

DESIGN OF CLAY MASONRY

for Wind & Earthquake



Acrobat Edition



Photograph by Bart Maiorana, courtesy Cox Richardson





Design of Clay Masonry for Wind and Earthquake

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Although Australia has relatively low levels of seismic activity by world standards, the Newcastle earthquake of 1989 highlighted the need for appropriate masonry design and detailing in earthquake-prone areas.

Similarly, wind loading can be severe in some parts of Australia, especially on the upper levels of multi-storey structures.

This manual provides guidance for the design of unreinforced clay masonry to resist such forces, following the procedures set out in the *Masonry Structures Code*, AS 3700–1998.

The information is applicable 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 multi-storey, framed construction with masonry infill.

Also covered are material properties for clay masonry, general arrangement of structures, out-of-plane lateral loading and in-plane shear loading, and detailing of ties, connections and joints.

There are worked examples for various cases, as well as design charts for lateral loading (using equivalent static loads for both wind and earthquake) and a range of typical details for resisting wind and earthquake forces.

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1. Introduction

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 \(AS 3700\)](#)¹. 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](#)².

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 level of load increases significantly in the upper stories of multi-storey buildings.

Unreinforced masonry has poor seismic performance because it is heavy and brittle 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 multi-storey, 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.

Worked examples for various cases, design charts for lateral loading (using equivalent static loads for both wind and earthquake) and typical details for resisting wind and earthquake forces 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, with brick-veneer being the most common form in the eastern states. Full-brick (cavity) construction is popular in Western Australia and single-leaf construction using hollow units is widely used 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-walled 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 cavity 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 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 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 concrete or steel frame 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 with reinforced concrete or steel frames. 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 is an infill wall tied to the frame. Design for composite action between frames and infill walls is beyond the scope of this manual. 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 walls in the upper storeys of multi-storey buildings may be subjected to high wind loads because of their height above the ground and this will usually govern their design.

3. Masonry Elements

Various types of masonry elements are used to make up a typical masonry structure. These include walls (that may be of veneer, cavity, solid or diaphragm construction), piers and freestanding elements such as parapets and chimneys. These elements behave in different ways and their design must take their particular characteristics into account. 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 masonry choice for such 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 backup system, as well as the stiffness and location of wall ties. These factors affect the degree of load-sharing between the veneer and backup and the amount of load re-distribution 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 usually does not need to be designed. For design purposes it is sufficient to know the wall tie forces and corresponding loads on the backup frame or wall. Typical distributions of tie force derived from an [elastic analysis](#)³ 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 the figure.

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.

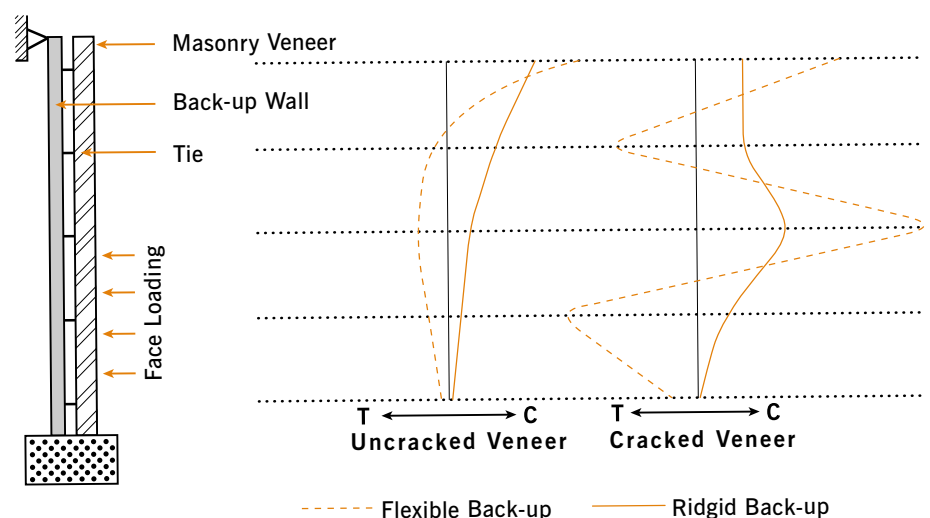


Figure 1. Variation of tie forces for flexible and rigid backups

This explains the logic of deemed-to-comply rules that 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 re-distribution 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 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.

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 in this 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.

3.3 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 that 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.

The design of infill panels that are isolated from the frame is 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.

3.4 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 5.4.3](#)).

4. Material Properties

Fired clay bricks have been in use for at least 3500 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 Australian standard governing the manufacture of masonry units is [AS/NZS 4455](#)⁴. 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](#)⁵. 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 the required flexural tensile bond strength is achieved.
- Lateral modulus of rupture sufficient for the required flexural tensile 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) are of vital importance for the achievement of the required tensile bond strength.

It is essential for the job specification to refer to the type of cement assumed in the design if the mix proportions are specified. For example AS 3700 classifies the following three mortars as M3:

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

A 1:1:6 mortar with GB cement is not deemed-to-satisfy the M3 classification. It is recommended that the job specification simply refer to the classification (such as M3) assumed in the design, allowing the actual mortar composition to be determined on site.

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, in both flexure and 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 per cent characteristic value, which means that 95 per cent of all masonry in the building should be stronger than this design value and only 5 per cent will be weaker. This bias is taken into account in setting the required safety factors. Special Masonry, that is with strengths higher than 0.2 MPa, requires quality control testing during construction to verify the required strengths are being achieved. Design values as high as 1 MPa can be taken for 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, that is accounted for by a shear factor prescribed in AS 3700 ([see Section 7.2](#)).

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 be properly installed. The standard covering wall ties is [AS 2699](#)⁶ with [AS 2975](#)⁷ governing other connectors and accessories. 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.

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 classification required for each particular loading situation ([see Section 8.2](#)). This required classification 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). Pending the publication of a revised

AS 2699, the means of satisfying these rating requirements are given in AS 3700 Appendix F. 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 2904](#)⁸. 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. The earthquake loading code [AS 1170.4](#)⁹ was amended in 1994 to permit friction on these planes to be considered as providing the necessary restraint for general structures, where the design shows the requirements are satisfied. AS 3700 provides friction shear factors for the common DPC materials to allow these calculations to be made ([see Section 8.4](#)). 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. General Design Aspects

5.1 Loading conditions

Wind and earthquake produce horizontal lateral loads on a structure, which generate in-plane shear loads and out-of-plane face loads on individual members. While both loading types generate horizontal forces, they are different in nature. Wind loads are applied directly to the surface 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.

5.1.1 Wind loading

Wind 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 northern cyclonic regions. 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 Code \(AS 1170.2\)](#)¹⁰. This code 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 [Wind Loads for Housing \(AS 4055\)](#)¹¹. This system uses regions, terrain categories, shielding categories and topographic

categories to determine the wind classification. Provided the structure is within certain restrictions on height, shape and slope of roof, it is classified as N1 to N6 for non-cyclonic regions and C1 to C4 for cyclonic regions. This classification then determines the design wind speeds for the ultimate limit state, ranging 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.

The [following table](#) is provided as a guide to wind pressures when the wind classification for a housing site is known. It is not intended to replace the proper use of AS 4055. The pressures are for ultimate-strength limit-state design and are derived using the coefficients applicable to the worst case for general wall areas given in AS 4055 Appendix B.

Wind acting on a structure causes three main effects that must be accounted for in design: out-of-plane bending, in-plane shear and uplift causing direct tension. Design for the first two is covered in [Section 6](#) and [Section 7](#). 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 in design as the tensile strength of the material is considered to be zero in such circumstances.

Table 1. Design wind pressures for housing (based on AS 4055)

Wind classification	Ultimate wind speed (m/sec)	Design wind pressure (kPa)
N1	34	0.7
N2	40	1.0
N3	50	1.5
N4	61	2.2
N5	74	3.3
N6	86	4.4
C1	50	2.0
C2	61	3.0
C3	74	4.4
C4	86	6.0

5.1.2 Earthquake loading

Earthquake loading is the force generated by horizontal and vertical ground movements due to 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 and background information is given in its [Commentary](#)¹². Until the advent of this new earthquake loading code in 1993, unreinforced masonry structures were usually designed for dead, live and wind loads, with seismic loading often not being considered. The 1989 Newcastle earthquake highlighted the need for seismic design and now, with the incorporation of AS 1170.4 into the *Building Code of Australia*, consideration of seismic effects is mandatory for all structures. Because of the low levels of Australian seismicity, it is feasible to use

properly detailed and constructed unreinforced masonry in most areas. 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. Unreinforced masonry (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. As with 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. To this end the influence of flashings, membrane-type damp-proof courses and slip joints must be considered.

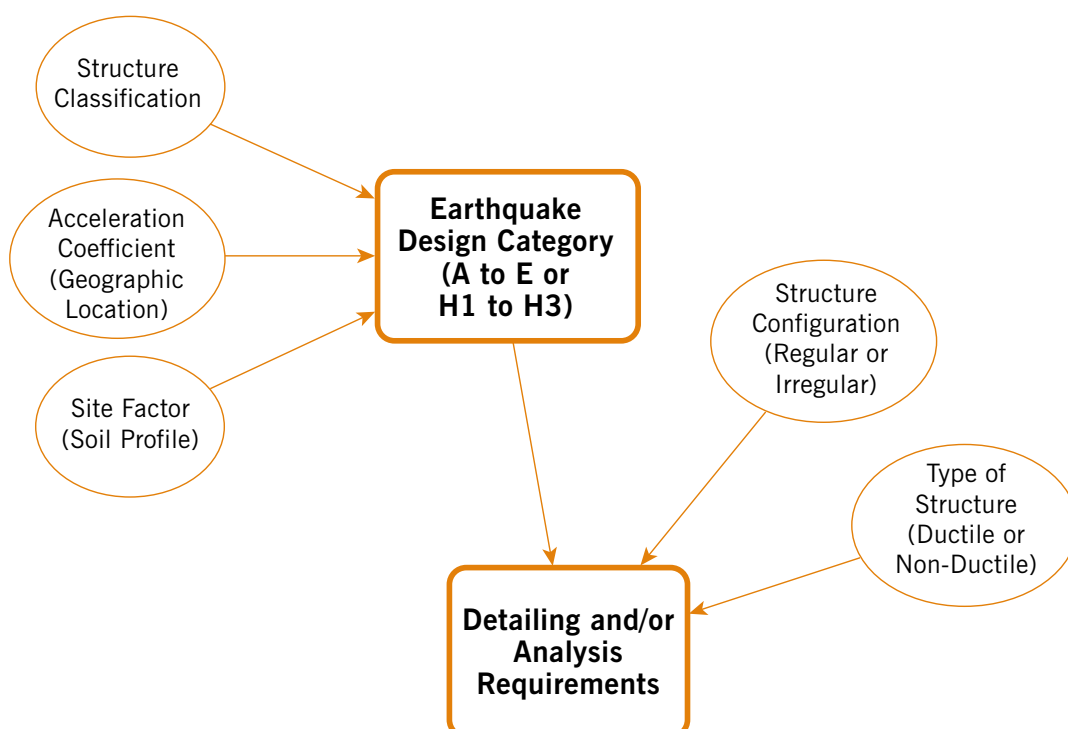
For simplicity, earthquake loading can be converted to equivalent static forces with appropriate allowance for the dynamic characteristics of the structure, foundation conditions, etc.

This approach is sufficient for most masonry structures that normally have a short fundamental period and low dynamic response.

AS 1170.4 provides load combinations for seismic design against strength and stability limit states. Serviceability is not considered for seismic design but must be considered in detailing connections for serviceability performance.

In the application of AS 1170.4 a structure classification, acceleration coefficient and site factor are used. The structure classification for general structures is related to the importance of the structure and its post-earthquake function. The acceleration coefficient (a) is determined by the geographic location of the structure and the site factor (S) depends on the soil profile of the site.

Figure 2. Application of AS 1170.4



5. General Design Aspects

Table 2. Design and detailing requirements for unreinforced masonry in general structures

Category	Analysis	Max. storeys	Detailing
A	None	4	Note 1
B	Static or dynamic	Regular: 4 Irregular: 3	Note 1 Note 1
C	Static or dynamic	3	Notes 1 & 2
D	Regular: static or dynamic Irregular: dynamic	2	Notes 1 & 2
E	Unreinforced masonry not permitted	Unreinforced masonry not permitted	Unreinforced masonry not permitted

Note 1. Detailing requirements–

- Load paths, ties and continuity
- Connections designed for 0.05 times gravity load
- Wall anchorage force $5a_s$ kN/m for Category A (but not less than 0.8 kN per metre), $10a_s$ kN/m for Category B

Note 2. Additional detailing requirements–

- More severe requirements for ties and continuity
- Specific diaphragm design requirements
- Ductility requirements on bearing wall connections
- Openings in shear walls and diaphragms to be considered
- Footing tie requirements

Table 3. Design and detailing requirements for domestic structures

Category	Type of construction	
	Ductile	Non-ductile
H1	No analysis or detailing	No analysis or detailing
H2	No analysis or detailing	Detailing only*
H3	Detailing only*	Static analysis & detailing*

* Detailing requirements –

- All parts of the structure to be tied together in the horizontal and vertical planes
- Beam and truss connections 5 per cent of gravity load reaction for H2, 7.5 per cent of gravity load reaction for H3
- Wall anchorage to transmit $10a_s$ kN/m

The structure classification and the product of the acceleration coefficient and the site factor determine a design category, ranging from A to E for general structures and H1 to H3 for domestic structures. This design category determines the type of analysis required by AS 1170.4, as described below.

In addition to the design category, the structure configuration (regular or irregular) and the ductility of the structure influence the requirements. AS 1170.4 describes various features that determine a structure as being irregular.

The overall process involving these various factors is summarised in [Figure 2](#).

Design and detailing requirements become more stringent as the earthquake design category changes from A to E for general structures and from H1 to H3 for domestic structures. A summary of the requirements for unreinforced masonry general structures is given in Table 2 and for domestic structures in Table 3. Detailing only is required for the less severe categories, with an increasing requirement for static or dynamic analysis and full design for the more severe cases.

Table 2 also shows the height limits imposed on unreinforced masonry structures. All loadbearing masonry structures in excess of the limits shown require the use of reinforced masonry for the structural system. In the most severe case (Category D), unreinforced masonry is limited to two storeys. 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 (they are typically in categories A, B, or C).

Design requirements (domestic)

No analysis is required for domestic structures in categories H1 and H2 because the system already in place to resist lateral wind load should provide sufficient wall, floor and roof diaphragms to resist horizontal earthquake loading. Analysis is required for non-ductile construction, such as solid and cavity walls of unreinforced masonry, in earthquake design category H3. The total base shear, however, has been simplified to 15 per cent of the total gravity loads (see Clause 3.4.2 of AS 1170.4).

Torsional effects need not be considered for regular domestic structures (defined as rectangular structures with a ratio of length to width not exceeding 1.2). For irregular domestic structures, torsional effects are catered for by increasing the design load effects by 25 per cent (see Clause 3.4.4 of AS 1170.4).

Detailing requirements (domestic)

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

There are no specific structural detailing requirements for H1 category for all types of construction and H2 category for ductile construction (see Clause 3.2 of AS 1170.4). Nevertheless it is good practice to ensure that all components are tied together.

Structural detailing requirements for non-ductile construction in H2 category and for all construction in H3 category include the following:

- Horizontal resistance must be provided for connections of beams and trusses to their supports (see Clause 3.3.1 of AS 1170.4).

- External walls must be anchored to roofs and floors for horizontal support and internal loadbearing walls must be restrained at their top and bottom (see Clause 3.3.2 of AS 1170.4).

Other measures can be taken to improve horizontal earthquake resistance, for example by incorporating sub-floor braces for discrete footings (see AS 1170.4 Appendix C, Clause C3.1 for guidance). Design of connectors is discussed in [Section 8.3](#).

Attention should be paid particularly to detailing of unreinforced 'non-structural' masonry components, as these 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 restraint to resist a force of $1.8a_s$ times the weight of the component. This is an important requirement applicable to all categories (see Clause 3.5 of AS 1170.4).

5.2 Structural behaviour

5.2.1 Wind loading

Traditionally, masonry structures were massively proportioned to provide stability and prevent tensile stresses. In the period following the Second World War, 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. When combined with the use of concrete floor systems acting as diaphragms, this 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 attach 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.

5.2.2 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. 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. Appendix A of AS 1170.4 provides guidance for the designer on features that result in a structure being classified as regular or irregular.

5. General Design Aspects

5.3 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 the masonry walls as loadbearing elements and those where the masonry forms an infill or a veneer applied to a structural frame.

5.3.1 Housing

The mechanisms of load transfer in masonry housing are different for cavity construction and masonry veneer. The inner leaf of cavity construction 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, that might either be of masonry or framed construction, act to transfer lateral shear forces to the footings.

For masonry veneer construction, the building frame supports all the applied forces (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 roof. Lateral shear resistance is provided by the frame that 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 AS 3700 minimum spacings will usually ensure 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 7.7.2(b)) to span divided by 300.

5.3.2 Framed structures

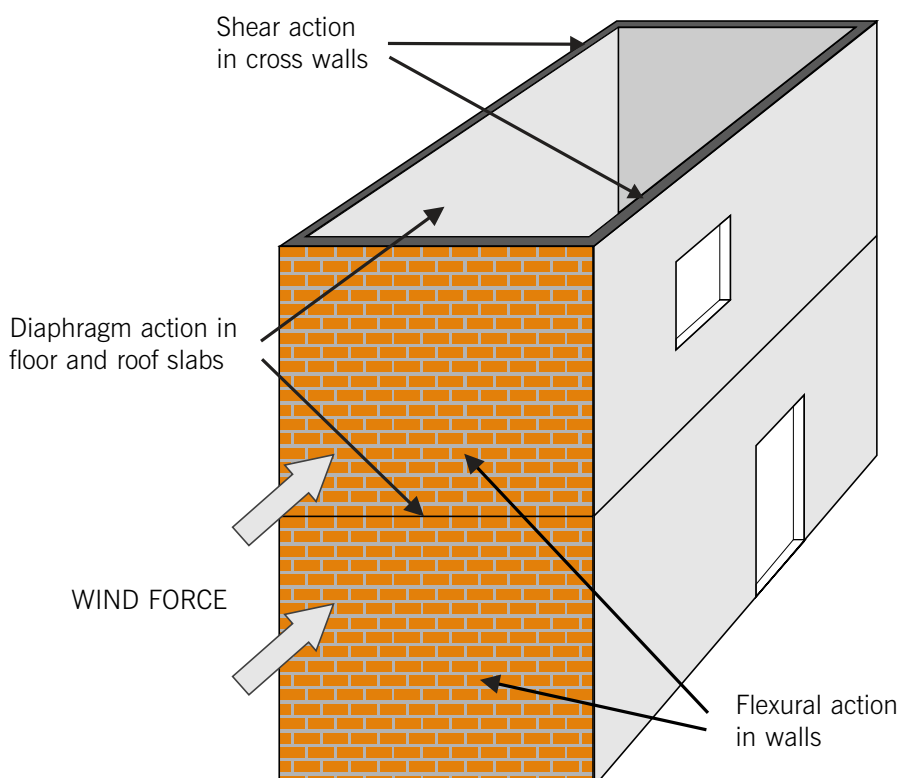
In a framed structure, load is transferred from the face-loaded walls to the framing members through their connections. The framing members then act together to resist the lateral force by sway action or braced-truss action, thereby transferring the force to the foundations. The important elements to 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.

5.3.3 Loadbearing structures

The basic mechanism of lateral load transmission for a loadbearing structure is shown in Figure 3. 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 out-of-plane flexural action, in-plane shear action and connections between structural elements.

Figure 3. Load transmission for a loadbearing structure



5.4 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.
- 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 one direction only, 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 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.

5.4.1 Slab/wall connections

To satisfy the requirements of AS 1170.4 a slab-wall connection must be capable of transmitting a horizontal force of $10aS$ kN per metre length of wall (where a is the acceleration coefficient and S is the

site factor). 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 8.4](#).

5.4.2 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 8.4](#).

5.4.3 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 with a maximum value of 2.0 (see Clause 5.4.2 of AS 1170.4). This is a simplification of what is quite a complex phenomenon.

Grouted and reinforced cavity construction or grouted and 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](#)¹³.

6. Design of Walls for Out-Of-Plane Load

6.1 Introduction

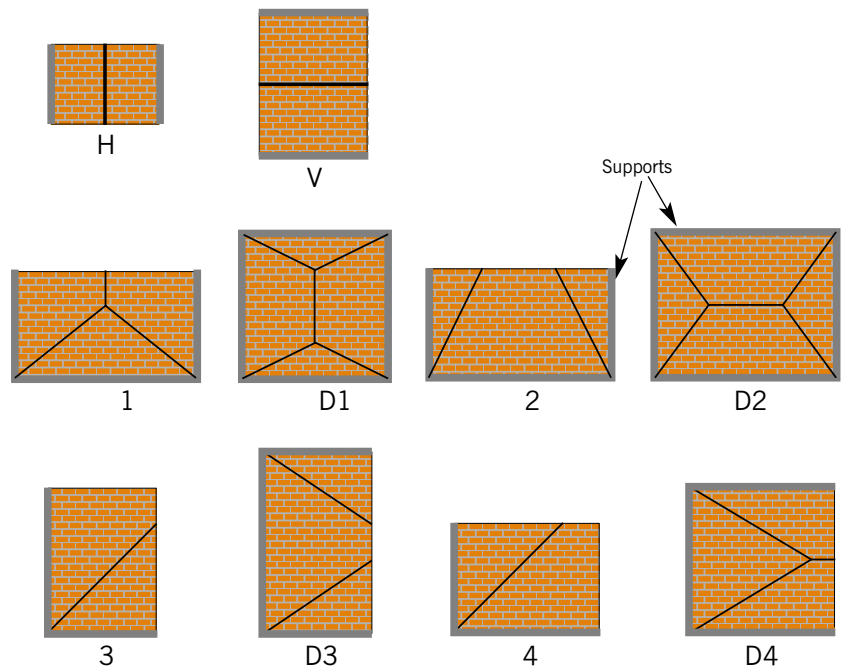
For lateral out-of-plane loading, whether it arises from wind or earthquake, the response of the structural element is calculated by considering the load as an equivalent static uniform pressure. Although there is a fundamental difference in the type of load caused by wind and earthquake, there is no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

Masonry elements subjected to out-of-plane loading resist the loads 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 5.1](#).

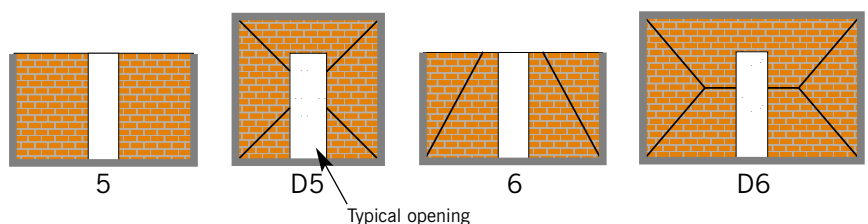
Masonry walls behave differently under simple bending in one direction, for example between top and bottom supports, and bending in two directions, such as when the wall has support on two or more adjacent edges. This is a result of the flexural action of plates under distributed loading and the different 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 4. Idealised crack patterns for various wall configurations

Walls without openings:



Walls with openings:



This manual's design charts give permissible lengths and heights for walls with a range of support conditions and for various wind pressures. Masonry walls crack in particular patterns under face loading, depending on the support conditions and dimensions. [Figure 4](#) shows the idealised cracking patterns that occur for various wall configurations, with edge supports indicated by darker shading. 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.

Two types of edge restraint are considered in the design charts: lateral restraint and full rotational restraint. The latter can be referred to as a fixed edge. The other condition considered is a free edge. The two types of restraint are defined as:

- *Lateral restraint* occurs when the edge of the masonry wall only has 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 incorporated within the wall provide such a case and the bed joint at the base of a wall is considered to be of this type.
- *Edge fixity* occurs when the edge of the wall is restrained against 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. 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. 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.

6.2 One-way vertical bending

The most common form of one-way bending is 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 mechanism results in a brittle failure. When a bed joint crack occurs in masonry acting in this way the joint has 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.

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

Where –

ϕ = Capacity reduction factor (0.6 for bending).

k_{mt} = 1.0 for clay masonry.

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 (usually based on 80 per cent of the dead load).

However the amount of vertical compression force f_d that can be used to enhance the bending resistance is limited to $2\phi k_{mt} f'_{mt}$, 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:

$$M_{cv} = f_d Z_d \dots\dots\dots(2)$$

Where f_d is limited to 0.36 MPa.

The use of this design procedure is illustrated with a worked example in [Section 9.1](#) and a design chart is given in [Section 10.1](#).

6. Design of Walls for Out-Of-Plane Load

6.3 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:

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

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

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 previously. 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 perpend. A third expression given in AS 3700 has the effect of limiting the amount of vertical compression force f_d , used to enhance the bending resistance, to f'_{mt} , that is 0.2 MPa for most masonry.

The use of this design procedure is illustrated with a worked example in [Section 9.2](#) and a design chart is given in [Section 10.2](#).

6.4 Two-way bending

6.4.1 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. Each of these cracking patterns divides the wall panel into a number of sub-plates that 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 now adopted in the [AS 3700 design provisions](#)¹⁴.

Some idealised cracking patterns are shown in [Figure 4](#). 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 horizontal 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 from at or near the corner. These cracks form in the bed and perpendicular joints and their angle is therefore governed by the length-to-height dimensions of the masonry units. The 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 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.

6.4.2 Virtual work method

The lateral load design method introduced in the revised 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 in 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 that 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 perpendiculars or from the stepped shearing failure alternating between the mortar joints and perpendiculars. 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 perpendiculars. 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 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 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 that 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 –

$$f'_t = \text{The equivalent torsional strength} = 2.25 \sqrt{f'_{mt}}$$

$$Z_t = \text{The equivalent torsional section modulus, measured normal to the diagonal crack line, as calculated by the expressions given in AS 3700 (Clause 7.4.4).}$$

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 4](#)). 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.

6. Design of Walls for Out-Of-Plane Load

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

Application of 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_d = The predicted lateral load capacity of the panel.

a_f = The aspect factor depending on the geometry.

L_d = Design length (see below).

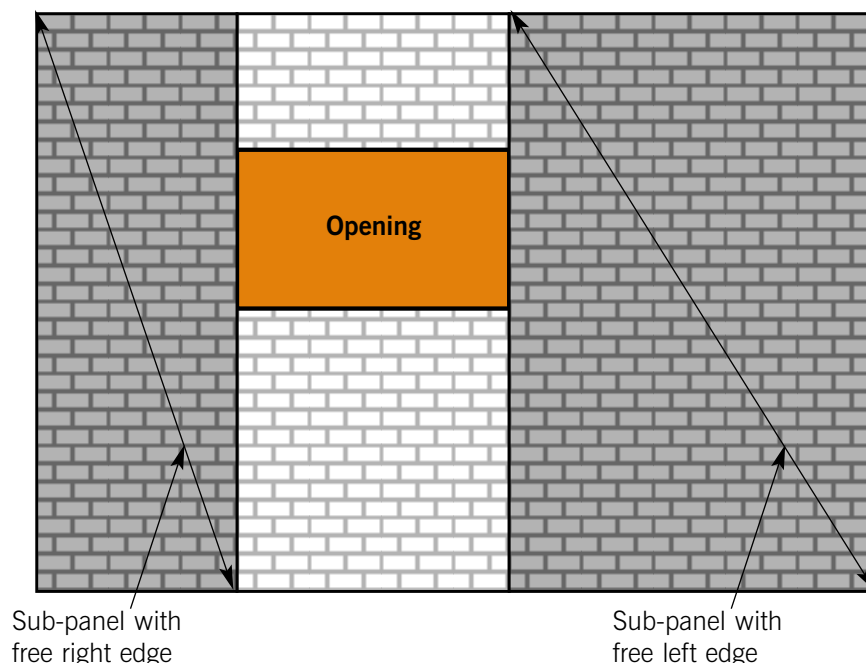
k_1 & k_2 = Coefficients depending on the edge restraints and the geometry.

M_{ch} and M_{cd} are as defined previously.

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 5. 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. However when designing using the charts in this manual, it is not required to evaluate these other factors.

Design charts

The equations given above are used as the basis for the design charts. Charts for walls without openings assume no rotational restraint at the sides. Those for walls with openings are given for cases with and without rotational restraint at the sides. 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 height for a range of loadings and a particular set of support conditions. The following procedure is followed in using the design charts:

- Determine the type of panel to be used (the type of masonry and its properties).
- Determine the support conditions.
- Calculate the face load on the panel from the wind and earthquake codes.
- Use the appropriate chart to find the design height and design length. (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).

The use of this design procedure with the aid of the design charts is illustrated with a worked example in [Section 9.3](#). The charts are given in [Section 10.3](#).

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.

6.5 Veneer walls

The veneer in a veneer wall 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. Usually this will not be critical for the maximum tie spacings specified in AS 3700.

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 6.4](#)). 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 8.2](#)).

A typical example of design for a veneer wall on a flexible backup is shown in [Section 9.4](#).

6.6 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 must be assessed and each leaf designed individually as set out in [Section 6.4](#).

Strength of the wall ties is no longer covered by deemed-to-satisfy provisions of AS 3700 and must always be justified ([see Section 8.2](#)).

A typical example of cavity wall design, where it behaves as a veneer on a stiff backup, is shown in [Section 9.5](#).

7. Design of Walls for In-Plane Load

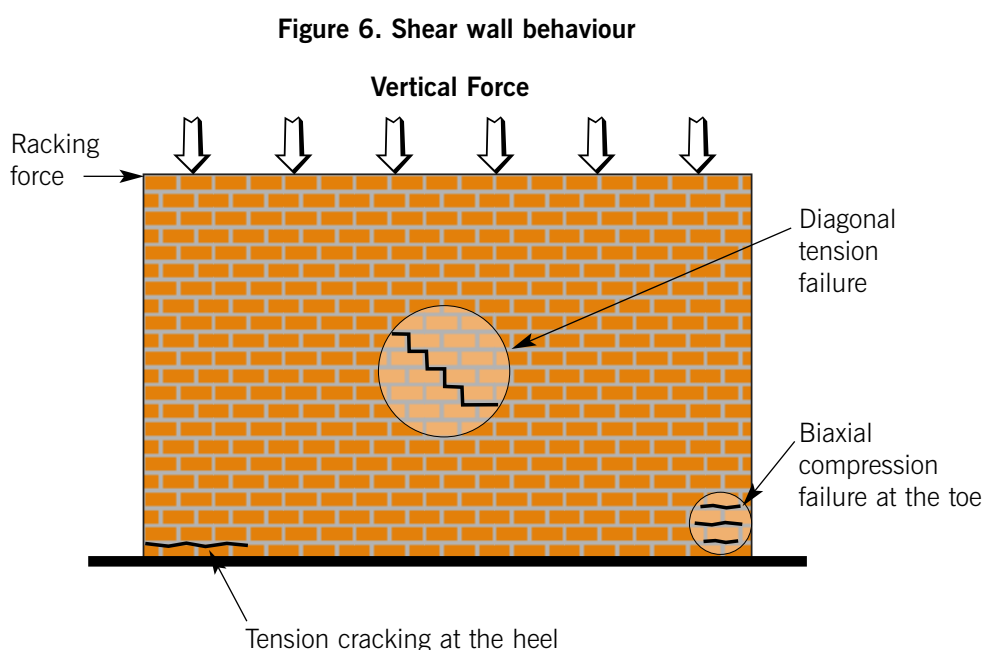
7.1 Introduction

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 5.1](#).

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. Consideration of this local state of stress is necessary if local and progressive failures are to be predicted.

[Figure 6](#) 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 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 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.

7.2 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:

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

Where—

- V_d = The design shear force.
- ϕ = Capacity reduction factor (0.6 for shear in unreinforced masonry).
- f'_{ms} = Characteristic shear strength of the masonry (see Section 4.3).
- A_{dw} = Bedded area.
- k_v = Shear factor (0.3 for bed joints in clay masonry).
- f_d = Simultaneously-acting design compressive stress on the bed joint (not greater than 2 MPa) – based on the non-removable dead load and taken as 80 per cent of the full dead load.

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

For earthquake loading, the shear design equation becomes –

$$V_d \leq \phi f'_{ms} A_{dw} + 0.9k_v f_{de} A_{dw} \dots\dots\dots(8)$$

Where –

- f_{de} = The design compressive stress on the bed joints under 'gravity load'.

The other symbols are as before.

The difference with this expression is that the superimposed compressive stress providing the friction component is derived using the same gravity load that is used to calculate the earthquake (inertia) force. That is, the load considered is the dead load plus a proportion of the live load (see AS 1170.4). The friction component is further reduced by 10 per cent to allow for upward acceleration that might be present in an earthquake.

As well as checking for shear failure on the bed joints using the above expressions, it is necessary to 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 8.4](#)).

A typical example of shear wall design is shown in [Section 9.6](#).

8. Design of Wall Ties, Connectors and Joints

8.1 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 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.

The earthquake loading code AS 1170.4 requires all roofs to be positively attached with a system capable of transmitting a horizontal force of 5 per cent to 7.5 per cent of the gravity load (depending on the category). 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.

8.2 Wall tie design

Unlike the previous version of AS 3700, the 1998 edition contains rational provisions for design of wall ties. It no longer gives deemed-to-satisfy spacings for strength. The overall maximum spacings are 600 mm horizontally and vertically and the first row of ties is required to be within 250 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 spacings, the ties must be designed to resist the applied forces. A full analysis to determine the tie forces, that 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 per cent 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.

Table 4. Maximum wall pressures (kPa) for Type A ties at 600 mm centres

Construction	Tension			Compression		
	Light duty	Medium duty	Heavy duty	Light duty	Medium duty	Heavy duty
Veneer with flexible backup (2.4 m wall height)	0.94	1.88	4.69	1.09	2.19	5.63
Veneer with stiff backup	0.58	1.15	2.88	0.67	1.35	3.46
Cavity wall (both leaves supported)	0.75	1.50	3.75	0.88	1.75	4.50

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 250 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 4](#) 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.

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.

Typical examples of wall tie design are shown in [Sections 9.4 and 9.5](#).

8.3 Design of connectors

Connectors used to tie masonry walls to frames and other supporting elements must be designed to resist 125 per cent 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 A3.5). 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 2975. Connectors must also comply with the durability requirements ([see Section 8.5](#)). 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 they must be rated at least medium duty, spaced at no more than 200 mm horizontally and 300 mm average (400 mm maximum) vertically. The vertical joint must be filled with mortar. Other connectors of equivalent strength 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 7.5.3. 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.

8.4 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.

The earthquake loading code AS 1170.4 requires that, for general structures of design category B, walls must be restrained at each floor level where the floor is relied on to provide lateral support to the wall. This provision of the code was modified in Amendment 1 from the previous requirement that positive anchors should be provided at each floor. It is therefore now possible to rely on the friction capacity of the wall/floor connection to provide the necessary restraint. Anchors must still be used to connect the roof to the walls and to connect both roofs and floors for design category C, D and E structures. Domestic structures also require positive connections at intermediate floor levels. Typical roof connection details are shown in [Section 11](#).

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 7.2](#)) 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 –

$$V_d \leq 0.9k_v f_{de} A_{dw} \dots\dots\dots(9)$$

Where –

k_v = Shear factor

f_{de} = The design compressive stress under 'gravity load' ([see Section 7.2](#))

A_{dw} = Bedded 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](#)¹⁵ and [dynamic loading](#)¹⁶. 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.

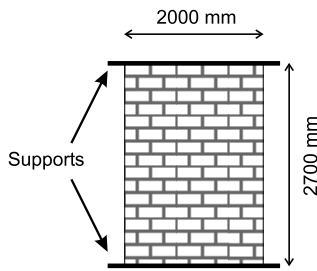
8.5 Durability of ties and connectors

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.

All ties and connectors used with masonry are required to meet durability requirements set out in AS 3700. These requirements are expressed in terms of a classification of exposure environments, ranging from mild to severe marine, and explanatory material is given in AS 3700 Appendix E. The accessories are given a rating from R0 to R5, depending on their materials and other characteristics, and a given exposure environment requires an appropriate durability rating. Pending completion of a revision of the standard for ties and accessories AS 2699, the performance requirements and deemed-to-satisfy provisions for durability of ties and connectors are given in AS 3700 Appendix F.

There are many reports of ties deteriorating markedly after only a short time in the structure, the most notable recent example being after the [Newcastle earthquake](#)¹⁷. In recent years, stainless steel ties have become more common and polymer ties have been developed. The revised version of AS 2699 will contain clearly stated performance requirements and a method of test for corrosion resistance of metal ties. These measures, combined with an increased awareness by designers of the importance of tie durability, should lead to fewer problems in the future.

9. Worked Examples



9.1 One-way vertical bending

Example: 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 –

$$Z_d = \frac{bt^2}{6} = \frac{1000 \times 110^2}{6} = 2.02 \times 10^6 \text{ mm}^3 \text{ per metre width}$$

Density for clay masonry = 0.19 kN/m² per 10 mm thickness (AS 1170.1).

Therefore, compressive stress at mid-height due to self-weight (based on 80 per cent dead load) –

$$f_d = \frac{0.8 \times [(0.19 \times 11) \times 1.35] \times 1000}{110 \times 1000} = 0.021 \text{ MPa}$$

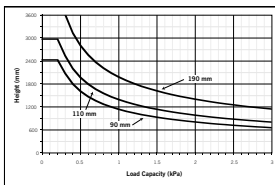
Vertical bending capacity –

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

$$= [0.6 \times 1.0 \times 0.2 \times 2.02 \times 10^6 + 0.021 \times 2.02 \times 10^6] \times 10^{-6} = 0.28 \text{ kN.m per metre width}$$

For two leaves of 110 mm masonry (assumed to share the load equally) –

Moment capacity = 0.56 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 10.1) –

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

9.2 One-way horizontal bending

Example: 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 –

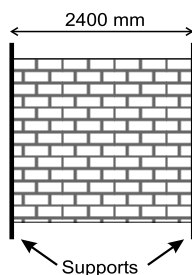
$Z_d = 2.02 \times 10^6 \text{ mm}^3 \text{ per metre width}$ (see Example 9.1),

and Z_u , Z_p are 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) –

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

$$= 1.0$$



9. Worked Examples

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

Horizontal bending capacity –

$$M_{ch} = 2.0 \phi k_p \sqrt{f'_{mt}} Z_d$$

$$= (2.0 \times 0.6 \times 1.0 \times \sqrt{0.2} \times 2.02 \times 10^6) \times 10^{-6}$$

$$= 1.08 \text{ kN.m per metre width (Equation (3))}$$

Or –

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

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

$$M_{ch} = 0.6 \times (0.44 \times 0.8 \times 2.02 \times 10^6 + 0.56 \times 0.2 \times 2.02 \times 10^6) \times 10^{-6}$$

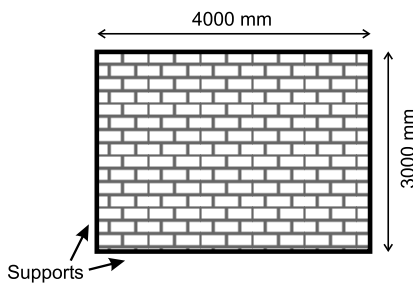
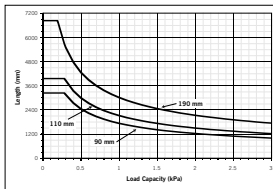
$$= 0.56 \text{ kN.m per metre width (Equation (4))}$$

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 10.2) –

Assume a 110 mm wall.

For a length of 2.4 m, the load capacity is 0.78 kPa. \therefore OK



9.3 Two-way bending (single-leaf wall)

Example: 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 $L_d = 2000$ mm (half the actual length).

Design height $H_d = 1500$ 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 7.4 –

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

Restraint factors R_{f1} and R_{f2} are both 0, therefore –

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

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

$M_{ch} = 0.56$ kN.m per metre width (see Example 9.2).

Equivalent torsional strength –

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

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 –

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

$$= \frac{2 \times 69.9^2 \times 110^2}{(3 \times 110 + 1.8 \times 69.9)} \left/ \left((230 + 10) \sqrt{1 + 0.717^2} \right) \right. = 878 \text{ mm}^3 \text{ per mm crack length}$$

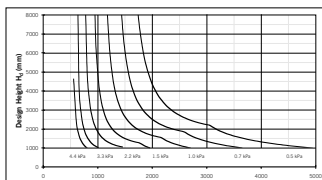
Diagonal moment capacity –

$$M_{cd} = \phi f'_t Z_t = (0.6 \times 1.01 \times 878 \times 1000) \times 10^{-6} = 0.53 \text{ kN.m per metre crack length}$$

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 10.3.1](#) 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

9.4 Veneer wall

Example: 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.

Design tie force based on 20 per cent 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 = 0.9 (AS 3700 Table 4.1).

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

$$\therefore \text{Tie capacity} = 0.9 \times 0.6 = 0.54 \text{ kN} \quad \text{OK}$$

The top row of ties must be within 250 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 250 mm of the edge. Ties in the remainder of the wall are spaced at 600 mm in both directions.

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

9.5 Cavity wall

Example: Design a cavity wall 3 m long and 3 m high for a multi-storey building. The inner leaf is tied to a structural frame on all four sides 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 7.8.4).

9. Worked Examples

The force to be transmitted by the ties derives from the outer leaf and the design tie force is based on 130 per cent of the load on a tributary area for one tie.

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 = 0.9 (AS 3700 Table 4.1).

Load capacity for medium duty ties in compression = 0.70 kN (AS 3700 Table 3.5).

Therefore the tie capacity = $0.9 \times 0.7 = 0.63 \text{ kN}$, which is greater than the applied force of 0.47 kN. \therefore OK

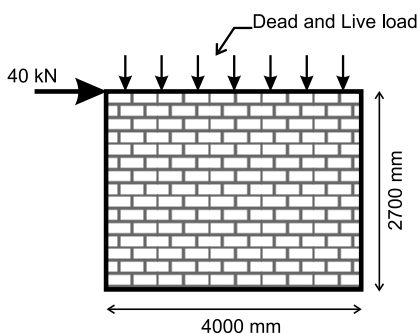
Ties throughout the wall are spaced at 600 mm in both directions, with the first row located within 250 mm of all edges, supports and around openings.

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 of 2.2 kPa (Chart 10.3.1). This is greater than the total applied load of 1.5 kPa. \therefore OK

A more refined design could be carried out by assessing the load distribution between the two leaves but this approach is beyond the scope of this manual.

9.6 Shear wall

Example: 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 –

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

Bedded area –

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

Therefore, shear bond capacity –

$$V_0 = \phi f'_{ms} A_{dw} = (0.6 \times 0.25 \times 0.44 \times 10^6) \times 10^{-3} = 66.0 \text{ kN}$$

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 80 per cent of dead load) –

$$f_d = 0.8 \times \left(\frac{50 \times 10^3}{110 \times 1000} \right) = 0.36 \text{ MPa}$$

Therefore, shear friction capacity –

$$V_1 = k_v f_d A_{dw} = (0.3 \times 0.36 \times 0.44 \times 10^6) \times 10^{-3} = 47.5 \text{ kN}$$

Total shear capacity $V_d = V_0 + V_1 = 66.0 + 47.5 = 113.5 \text{ kN}$, which is greater than the applied shear force of 40 kN. \therefore OK

Sliding at the base of wall is acceptable because the shear factor is the same (0.3) and additional frictional resistance is provided by self-weight.

Check for crushing at the toe –

Density = 0.19 kN/m² per 10 mm thickness (AS 1170.1)

Stress due to compression (using 1.25 times dead load, including self-weight, plus 40 per cent live load)

$$= 1.25 \times \left(\frac{50 \times 10^3 + 2.7 \times 0.19 \times 11 \times 10^3}{110 \times 1000} \right) + 0.4 \times \left(\frac{15 \times 10^3}{110 \times 1000} \right) = 0.69 \text{ MPa}$$

Overturning moment due to wind = 40 × 2.7 = 108 kN.m

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

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

Therefore, net stress at the toe = 0.69 + 0.37 = 1.06 MPa compression

Allowing for a Phi factor of 0.45, this requires f'_{mb} of 2.4 MPa, which is satisfied by units of strength 5 MPa and M2 mortar (see AS 3700 Table 3.1).

∴ OK

Check for tension at the heel –

Stress due to compression (using 80 per cent dead load, including self-weight)

$$= 0.8 \times \left(\frac{50 \times 10^3}{110 \times 1000} + 2.7 \times 0.019 \right) = 0.40 \text{ MPa}$$

Therefore, net stress at the heel = 0.40 – 0.37 = 0.03 MPa compression. ∴ OK

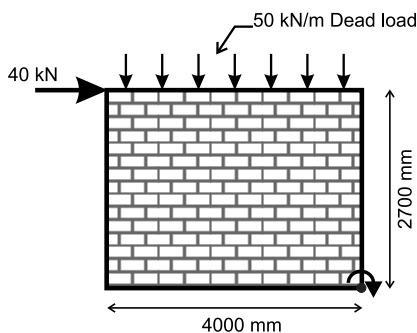
Check for overturning about the toe –

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

Force resisting overturning (using 80 per cent dead load, including self-weight) –

$$= 0.8 \times (50 \times 4.0 + 4.0 \times 2.7 \times 11 \times 0.19) = 178 \text{ kN}$$

Therefore resisting moment = 178 × 2.0 = 356 kN.m and this is greater than the overturning moment. ∴ OK



10. Design Charts

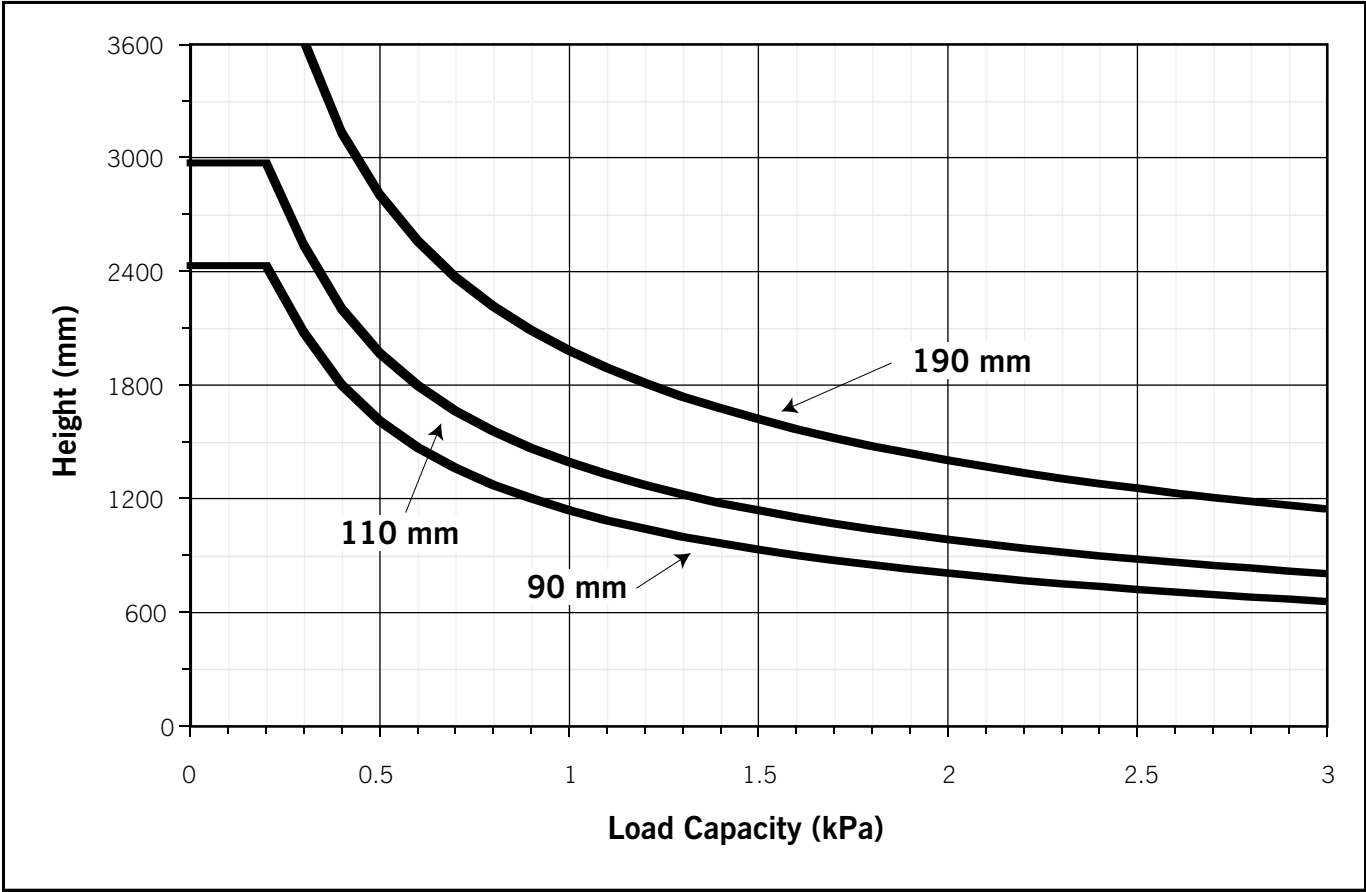
Limitations for all charts:

- They apply to single-leaf walls without engaged piers.
- 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).
- 30 mm face-shell bedding is assumed for the 190 mm units.
- Robustness cut-offs for one-way bending are for other than slab loading; robustness for two-way bending should be checked separately.
- All supports are simple (without rotational restraint) unless otherwise indicated.

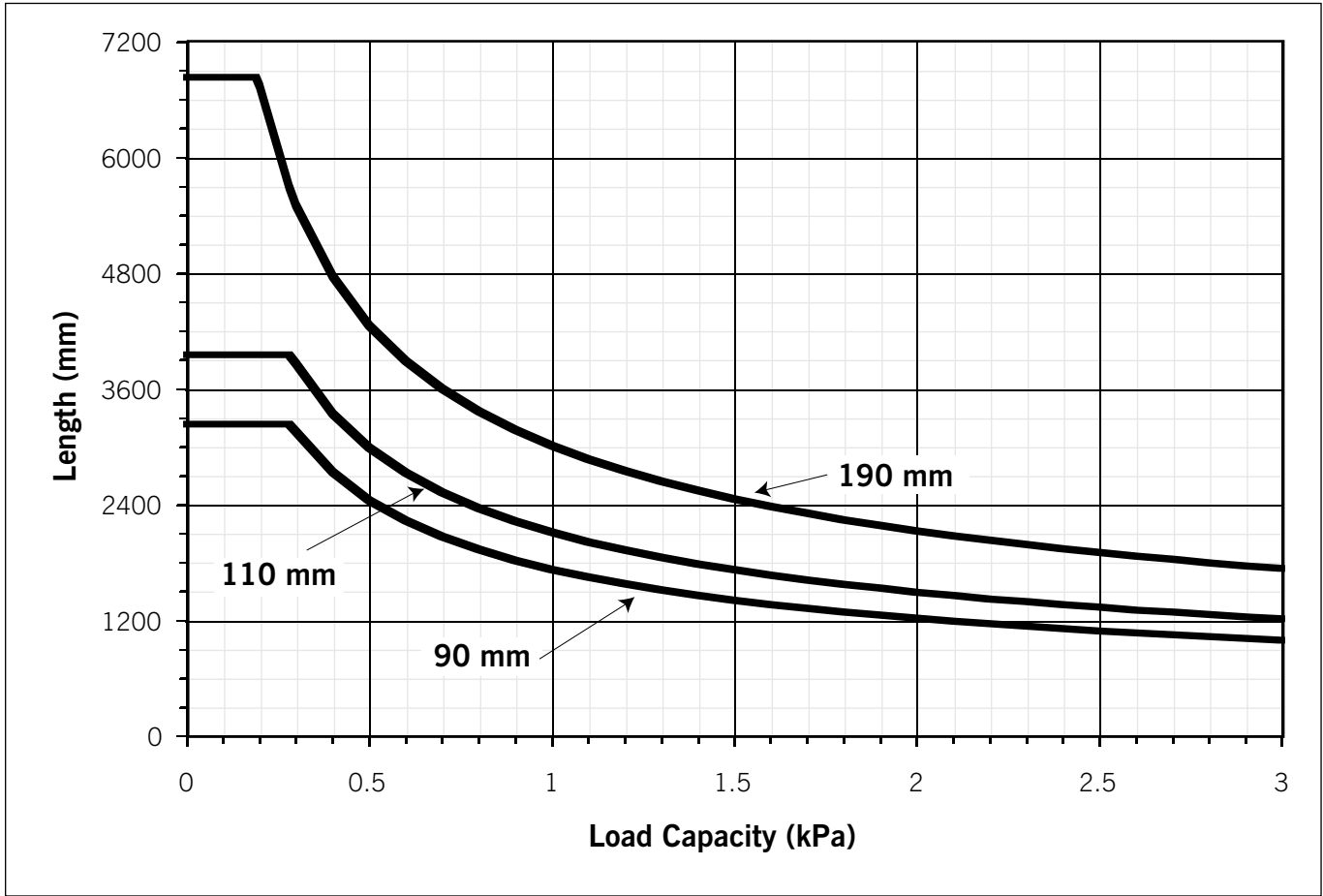
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10.1 One-way vertical bending



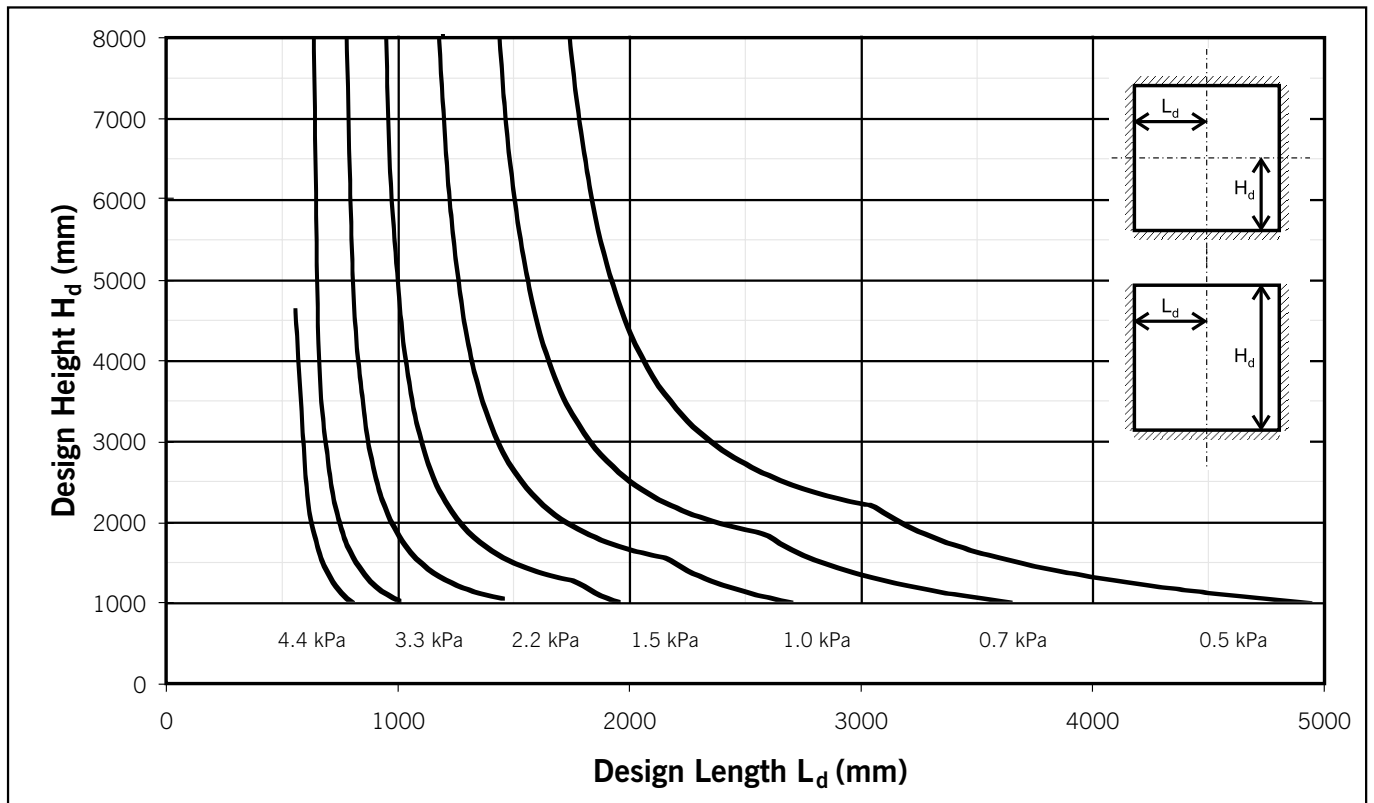
10.2 One-way horizontal bending



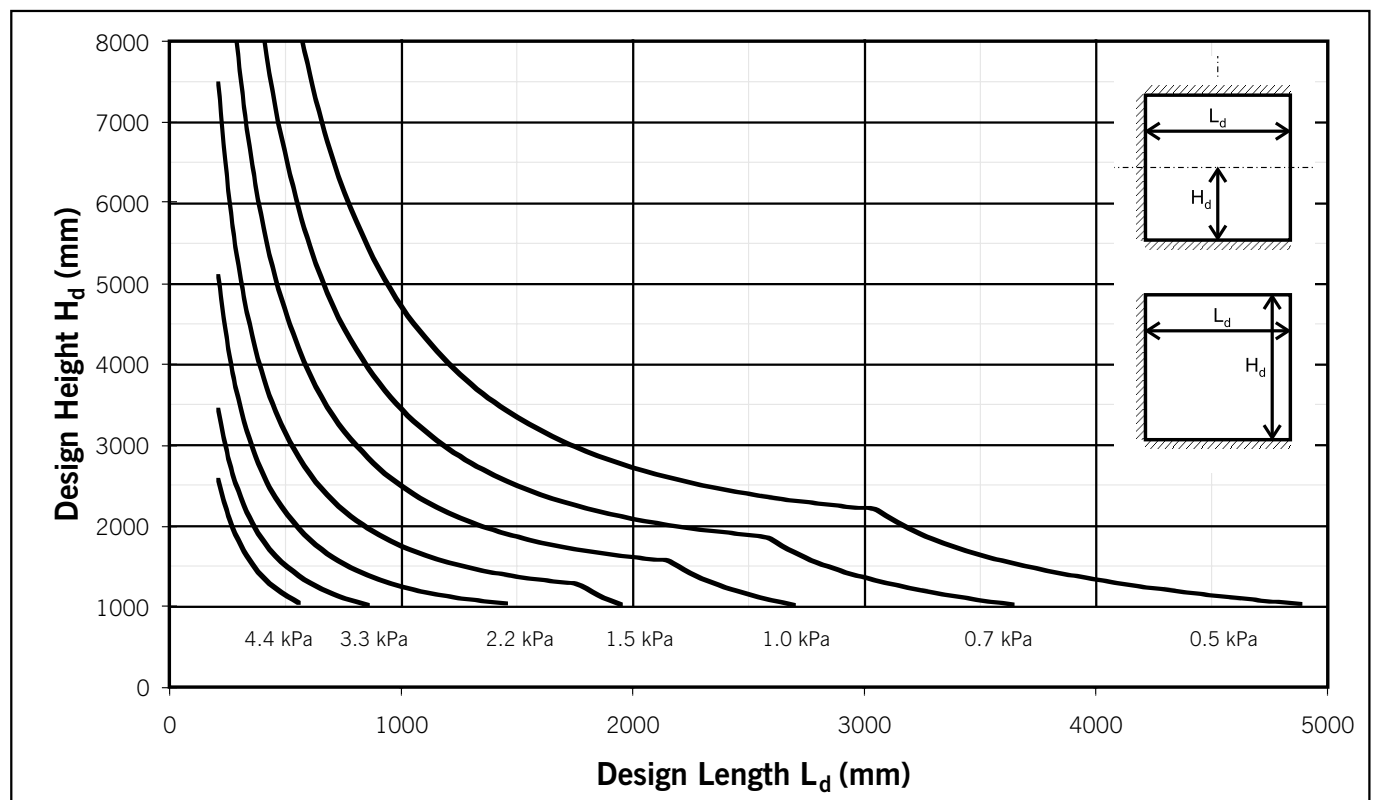
10.3 Two-way bending

10.3.1 110 mm without openings

Both sides supported

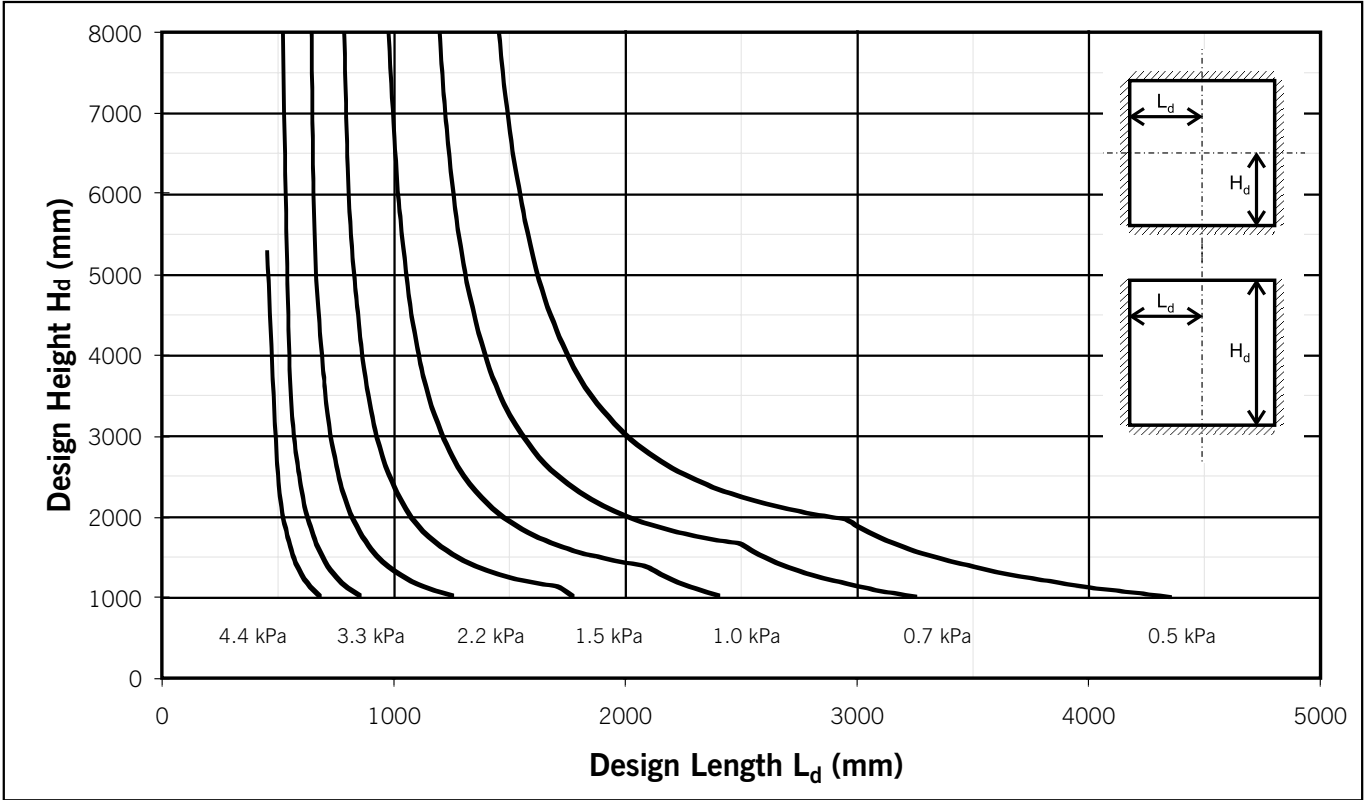


One side supported

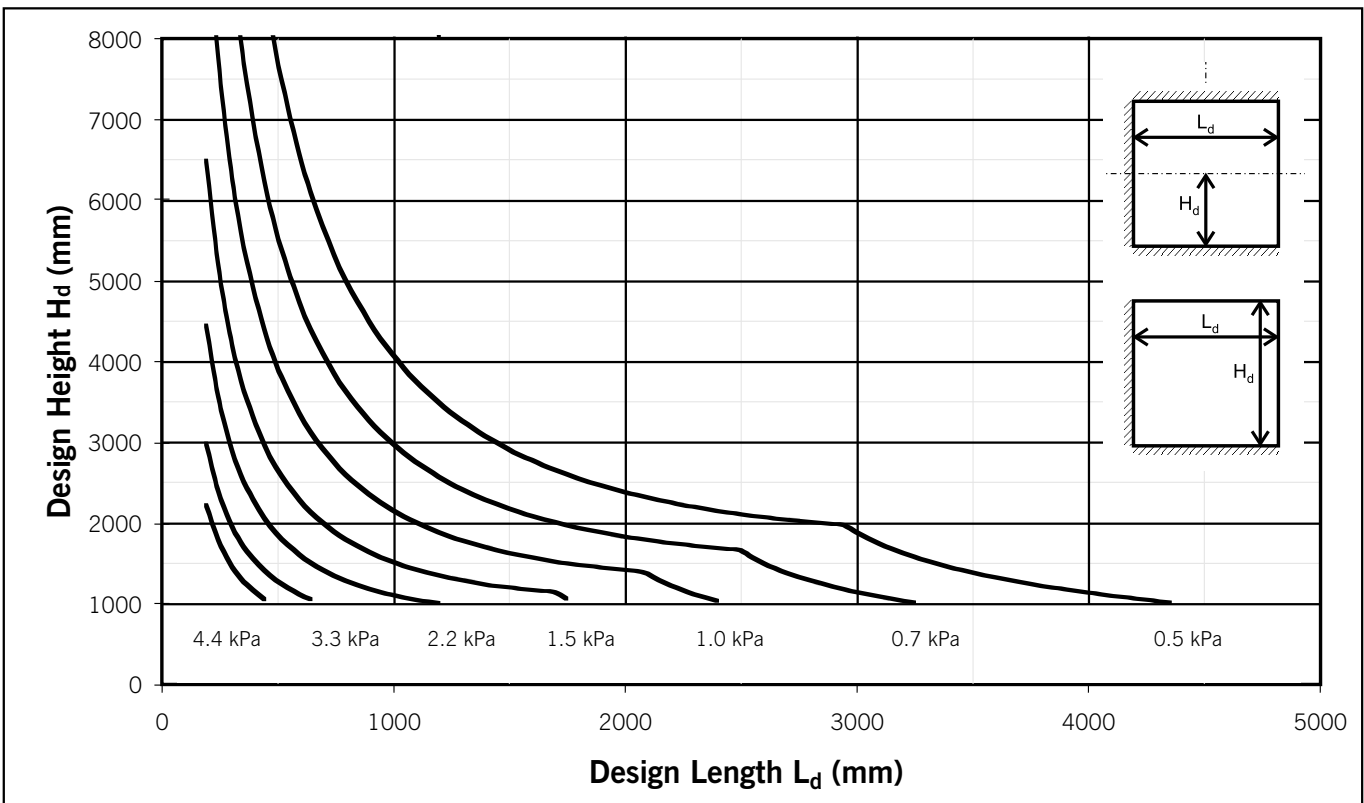


10.3.2 90 mm without openings

Both sides supported

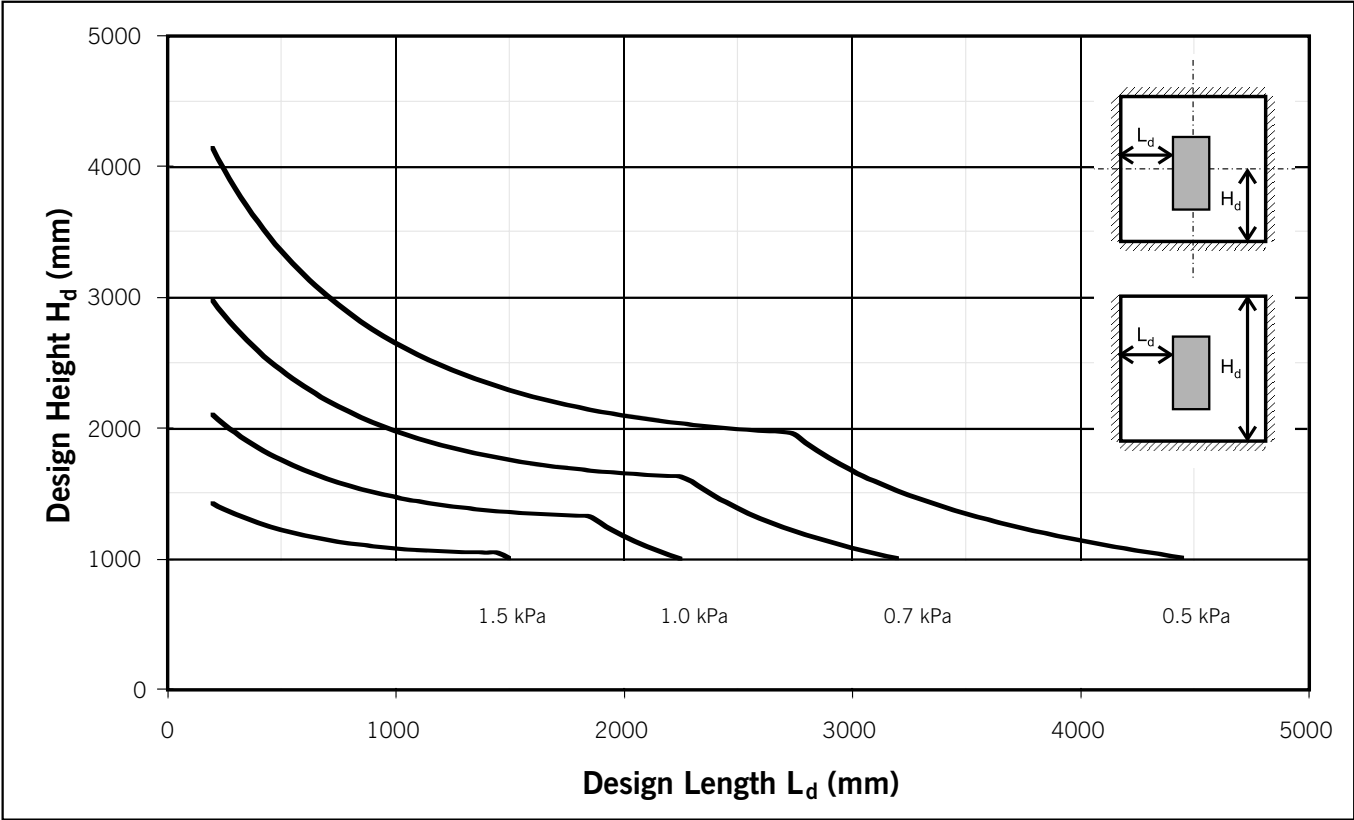


One side supported

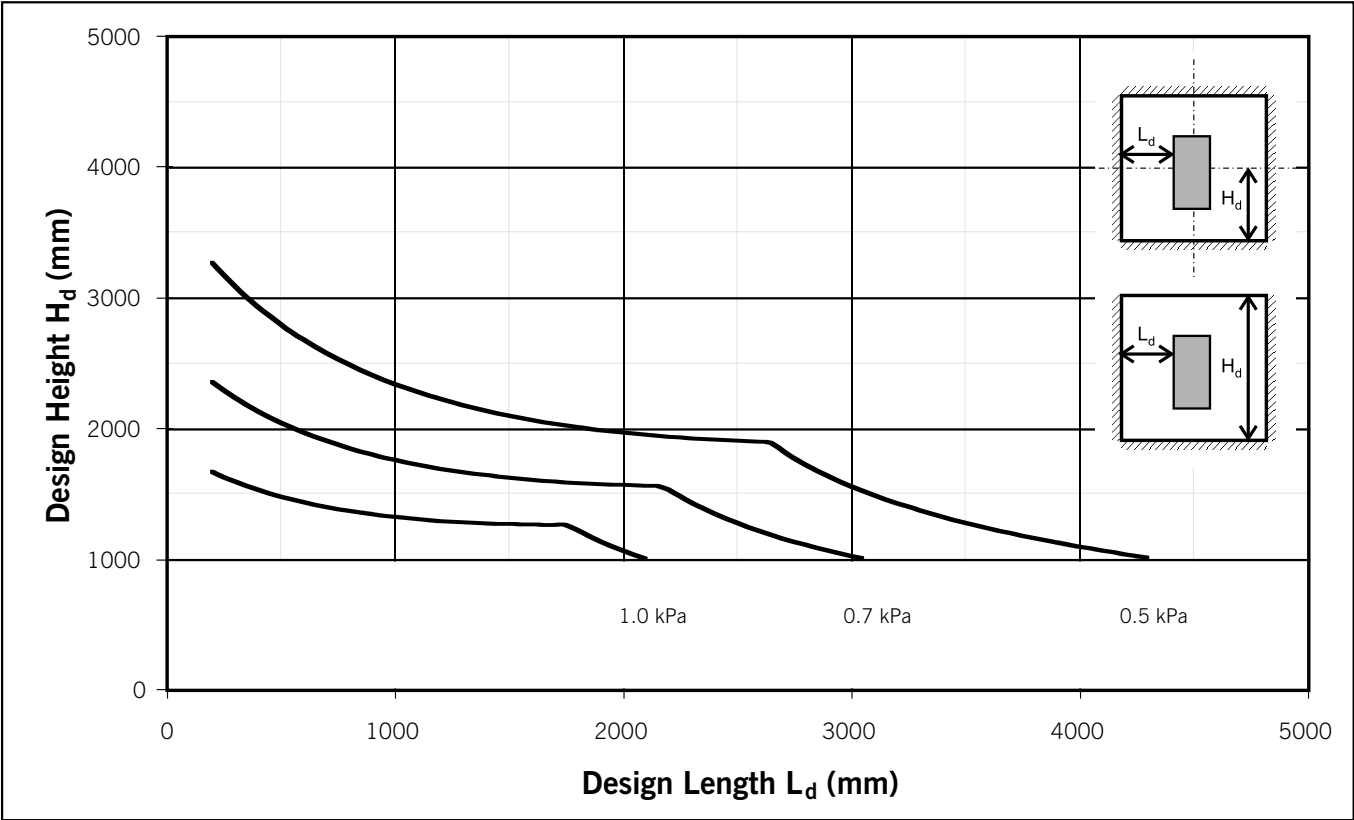


10.3.3 110 mm with openings (no rotational restraint at sides)

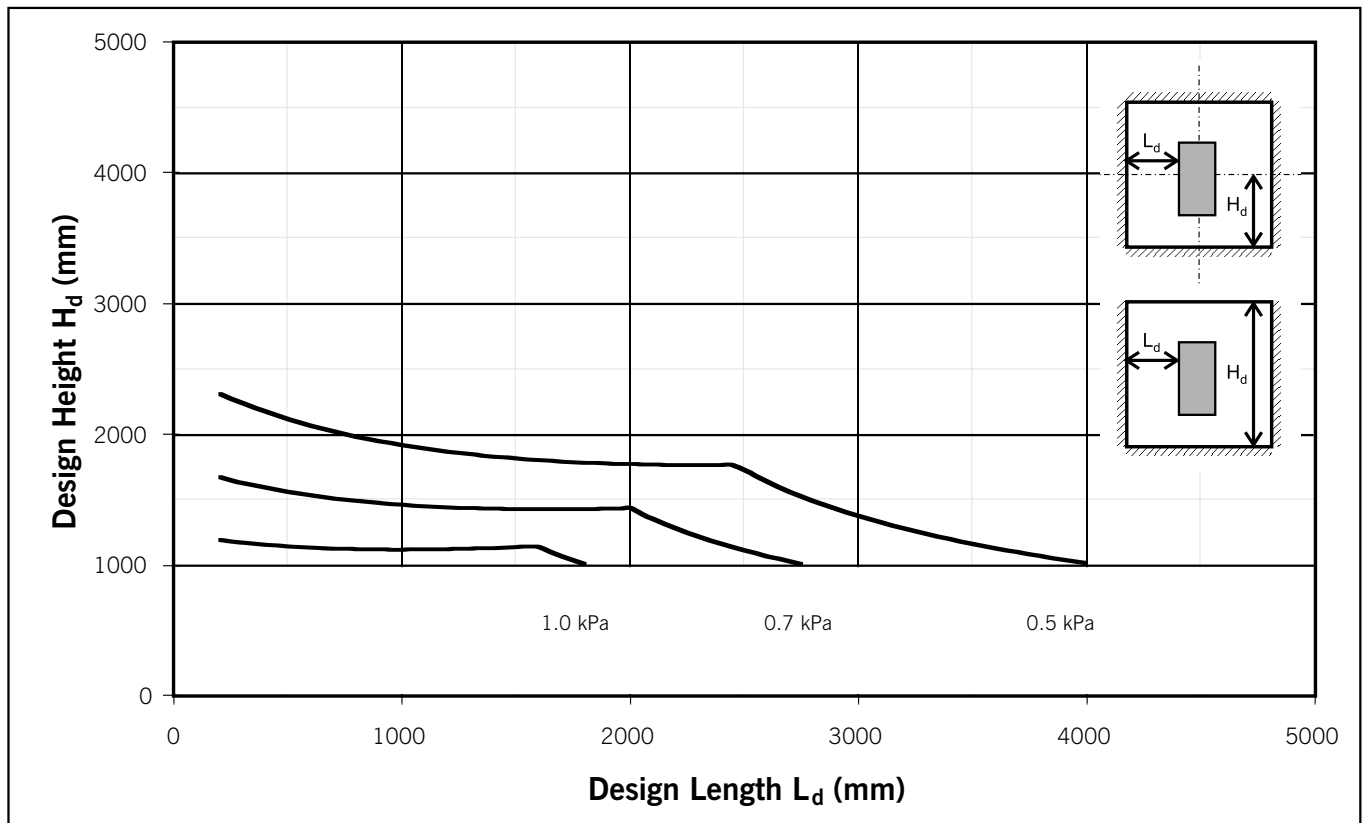
900 mm wide opening (any height), both sides supported



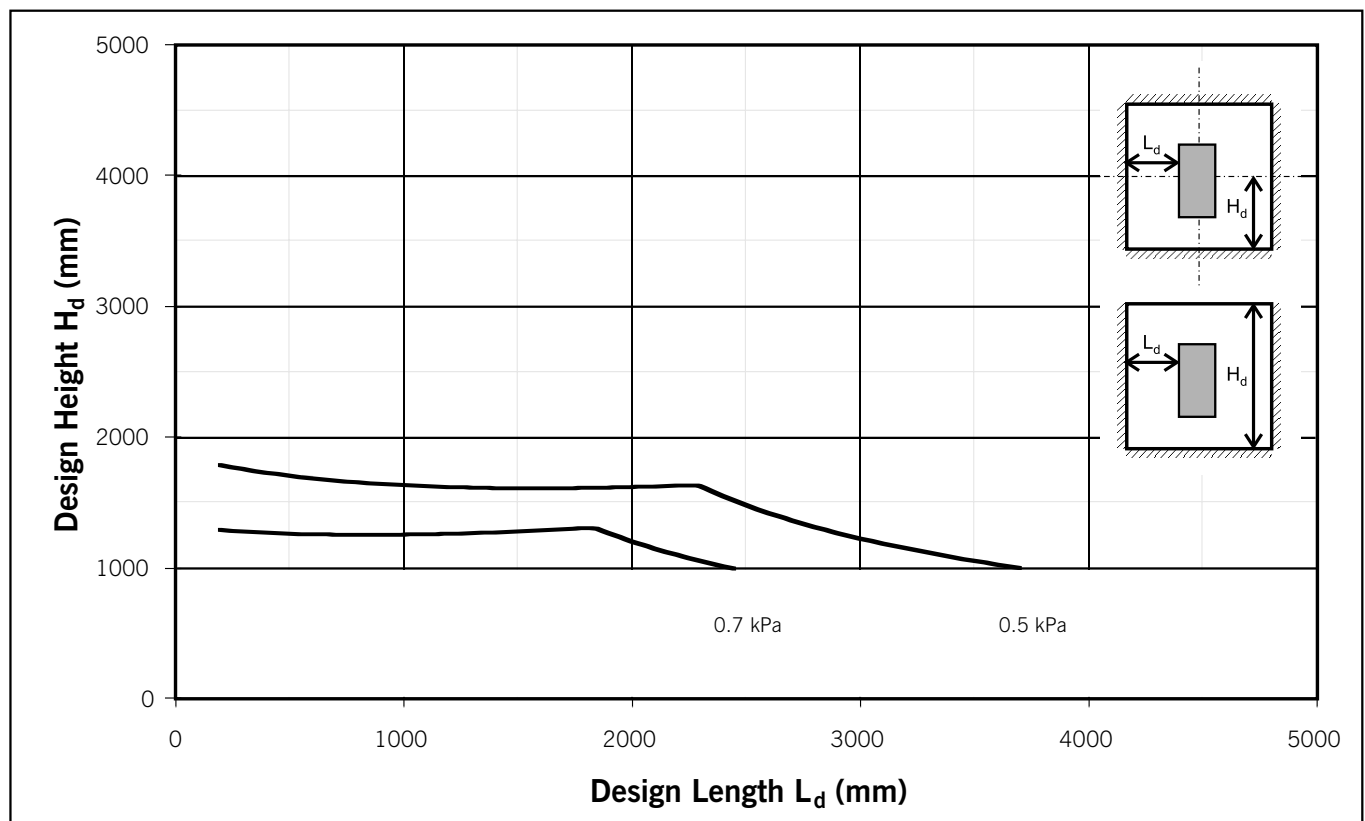
1200 mm wide opening (any height), both sides supported



1800 mm wide opening (any height), both sides supported

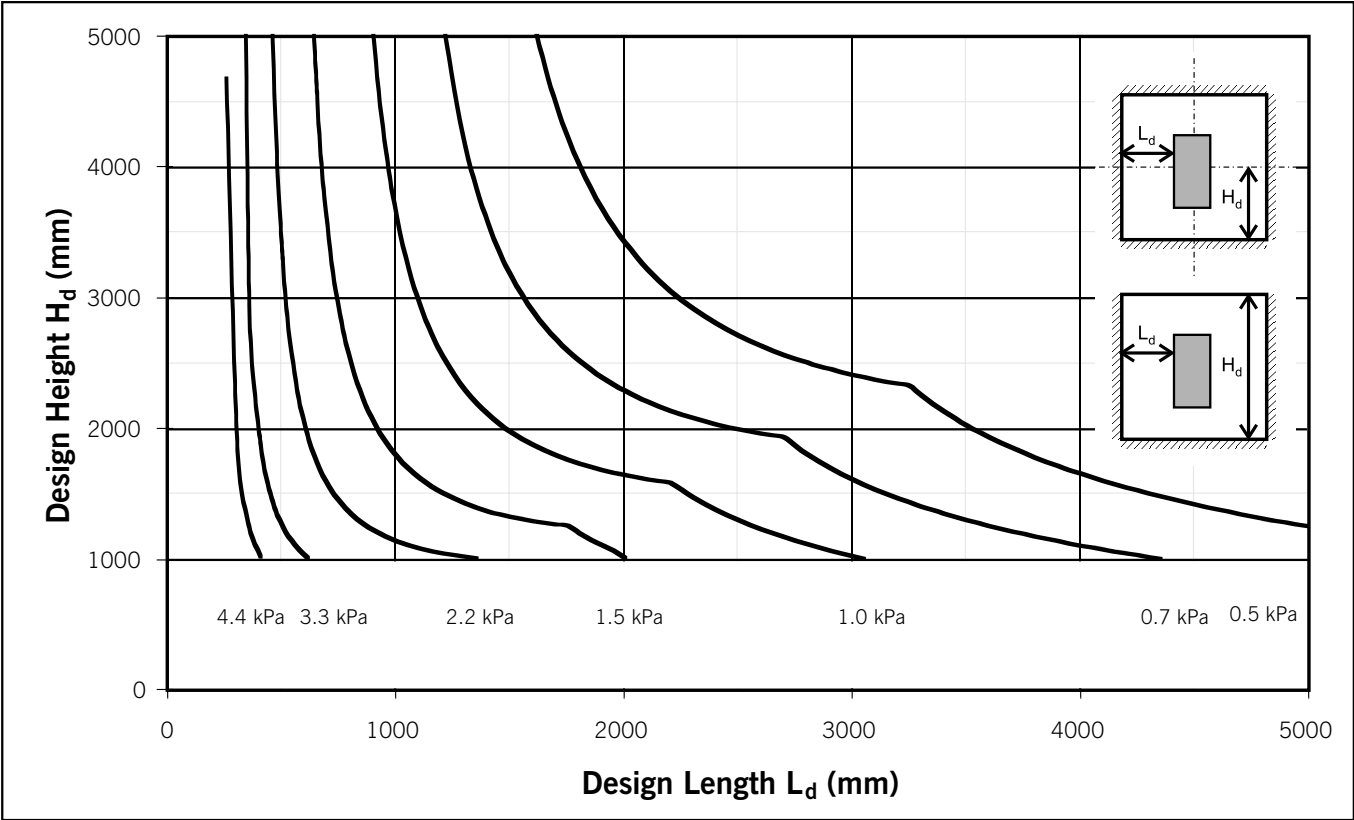


2400 mm wide opening (any height), both sides supported

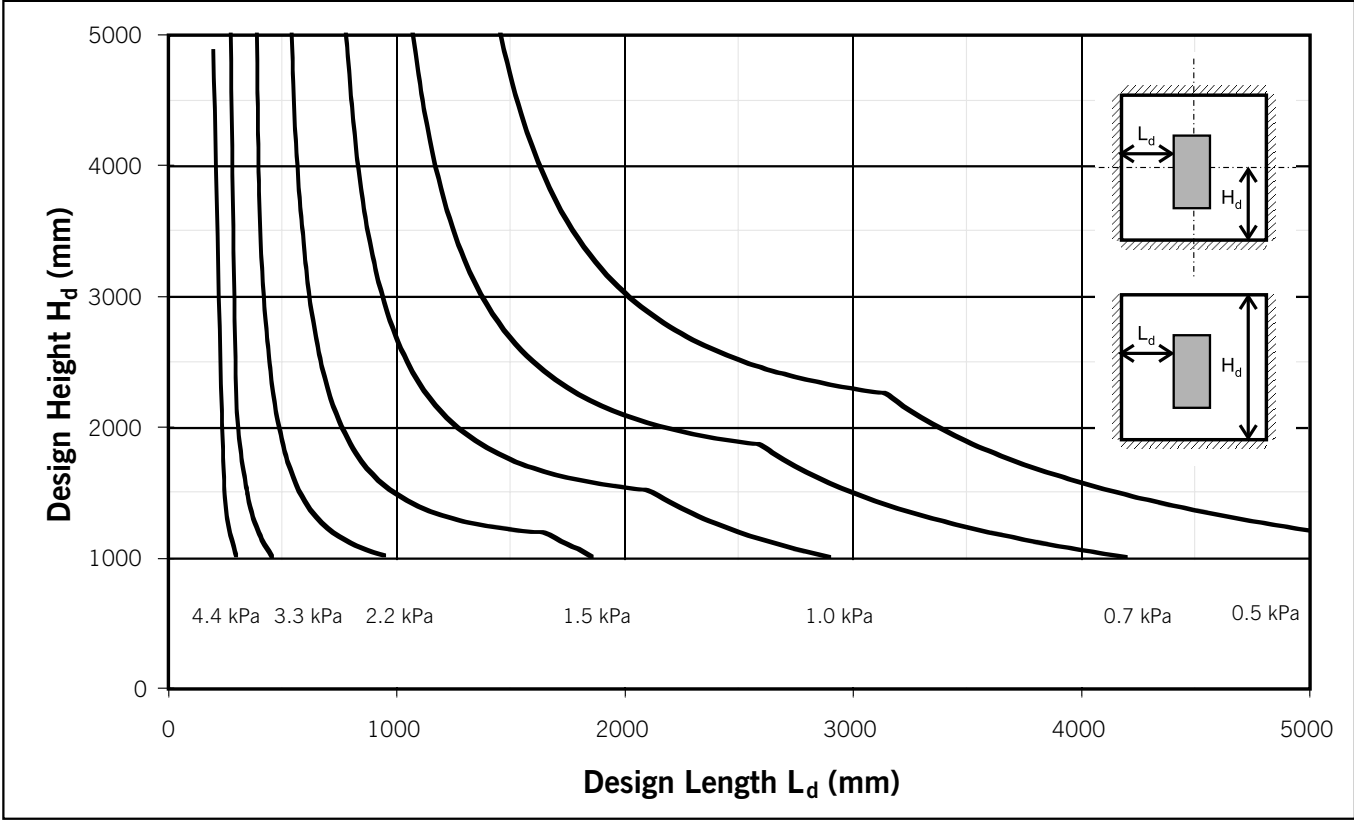


10.3.4 110 mm with openings (rotational restraint at sides)

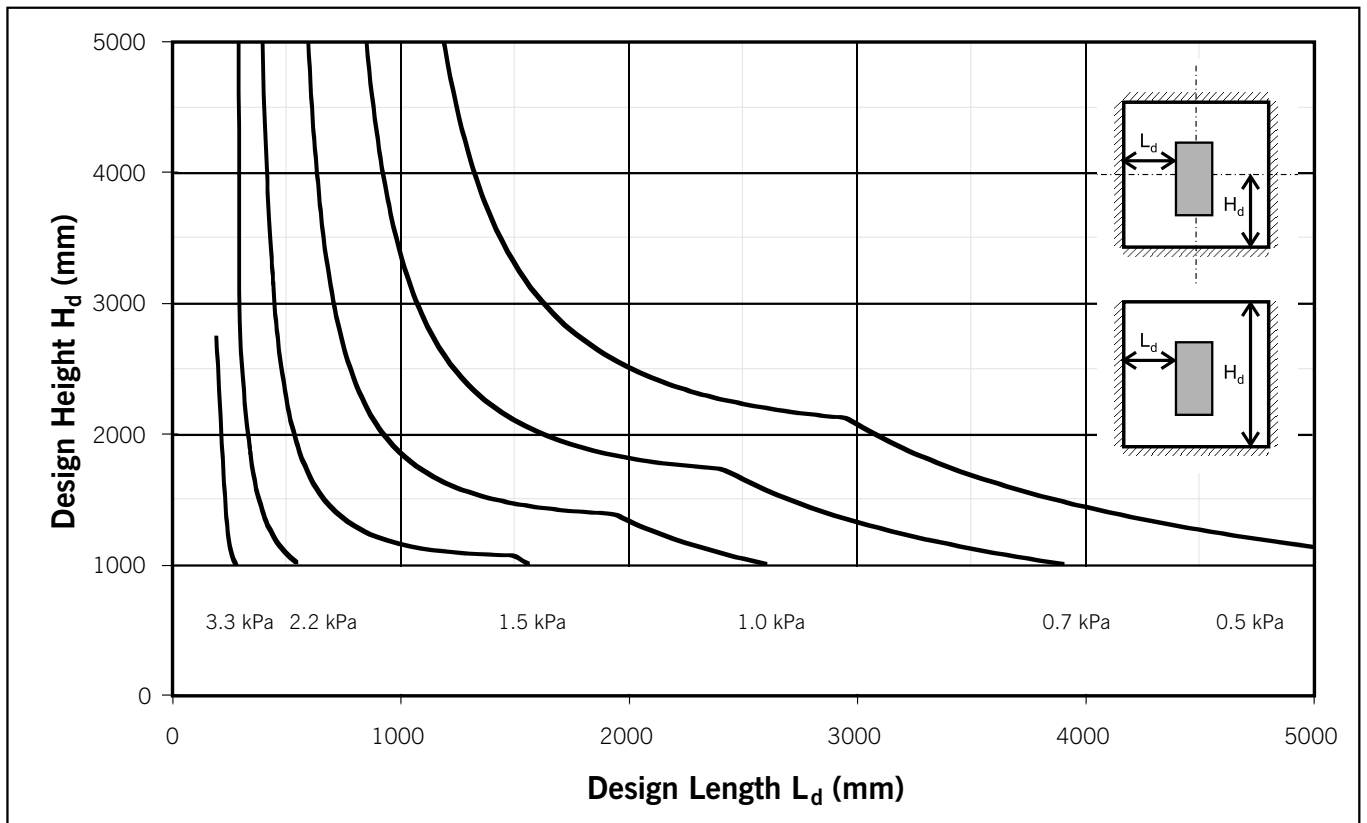
900 mm wide opening (any height), both sides supported



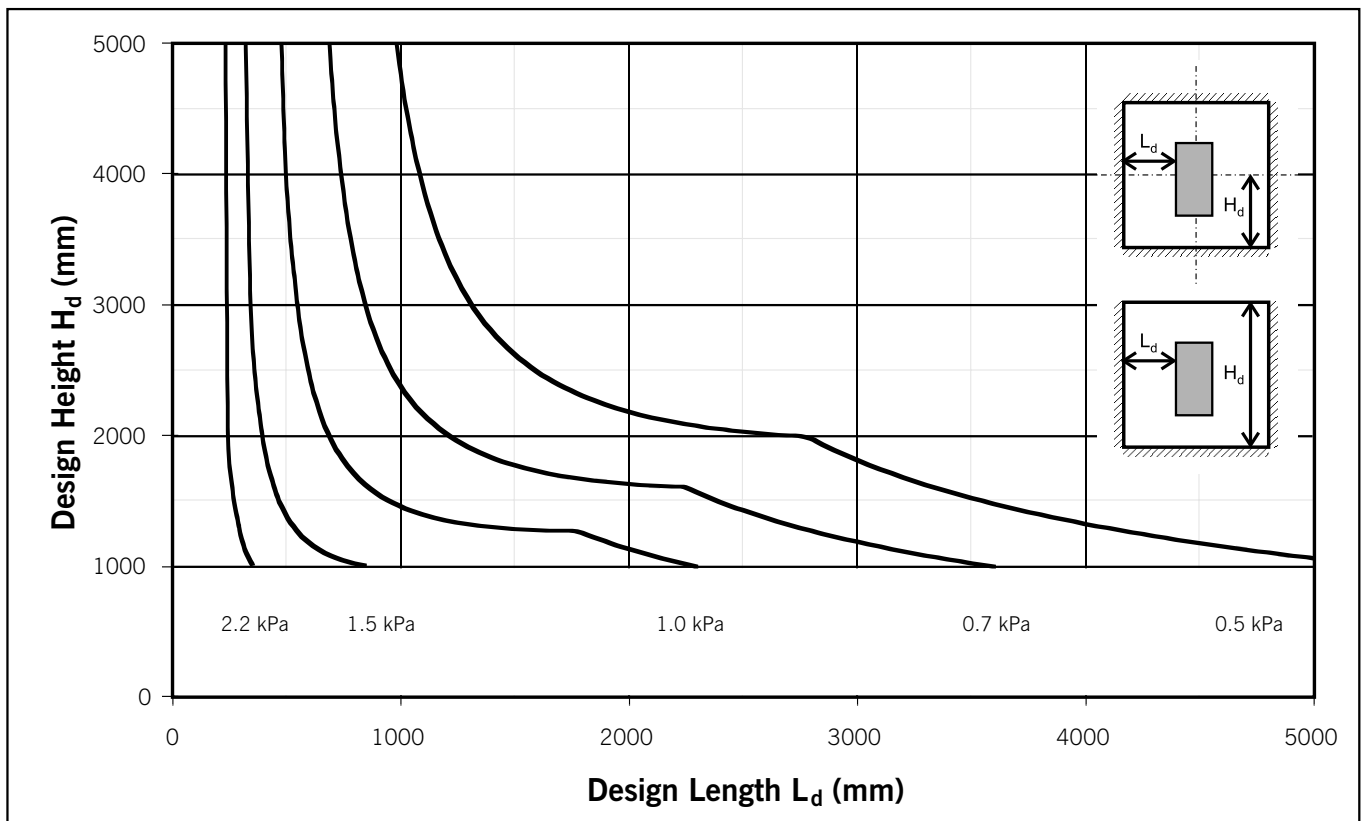
1200 mm wide opening (any height), both sides supported



1800 mm wide opening (any height), both sides supported

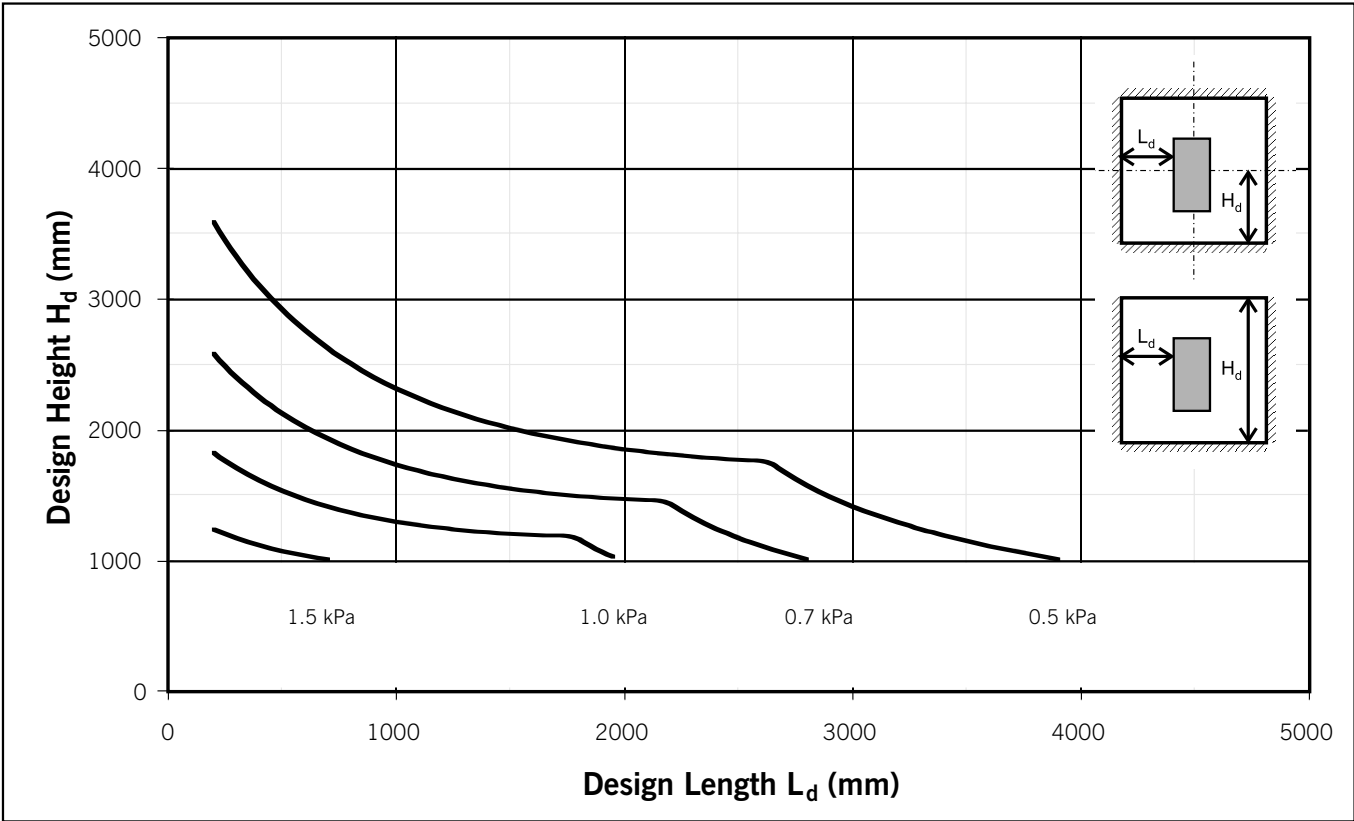


2400 mm wide opening (any height), both sides supported

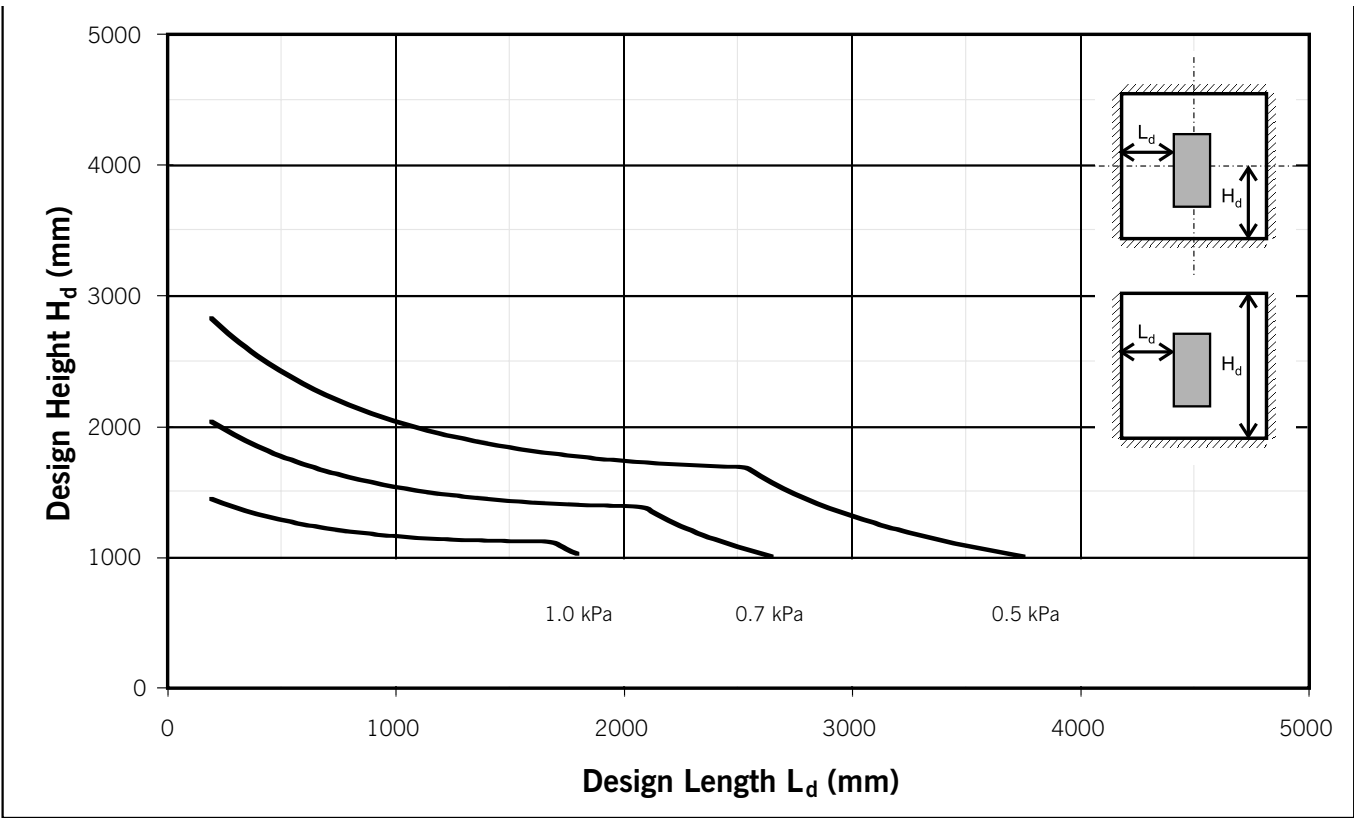


10.3.5 90 mm with openings (no rotational restraint at sides)

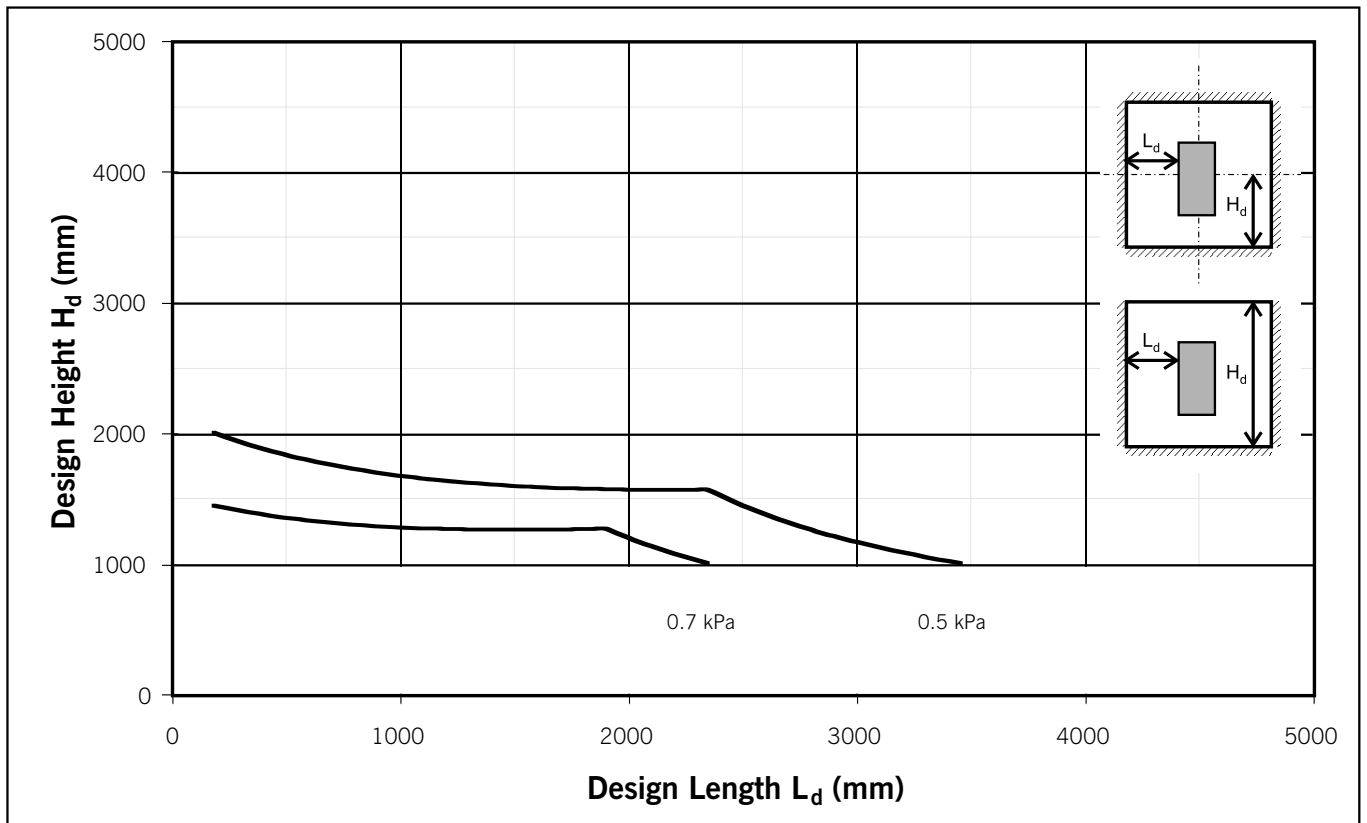
900 mm wide opening (any height), both sides supported



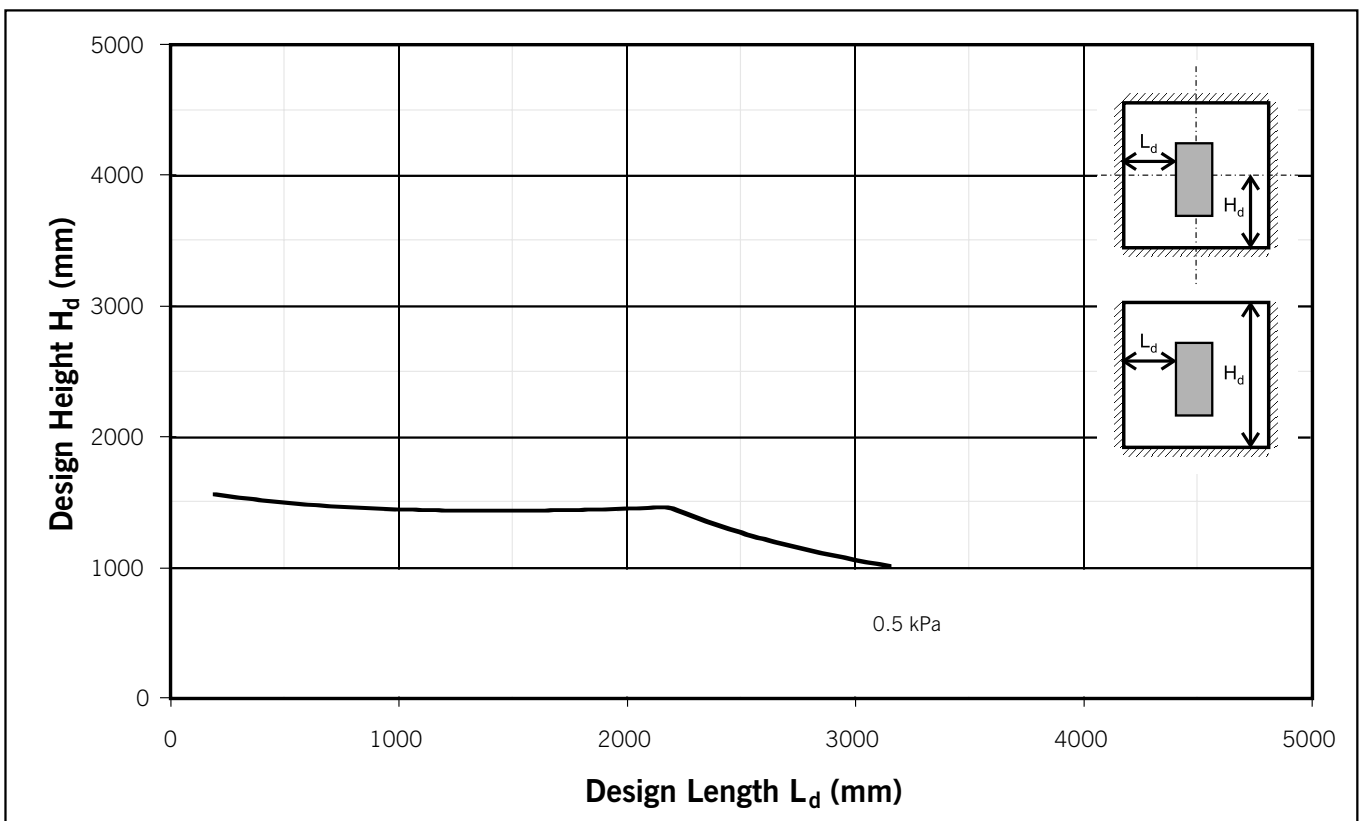
1200 mm wide opening (any height), both sides supported



1800 mm wide opening (any height), both sides supported

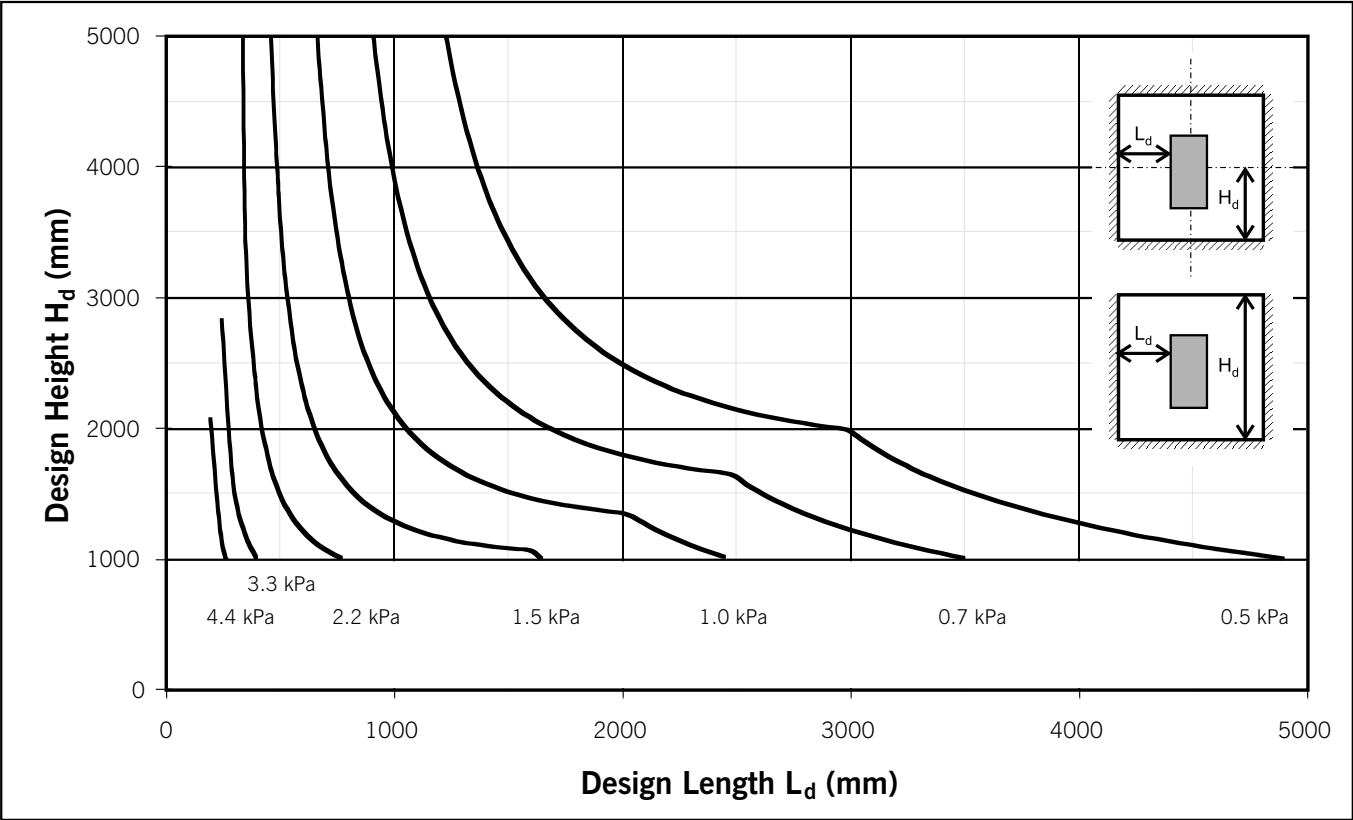


2400 mm wide opening (any height), both sides supported

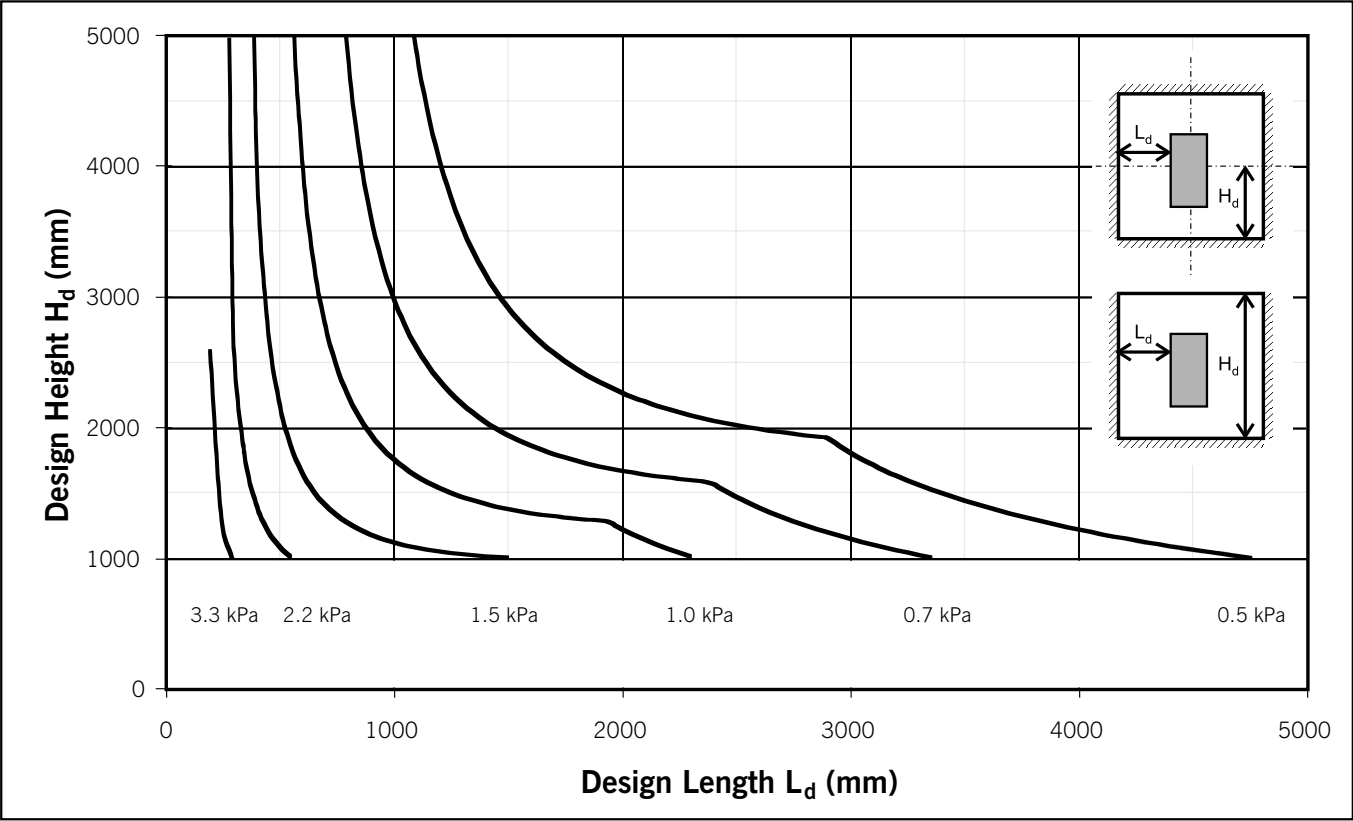


10.3.6 90 mm with openings (rotational restraint at sides)

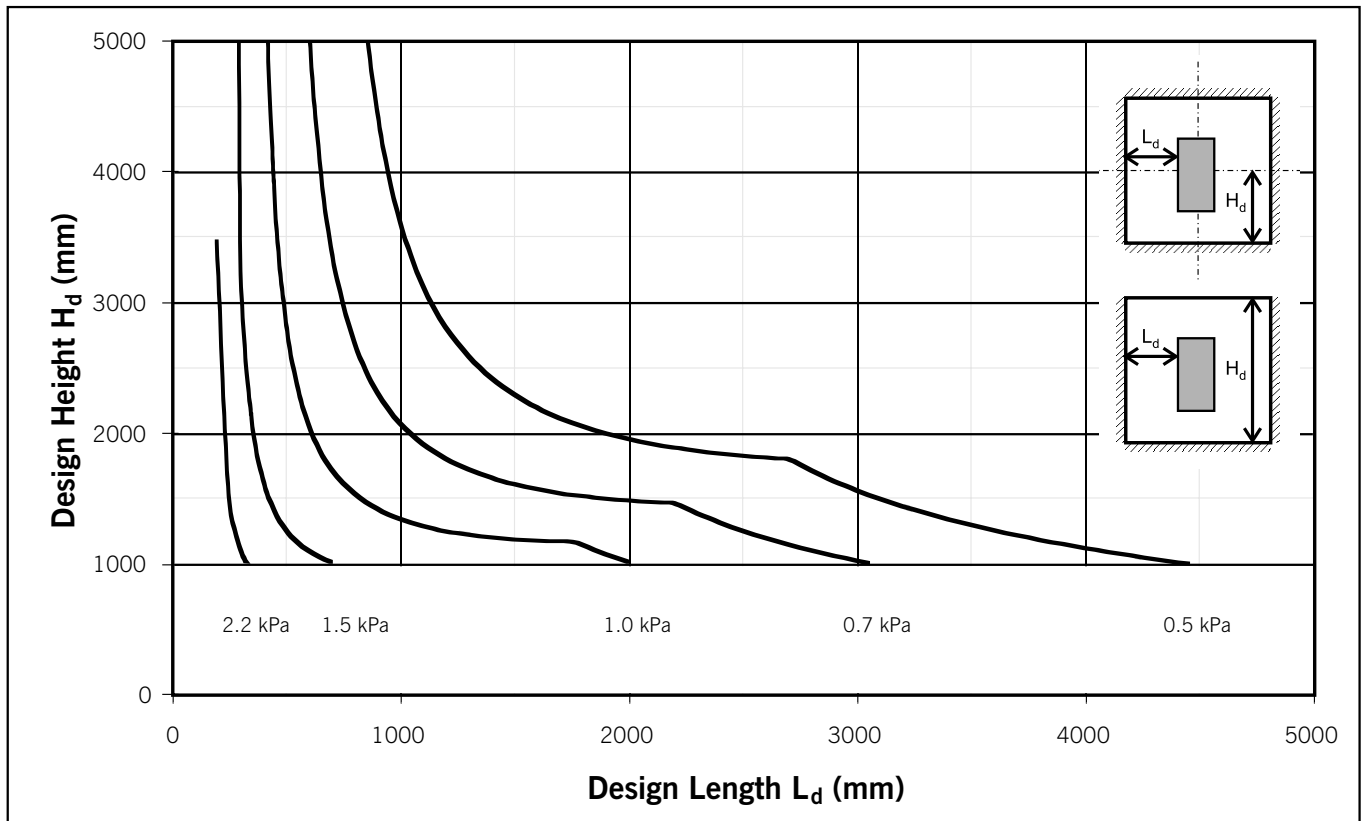
900 mm wide opening (any height), both sides supported



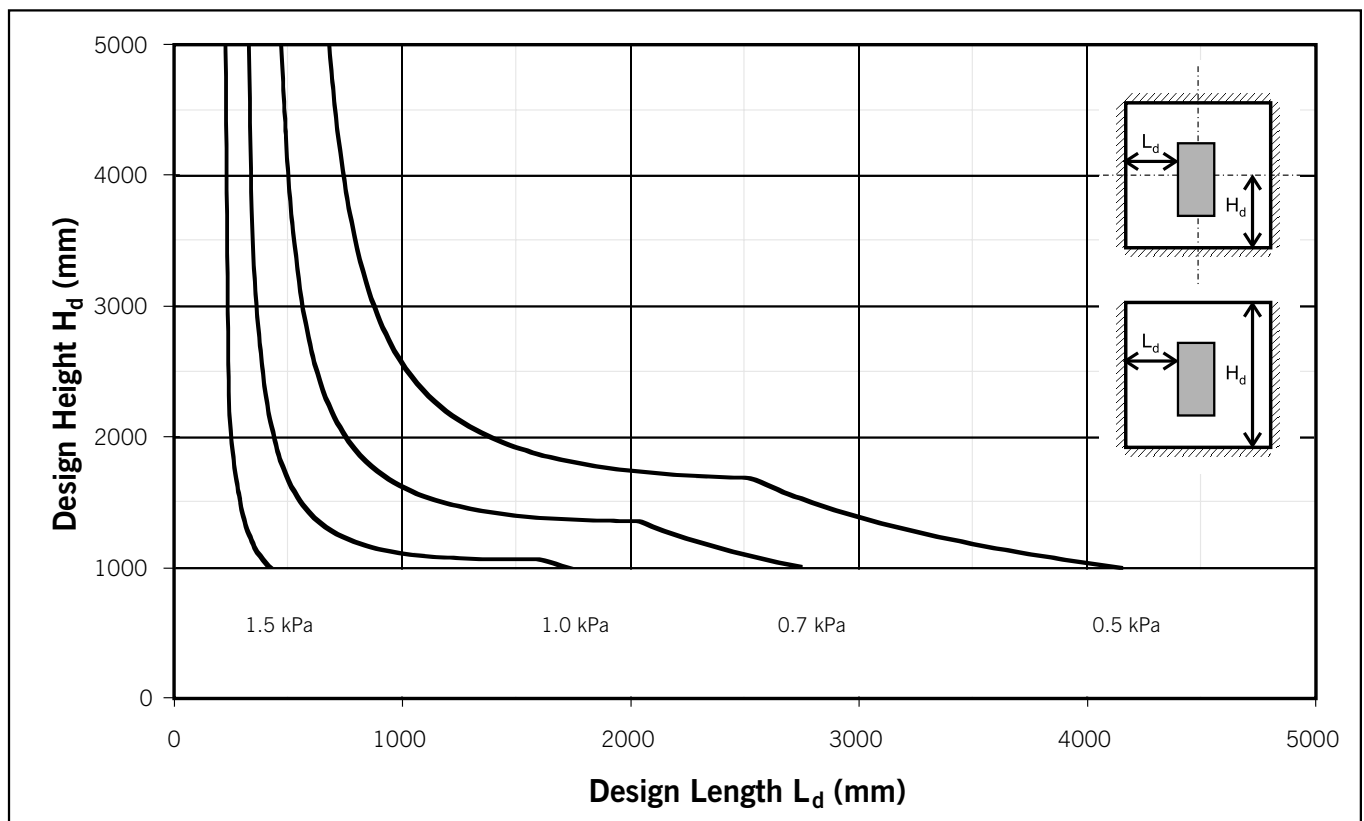
1200 mm wide opening (any height), both sides supported



1800 mm wide opening (any height), both sides supported



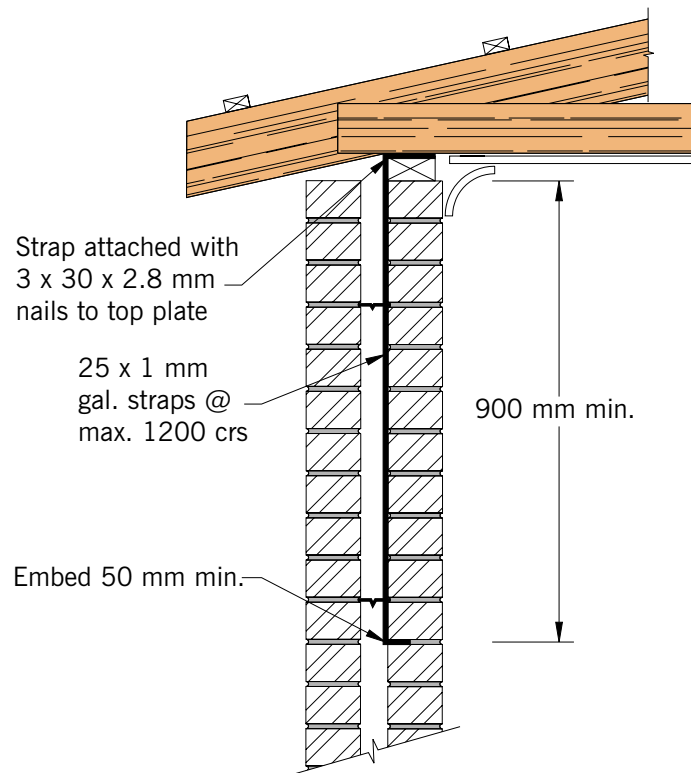
2400 mm wide opening (any height), both sides supported



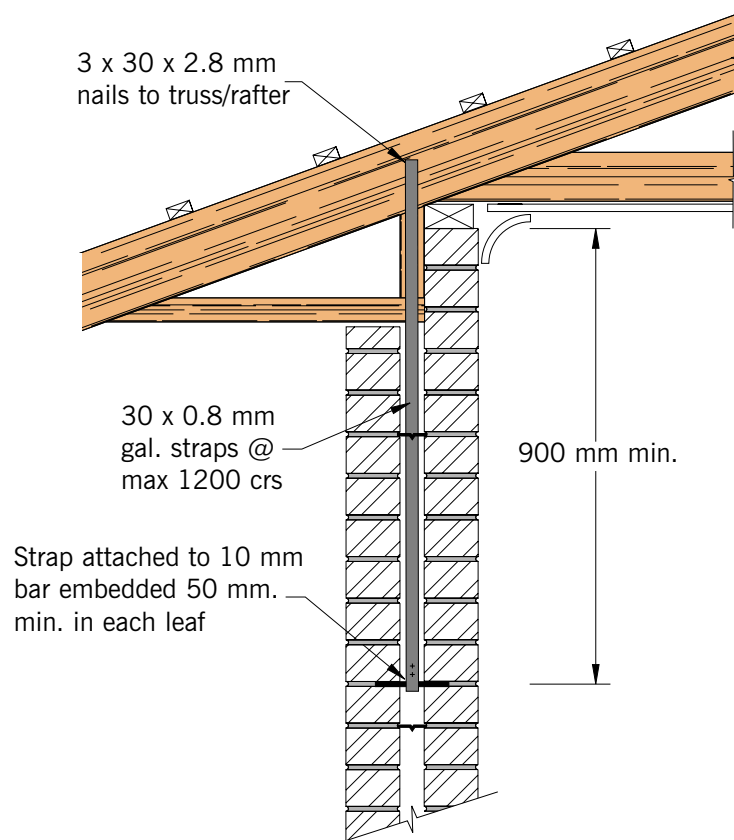
11. Typical Details

Note: These details are indicative only and may need to be modified to suit local conditions.

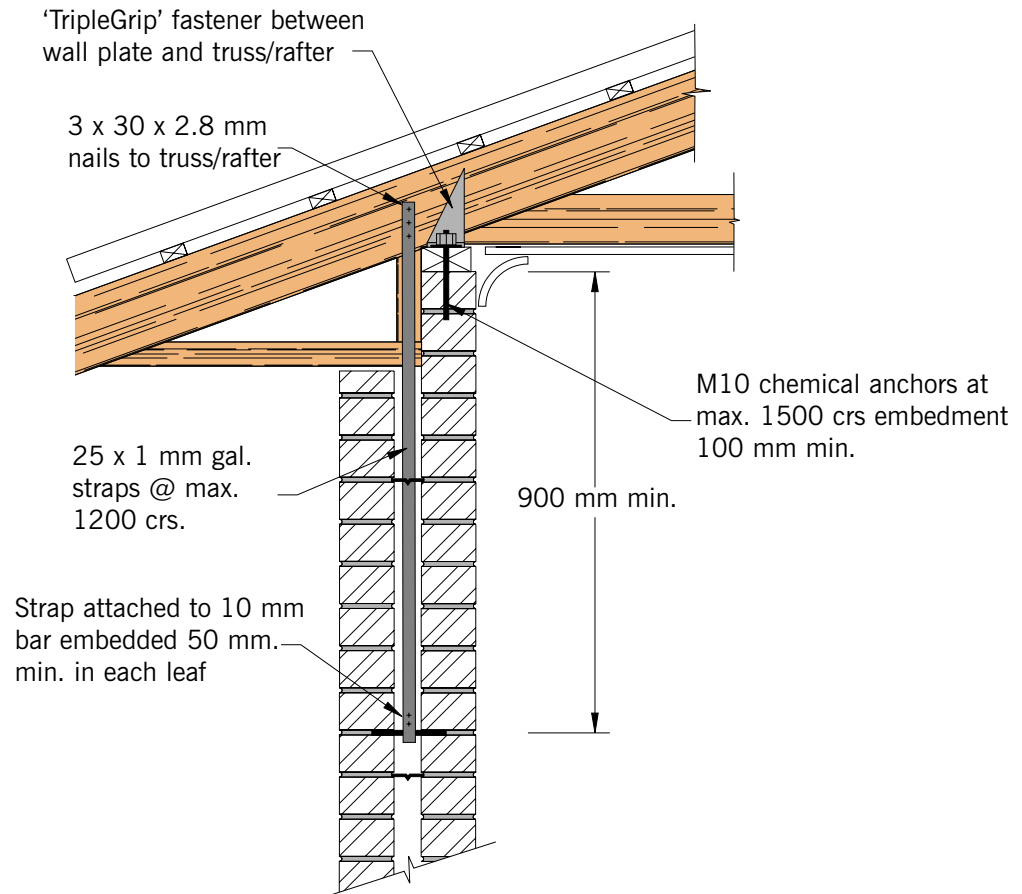
Detail 1. Roof tie-down - cavity wall (low wind)



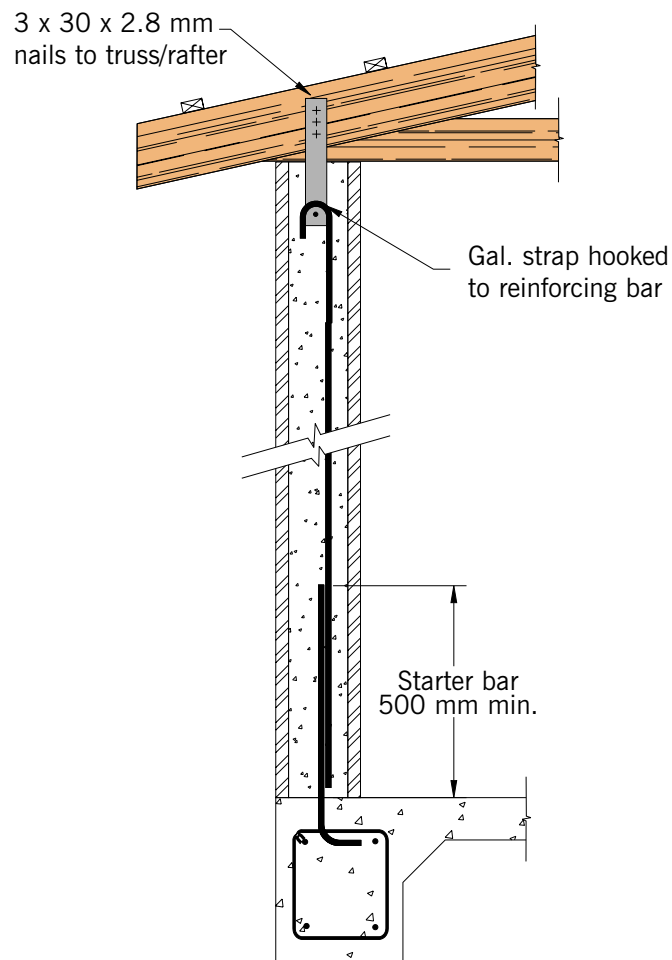
Detail 2. Roof tie-down - cavity wall (high wind - not cyclonic)



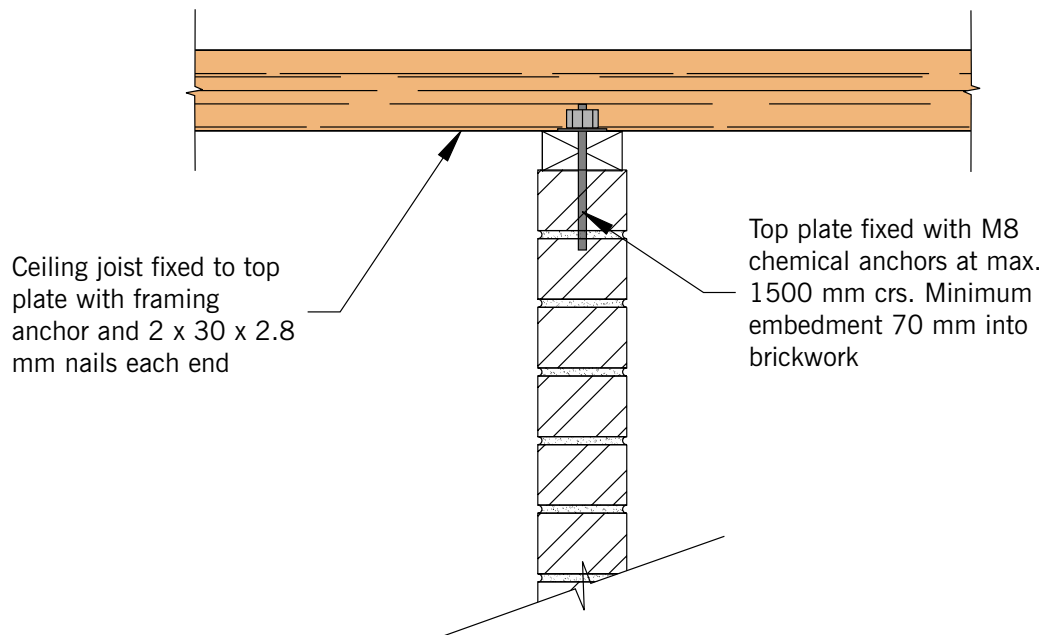
Detail 3. Roof tie-down - cavity wall (earthquake)



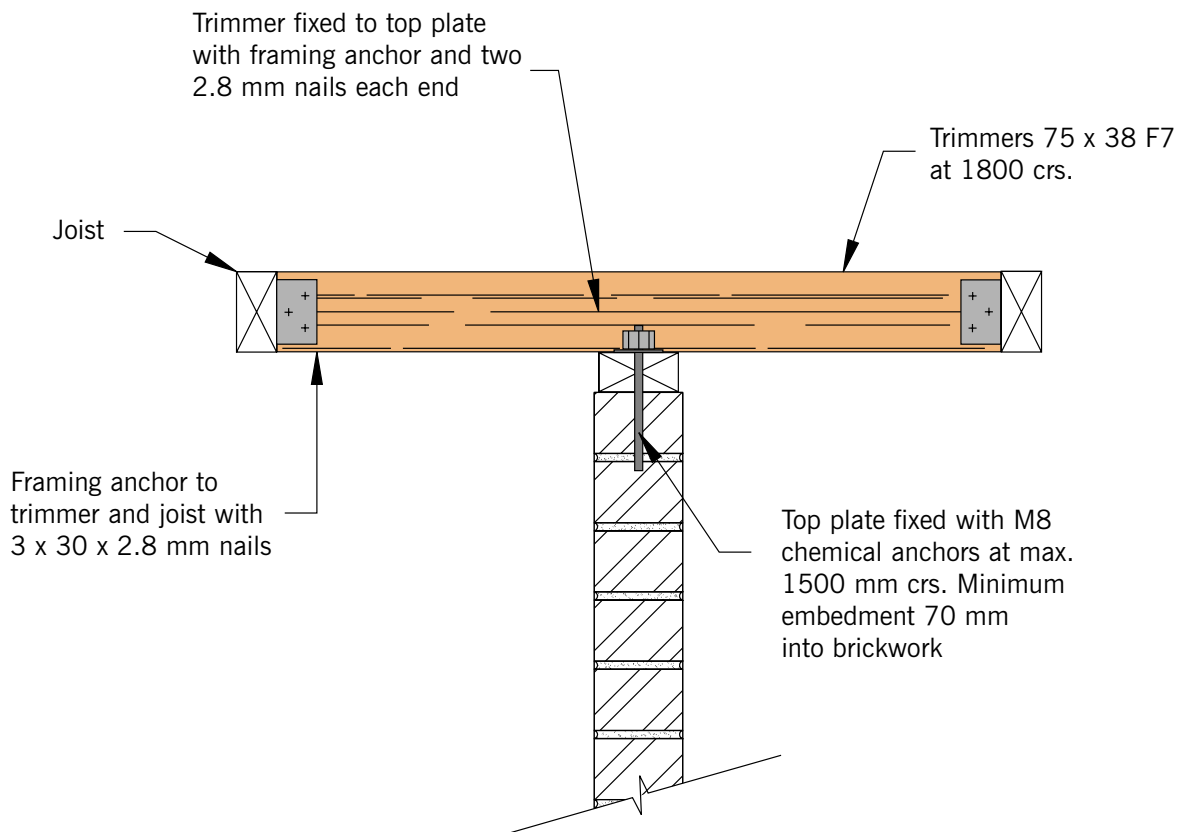
Detail 4. Roof tie-down - hollow-unit wall (earthquake and high wind)



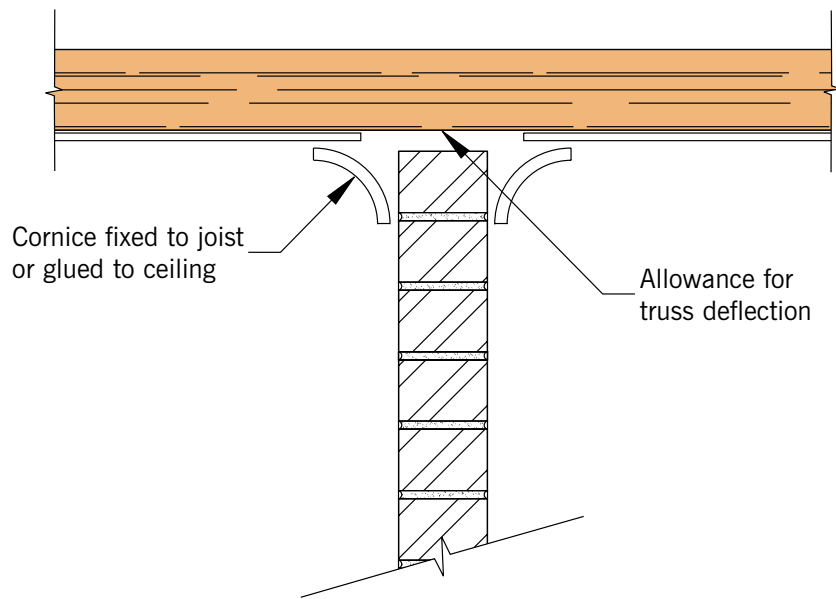
Detail 5. Internal wall - ceiling joist perpendicular to wall



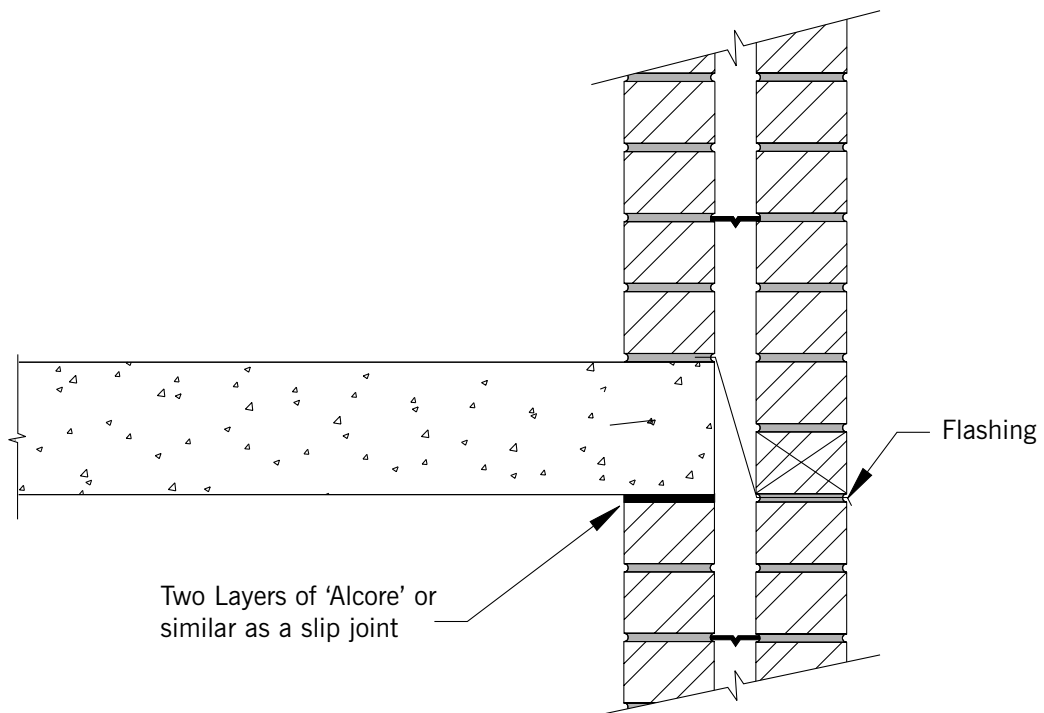
Detail 6. Internal wall - ceiling joist parallel to wall



Detail 7. Cornice support (low wind)



Detail 8. Wall-slab junction



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