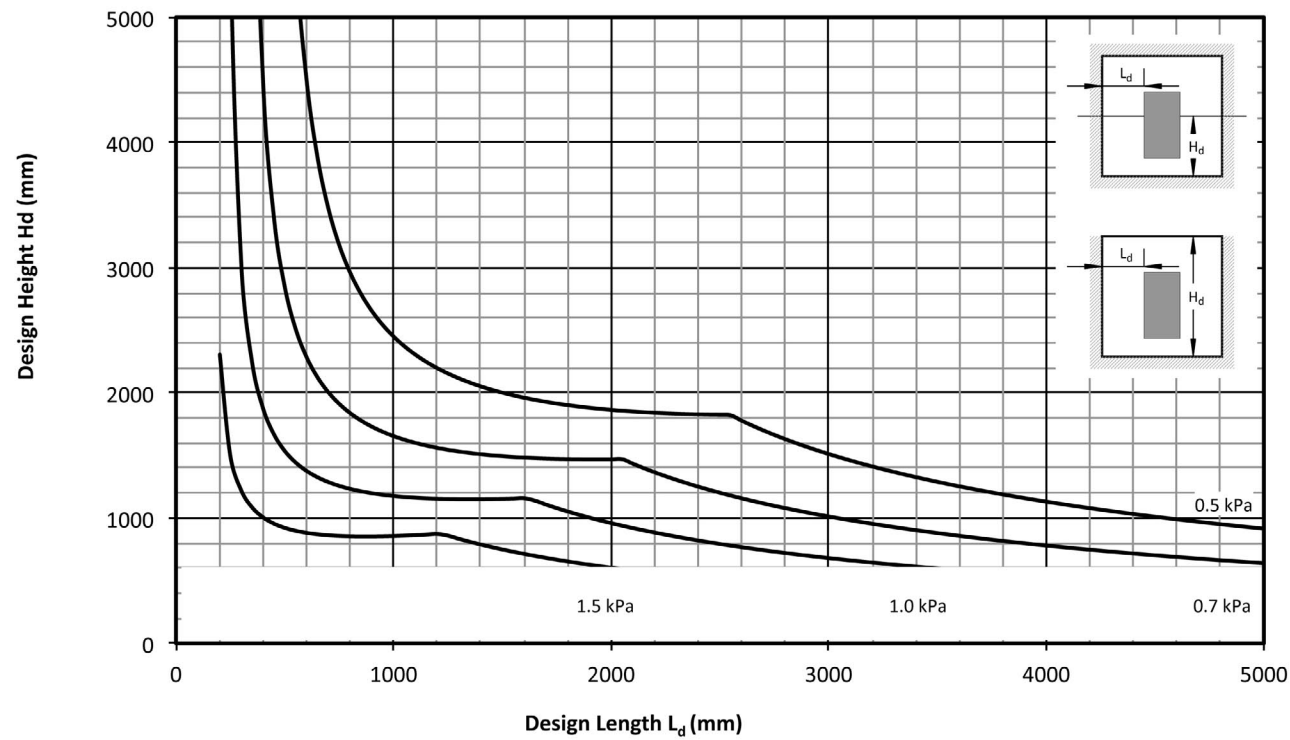
1800 mm Wide Opening (Any Height), Both Sides Supported



1200 mm Wide Opening (Any Height), Both Sides Supported

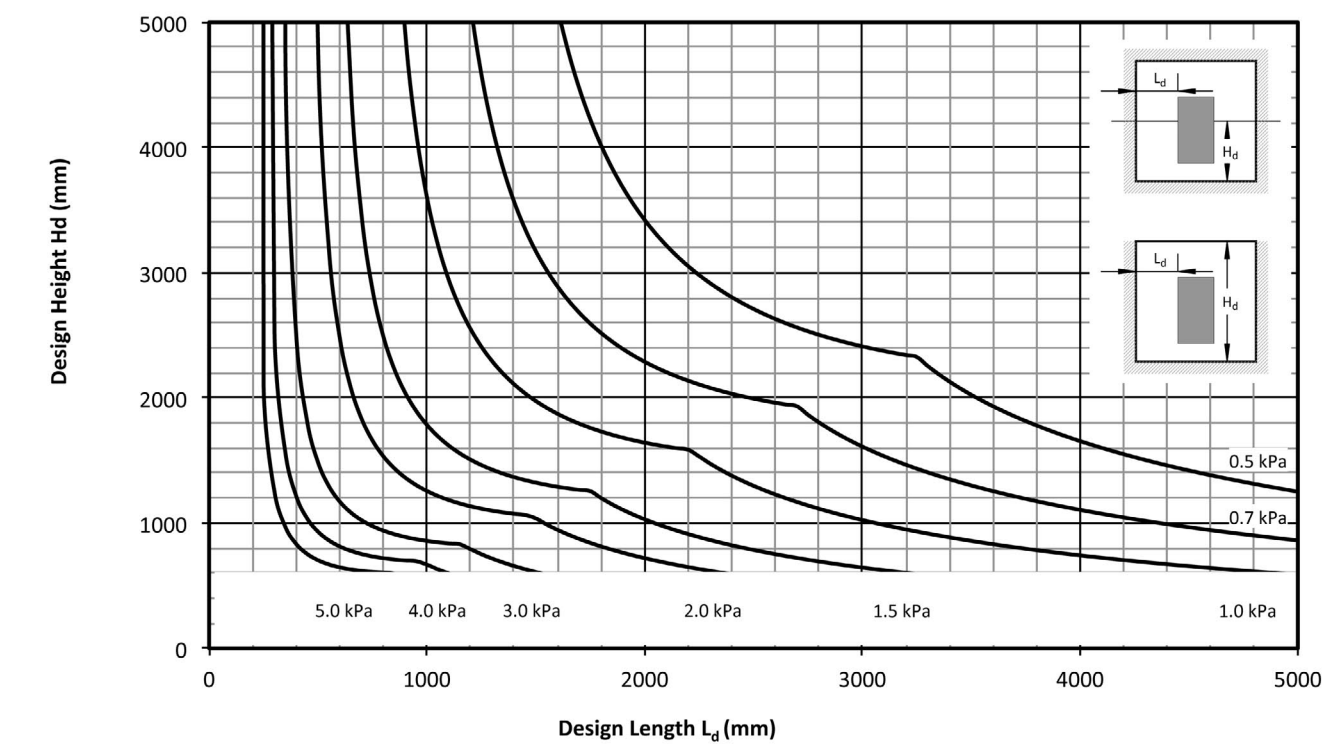


2400 mm Wide Opening (Any Height), Both Sides Supported

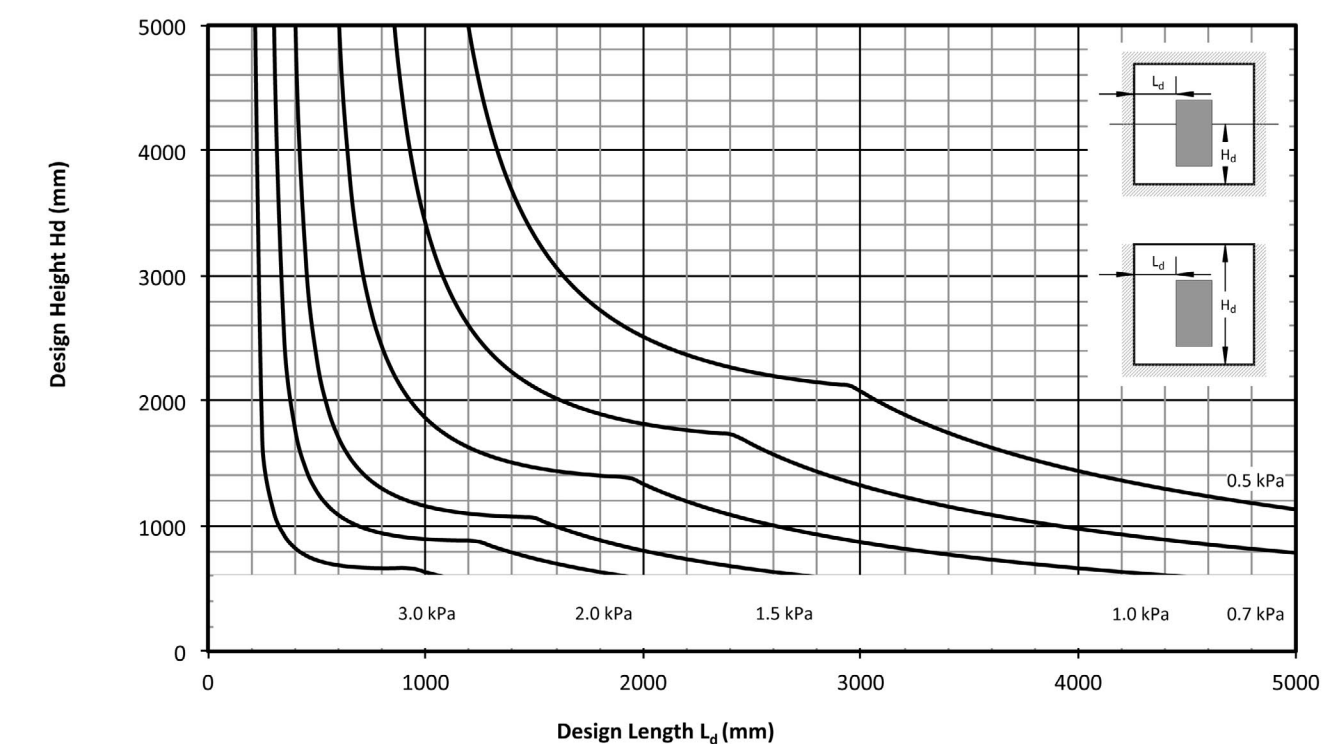


11.4.3 110 mm with openings (full rotational restraint at the sides - factor 1.0)

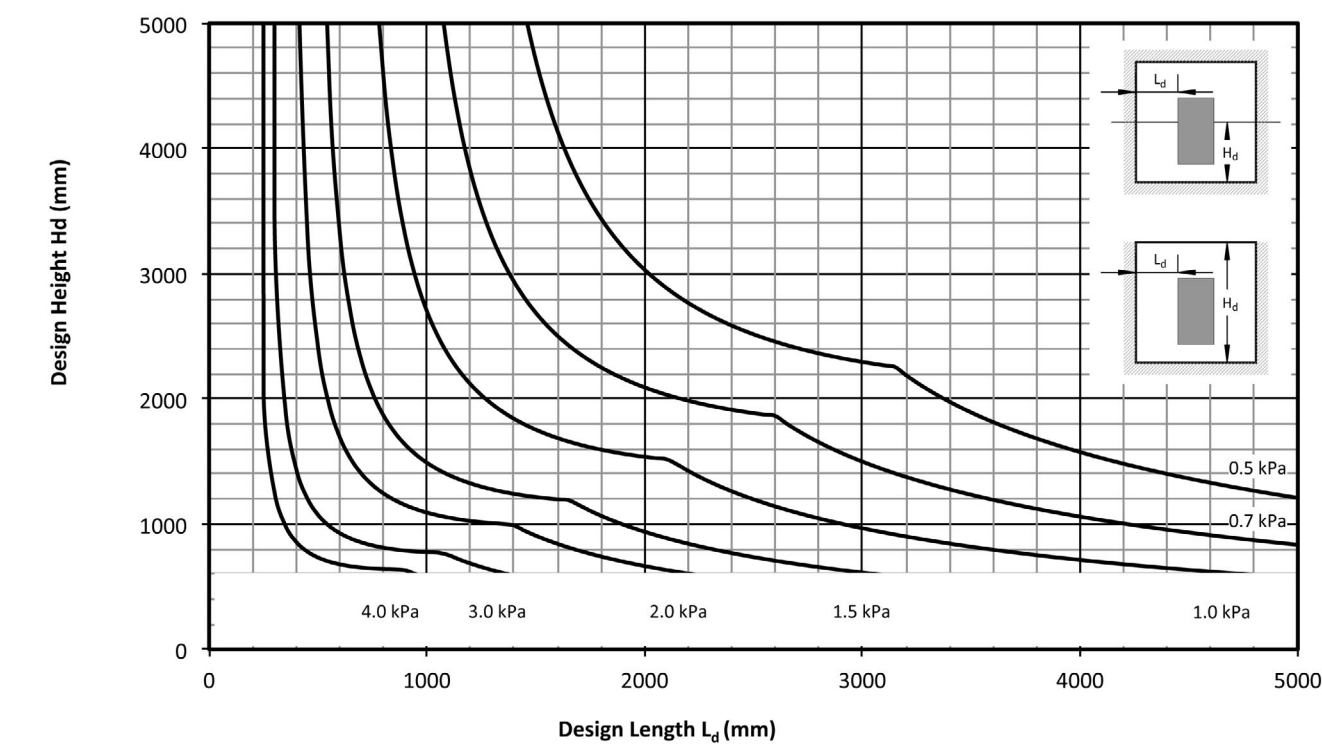
900 mm Wide Opening (Any Height), Both Sides Supported



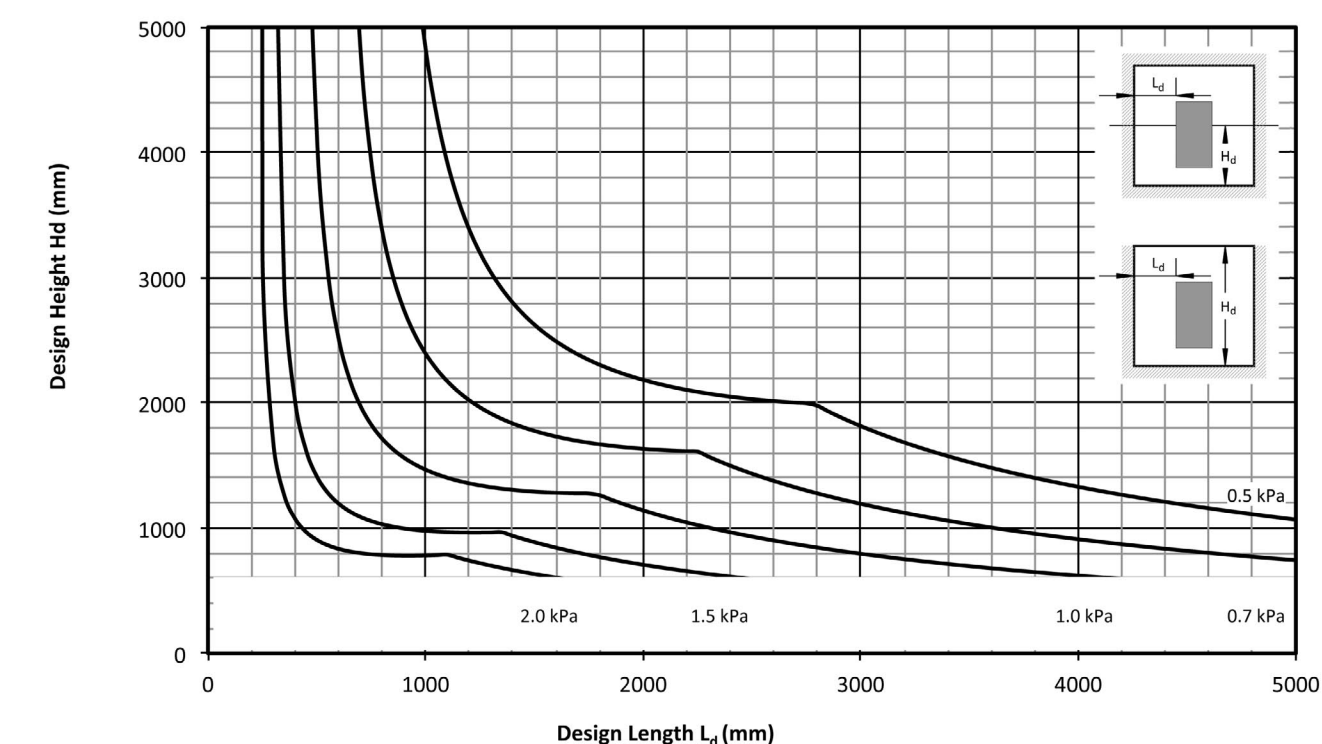
1800 mm Wide Opening (Any Height), Both Sides Supported



1200 mm Wide Opening (Any Height), Both Sides Supported



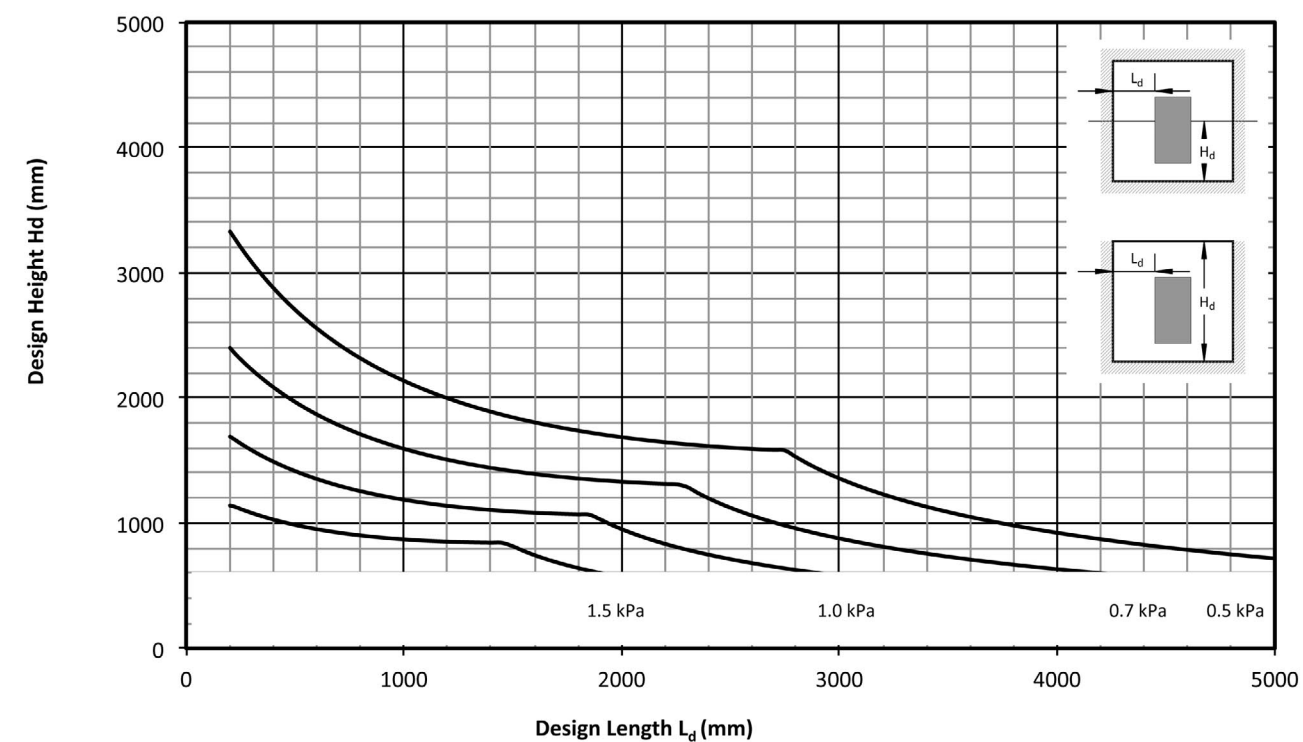
2400 mm Wide Opening (Any Height), Both Sides Supported



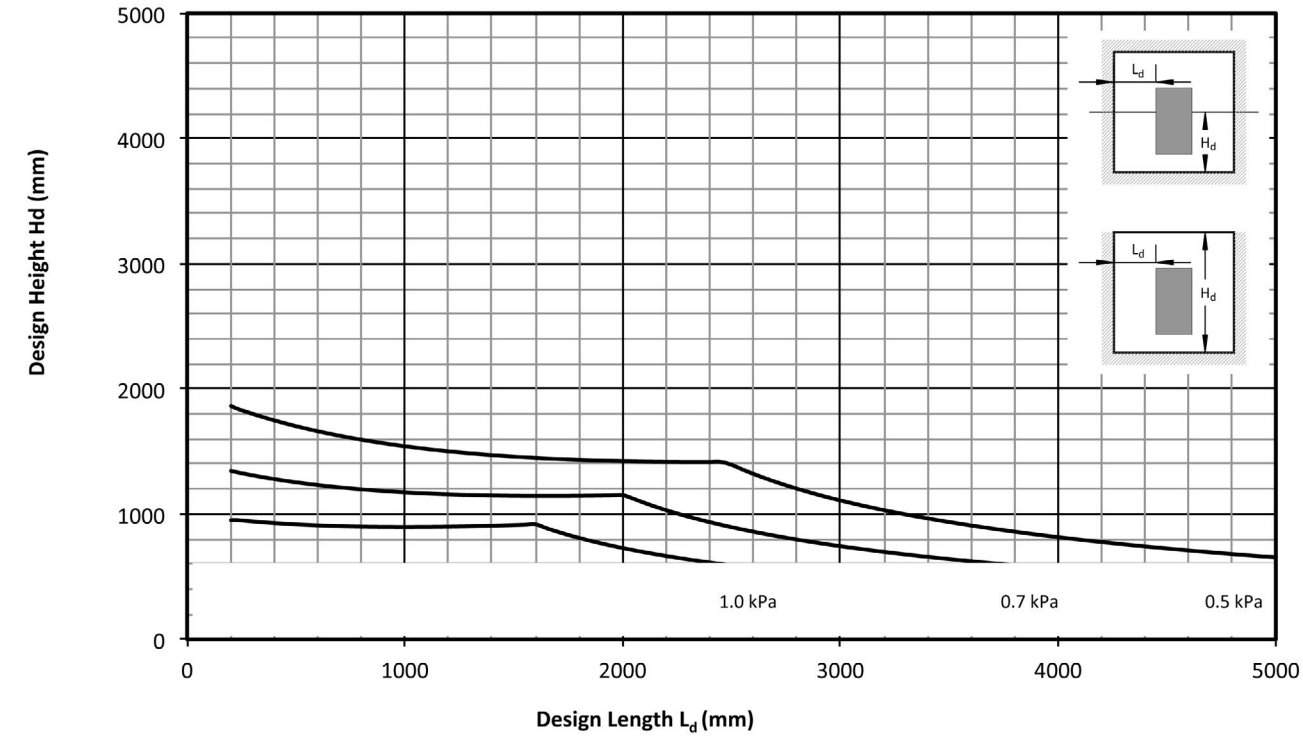


11.4.4 90 mm with openings (no rotational restraint at the sides)

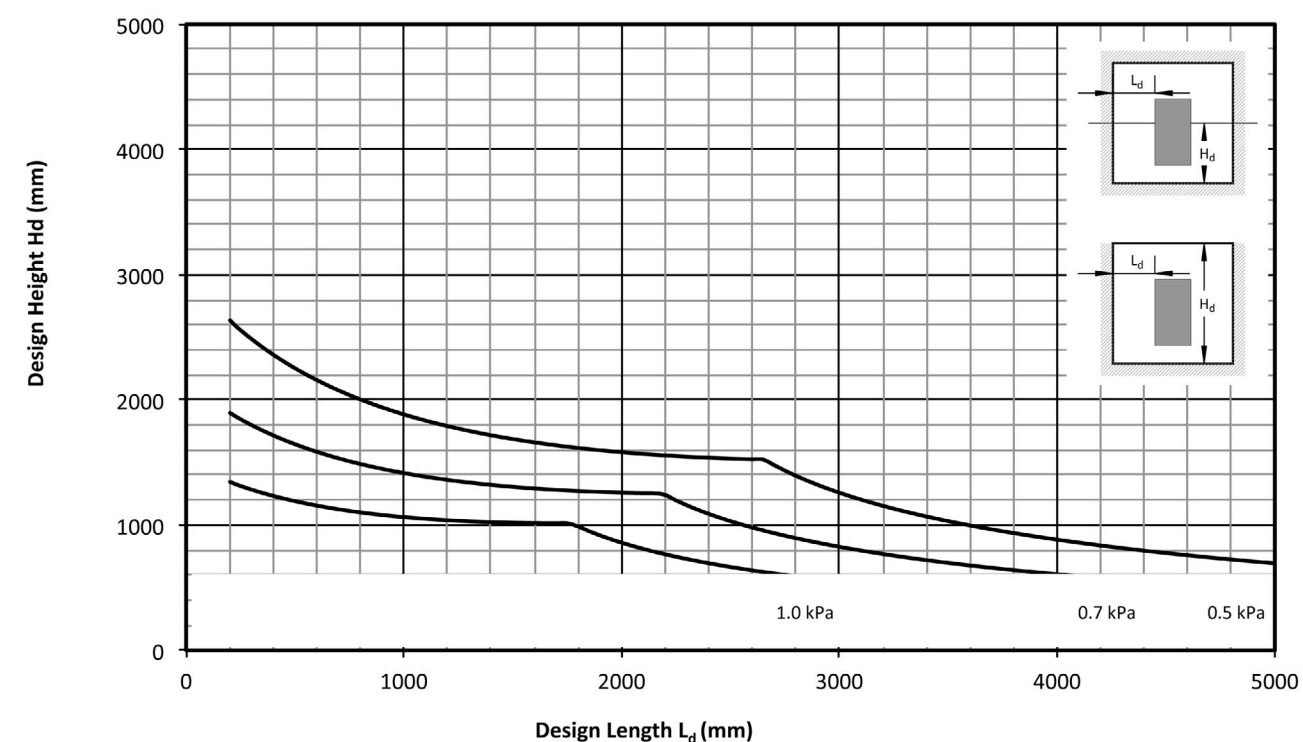
900 mm Wide Opening (Any Height), Both Sides Supported



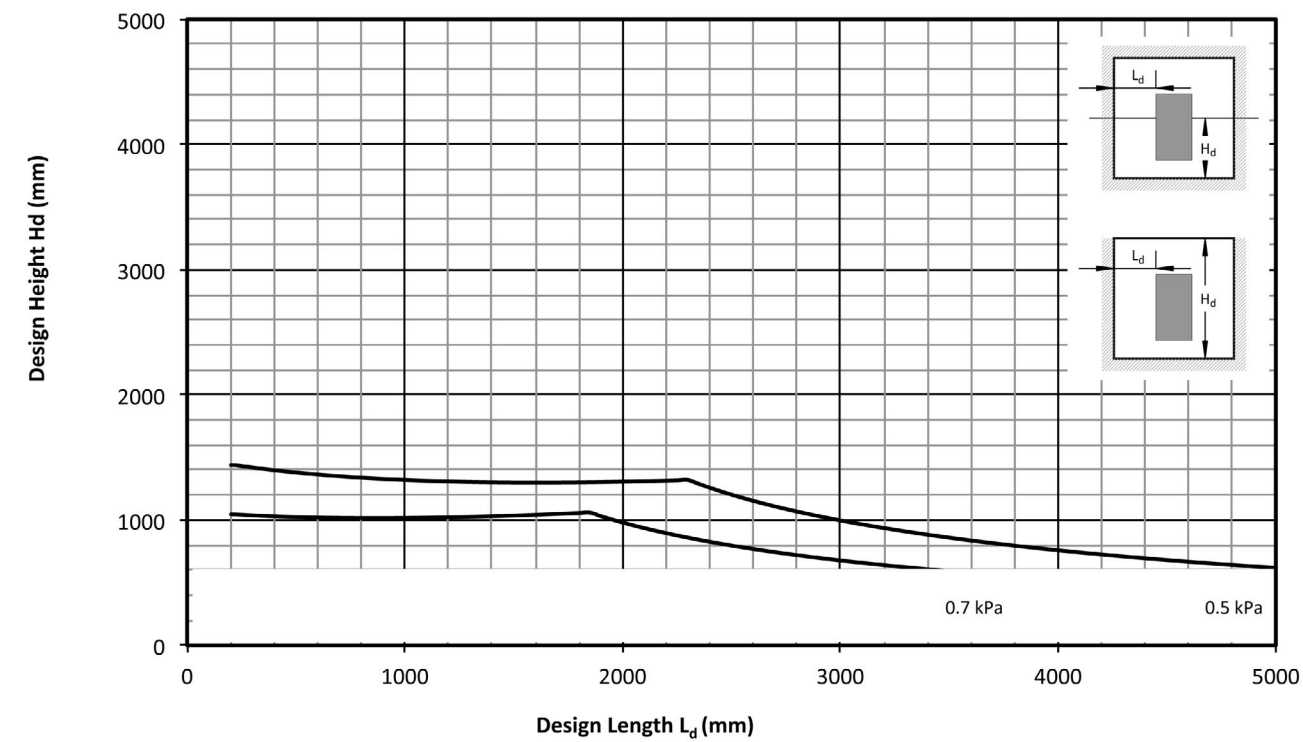
1800 mm Wide Opening (Any Height), Both Sides Supported



1200 mm Wide Opening (Any Height), Both Sides Supported



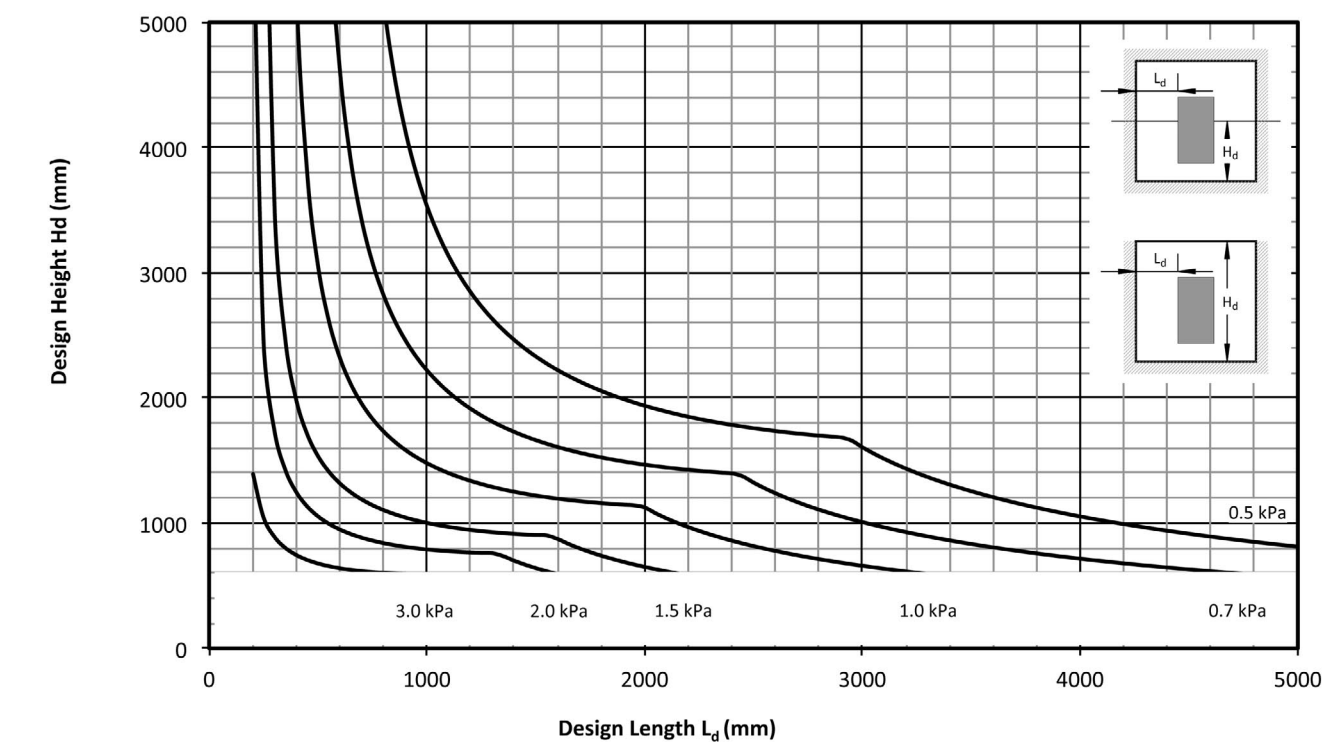
2400 mm Wide Opening (Any Height), Both Sides Supported



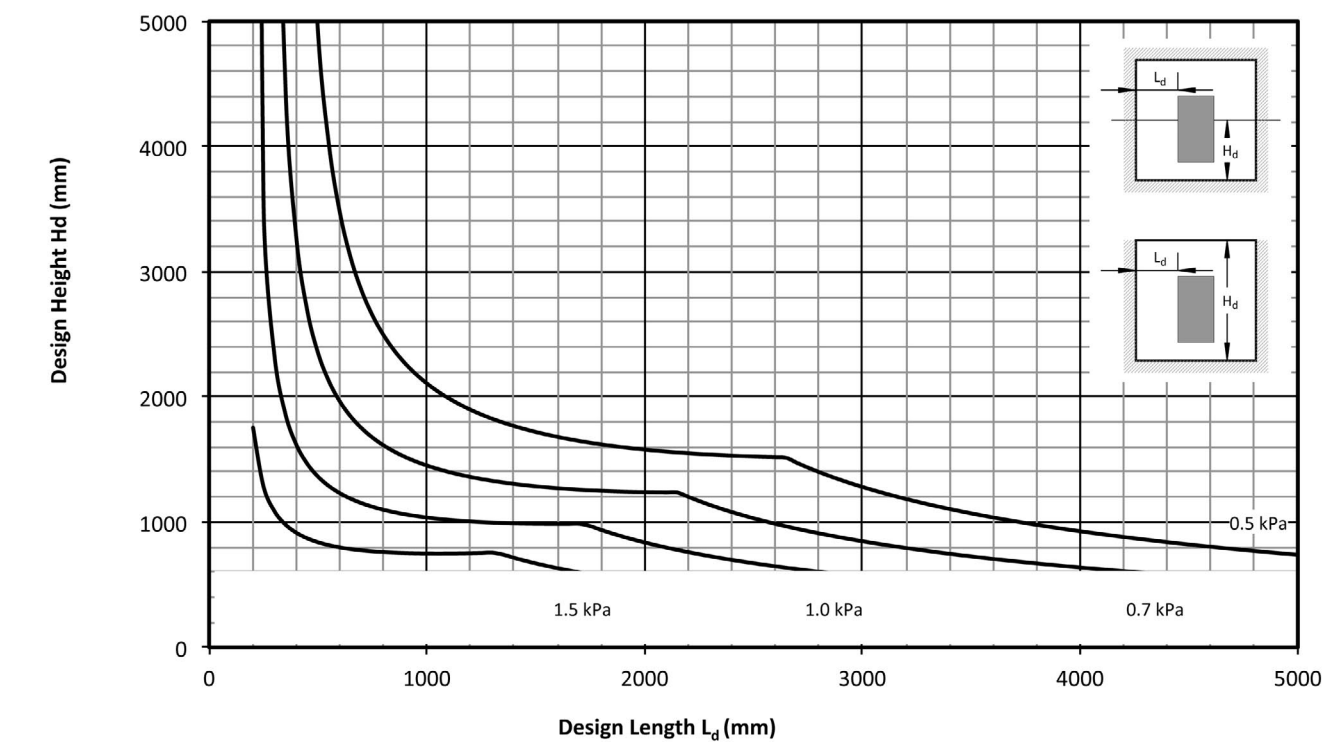


11.4.5 90 mm with openings (partial rotational restraint at the sides - factor 0.5)

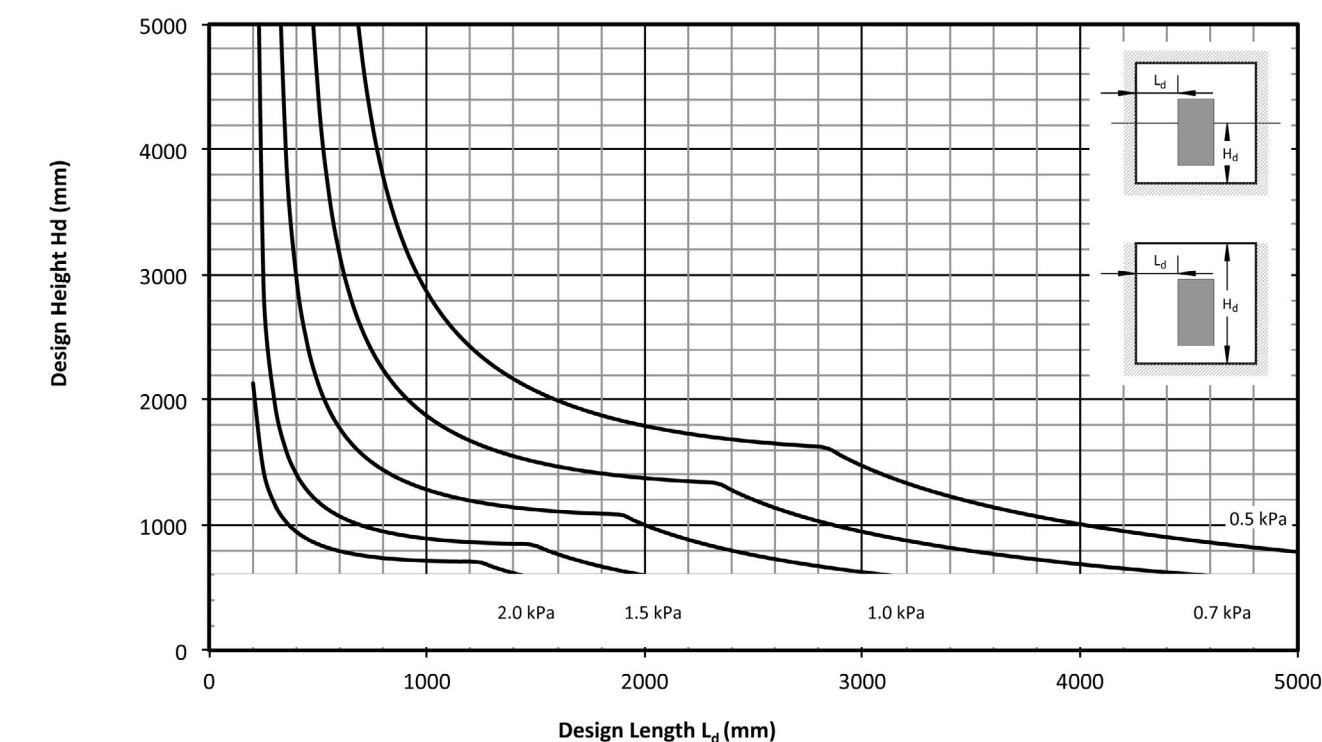
900 mm Wide Opening (Any Height), Both Sides Supported



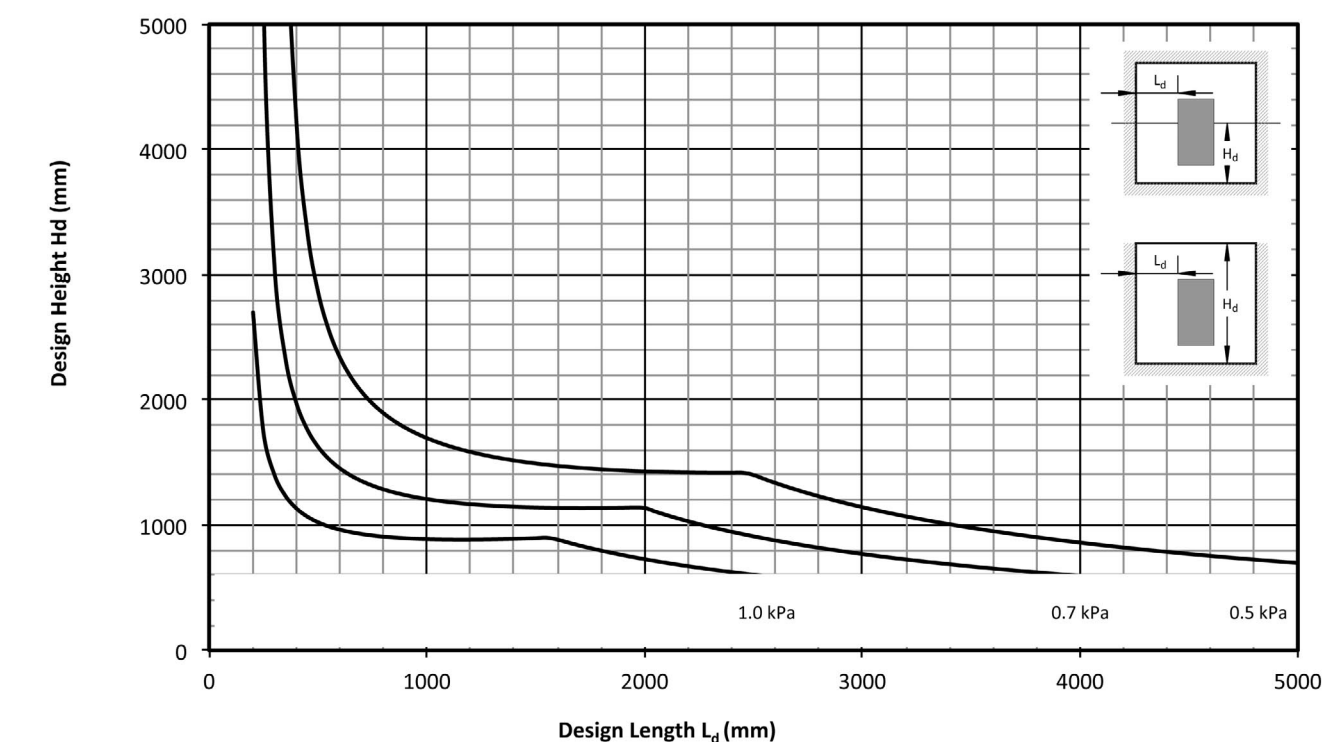
1800 mm Wide Opening (Any Height), Both Sides Supported



1200 mm Wide Opening (Any Height), Both Sides Supported

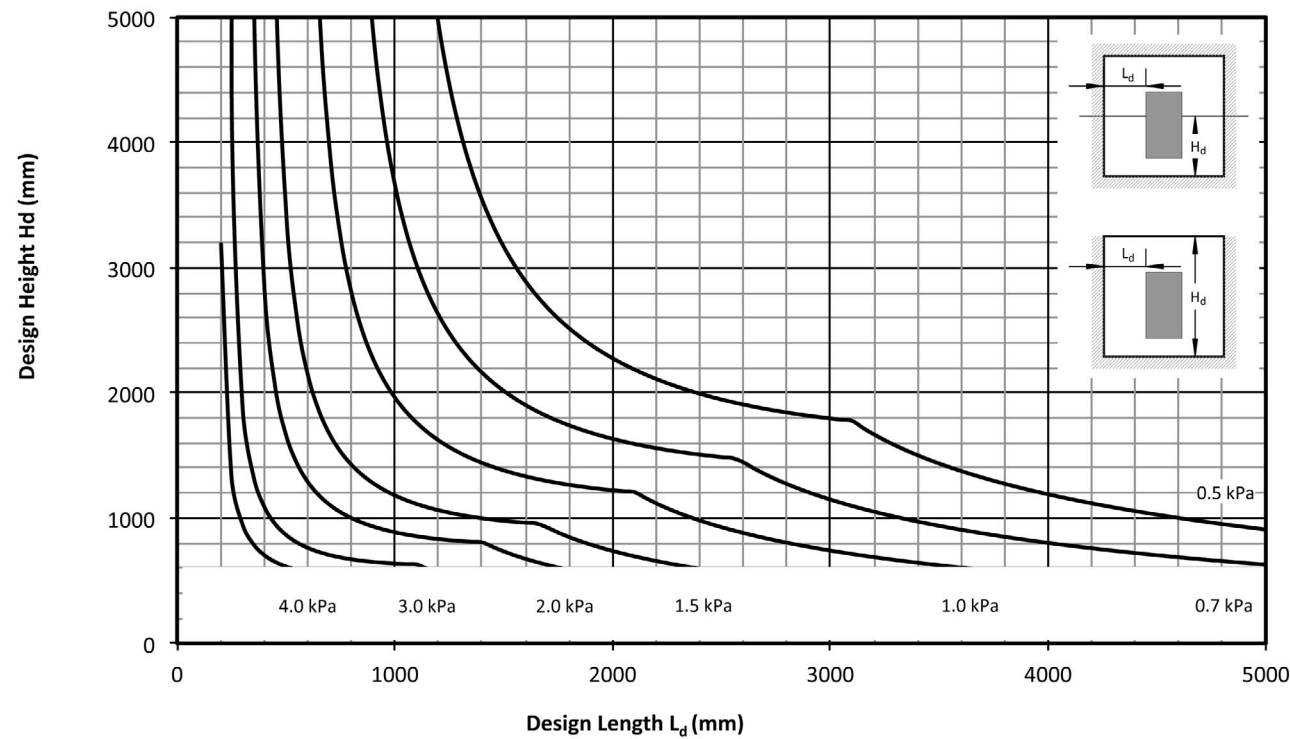


2400 mm Wide Opening (Any Height), Both Sides Supported

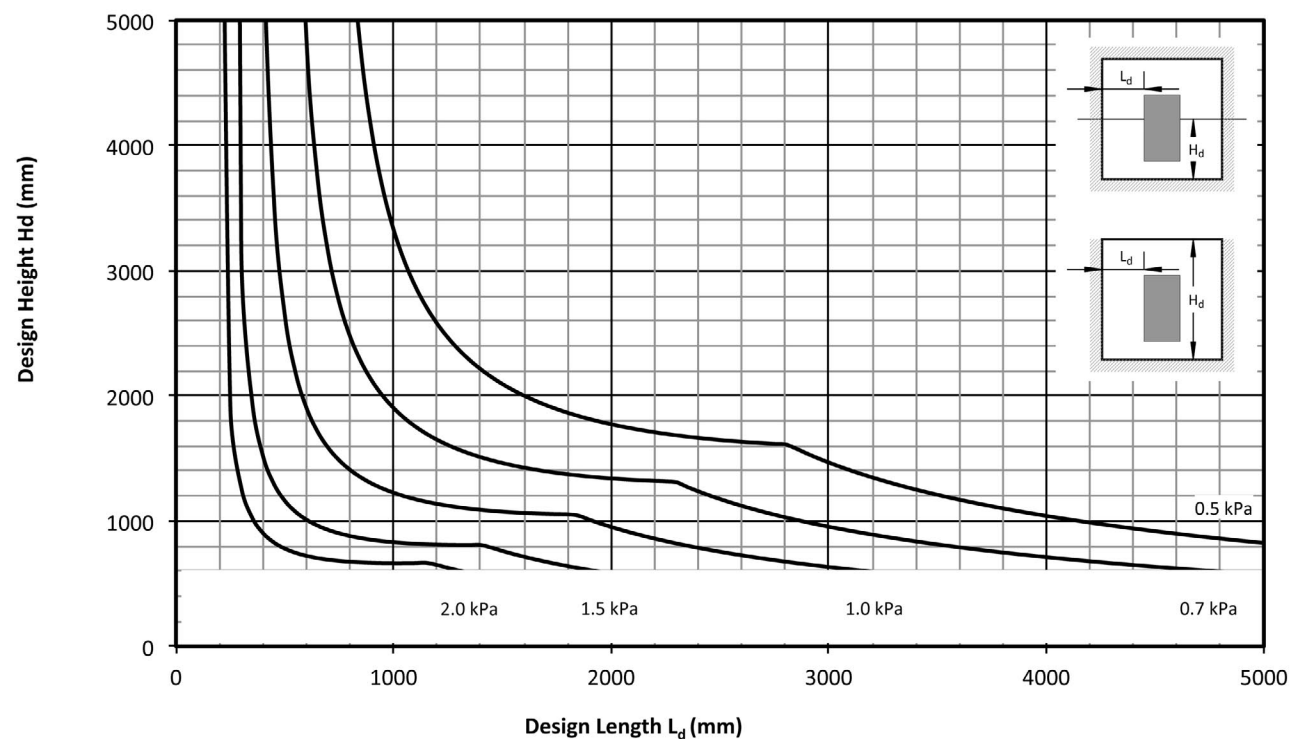


11.4.6 90 mm with openings (full rotational restraint at the sides - factor 1.0)

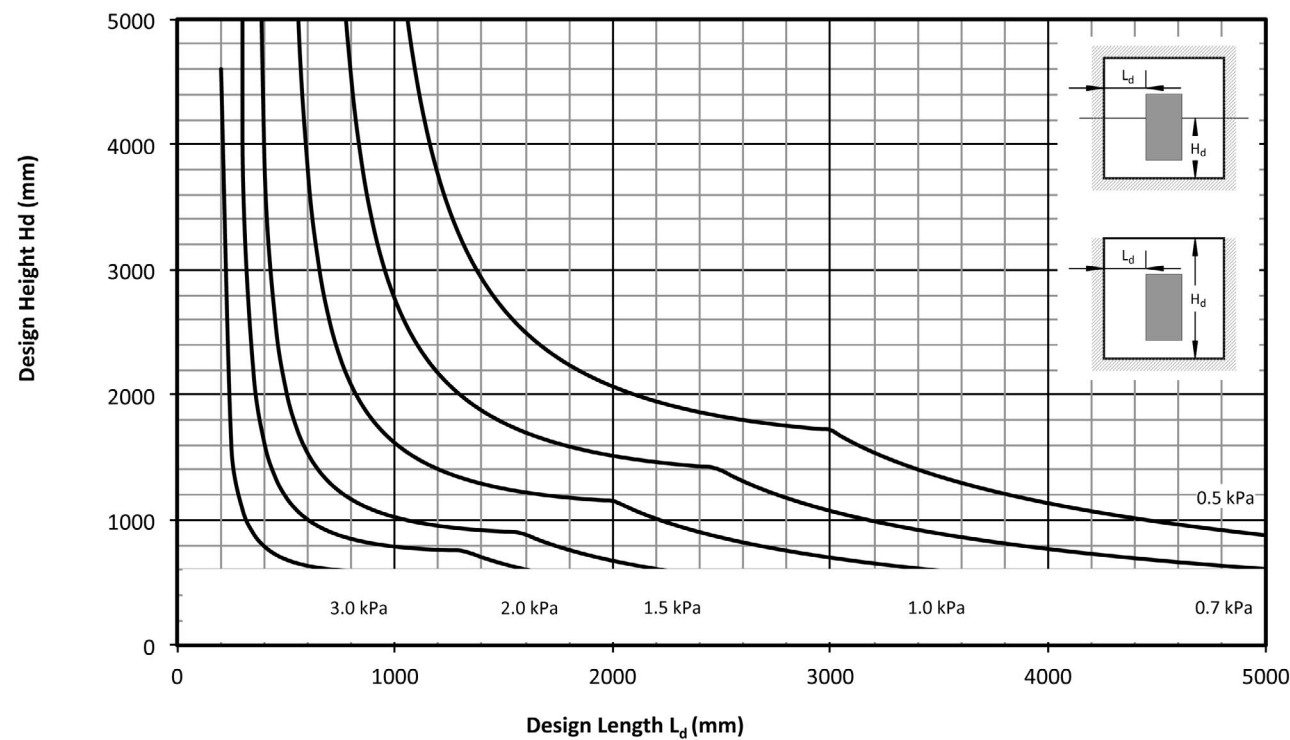
900 mm Wide Opening (Any Height), Both Sides Supported



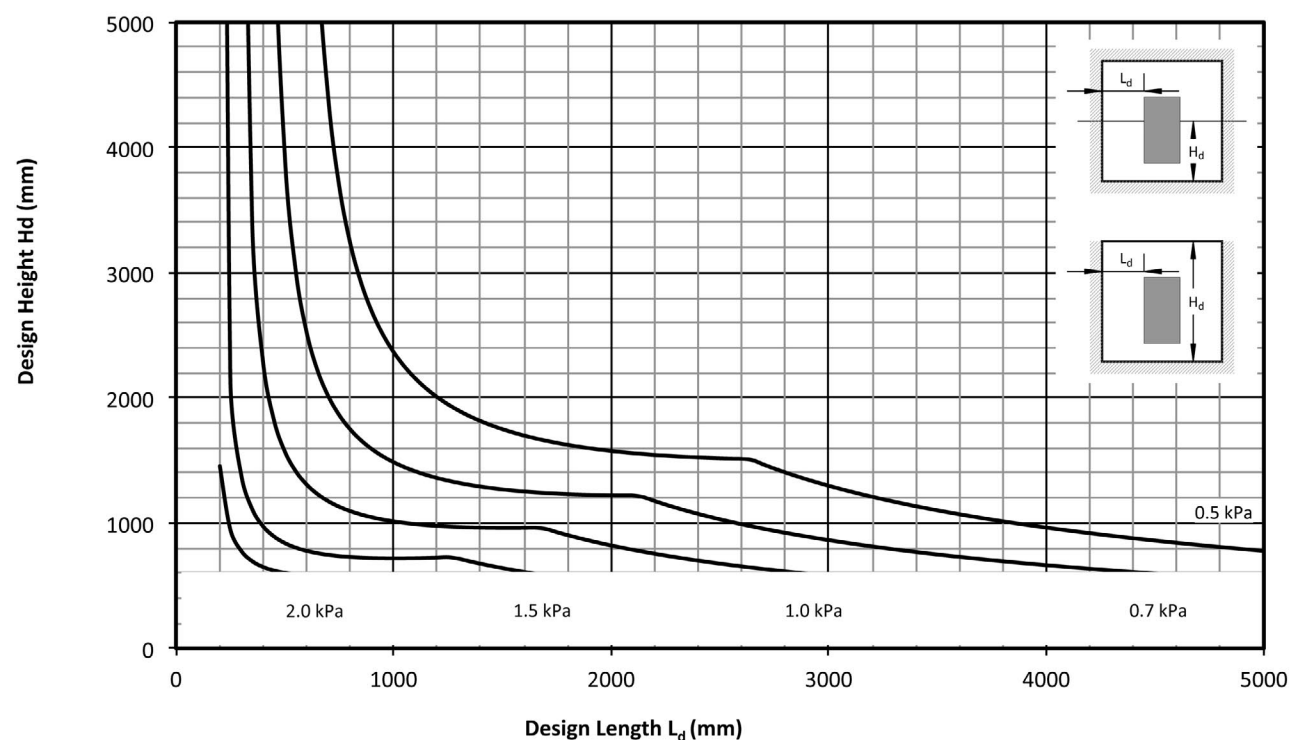
1800 mm Wide Opening (Any Height), Both Sides Supported



1200 mm Wide Opening (Any Height), Both Sides Supported



2400 mm Wide Opening (Any Height), Both Sides Supported



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7	AS/NZS 4456:2003 <i>Masonry units, segmental pavers and flags – Methods of test</i> , Standards Australia, Sydney, 2003.	20	Lawrence, S.J. & Marshall, R.J. <i>The New AS 3700 Approach to Lateral Load design</i> , Proceedings of the Fifth Australasian Masonry Conference, Gladstone, 1998.		
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