



Manual 6

Design of Clay Masonry for Compression



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Masonry Contractor: Harris Bricklaying
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Masonry walls in a structure can resist loads by compression, shear or flexure, depending on the type of structure and the location of the walls within it. These actions can occur in combination, but there is usually one action that governs the behaviour of each particular wall. This manual deals with design for compression loading, while design for shear and flexure are dealt with in the Think Brick Australia Manual 4, *Design of Clay Masonry for Wind & Earthquake*¹.

As well as design for compression, shear and flexure forces, all walls should be checked for robustness to ensure that they are serviceable in the absence of specific applied loading. Design for robustness is dealt with in Think Brick Australia Manual 7, *Design of Clay Masonry for Serviceability*².

This manual follows the procedures set out in Masonry Structures (AS 3700)³ for the design of unreinforced clay masonry to resist compressive forces. 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 commentary⁴.

This manual covers the general arrangement of load-bearing masonry structures, material properties for clay

masonry, specific design procedures for vertical loading, both simple and refined, design for concentrated loads, and composite action between walls and beams. It covers walls and piers to predominantly vertical loading. Design charts for vertical loading and worked examples for typical cases are included.

Unreinforced load-bearing masonry structures greater than four storeys in height are common in other parts of the world (for example in Europe) and have been built in Australia in the past. Modern Australian codes and standards permit the use of unreinforced load-bearing masonry in structures up to 15 m in height, provided earthquake loading is properly considered. Multiple-occupancy domestic units of load-bearing masonry up to four storeys in height are common in Australia and two-storey semidetached townhouses are becoming increasingly popular. In these buildings, the masonry walls and piers usually support concrete floor slabs and the roof structure, without any separate building frame. 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, and this requires a cellular form of structure. Wall sizes in these structures are usually determined by their capacity to resist vertical load.

2. General principles

Load-bearing walls are defined as those that carry significant vertical load in addition to their own weight. In other words, they rely on their compressive load resistance to support other parts of the structure. Buckling and crushing effects, which depend on the wall slenderness and interaction with the elements supported by the wall, determine the compressive capacity of each individual wall. The shape of the units, particularly the presence and size of hollow cores, influences the compressive strength of the masonry. External load-bearing walls will usually be of cavity construction to ensure adequate penetration resistance.

Masonry piers can either be isolated (supporting a slab) or engaged (providing enhanced load resistance to a wall). Isolated piers are designed for compressive load capacity in the same way as load-bearing walls. The effect of engaged piers in stiffening a wall is taken into account by the use of an effective thickness for the wall/pier combination.

Strength properties of masonry can be difficult to predict from known properties of the mortar and masonry units, because of the relatively complex interaction of the two component materials. For this reason, values given in AS 3700 for masonry strengths based on these properties are conservative. Whenever practical, it is better to measure the strength of masonry by testing small specimens (stack-bonded prisms) than to use the conservative AS 3700 figures. Tests will always give a more accurate measure of properties than can be obtained from typical values. This can be worthwhile on larger projects, and classifies the masonry as *Special Masonry* (see Section 2.2.4).

Masonry as a material has inherently variable properties. This variability arises because of natural variations in materials, and manufacturing processes and the site conditions, and workmanship. Partial safety factors used for design make allowance for this variability and ensure that the resulting structures are safe and serviceable,

provided that the workmanship is adequate. Masonry is sometimes considered to be an architectural rather than a structural material and this can mean that supervision is not as thorough as it is for structural elements. The importance of supervision of construction cannot be overestimated for loading masonry, especially when it is classed as Special Masonry. The potential economic benefits in the use of structural materials are great when the full strength of the material is utilized in design and ensured by adequate supervision.

2.1 Behaviour of masonry elements in compression

In load-bearing masonry buildings, the masonry is basically designed to act in compression, with the load-bearing walls transmitting vertical loads from storey to storey down to the foundation. Lateral loads caused by wind and earthquake are transmitted to the foundation by shear wall action and also produce bending moments in the walls that are normal to the direction of the loads. Further bending moments arise in the walls from the floor loading, which is transmitted to the walls because of partial joint fixity where the floors rest on the walls. Full or partial frame action in the structure is therefore mobilised, with the degree of joint fixity depending on the level of pre-compression (see Figure 1). The masonry walls are therefore subjected to combined compressive and bending actions. These combined compressive forces and bending moments acting simultaneously can be expressed as a compressive force acting at an equivalent eccentricity, as detailed in Section 3.

Depending on the relative signs of the eccentricity at opposite ends of the wall, the combined loading will give rise to either single or double curvature (see Figure 2). A wall subjected to double curvature has a much higher resistance to buckling than a wall in single curvature, because of the reduced second-order (P- δ) effect and this can be taken account of in design.

Figure 1. Frame action in a load-bearing building (typical section)

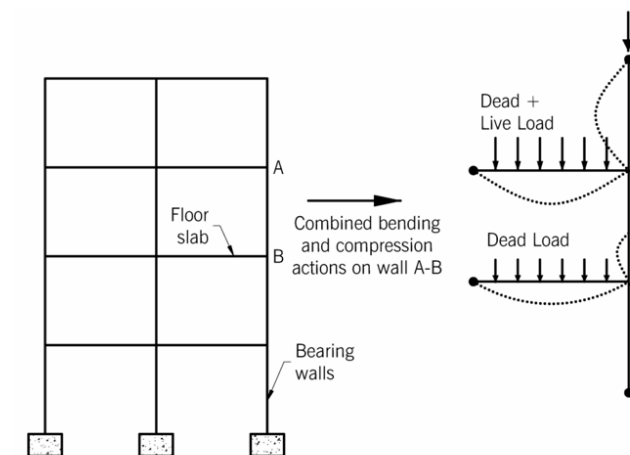
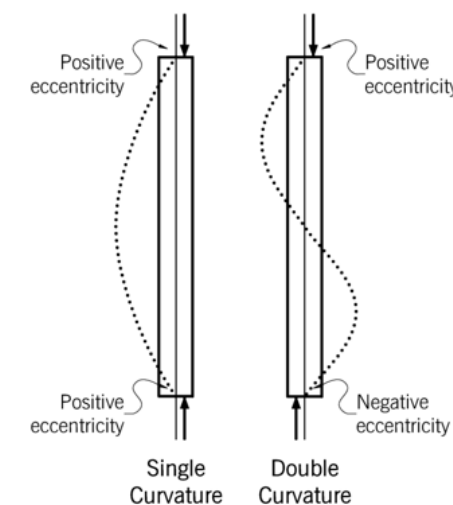


Figure 2. Single and double curvature in compression

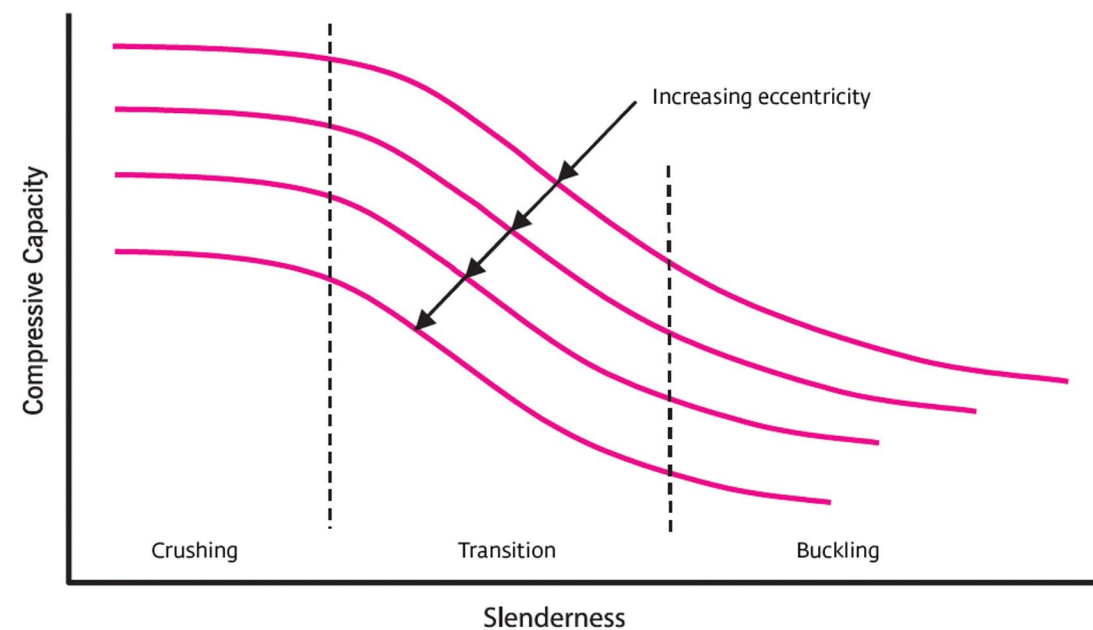


There are two possible modes of failure for masonry members under eccentric compression. These mechanisms are quite different and would rarely occur in this pure form; most walls and piers, if overloaded in a building, would fail by a combination of both effects. The first mechanism is crushing, which would only be observed in short stocky members, and the second mechanism is buckling, which would occur in long slender members. The transition between these two extremes is shown schematically in Figure 3, with the various curves showing the effect of increasing eccentricity of loading. The calculation of member capacity therefore requires both the compressive strength of the masonry and the effects of buckling to be taken into account.

In general, the compressive load capacity of a masonry wall or pier depends on four factors:

- The characteristic compressive strength of the masonry.
- The cross-sectional area resisting the load.
- The slenderness.
- The effective eccentricity of loading at each end and the resulting deflected shape (single or double curvature).

Figure 3. Effect of slenderness and eccentricity on compressive capacity



2.2 Compressive strength of masonry

2.2.1 Action in compression

When slenderness effects are eliminated, the compressive strength of masonry is a function of the masonry units and the mortar. The compressive strength of masonry is usually less than that of the units alone, because of the influence of the mortar joints. When considering the action of masonry in compression, a distinction must be made between solid and hollow masonry units. These are the terms used in AS 3700 as follows:

- Solid masonry units, which usually have frogs or cores, are laid in a bed of mortar covering the full bed face area (called *full bedding*).
- Hollow masonry units, which usually have a large proportion of hollow cores, are laid with strips of bedding mortar on the face shells and no mortar on the webs (called *face-shell bedding*).

Solid masonry units

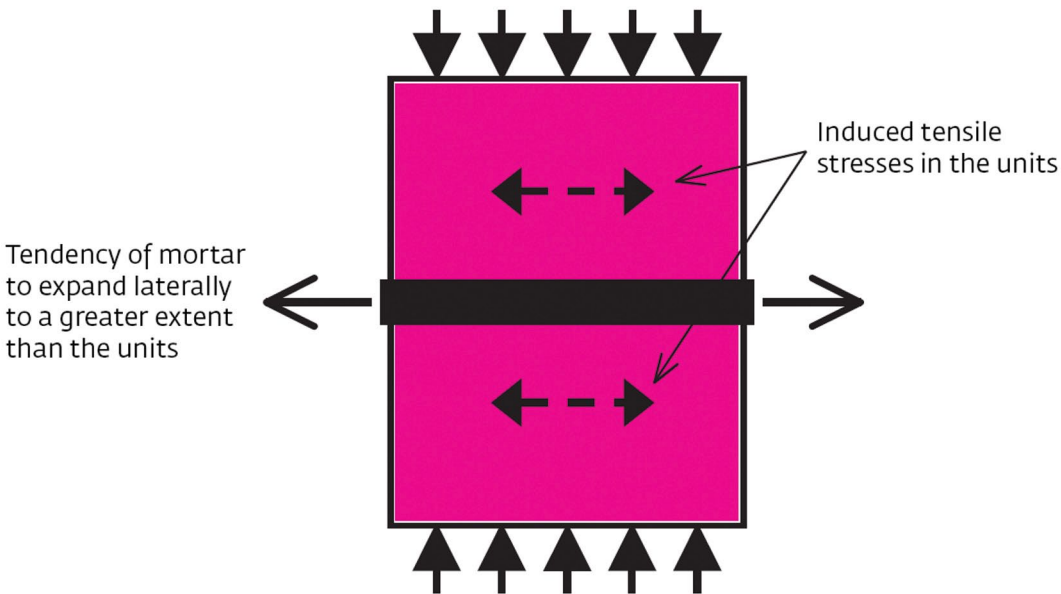
When masonry constructed with solid units is subjected to direct compression and loaded to its full capacity, failure will be initiated by the lateral expansion of the mortar joints (the Poisson's Ratio effect), which causes vertical tensile cracks to form in the masonry units. While both materials expand laterally, the mortar

joints are weaker and therefore more flexible than the units are, and expand to a greater extent. This induces tensile stresses in the units and corresponding lateral compressive stresses to the mortar (see Figure 4).

The measured compressive strength is therefore affected by the following:

- Tensile strength of the masonry unit. For clay units, the material itself, the degree of firing, and the coring pattern all affect this property.
- Mortar joint thickness. In highly stressed load-bearing walls, thicker mortar joints would increase the splitting force on the units. An appropriate correction factor can be applied, but the joint thickness is best limited to 10 mm in these structures.
- Mortar strength. Because mortar with a low strength generally has a lower elastic modulus, it has a greater lateral expansion under load and therefore a higher tendency to split the units. It is therefore important to use a mortar with adequate strength in load-bearing masonry construction. If the mortar strength is very low, then crushing of the mortar itself might occur even before tensile failure is induced in the units.

Figure 4. Behaviour of masonry in compression



Hollow masonry units

For hollow units, the compressive strength of masonry is based upon face-shell bedding and when prism tests are carried out with hollow units they can use this form of loading. In this case, the mode of failure differs from that applying for solid and cored units. Load applied through the face shells is transferred into the webs, causing transverse tensile stresses. This causes web splitting followed by crushing of the mortar joints and spalling of the face shells as a result of the subsequent stress redistribution. Because of this different mode of behaviour, the strength of hollow unit masonry is less affected by mortar strength than is the strength of solid and cored unit masonry.

Grouted hollow masonry

Where hollow units are filled with grout, the compressive strengths of the blocks and the grout cannot be simply added. The grout, which is usually less stiff than the units, has a greater level expansion under compressive load and tends to burst the units. AS 3700 allows for this when determining the basic compressive capacity.

2.2.2 Masonry units

When a prism of any material is tested in compression, the rigid platens of the testing machine, through friction, confine the upper and lower faces and resist the tendency of the material to spread under load (see Figure 5). This phenomenon has the effect of enhancing the apparent strength of the material. The platen restraint effect varies

with the relative height to width of the masonry unit. Because masonry units are available in a wide range of sizes and shapes, the effect is not the same for all units and must be corrected so that all unit shapes are treated equally for design.

Figure 5. Platen restraint effect for masonry in compression

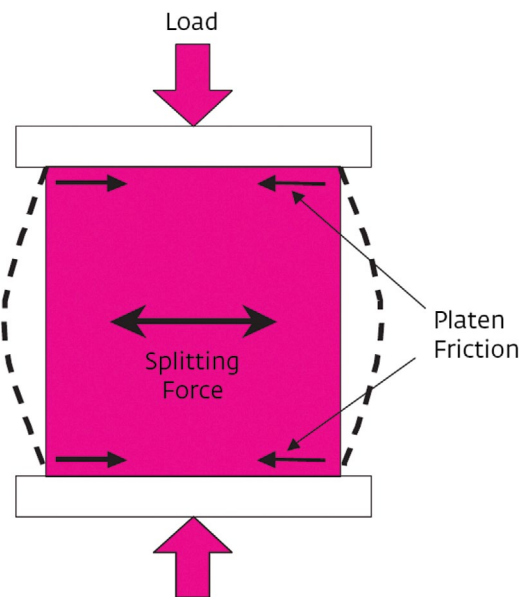
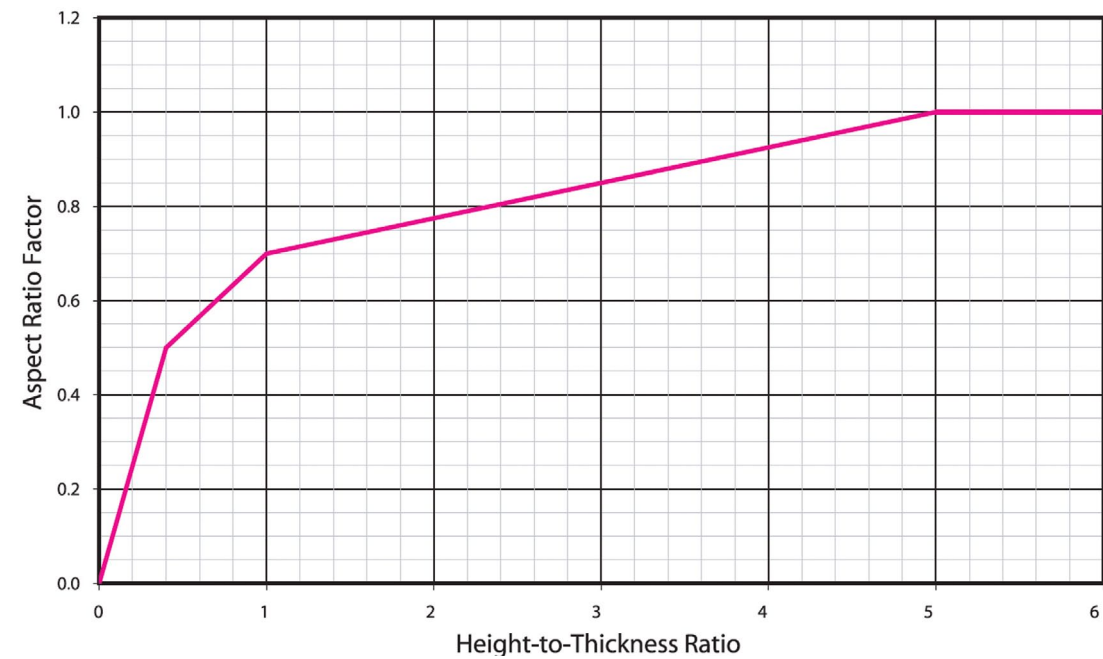


Figure 6. Aspect ratio factor for masonry in compression



The compressive strength for units used in design is called an unconfined strength, which means that the effects of platen restraint have been eliminated. An adjustment factor based on the height-to-width ratio of the unit is used. There is no international standard method of adjusting for this effect, and this makes comparisons between various national standards difficult. AS 3700 adopts the strength corresponding to a prism of material with height equal to five times its width as a standard basis for this adjustment. This assumes that the platen restraint effect is negligible at these proportions and that the effects of load eccentricity have not yet become significant. The correction factor is called the aspect ratio factor and is tabulated in AS 3700 Appendix C (see Figure 6).

For hollow units, the aspect ratio factor is based on the height-to-thickness ratio of the face shell, which is usually greater than 5. The factor is usually 1.0.

The dimensions used for finding the aspect ratio factor are the work size dimensions of the unit, that is, the nominal dimensions specified by the manufacturer. This is particularly important for hollow units, where the face shells might vary in thickness (taper from top to bottom) but the manufacturer nominates a single work size dimension. This ensures that any compressive strength calculations are made on the same basis as that used by the manufacturer to rate the strength of the units.

The standards require compressive strength for hollow units to be determined by testing under load applied to the face shells only, to simulate the way in which they will be laid. This ensures that the failure mechanisms in compression for the test and the units in the structure would be the same.

The symbol used for the characteristic unconfined compressive strength of units is f'_{uc} . As for other structural design properties, the compressive strength used in design is the 95% characteristic value. This strength is expected to be exceeded by 95% of the material, ensuring a very low risk of structural failure.

Typical characteristic strengths (f'_{uc}) for clay masonry units range from 12 MPa to 40 MPa or more.

2.2.3 Masonry strength

Masonry strength depends on the interaction between the units and the mortar. Mortar strength (and therefore masonry strength) is usually lower than the unit strength.

As for the case of the masonry units, an unconfined characteristic compressive strength of the masonry based on a height-to-thickness ratio of five is used for design. This compensates for platen restraint effects in testing and brings all masonry types to a common basis for comparison.

Consistent with the behaviour mechanism described in Section 2.2.1, strength tests on masonry prisms have shown a strong correlation with the strength of the masonry units and the mortar composition. AS 3700 gives characteristic compressive strengths for masonry with various unit strengths and mortar compositions in Table 3.1 and these are shown graphically in Figure 7. The AS 3700 table is based on the characteristic unconfined compressive strength of the unit, the material of manufacture, and the classification of mortar. The values have been established from a lower bound fit to a wide range of tests carried out in Australia and will be adequate for most cases without confirmation by testing. While it will usually be possible to obtain a higher value of design strength by carrying out tests, this would require precise knowledge at the design stage of the type of unit to be used on the job. It would also require the masonry to be classified as *Special Masonry* for compressive strength and site control tests would then be required during construction (see Section 2.2.4). This is difficult, but the potential benefit from the higher strength will sometimes justify accepting this higher level quality control regime.

Figure 7. Compressive strength of masonry for various unit strengths

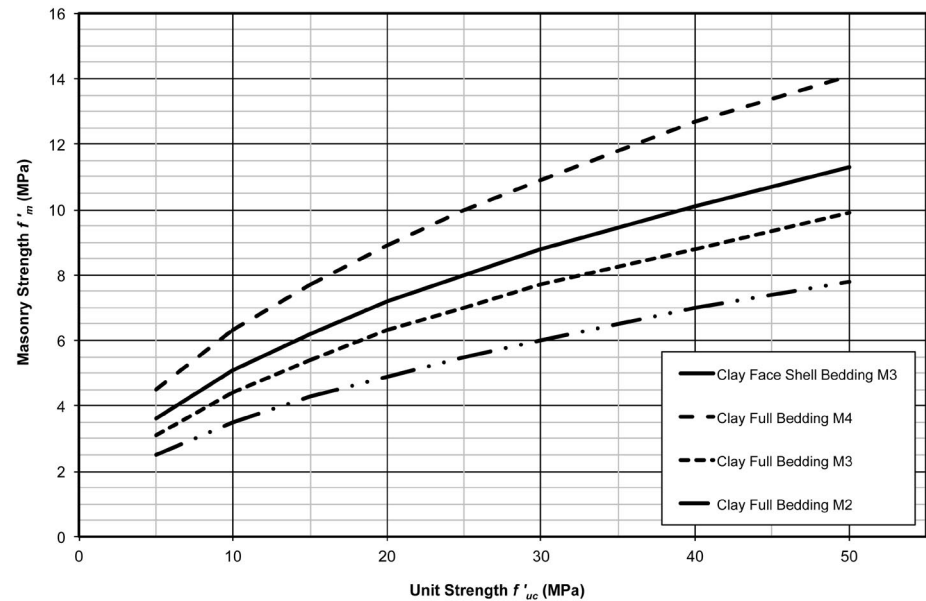


Figure 8. Factor to adjust for unit height/joint thickness



The compressive strength of the masonry is further adjusted in AS 3700 (Clause 3.3.2) by a factor, k_n , that allows for the effect of the mortar joint thickness by relating it to masonry unit height (see Figure 8). This factor is 1.0 for traditional brick-sized units of 76 mm height with mortar joints of 10 mm thickness, because the values in AS 3700 (shown in Figure 7) have been based on these dimensions. For taller units, where a smaller number of joints will be used in a given wall height (or where the joint is thinner in relative terms) the strength is enhanced. Conversely, for less-tall units the strength will be reduced because of the higher number of joints. The resulting characteristic compressive strength of the masonry is referred to by the symbol f'_m .

2.2.4 Special masonry

Whenever a designer uses higher design strength than the default value given in AS 3700, the corresponding masonry is classified as *Special Masonry*. This means that site testing for quality control becomes mandatory for that masonry.

For determination of design strength, tests must be carried out at the design stage, using the same materials (units and mortar) and the same workmanship as that intended for the actual construction. These pre-construction tests are conducted as specified in 3700 Appendix C and the results are used to determine a characteristic strength in accordance with AS 3700 Appendix B. By following this procedure, the full strength of the material in compression can be utilised and this can often be significantly more than the minimum compressive strength guaranteed by the manufacturer.

During construction of masonry classed as *Special Masonry*, tests must be carried out for quality control and to assess compliance. The test regime is specified in AS 3700 Clause 12.7. The standard specifies the rate of sampling as one sample per storey height or one sample per 400 m² of walling, whichever gives the greater number of samples, with at least two samples for any job. The number of specimens in a sample is at least three for determination of compressive strength. This quality control testing is in accordance with AS 3700 Appendix C, as for tests to determine the design strength, but in this case, the assessment of test results is based on the average strength of the test sample.

For quality control purposes, the average test unit is compared with the target strength. If the strength falls below the target then reasons should be sought and remedial action taken. This is purely for the purpose of control on site and is quite distinct from the assessment of compliance, although the same test samples can be used for both purposes. The target strength specified by AS 3700 is $1.4 f'_m$ for compression, where f'_m is the design characteristic strength.

For assessment of compliance of *Special Masonry*, each individual test sample represents a segment of masonry on the job and this relationship should be tracked and recorded on site. Then if the result of any sample falls below a specified strength level the whole of the masonry represented by that sample is deemed not to comply with AS 3700.

The use of testing to control construction of *Special Masonry* is described in Think Brick Australia Manual 10, *Construction Guidelines for Clay Masonry*⁵, and illustrates with sample charts for quality control and assessment of compliance.

2.3 Effect of engaged piers

The thickness coefficient for engaged piers reflects the stiffening effect that piers have in increasing the buckling resistance by increasing the effective radius of gyration of the section. This coefficient depends upon the relative pier thickness, pier width and pier spacing. When applied to the wall thickness, it gives an effective thickness of a rectangular section having an equivalent radius of gyration to the 'T' section.

For the stiffening to be effective, it requires full engagement of the pier, with bonding or tying as required by AS 3700 Clause 4.11.

Values are tabulated in AS 3700 Table 7.2 for overall thickness (of wall and pier combined) of two and three times the wall thickness alone. Figure 9 shows these values plotted against the ratio of pier spacing (centre to centre) and pier width. The pier width and spacing are illustrated on Figure 10.

Figure 9. Thickness coefficients for engaged piers

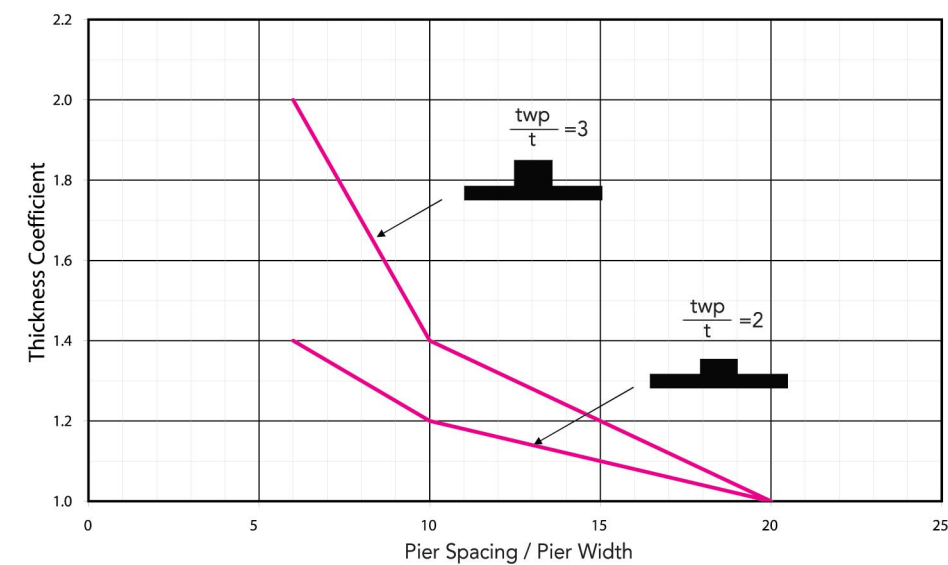
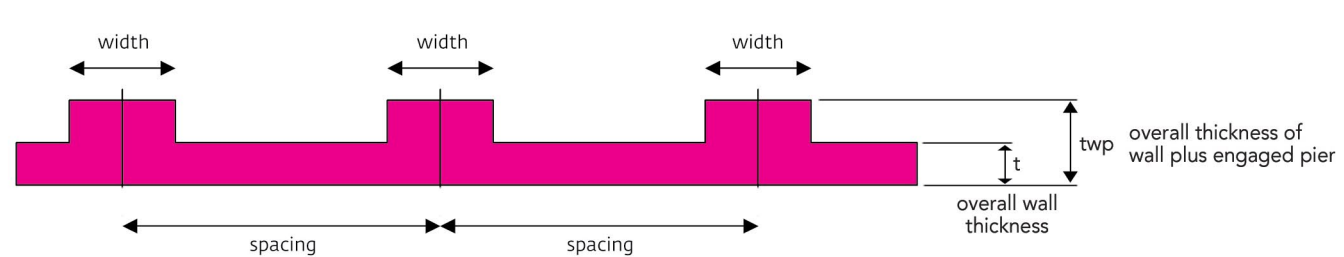


Figure 10. Width and spacing of engaged piers



3. General basis of design for compression

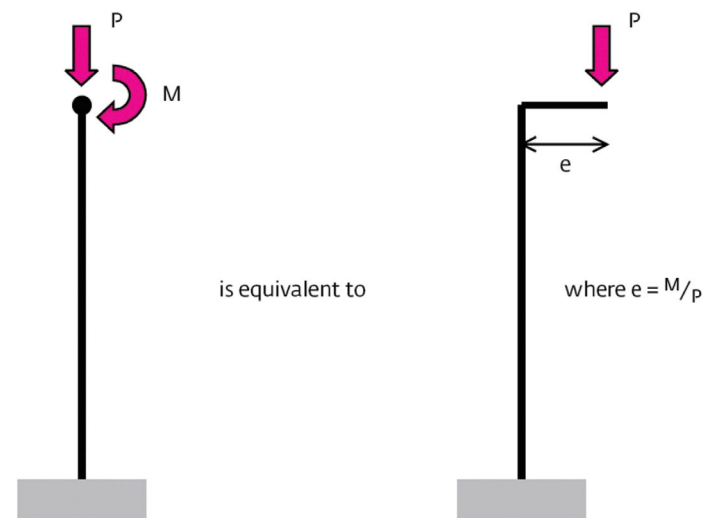
Compressive strength design in AS 3700 uses a tiered approach comprising two levels- design by simple rules and refined calculation. Both use the same general approach, but they differ in the method of allowance for slenderness and eccentricity effects. The refined calculation method is more specific about these factors and can produce more efficient designs, but at the expense of greater calculation effort.

Whereas some masonry codes in other parts of the world deal with compression and bending effects acting together by considering an interaction diagram for the behaviour of material, AS 3700 deals with it by an equivalent eccentricity approach, in which vertical load is considered to act at equivalent eccentricities at the top and bottom of the wall. These equivalent eccentricities incorporate the effects of any imposed bending actions.

AS 3700 sets out the general basis of design for unreinforced masonry in Clause 7.2. The approach is based on the assumption that design for bending due to lateral forces and design for light compression loads can be treated separately, ignoring any interaction. Consequently, provided the compressive stress is less than or equal to three times the characteristic flexural tensile strength (f'_{mt}), design for bending due to lateral forces can be carried out using the lateral load provisions in AS 3700 (see Manual 4, *Design of Clay Masonry for Wind and Earthquake*). Under these circumstances, the compression design excludes forces and moments arising from transient out-of-plane loads. In practice, this calculation will usually be carried out using simple rules procedure (see section 5).

Where the level of compressive stress exceeds the limit of three times f'_{mt} , the bending provisions in AS 3700 do not apply and compression design must include all forces and moments. This is handled by converting the bending moments to vertical loads at equivalent eccentricities. Thus a concentric load P acting simultaneously with a moment M will be replaced by the load P acting at the eccentricity e , where $e = M/P$, (see Figure 11). Design for compression in these situations can use the refined calculation procedure (see Section 6) but, in practice, the common cases would be carried out more quickly (and conservatively) using the simple rules procedure (see Section 5).

Figure 11. Equivalent eccentricity approach to combined compression and bending



The flow chart shown in Figure 12 summarises the general basis of design as set out in AS 3700 Clause 7.2 and discussed above.

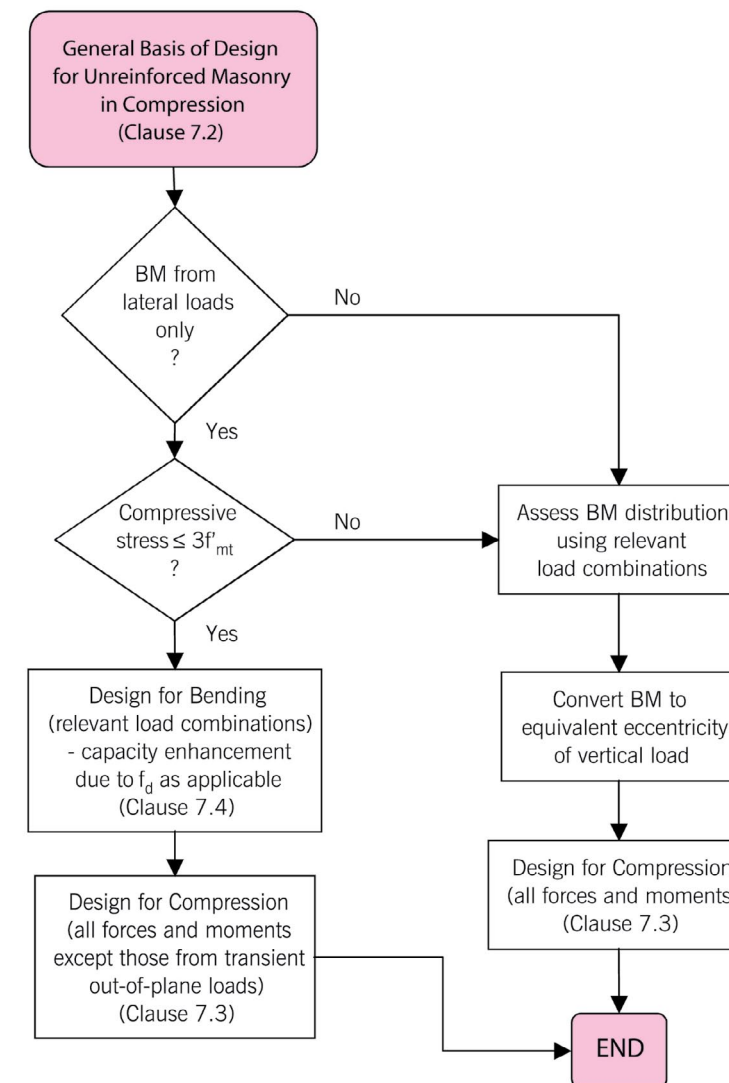
Both the slenderness and the eccentricity effects are incorporated into a single set of reduction factors, which are provided separately for the simplified and refined calculation methods. These reduction factors are applied to a basic compressive capacity (see Section 4) to give the load capacity of the member being designed.

The simplified method is sufficient for most cases commonly encountered and design charts for this approach are provided in this manual (see Section 5.2). In the simplified method, the end eccentricities are dealt with by defining three common cases of end restraint and tabulating slenderness reduction factors for these cases, based on all members being in single curvature (a conservative assumption in some cases). Unlike the refined method, this approach uses a single combined factor to cover lateral buckling and crushing failure. This simplified approach is defined in Section 5.

The method of refined calculation requires the assessment of end eccentricities and takes full account of the strengthening effect of double curvature, where the eccentricities at two ends of a member are of opposite sign. A full range of slenderness coefficients is provided, depending on the conditions of the wall or pier. The refined method uses separate reduction factors

for lateral buckling and for crushing failure. The method will therefore give the most efficient design in any given situation. It can provide a substantial strength increase in comparison with the simplified method but it can be complex to apply. The refined calculation method is described in Section 6.

Figure 12. Flow chart for the general basis of the compression design



4. Basic compressive strength capacity

Both the simplified and refined methods use a basic compressive strength capacity, which is derived as follows for ungrouted masonry:

$$F_o = f'_m A_b$$

Where:

ϕ = Capacity reduction factor

= 0.75 for compression in unreinforced solid or cored masonry

= 0.5 for compression in hollow or grouted masonry (See AS 3700 Clause 4.4)

f'_m = Characteristic compressive strength of the masonry

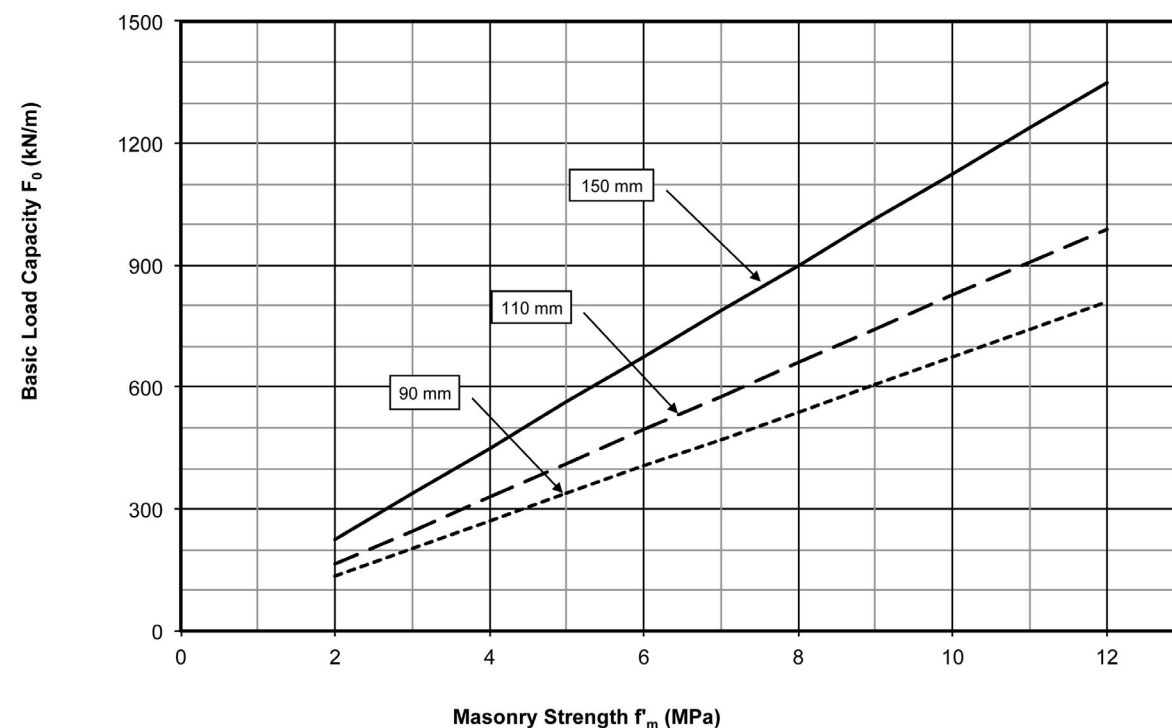
A_b = Bedded area

The design characteristic compressive strength f'_m is either obtained from the default values given in AS 3700 Table 3.1 or from tests on the actual materials and methods to be used in the construction (see Section 2.2).

The bedded area A_b is calculated as specified in AS 3700 Clause 4.5.4, allowing for any raking of the mortar joints. It is based on full bedding for solid or cored units and face shell bedding for hollow units. The length and width of the units used to calculate this area should always be based on manufacturer's work size dimensions, not on actual measurement of individual masonry units, as explained in Section 2.2.2.

Basic compressive load capacities are shown in Figure 13 for wall thickness of 90 mm, 110 mm, and 150 mm and for a range of masonry compressive strength. The calculations are based on the full bedded area with no joint raking.

Figure 13. Basic factored compressive load capacity



5. Design by simple rules

5.1 Procedure

It can be difficult to establish the end eccentricities for a given set of wall support conditions and AS 3700 provides a simplified means of design to cover the most common cases without the need to make the assessment. In this approach, the slenderness and eccentricity factors are combined into a single set of coefficients, which are tabulated for three common end support cases. In doing so, simplifying but conservative assumptions are made for end restraint conditions and the deflected shape of the wall under the applied end moments. Any additional support provided along the vertical edges of the wall is ignored. This is called design by simple rules.

The design equation for the simplified method is:

$$F_d \leq k F_o$$

Where:

F_d = Design compressive strength capacity of the wall

k = Slenderness and eccentricity reduction factor (obtained from AS 3700 Table 7.1)

F_o = Basic compressive strength capacity (see Section 4)

To find the slenderness and eccentricity reduction factor it is necessary to establish the slenderness ratio of the wall. This is derived as follows (see AS 3700 Clause 7.3.3.4).

Simplified slenderness ratio

For a wall the simplified slenderness ratio is:

$$S_{rs} = \frac{a_v H}{k_t t}$$

Where:

S_{rs} = simplified slenderness ratio

a_v = Slenderness coefficient for vertical span

= 1.0 for top edge support
= 2.5 otherwise

H = Clear height between the supports or the overall height of the wall

k_t = Thickness coefficient for engaged piers - see Section 2.3

= 1.0 for no engaged piers

t = Thickness of the leaf

Consequently for walls with top edge support and where there are no engaged piers, the slenderness ratio is simply the height-to-thickness ratio.

If there is no support, the slenderness ratio is based on an equivalent height of 2.5 times the actual height. Such a cantilevered wall, with a vertical load applied and yet no lateral support at the top is rare and its size is severely restricted by the high slenderness ratio.

Once the slenderness ratio is calculated, the slenderness and eccentricity reduction factor is found (see AS 3700 Table 7.1). These factors are tabulated for three different end support conditions, thus avoiding the need for the designer to calculate end eccentricities. Slenderness ratios up to 36 are provided for.

End support conditions

Figure 14, Figure 15 and Figure 16 illustrate the three end support conditions covered by the simple method. These are a wall supporting a concrete slab, a wall supporting a roof or floor other than a concrete slab, and a wall with a side-attached load (such as a floor) and at least one storey height of masonry above that level. In the last case, the wall must be at least 140 mm thick. Note that strengthening provided by any vertical edge support is ignored.

Figure 17 shows the slenderness reduction factors for the three support cases illustrated on Figure 14, Figure 15 and Figure 16. Note the higher factors for a wall loaded by a concrete slab, compared with those for a wall supporting another type of load. This is because the concrete slab imposes a greater degree of end restraint, causing the equivalent load to act at a smaller eccentricity than for other types of load. The factors for a load attached at the side of the wall are very low because of the high eccentricity. In this case, since the load being applied to the wall is outside the wall thickness, the load from above is critical in moving the resultant of the two forces

within the wall thickness and thus avoiding inherent instability. This is the reason for the requirement of at least one storey of masonry (and thus superimposed load) above the level under consideration.

In the past, it was assumed that the two leaves of a cavity wall provided mutual support against buckling and an equivalent thickness based on a combination of the two leaf thicknesses was used for such a case. However, later research showed that each leaf acts independently and there is no mutual support available to enhance load capacity. AS 3700 therefore requires that each leaf be designed separately for its share of the vertical load. In the majority of cases, the vertical load is taken entirely by the inner leaf of the cavity wall and the outer leaf acts only as cladding.

Figure 18 is a flow chart summarising the design process for compression by simple rules.

Worked Example 9.1 shows the design by simple rules of a wall

Figure 14. Wall loaded by a concrete slab

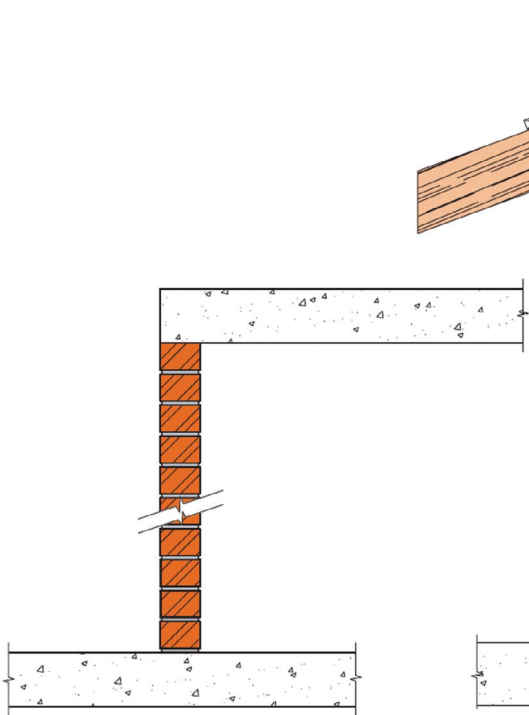


Figure 15. Wall loaded by other than a slab

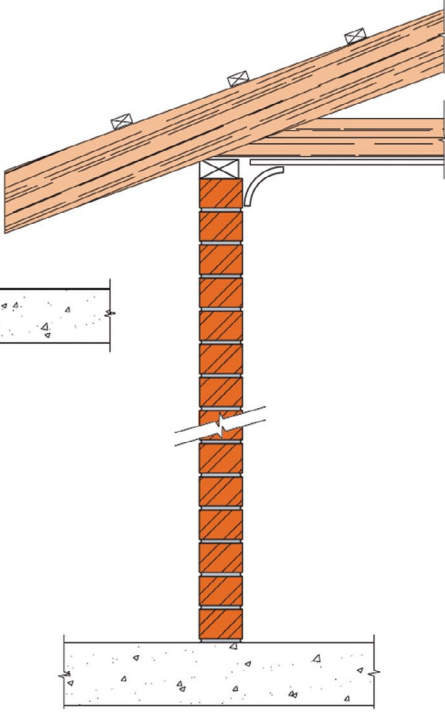


Figure 16. Wall loaded on one face

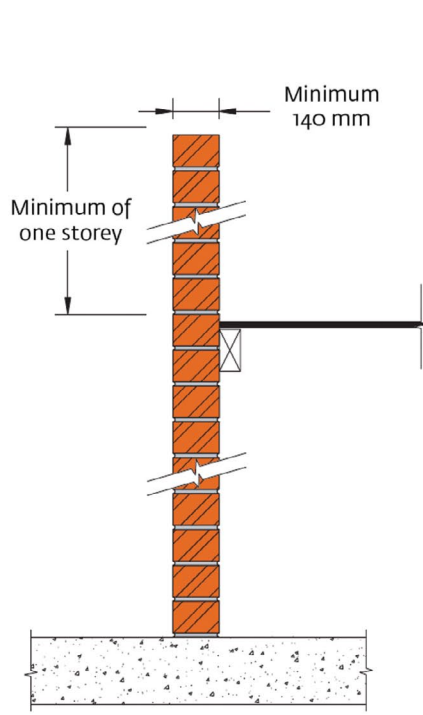


Figure 17. Slenderness reduction factors for design by simple rules

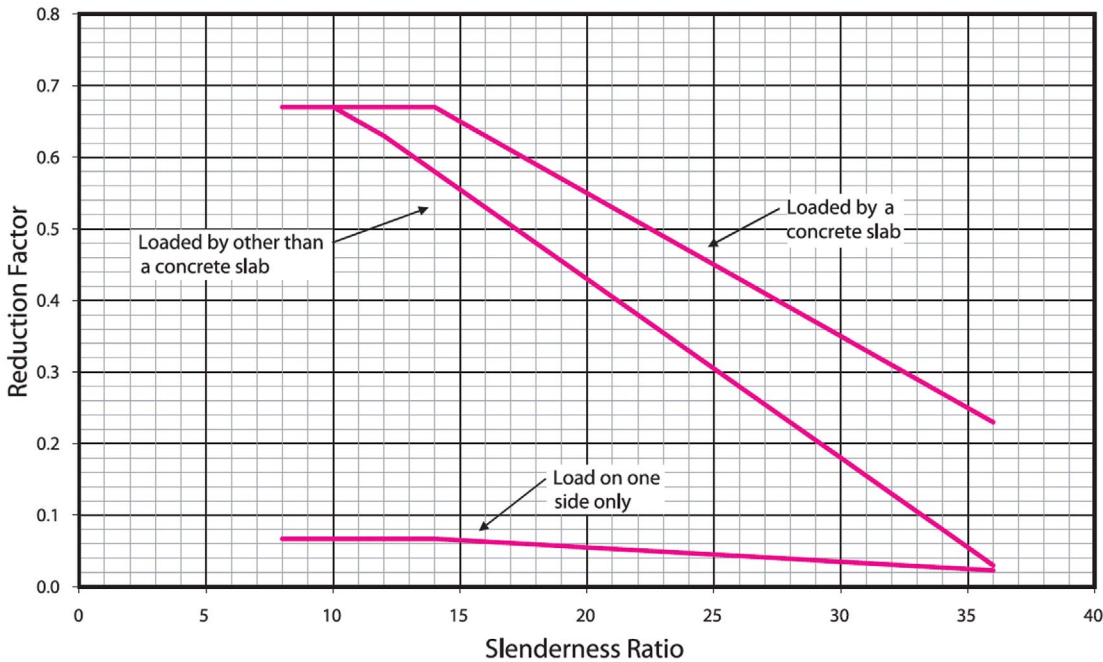
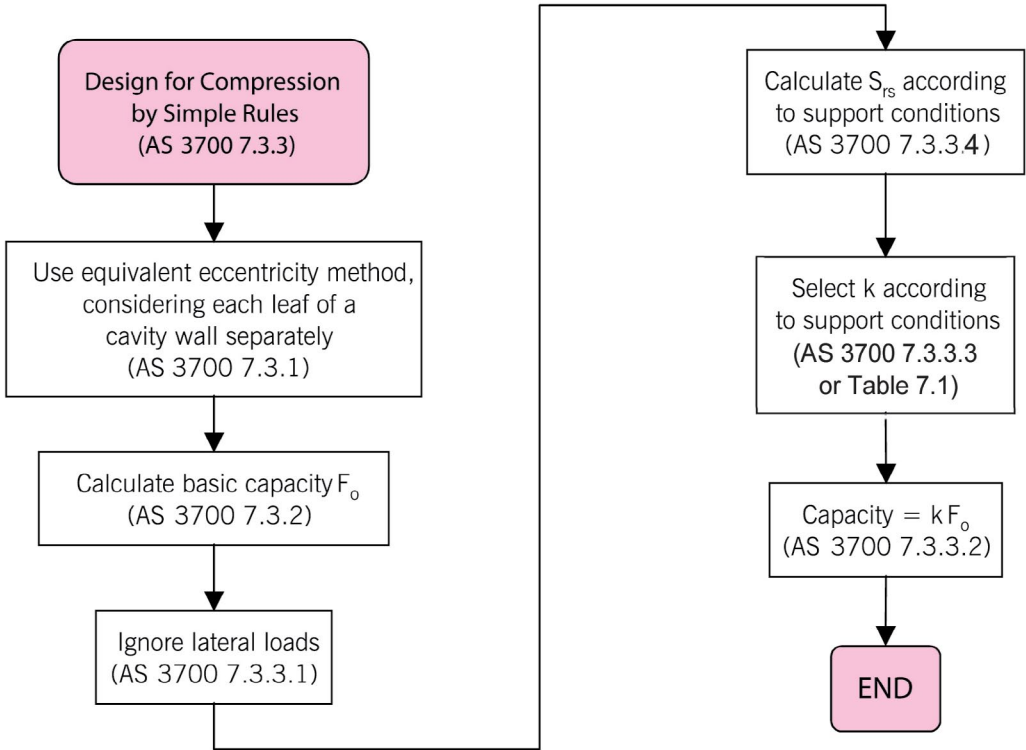


Figure 18. Flow chart for compression design by simple rules



5.2 Design charts

The following design charts can be used for design according to simple rules for walls with slab or other loading and with supports top and bottom. For the use of these charts, the loading conditions must correspond with one of the cases illustrated in Figure 14, Figure 15 and Figure 16. The applicable case is indicated on each chart by an icon.

Charts are provided for thicknesses of 90 mm, 110 mm and 150 mm in combination with heights of 2.4 m and 3.0 m. For the case of face loading, the only thickness provided for is 150 mm because AS 3700 requires the wall to be at least 140 mm for this loading case.

Figure 19. Compressive load capacities for walls 90 mm thick, and loaded by a concrete slab

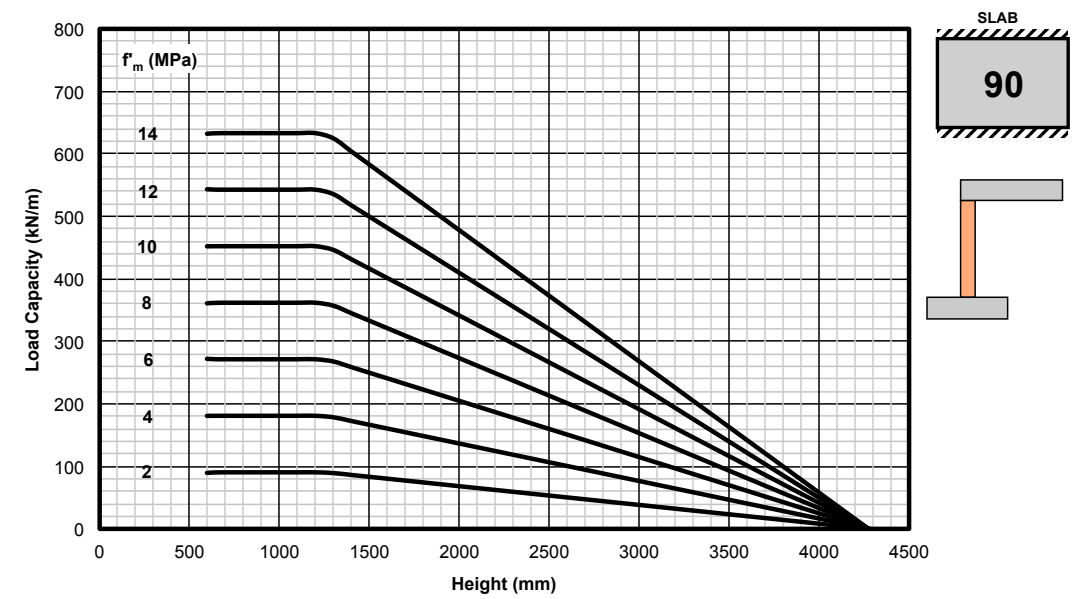


Figure 20. Compressive load capacities for walls 2.4 m high and 90 mm thick, with three edges supported and loaded by a roof or floor other than a slab

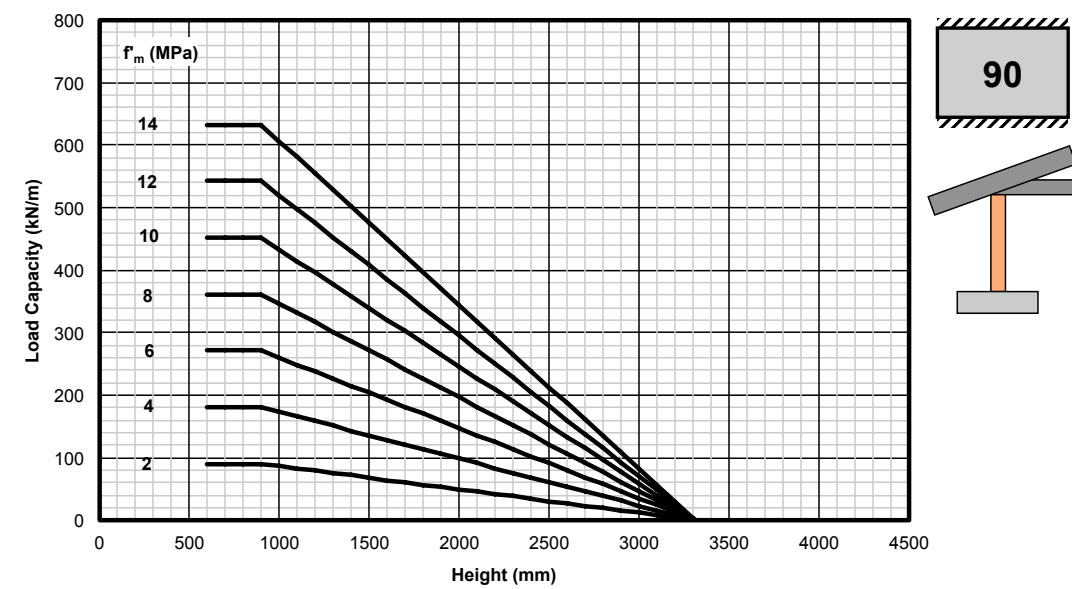


Figure 21. Compressive load capacities for walls 110 mm thick, and loaded by a concrete slab

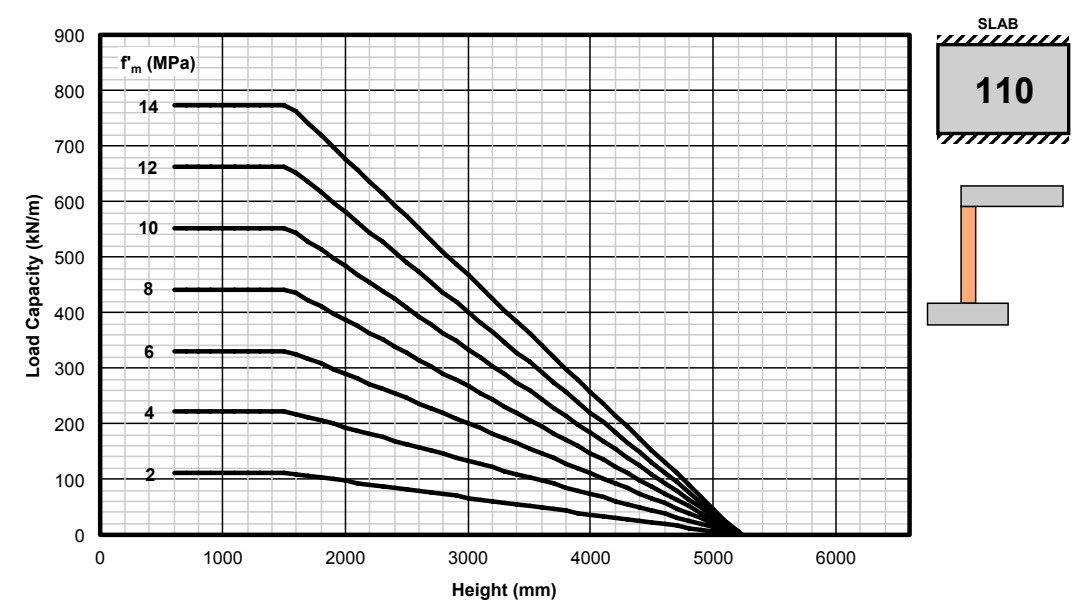


Figure 23. Compressive load capacities for walls 150 mm thick, and loaded by a concrete slab

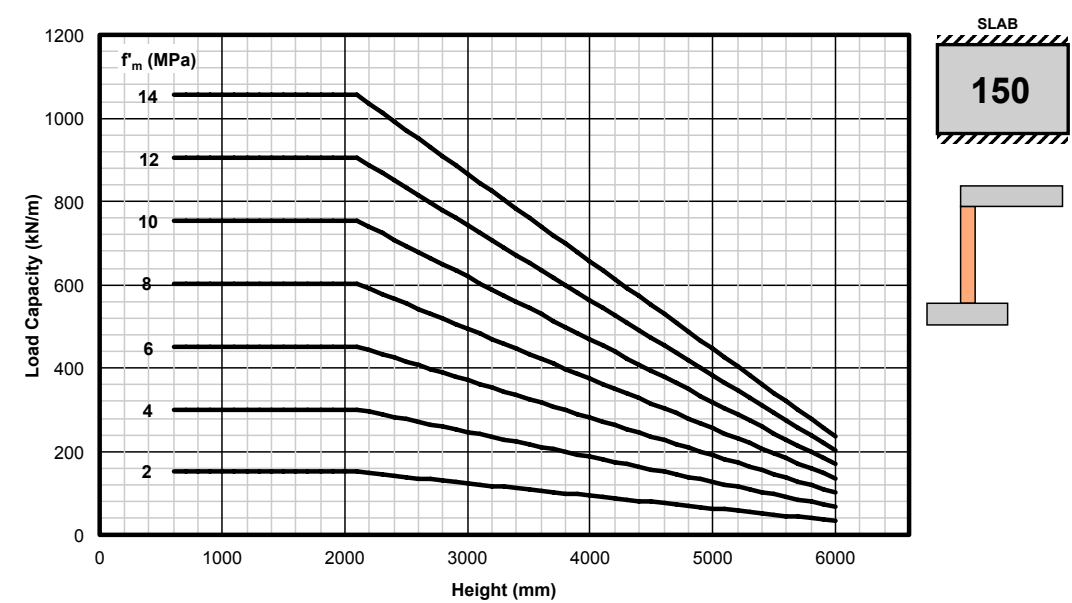


Figure 22. Compressive load capacities for walls 110 mm thick, and loaded by a roof and floor other than a slab

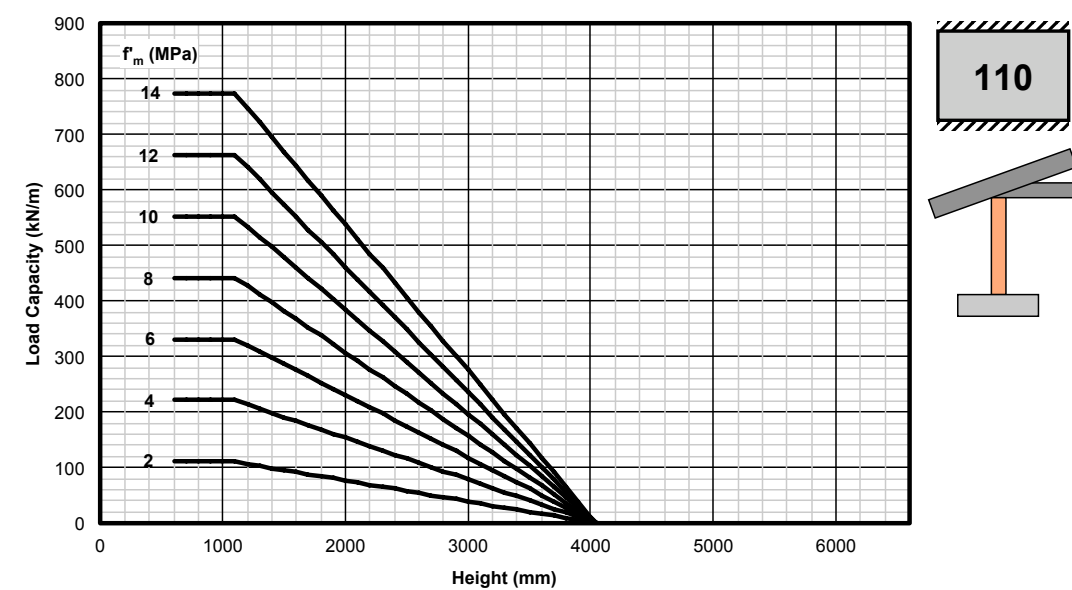
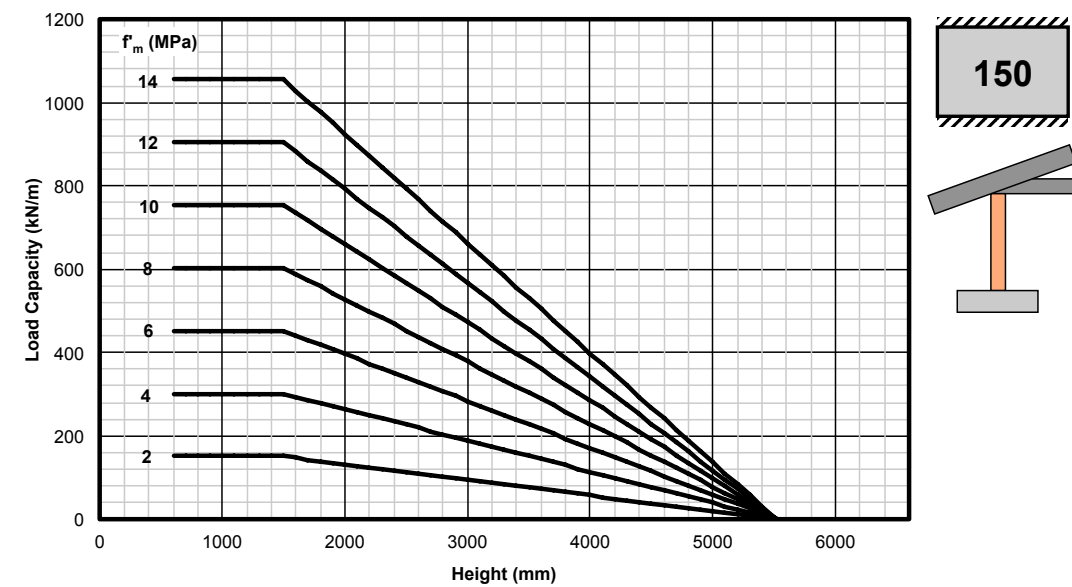
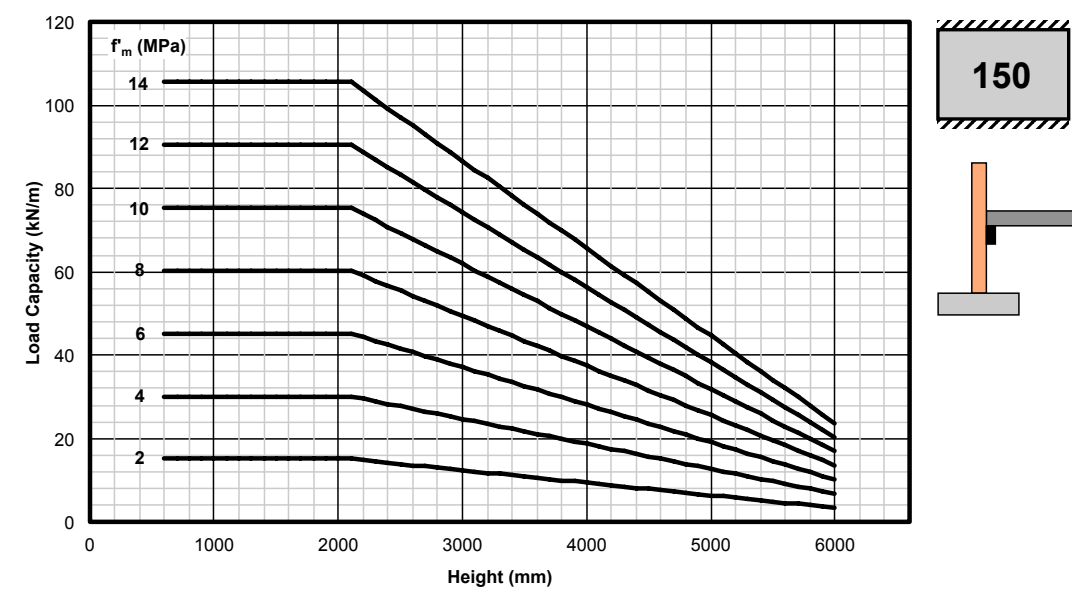


Figure 24. Compressive load capacities for walls 150 mm thick, and loaded by a roof or floor other than a concrete slab



6. Design by refined calculation

Figure 25. Compressive load capacities for walls 150 mm thick, and loaded by floor attached to the face



In the method of design by refined calculation, the slenderness and eccentricity reduction factors are calculated considering the actual supports of the wall and making an assessment of eccentricities at the top and bottom and their effect on the wall deflected shape and buckling capacity. Figure 26 illustrates the idealised buckling modes in walls depending on their support conditions.

To illustrate how the slenderness ratio is influenced by the wall length and support conditions, example slenderness ratios are plotted in Figure 27 (for walls 2400 mm high and 110 mm thick). It can be seen that the slenderness ratio is limited once a certain length is reached.

Since there is some doubt about the maintenance of the integrity of the side supports or returns of the wall up to the point of buckling failure, AS3700 limits the assumption of two-way panel action to lightly loaded walls where the design compressive force (F_d) is less than 20% of the basic compressive capacity (F_o). For other walls, this effect is ignored, and one-way action is assumed.

Figure 26. Panel action in walls under light compressive loading

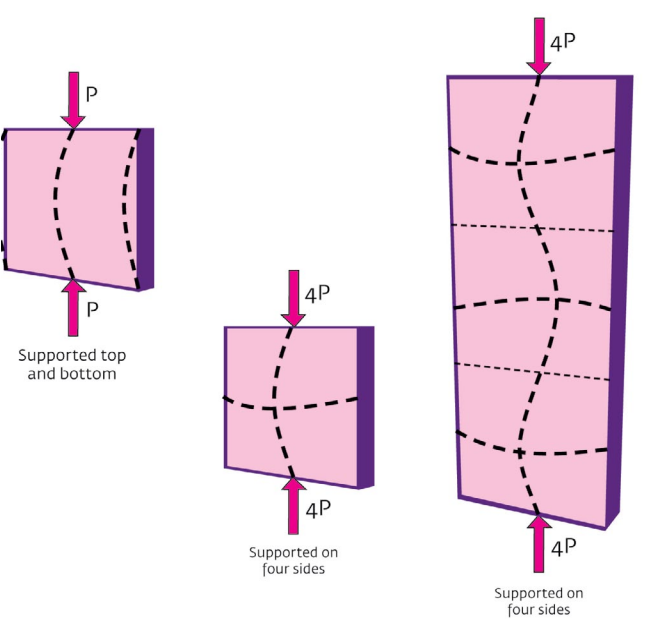
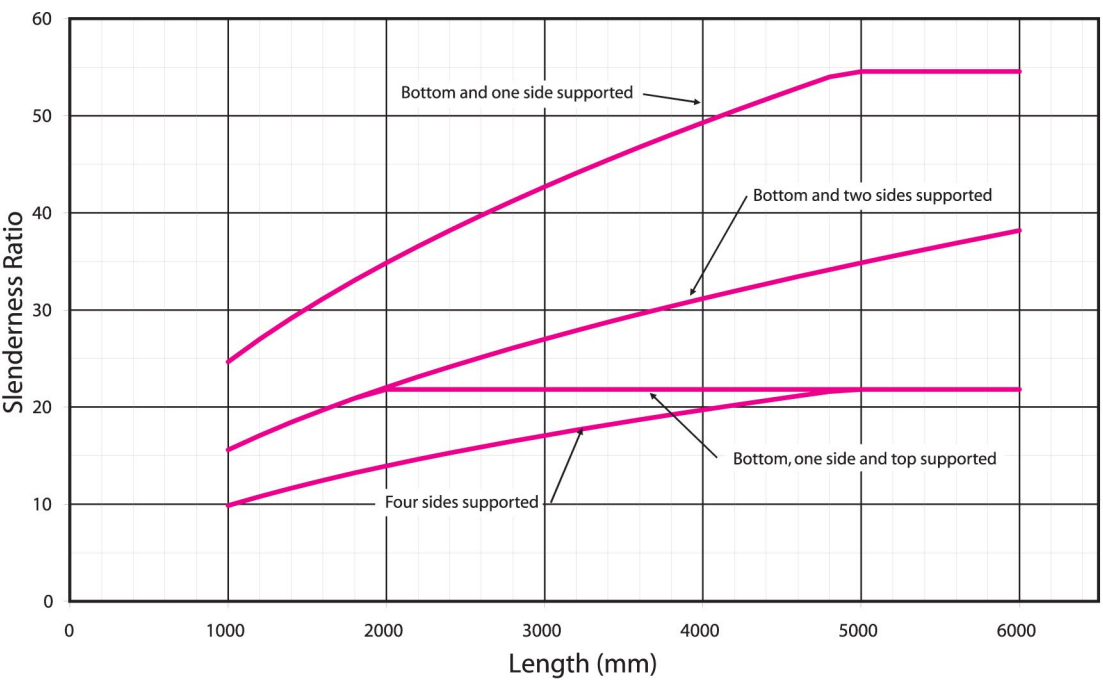


Figure 27. Slenderness ratios for various length and support conditions (walls 2.4m high and 110mm thick)



For lightly loaded walls, (where is F_d less than $0.20F_o$), the slenderness ratio is calculated using a vertical coefficient a_v and a horizontal coefficient a_h in combination with the height and length of the wall. The stiffening effect of any engaged piers is also considered. The horizontal coefficient depends on whether one or both vertical edges of the wall are supported. It is never considered justified to assume any degree of rotational restraint to the vertical edges of a wall. For all other walls, horizontal effects are ignored and a_h is taken as zero.

For a support to qualify as effective it must satisfy the AS 3700 conditions for load resistance and stiffness. The vertical coefficient is calculated taking into account the degree of lateral support and rotational restraint at the top and bottom of the wall, rather than being based on a few common cases as it is for simplified design. The calculation of slenderness ratios for the refined method is described and illustrated in Section 6.1

In addition to calculating the slenderness ratio, the refined calculation method requires an assessment of end

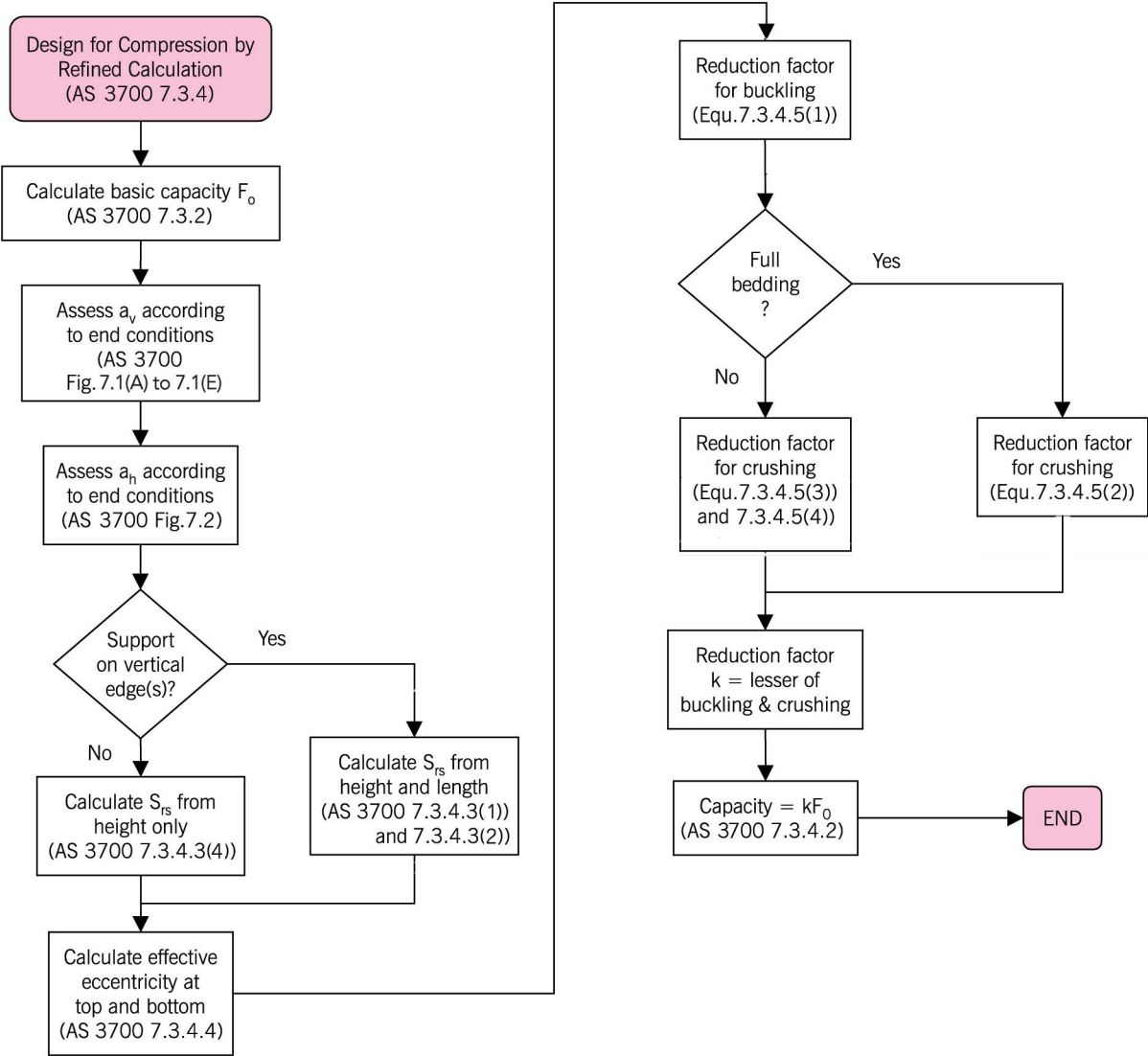
eccentricities at the top and bottom of the wall. AS 3700 permits this to be done by a rigorous calculation method or by certain simplifying assumptions. The assessment of eccentricity is illustrated in Section 6.2.

Once the slenderness ratio and end eccentricities have been determined, the reduction factor is calculated as the lesser of the two factors. The first of these accounts for the possibility of lateral buckling of the wall and the second accounts for the possibility of local crushing in the masonry. The worst of these two possibilities will govern in any particular case. The calculation of slenderness and eccentricity reduction factors is illustrated in 6.3.

Figure 28 is a flow chart summarising the design process for compression by refined calculation.

Worked example 9.2 shows the design of a wall in a residential walk-up building, illustrating that a more efficient solution can be achieved by refined calculation than by simple rules.

Figure 28. Flow chart for compression design by refined calculation



6.1 Slenderness ratio

For walls designed by refined calculation, the slenderness ratio takes into consideration a range of horizontal and vertical support conditions for the wall or pier, through the calculation of the vertical and horizontal slenderness coefficients, or a_v and a_h .

The possible values for a_v are as follows:

- 0.75 Where the wall is assessed as having lateral support and partial rotational restraint at the top and bottom.
- 0.85 Where the wall is assessed as having lateral support at both top and bottom and partial rotational restraint at only one of them.
- 1.0 Where the wall is assessed as having lateral support and no rotational restraint at both top and bottom.
- 1.5 Where the wall is assessed as having lateral support and partial rotational restraint at the bottom and only partial lateral support at the top.
- 2.5 Where the wall has no support at the top, in other words is freestanding.

Cases of walls where the various support and restraint conditions are deemed to be satisfied are illustrated in AS 3700 Figures 7.1 (A) to 7.1 (E).

The possible values of a_h are:

- 1.0 Where the wall is laterally supported along both vertical edges regardless of the rotational restraint)
- 2.5 Where a wall is laterally supported along one vertical edge, and unsupported along its other vertical edge.

Cases of walls where the various support conditions are deemed to be satisfied are illustrated in AS3700 Figure 2.

Figure 29 and Figure 30 show the slenderness ratios applicable for walls 2400mm high and 110 mm thick with support on three sides and four sides respectively. The limitation of slenderness ratio based on height and the effect of the slenderness coefficient, which depends on the support conditions at the top and bottom, can be clearly seen.

Figure 29. Slenderness ratios for walls 2.4 m high and 110 mm thick, with three edges supported

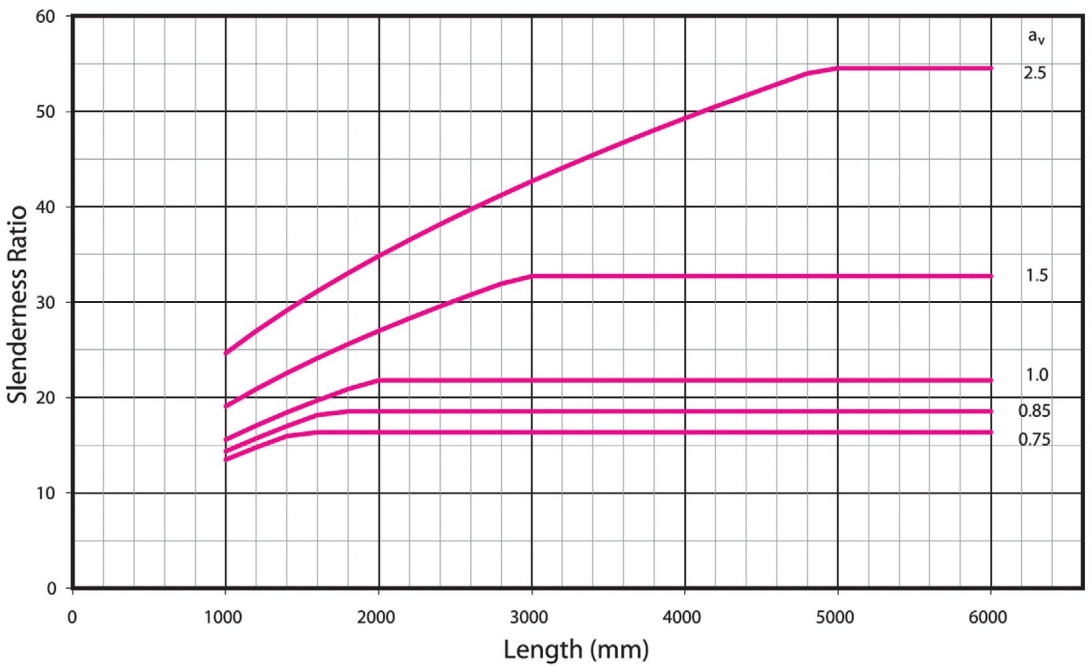
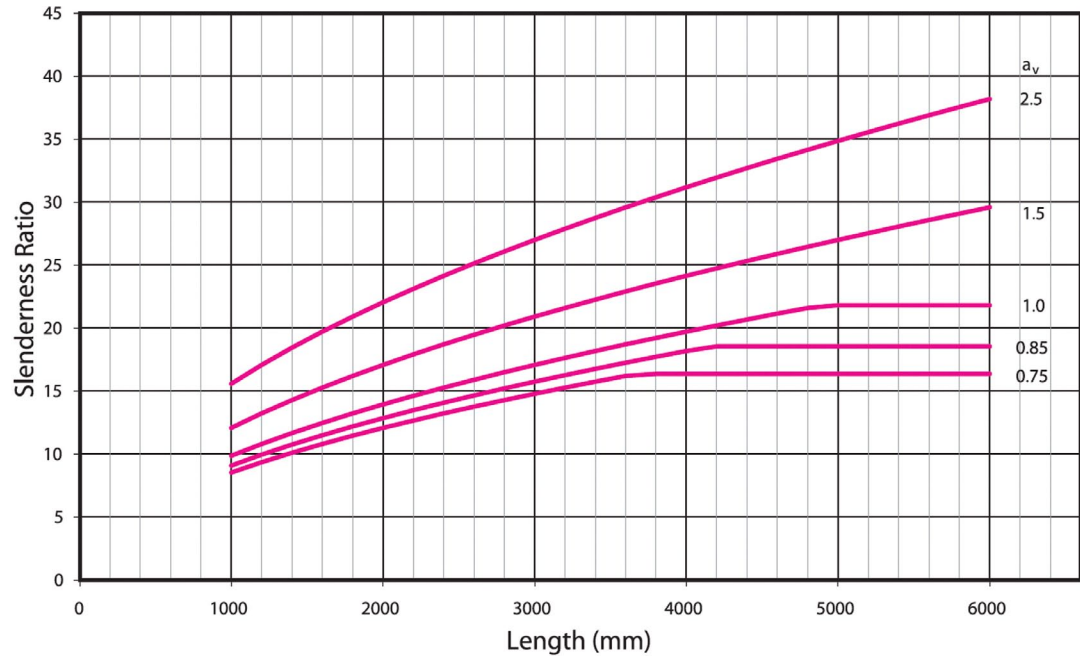


Figure 30. Slenderness ratios for walls 2.4 m high and 110 mm thick, with four edges supported



6.2 Assessment of eccentricity

The effective eccentricity at a joint between a masonry wall and a beam or slab that it supports is determined by the relative stiffness of the members and the degree of joint fixity that exists. Factors that influence the effective eccentricity include local crushing in the joint, opening up of the joint (relative rotation between wall and floor slab), non-linear material properties and two-way slab action. The degree of joint fixity is also influenced strongly by the magnitude of compressive load transmitted from the levels above.

Ideally for a wall supporting a slab or beam, with additional masonry above the level under consideration, assessment of eccentricity requires an equivalent frame analysis, which involves calculation of masonry stiffness above and below and calculation of the slab or beam stiffness and the relative rotation between it and the wall. This calculation is relatively complex and AS 3700 permits a simple assumption that, where a floor or roof frames into a wall, its load can be considered to act at an eccentricity of one-third of the bearing thickness. If

the floor or roof is continuous across the wall then each side is considered to be supported on half of the bearing thickness. In this approach, the vertical load coming from the level above is assumed to be axial at the level under consideration.

Figure 31 illustrates these simplified assumptions for assessment of eccentricity. For the case of a slab framing into either a solid wall or one leaf of a cavity wall (shown on the left) the resultant eccentricity is:

$$e = \frac{W_2 \times \frac{t_w}{6}}{W_1 + W_2}$$

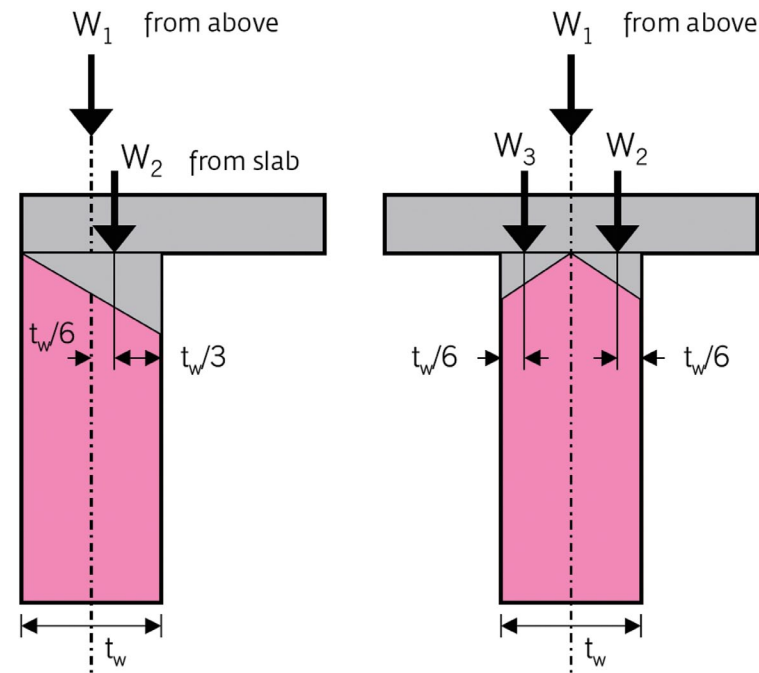
For the case of a slab continuous across a solid wall (shown on the right) the resultant eccentricity is:

$$e = \frac{(W_2 - W_1) \times \frac{t_w}{6}}{W_1 + W_2 + W_3}$$

In cases where the vertical stress coming into the joint from the wall above exceeds 0.25 MPa, a frame analysis can be carried out to determine the moment, and hence the equivalent eccentricity, at the joint. This frame analysis can take into account a partial fixity of the joint between the masonry and the supported member. For this frame analysis the remote ends of the members attached to the wall being designed can be assumed as pinned (see Figure 1). The commentary to AS 3700 illustrates some typical joint fixity factors from research on walls compressed between concrete slabs. These factors can be used to modify the results of a rigid-frame analysis.

In extreme cases, very high eccentricities can arise. These can be dealt with by inserting a strip of flexible packing material under the slab at the edge of the wall. This has the effect of permitting a higher joint rotation and relieving some of the effect of the eccentric load on the wall below the slab. For cases where high eccentricity is a risk, it is considered good practice to restrict the maximum eccentricity to one-third of the wall by this method.

Figure 31. Simplified assessment of eccentricity



6.3 Reduction factors for slenderness and eccentricity

Reduction factors for slenderness and eccentricity are calculated from several equations given in AS 3700 Clause 7.3.4.5. The equations for lateral instability (buckling) take into account slenderness ratio, the ratio of the larger end eccentricity to wall thickness (e_1/t_w), and the ratio of the smaller eccentricity to the larger end eccentricity (e_2/e_1) that is, the deflected shape of the wall. There is a lower cut-off eccentricity of 5% of the wall thickness, which is considered to be the inherent limit of construction accuracy. In other words, no wall is assumed to have perfectly aligned end bearings.

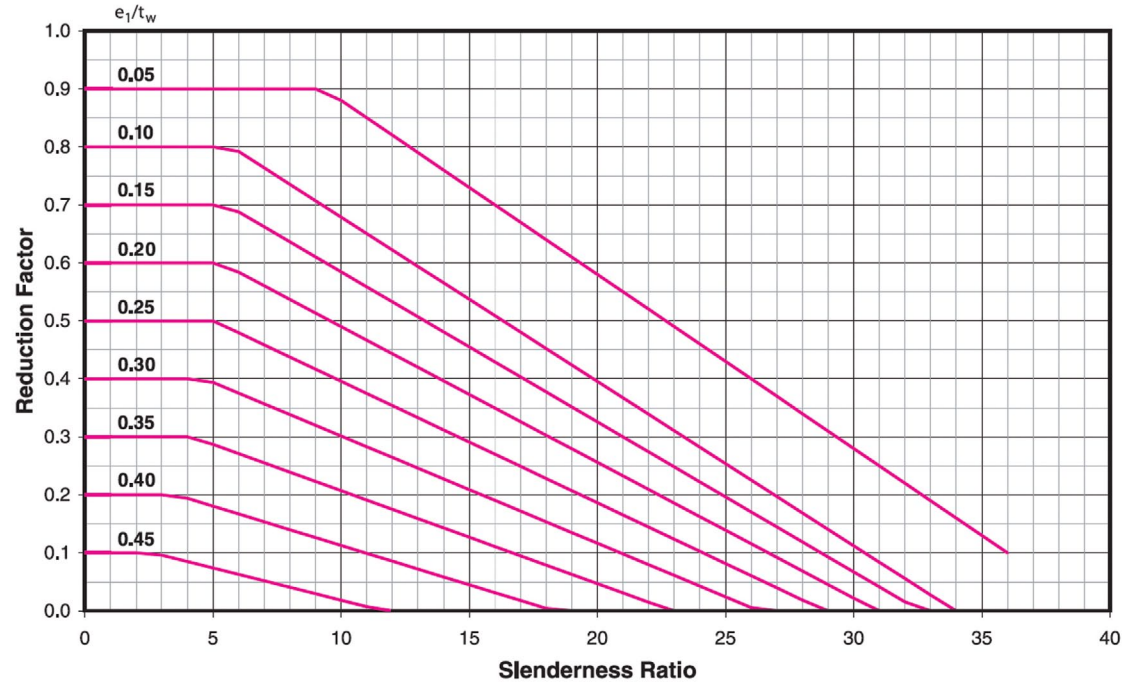
If the wall has one or both vertical edges supported, the slenderness ratio is calculated assuming panel action. It therefore cannot deform in double curvature and the reduction factor for lateral instability is based on $e_2/e_1 = 1$.

The equations for local crushing take into account the eccentricity ratio (for the larger end eccentricity) and, in the case of hollow units with face-shell bedding, the ratio of face-shell thickness to wall thickness.

The overall reduction factor for any given case is the lower of the separate factors for instability and local crushing.

Figure 32 shows the overall reduction factors for walls with supports to one or both vertical edges (that is, single curvature) and built with solid units, plotted against slenderness ratio. A range of eccentricity ratios from the minimum of 0.05 to the practical maximum of 0.45 is shown. The horizontal cut-off at the left-hand side of each line of the chart corresponds to the factor for local crushing.

Figure 32. Slenderness and eccentricity reduction factors for walls with edge supports (single curvature)

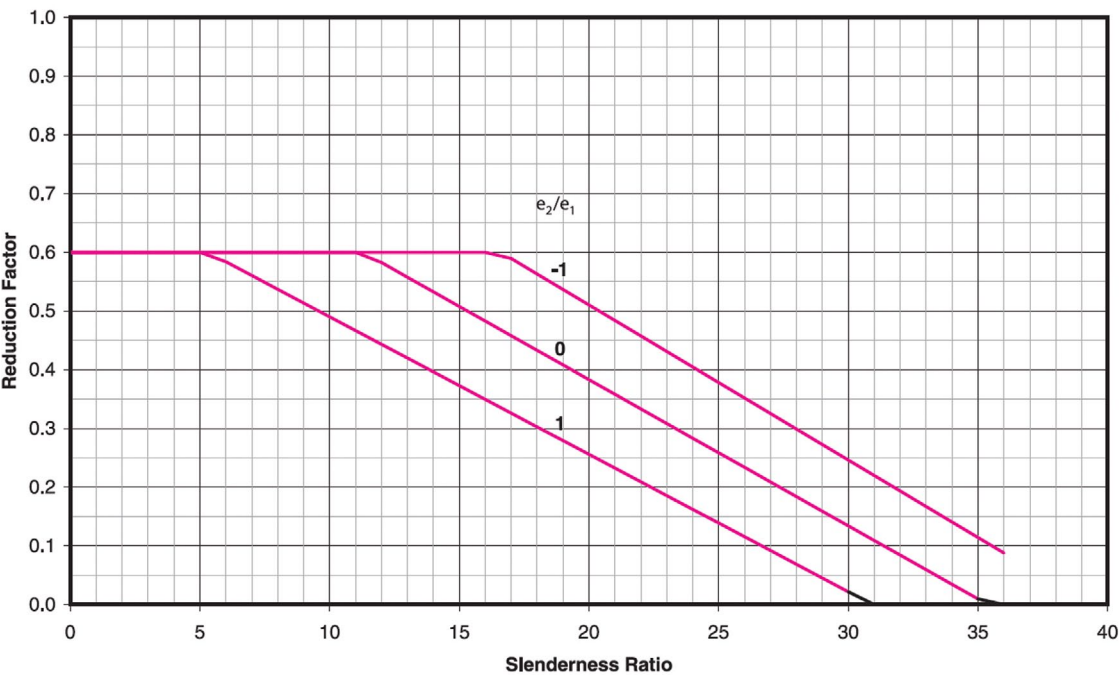


Similar relationships hold for walls with no support on their vertical edges, including cases of equal eccentricity at top and bottom, eccentricity at either top or bottom only, and opposite eccentricity at top and bottom (leading to double curvature). A typical example is shown in Figure 33 for walls built with solid units where the

ratio of the larger end eccentricity to the wall thickness is 0.2 and the eccentricity at the other end is equal but opposite in sign (double curvature), zero, or equal having the same sign (single curvature). The strengthening effects of double curvature can be seen.

7. Concentrated loads

Figure 33. Slenderness and eccentricity reduction factors for $e_1/t_w = 0.2$ showing the effect of end restraints and double curvature



When a concentrated load is applied to a masonry element, there is an enhancement of compressive (bearing) capacity immediately beneath the load because of the restraining effect of the surrounding, more lightly stressed, material. In AS 3700 a strength enhancement factor is used when checking the capacity of walls and piers supporting this type of load. Although the basic compressive capacity is used for design, failure under this type of loading would usually be by splitting as a result of induced tensile stress. The strength enhancement depends on the size of the loaded area as a proportion of the area of the member, as well as the location of the load, measured as the closest distance to the end or side of the member. Empirical equations are given in AS 3700 Clause 7.3.5.4 for this strength enhancement factor.

In addition to checking for crushing, the overall performance of the wall or pier must also be checked, assuming the concentrated load to be spread over a dispersion zone at mid-height.

This treatment of concentrated loads applies equally to situations where a load is supported by a wall or pier, in which the case load disperses downwards, and cases where a wall is supported on a small bearing area, in which case the load disperses upward through the wall. The discussion here concentrates on the more common case of a load bearing downwards on a wall.

In summary, design for concentrated loads requires two things:

- Check for crushing on the bearing area under the load (allowing for strength enhancement).
- Check for buckling in the dispersion zone at mid-height (overall member performance).

At the present time, the buckling check requires the refined approach to compressive strength design, the principles of which are outlined in Section 6.

The crushing capacity under a concentrated load is $k_b F_o$.

Where:

k_b = concentrated bearing factor

F_o = basic compressive strength capacity (see Section 4)

The effect of the code provisions is to limit the concentrated bearing factor k_b as follows:

- 1 for hollow masonry (because of the limited possibility for dispersion)
- not < 1 in any case
- not > 2 in any case
- not > 1.5 for a load at the end of a member (because of the limited possibility for dispersion)

This is achieved by taking the concentrated bearing factor as the lesser of the following equations:

$$k_b = 0.55 \frac{\left(1 + 0.5 \frac{a_1}{L}\right)}{\left(\frac{A_{ds}}{A_{de}}\right)^{0.33}}$$

and

$$k_b = 1.50 + \frac{a_1}{L}$$

Where:

a_1 = distance of bearing area from the end of the wall or pier

L = the length of the wall or pier

A_{ds} = the bearing area

A_{de} = the area of dispersion at mid-height (see Figure 35)

Figure 34 shows the concentrated bearing factors for solid, cored or grouted masonry walls or piers. The factor is plotted against the ratio of the loaded area A_{ds} to the mid-height area of the dispersion zone A_{de} . Two example cases are shown, namely the case of a concentrated load bearing at the centre of a wall $a_1/L=0.5$ and the case of a load bearing at the end of a wall $a_1/L=0$. The curves show the lower of the two equations in each case. All other cases will lie between these two extremes. The factor k_b is always greater than or equal to one.

Figure 34. Concentrated bearing factor for solid, cored or grouted masonry

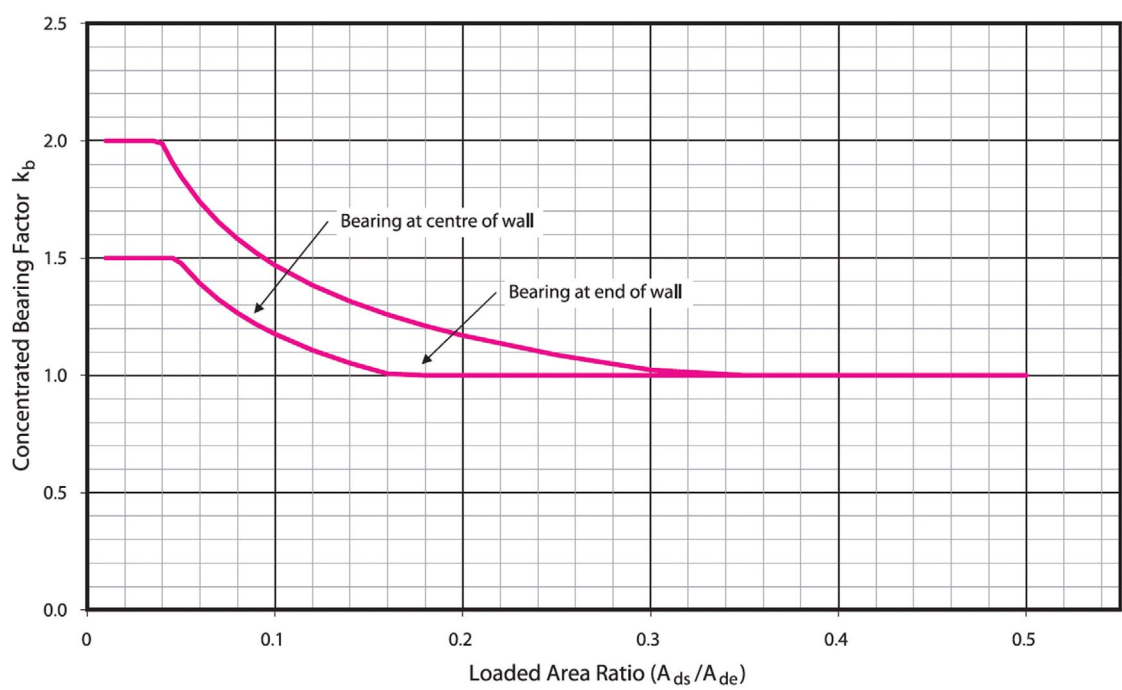
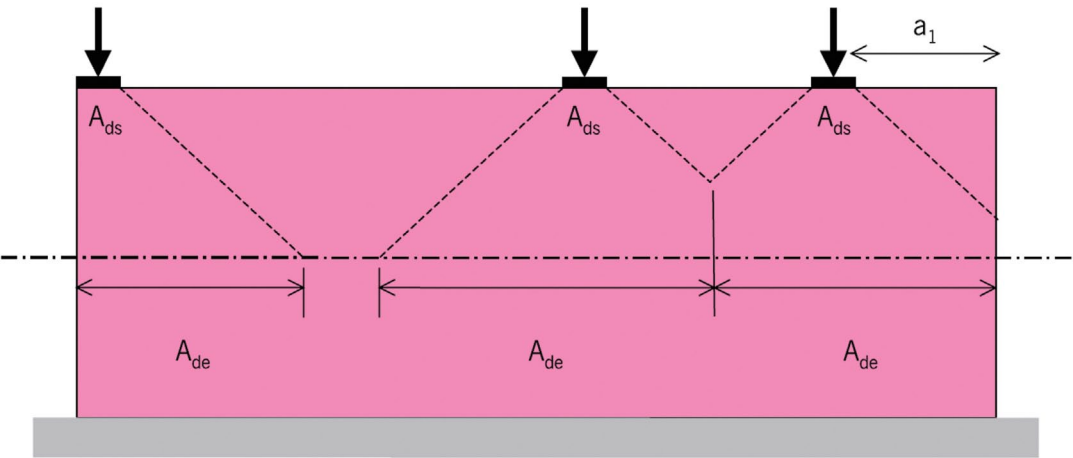


Figure 34 shows that the enhancement starts to operate when the loaded area falls below about 30% of the area in the dispersion zone for centre loading and about 15% of the area for end loading. Cut-offs apply for loaded areas below about 5% of the dispersion zone area.

Dispersion zones are calculated based on a 45° angle

of dispersion. They must also not overlap each other or extend beyond the end of a member. Various cases are illustrated on Figure 35. The area of the member in the dispersion zone at mid-height is the parameter A_{de} used for calculating the concentrated bearing factor and for checking the member against buckling under concentrated load.

Figure 35. Dispersion zones for concentrated loads acting on a wall



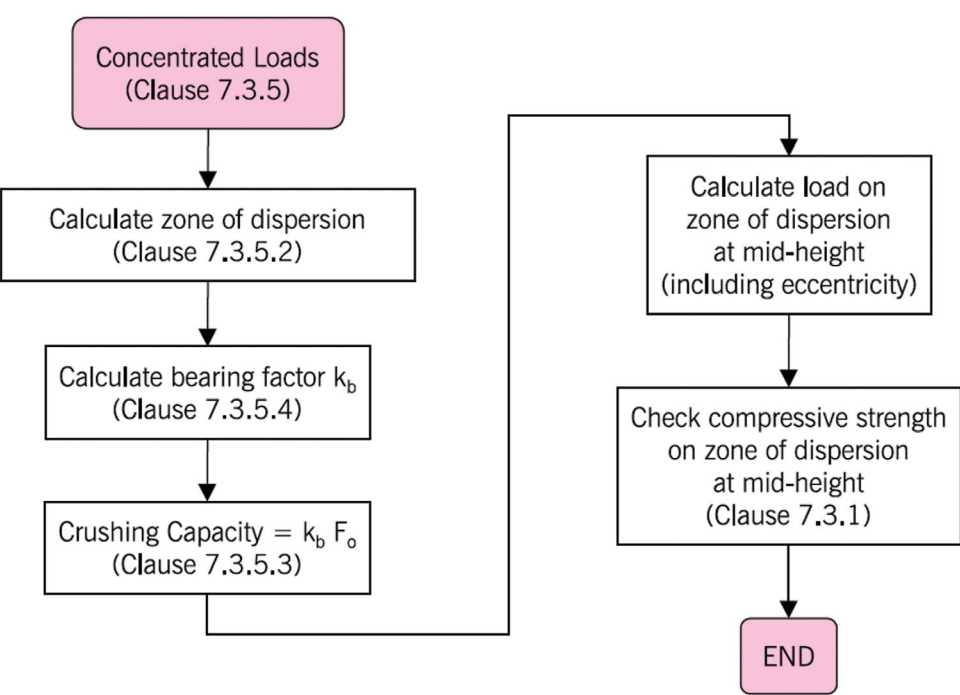
Buckling checks are carried out in accordance with the compressive load design (Section 6). The appropriate end eccentricity (or end support conditions) must be used as with the case of compressive load design.

The flow chart shown in Figure 36 summarises the procedures for concentrated load design.

Worked example 9.3 shows the design check of the bearing capacity for a beam supported on a masonry wall.

8. Composite action between walls and beams

Figure 36. Flow chart for concentrated load design



When a masonry wall is supported on a beam or slab of concrete or other material, the wall can act compositely with the supporting member to resist vertical loads. The resulting action is complex and depends on the relative stiffness of the wall and the supporting member. A common example of this is a lintel over an opening in a masonry wall, where the composite action is likely to be partial and is usually ignored. Another situation where composite action arises is for an infill wall supported on a slab or beam in a multistorey structure. For this case, assuming that the beam will support the full load of the wall can be unnecessarily conservative, and it can be advantageous to design for composite action.

A full design procedure for composite action is beyond the scope of this manual, but an outline of the behaviour and the important design parameters is given. Further information and a design procedure are discussed in *The Australian Masonry Manual*⁶.

8.1 Wall-beam interaction

A simple model of the interaction between a loaded wall and its supporting beam would be to consider that the beam supports only a triangular portion of the wall, and that the remainder of the masonry and the load from above is supported by arching between the supports. Figure 37 shows schematically this model behaviour.

However, such a model is of limited value because it does not provide information on the compressive stresses in the wall or an accurate picture of the forces in the beam, either of which can be critical to design. This model would significantly overestimate the forces in the beam.

Figure 38 shows a wall carrying vertical load supported on a beam and illustrates the main characteristics of the behaviour. Arching action develops in the wall, supported by a combination of the abutments (not shown in Figure 38) and the horizontal forces that develop at the wall-beam interface.

These horizontal forces are concentrated towards the ends of the beam and rely on friction developed at the interface. The resulting action is to develop a tensile force in the beam, superimposed on its bending action under the vertical load from the wall. As the vertical load progressively increases, a point is reached where the friction capacity is insufficient to maintain the horizontal force and composite action is lost. The masonry will then suffer tensile failure near the centre at the base of the wall and the beam will deflect excessively and fail.

The degree of arching depends on the relative stiffness of the wall and the beam. Both the flexural and axial stiffness of the beam are relevant. In the case of a stiff

Figure 37. Simple model of composite action behaviour

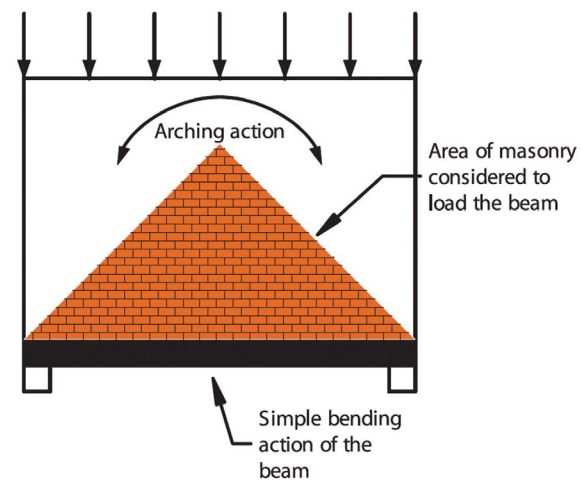
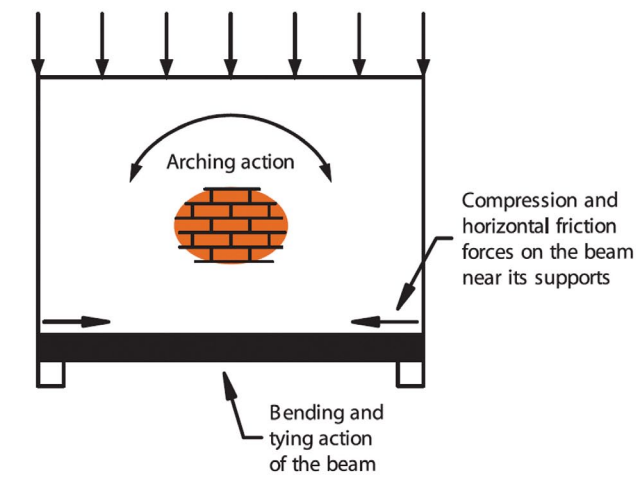


Figure 38. Composite action behaviour



wall and a very flexible beam, the latter is acting almost entirely as a tie to balance the horizontal compressive stresses in the wall. This is the action for a simple arch bar used over small openings. At the opposite extreme, where a flexible wall is supported on a stiff beam, the beam resists the load primarily by bending and the wall is subject to much lower stresses.

Another important consideration is the relative proportion of vertical load transmitted through the wall and that applied directly to the beam or slab. If the latter is high, the beam can separate from the base of the wall and the development of friction forces to promote arching can be compromised. The resulting horizontal crack at the base of the wall can be unsightly and the lack of sufficient arching might lead to overstressing of the masonry in the tension and result in a vertical crack at this point.

The main parameters for the design of this combination can be listed as follows:

- Vertical stress in the wall, resisted by the masonry compressive strength. The critical location for this action is adjacent to the supports and immediately above the beam. If the wall is heavily loaded in comparison to its compressive capacity this concentration of stress at the supports can lead to crushing.
- Axial force in the beam, resisted by its tensile strength. The tensile force in the beam will be a maximum at mid-span.
- Shear stress at the interface, resisted by friction between the masonry and the beam. The friction capacity of the interface is influenced by the materials of the wall and beam and the level of the compressive stress at the interface. Higher friction is usually likely for a concrete supporting member than for steel.

- Bending moment in the beam, resisted by its bending capacity. The beam must be designed for the combination of this bending moment and the superimposed tensile force.

In addition, the shear and tensile strengths of the masonry must be considered and the construction sequence can have a significant effect, because failure to prop the beam prior to building the masonry wall will increase the bending moments in the beam and reduce the capability for development of composite action.

Most design procedures for composite action are limited to cases where the height of the wall is at least 60% of the span of for the supporting member. Below this level, the composite action is usually considered to be partial and not reliable for design.

8.2 Composite action in walls with openings

Where walls with openings are seated on flexible supports, the resulting action depends on the location of the opening within the wall as well as its size. Central openings have only a small effect, because the wall has a capacity to arch above the opening and still develop the necessary composite action.

In the case of an opening near the ends of a wall (near the beam supports), the effect of the opening on the potential for composite action can be severe. In these cases, it has been suggested that the masonry between the opening and the end of the wall be designed as a column to withstand half of the vertical load transmitted through the wall.

Figure 39 shows schematically the distribution of vertical stresses imposed on a beam as a result of arching over a central opening and Figure 40 indicates the approximate shape of the distribution for an offset opening.

Figure 39. Composite action with a central opening

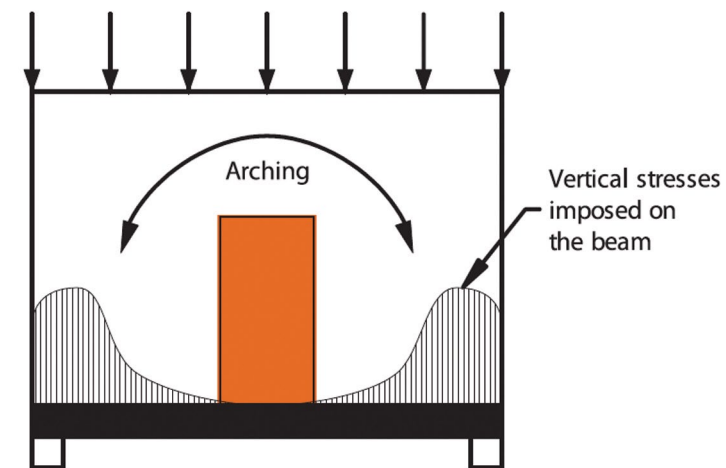
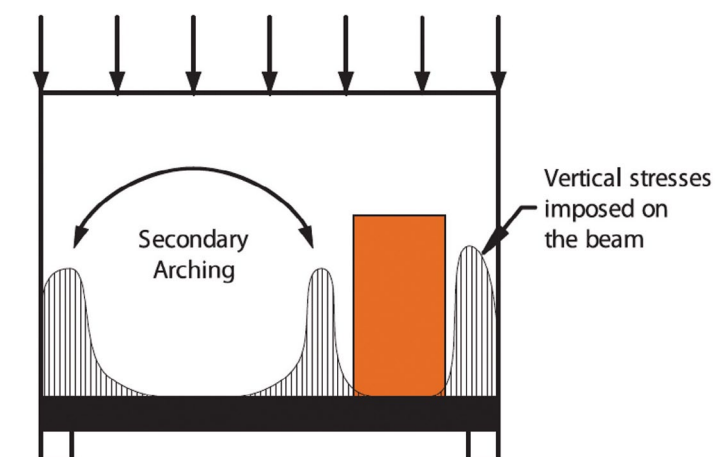


Figure 40. Composite action with an offset opening



9. Worked Examples

9.1 Simple wall

Design a wall 4m long and 2.7m high carrying a concrete slab with a total factored load on the wall of 50kN per metre. Support is provided by intersecting walls at the sides and the slabs above and below. The material is 90mm thick masonry with M3 mortar (1:1:6) and brick strength f'_{uc} of 20MPa.

Masonry strength:

$$f'_m = 6.3 \text{ MPa (AS 3700 Table 3.1).}$$

Bedded area:

$$\begin{aligned} A_b &= 1000 \times 90 \\ &= 90\,000 \text{ mm}^2 \text{ per metre} \end{aligned}$$

Basic compressive capacity (AS3700 Equation 7.3.2(1)):

$$F_o = \phi f'_m A_b$$

Capacity Reduction Factor $\phi = 0.75$ (AS3700 Table 4.1)

$$\begin{aligned} F_o &= 0.75 \times 6.3 \times 90\,000 \times 10^{-3} \\ &= 425 \text{ kN per metre} \end{aligned}$$

Slenderness ratio based on height (AS 3700 Equation 7.3.3.4):

$$S_{rs} = \frac{a_v H}{k_t t}$$

There are no engaged piers, so $k_t = 1.0$

Vertical slenderness coefficient $a_v = 1.0$

$$\begin{aligned} S_{rs} &= \frac{1.0 \times 2700}{1.0 \times 90} \\ &= 30 \end{aligned}$$

Wall capacity (AS 3700 Equation 7.3.3.2):

$$F_d \leq k F_o$$

Slenderness and eccentricity factor $k = 0.35$ (AS 3700 Table 7.1)

$$\begin{aligned} F_d &= 0.35 \times 425 \\ &= 149 \text{ kN per metre OK} \end{aligned}$$

9.2 Ground floor wall in a three-storey residential walk-up

Design a cavity wall 5m long and 2.7m high, supported on one vertical edge with a window at the other edge. The wall is at the ground floor level of an apartment building with a tiled roof and supports two higher floors having concrete slabs, which span 6m onto the inner leaf of the cavity wall and are subjected to floor live loads of 2kPa. The factored load on the top of the ground floor wall is 170kN per metre, comprising 100kN/m from above and 70kN/m from the first floor slab. The ground floor slab is on fill and isolated from the wall. The wall material is clay masonry with M3 mortar (1:1:6) and brick strength f'_{uc} of 20MPa.

Masonry strength:

$$f'_m = 6.3 \text{ MPa (AS 3700 Table 3.1).}$$

For a 110 mm leaf:

Bedded area:

$$\begin{aligned} A_b &= 1000 \times 110 \\ &= 110\,000 \text{ mm}^2 \text{ per metre} \end{aligned}$$

Basic compressive capacity (AS 3700 Equation 7.3.2 (1)):

$$F_o = \phi f'_m A_b$$

Capacity Reduction Factor $\phi = 0.75$ (AS3700 Table 4.1)

$$\begin{aligned} F_o &= 0.75 \times 6.3 \times 110\,000 \times 10^{-3} \\ &= 520 \text{ kN per metre} \end{aligned}$$

For the simplified method:

Slenderness ratio based on height (AS 3700 Equation 7.3.3.4):

$$S_{rs} = \frac{a_v H}{k_t t}$$

There are no engaged piers, so $k_t = 1.0$

Vertical slenderness coefficient $a_v = 1.0$

$$\begin{aligned} S_{rs} &= \frac{1.0 \times 2700}{1.0 \times 110} \\ &= 24.5 \end{aligned}$$

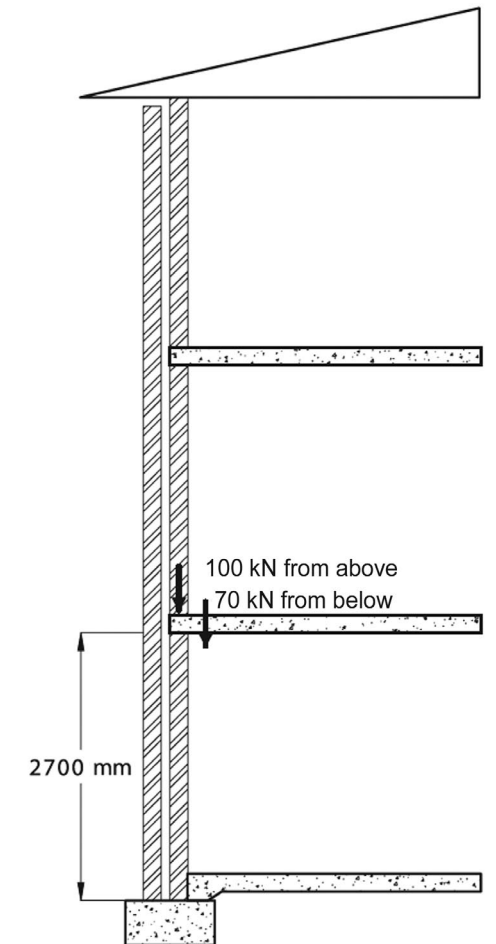
Slenderness and eccentricity factor (AS 3700 Clause 7.3.3.3 (a)):

$$\begin{aligned} k &= 0.67 - 0.02(S_{rs} - 14) \\ &= 0.67 - 0.02(24.5 - 14) \\ &= 0.46 \end{aligned}$$

Wall capacity (AS 3700 Equation 7.3.3.2):

$$F_d \leq k F_o$$

$$\begin{aligned} k F_o &= 0.46 \times 520 \\ &= 239 \text{ kN per metre OK} \end{aligned} \quad (> F_d = 170 \text{ kN per metre})$$



Try a 90 mm leaf:

Basic compressive capacity (AS 3700 Equation 7.3.2(1)):

$$F_o = \phi f'_m A_b$$

Capacity Reduction Factor $\phi = 0.75$ (AS3700 Table 4.1)

$$F_o = 0.75 \times 6.3 \times 90\,000 \times 10^{-3} \\ = 425 \text{ kN per metre}$$

Slenderness ratio based on height (AS 3700 7.3.3.4):

$$S_{rs} = \frac{a_v H}{k_t t}$$

Again, there are no engaged piers, so $k_t = 1.0$

Vertical slenderness coefficient $a_v = 1.0$

$$S_{rs} = \frac{1.0 \times 2700}{1.0 \times 90} \\ = 30$$

Slenderness and eccentricity factor (AS 3700 Clause 7.3.3.3 (a)):

$$k = 0.67 - 0.02(S_{rs} - 14) \\ = 0.67 - 0.02(30 - 14) \\ = 0.35$$

Wall capacity (AS 3700 Equation 7.3.3.2):

$$F_d \leq k F_o \\ k F_o = 0.35 \times 425 \\ = 149 \text{ kN per metre } \underline{\text{Too Low}} \quad (< F_d = 170 \text{ kN per metre})$$

For the refined calculation method:

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

F_d is greater than $0.20F_o$, therefore no panel action is considered.

Vertical slenderness coefficient $a_v = 0.75$ (AS 3700 7.3.4.3 (c) and Figure 7.1)

Slenderness ratio based on height (AS 3700 Equation 7.3.4.3 (4)):

$$S_r = \frac{0.75 \times 2700}{1.0 \times 110} \\ = 18.4$$

Assuming that the load from the slab acts at one-third of the bearing area from the loaded face of the wall (AS 3700 7.3.4.4), the eccentricity slab loading:

$$e_{\text{slab}} = \frac{t_w}{6} \\ = \frac{110}{6} \\ = 18.3 \text{ mm}$$

Load from above the first floor slab = 100kN (assumed axially applied).

Load from the first floor slab = 70kN/m acting at an eccentricity of 18.3mm.

Therefore, the effective eccentricity at the top of the wall:

$$e_1 = \frac{w_2 e_{\text{slab}}}{w_1 + w_2} \\ w_1 = 100 \text{ kN} \\ w_2 = 70 \text{ kN} \\ e_1 = \frac{70 \times 18.3}{100 + 70} \\ = 7.54 \text{ mm}$$

Eccentricity ratio :

$$\frac{e_1}{t_w} = \frac{7.54}{110} \\ = 0.069$$

At the base of the wall, no moment is introduced from the ground floor slab so $e_2 = 0$

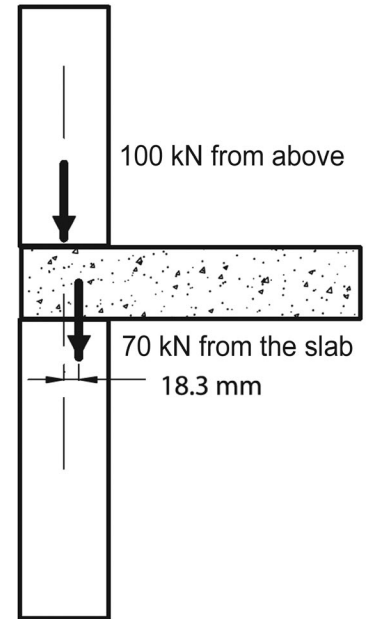
Therefore

$$\frac{e_2}{e_1} = 0$$

Reduction factor for instability (AS 3700 Equation 7.3.4.5 (1)):

$$k = 0.5 \left(1 + \frac{e_2}{e_1} \right) \left[\left(1 - 2.083 \frac{e_1}{t_w} \right) - \left(0.025 - 0.037 \frac{e_1}{t_w} \right) (1.33 S_r - 8) \right] \\ + 0.5 \left(1 - 0.6 \frac{e_1}{t_w} \right) \left(1 - \frac{e_2}{e_1} \right) (1.18 - 0.03 S_r) \\ = 0.5 \times [(1 - 2.083 \times 0.069) - (0.025 - 0.037 \times 0.069) \times (1.33 \times 18.4 - 8)] \\ + 0.5 \times [(1 - 0.6 \times 0.069) \times (1.18 - 0.03 \times 18.4)] \\ = 0.544$$

Note: this factor could be interpolated from AS 3700 Table 7.3.



Reduction factor for crushing (AS 3700 Equation 7.3.4.5 (2)):

$$\begin{aligned} k &= 1 - 2 \frac{e_1}{t_w} \\ &= 1 - 2 \times 0.069 \\ &= 0.862 \end{aligned}$$

Therefore the governing reduction factor $k = 0.544$

Wall capacity (AS 3700 Equation 7.3.4.2):

$$\begin{aligned} F_d &\leq kF_o \\ k F_o &= 0.544 \times 520 \\ &= 283 \text{ kN per metre } \underline{\text{OK}} \quad (> F_d = 170 \text{ kN per metre}) \end{aligned}$$

Try a 90 mm leaf:

Slenderness ratio based on height (AS 3700 Equation 7.3.4.3 (4)):

$$\begin{aligned} S_r &= \frac{a_v H}{k_t t} \\ &= \frac{0.75 \times 2700}{1.0 \times 90} \\ &= 22.5 \end{aligned}$$

The eccentricity of slab loading:

$$\begin{aligned} e_{\text{slab}} &= \frac{t_w}{6} \\ &= \frac{90}{6} \\ &= 15.0 \text{ mm} \end{aligned}$$

Therefore, the effective eccentricity at the top of the wall:

$$\begin{aligned} e_1 &= \frac{w_2 e_{\text{slab}}}{w_1 + w_2} \\ &= \frac{70 \times 15}{100 + 70} \\ &= 6.18 \text{ mm} \end{aligned}$$

Eccentricity ratio:

$$\begin{aligned} \frac{e_1}{t_w} &= \frac{6.18}{90} \\ &= 0.069 \end{aligned}$$

At the base of the wall, assume $e_2 = 0$

Therefore $\frac{e_2}{e_1} = 0$

Reduction factor for instability (AS 3700 Equation 7.3.4.5 (1)):

$$\begin{aligned} k &= 0.5 \left(1 + \frac{e_2}{e_1} \right) \left[\left(1 - 2.083 \frac{e_1}{t_w} \right) - \left(0.025 - 0.037 \frac{e_1}{t_w} \right) (1.33 S_r - 8) \right] \\ &\quad + 0.5 \left(1 - 0.6 \frac{e_1}{t_w} \right) \left(1 - \frac{e_2}{e_1} \right) (1.18 - 0.03 S_r) \\ &= 0.5 \times [(1 - 2.083 \times 0.069) - (0.025 - 0.037 \times 0.069) \times (1.33 \times 22.5 - 8)] \\ &\quad + 0.5 [(1 - 0.6 \times 0.069) \times (1.18 - 0.03 \times 22.5)] \\ &= 0.424 \end{aligned}$$

Reduction factor for crushing (AS 3700 Equation 7.3.4.5 (2)):

$$\begin{aligned} k &= 1 - 2 \frac{e_1}{t_w} \\ &= 1 - 2 \times 0.069 \\ &= 0.862 \end{aligned}$$

Therefore the governing reduction factor = 0.424

Wall capacity (AS 3700 Equation 7.3.4.2):

$$\begin{aligned} F_d &\leq kF_o \\ kF_o &= 0.424 \times 425 \\ &= 180 \text{ kN per metre } \underline{\text{OK}} \quad (> F_d = 170 \text{ kN per metre}) \end{aligned}$$

This illustrates that a better result can be achieved by using refined calculation because, using that approach, a 90mm leaf will resist the applied loads, whereas by the simple method a 110mm leaf is required.

10. References

9.3 Concentrated load

Check the bearing capacity for a beam resting on a 110mm leaf of clay masonry, 2.4m high and 3.0m long and applying a factored load 140kN. The length of the bearing is 200mm. The material is clay masonry with M4 mortar ($1:\frac{1}{2}:4\frac{1}{2}$) and brick strength f'_{uc} of 30MPa.

Masonry strength:

$$f'_m = 10.9 \text{ MPa (AS 3700 Table 3.1)}$$

Bedded area:

$$\begin{aligned} A_b &= 200 \times 110 \\ &= 22\,000 \text{ mm}^2 \text{ per metre} \end{aligned}$$

Basic compressive capacity (AS 3700 Equation 7.3.2(1)):

$$\begin{aligned} F_o &= \phi f'_m A_b \\ &= 0.75 \times 10.9 \times 22\,000 \times 10^{-3} \\ &= 180 \text{ kN per metre} \end{aligned}$$

Distance from the edge of the wall $a_1 = 0$

With load dispersion at 45° , the effective length of dispersion at mid-height is $1200+200$ mm, so the ratio of bearing area to area of dispersion:

$$\begin{aligned} \frac{A_{ds}}{A_{de}} &= \frac{200}{1400} \\ &= 0.143 \end{aligned}$$

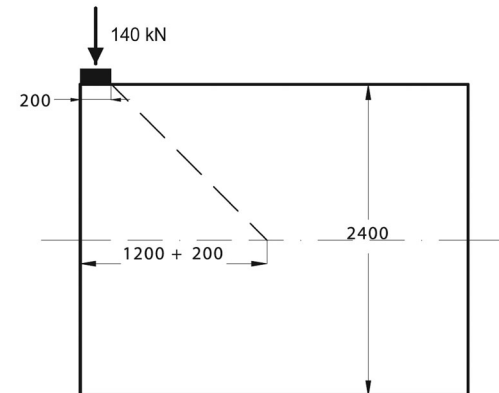
Concentrated bearing factor (AS 3700 Clause 7.3.5.4) is not less than 1.0 and is the lesser of 1.5 and (AS 3700 Equation 7.3.5.4 (1)):

$$\begin{aligned} k_b &= 0.55 \times \frac{(1 + 0.5 \frac{a_1}{L})}{(\frac{A_{ds}}{A_{de}})^{0.33}} \\ &= 0.55 \times \frac{1}{0.143^{0.33}} \\ &= 1.045 \end{aligned}$$

Therefore, bearing capacity (AS 3700 Equation 7.3.5.3)

$$\begin{aligned} k_b F_o &= 1.045 \times 180 \\ &= 188 \text{ kN} \quad \underline{\text{OK}} \quad (> F_d = 140 \text{ kN per metre}) \end{aligned}$$

Note: the overall adequacy of the wall under the concentrated load plus any other applied loads must also be checked using the refined calculation method as described in Section 6 and illustrated in Example 9.2



1. Manual 4, *Design of Clay Masonry for Wind & Earthquake*, Think Brick Australia, Sydney, 2013.
2. Manual 7, *Design of Clay Masonry for Serviceability*, Think Brick Australia, Sydney, 2013.
3. AS 3700 – 2011 *Masonry structures*, Standards Australia, Sydney, 2011.
4. AS 3700 – 2011 *Masonry structures – Commentary*, Standards Australia, Sydney, 2012.
5. Manual 10, *Construction Guidelines for Clay Masonry*, Think Brick Australia, Sydney, 2008.
6. Baker, L. R. Lawrence, S.J. & Page, A.W. *Australian Masonry Manual*, Joint Committee of the NSW Public Works Department and the Association of Consulting Structural Engineers of NSW, Sydney, 1991.

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