BRITISH STANDARD

BS 5628-2:2005

Code of practice for the use of masonry —

Part 2: Structural use of reinforced and prestressed masonry

ICS 91.080.30

British Standards

Committees responsible for this British Standard

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Brick Development Association
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British Precast Concrete Federation
CERAM Research Ltd.
Concrete Block Association
Construction Federation
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Mortar Industry Association Ltd.
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Contents

Foreword

This part of BS 5628 has been prepared under the direction of Technical Subcommittee B/525/6. It supersedes BS 5628-2:2000, which will be withdrawn on 31 March 2006.

This edition of BS 5628-2 introduces changes to reflect the recent publication of related European product standards. It does not reflect a full review or revision of the standard, although the opportunity has been taken to make minor technical changes in order to keep the standard up-to-date.

It has been assumed in the drafting of this code that the design of reinforced and prestressed masonry is entrusted to appropriately qualified and experienced persons, and the execution of the work is carried out under the direction of appropriately qualified supervisors.

As a code of practice, this British Standard takes the form of guidance and recommendations. It should not be quoted as if it were a specification and particular care should be taken to ensure that claims of compliance are not misleading.

Annexes A and D are normative. Annexes B, C and E are informative.

A British Standard does not purport to include all the necessary provisions of a contract. Users of British Standards are responsible for their correct application.

Compliance with a British Standard does not of itself confer immunity from legal obligations.

Summary of pages

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1 Scope

This part of BS 5628 gives recommendations for the structural design of reinforced and prestressed masonry constructed of brick or block masonry or masonry of manufactured stone or square dressed natural stone.

NOTE The dimensions of a member determined from strength considerations may not always be sufficient to satisfy requirements for other properties of the member such as resistance to fire and thermal insulation, and reference should be made to other appropriate standards.

2 Normative references

The following referenced documents are indispensable for the application of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

BS 410-1, *Test sieves — Technical requirements and testing — Test sieves of metal wire cloth.*

BS 410-2, *Test sieves — Technical requirements and testing — Test sieves of perforated metal plate.*

BS 3892-1, *Pulverized-fuel ash — Specification for pulverized-fuel ash for use with Portland cement.*

BS 4027, *Specification for sulfate-resisting Portland cement.*

BS 4449:1997, *Specification for carbon steel bars for the reinforcement of concrete.*

BS 4482, *Specification for cold reduced steel wire for the reinforcement of concrete.*

BS 4483, *Specification for steel fabric for the reinforcement of concrete.*

BS 4486, *Specification for hot rolled and hot rolled and processed high tensile alloy steel bars for the prestressing of concrete.*

BS 4729, *Specification for dimensions of bricks of special shapes and sizes.*

BS 4887-1, *Mortar admixtures — Part 1: Specification for air-entraining (plasticizing) admixtures.*

BS 5502-22, *Buildings and structures for agriculture. Code of practice for design, construction and loading.*

BS 5628-1:2005, *Code of practice for use of masonry — Part 1: Structural use of unreinforced masonry.*

BS 5628-3:2005, *Code of practice for use of masonry — Part 3: Materials and components, design and workmanship.*

BS 5896, *Specification for high tensile steel wire and strand for the prestressing of concrete.*

BS 6399-1, *Loading for buildings — Part 1: Code of practice for dead and imposed loads.*

BS 6399-2, *Loading for buildings — Part 2: Code of practice for wind loads.*

BS 6399-3, *Loading for buildings — Part 3: Code of practice for imposed roof loads.*

BS 6699, *Specification for ground granulated blastfurnace slag for use with Portland cement.*

BS 6744:2001, *Stainless steel bars for the reinforcement of and use in concrete — Requirements and test methods.*

BS 7979, *Specification for limestone fines for use with Portland cement.*

BS 8002, *Code of practice for earth retaining structures.*

BS 8110-1:1997, *Structural use of concrete — Part 1: Code of practice for design and construction.*

BS 8110-2:1985, *Structural use of concrete — Part 2: Code of practice for special circumstances.*

BS 8215, *Code of practice for design and installation of damp-proof courses in masonry construction.*

BS 8500-1, *Concrete — Complementary British Standard to BS EN 206-1 — Method of specifying and guidance for the specifier.*

BS 8500-2, *Concrete — Complementary British Standard to BS EN 206-1 — Specification for constituent materials and concrete.*

BS 8666, *Specification for scheduling, dimensioning, bending and cutting of steel reinforcement for concrete.*

BS EN 197-1:2000, *Cement — Composition, specifications and conformity criteria for common cements.*

BS EN 450-1, *Fly ash for concrete — Definition, specifications and conformity criteria.*

BS EN 771-1, *Specification for masonry unit — Clay masonry units.*

BS EN 771-2, *Specification for masonry units — Calcium silicate masonry units.*

BS EN 771-3, *Specification for masonry units — Aggregate concrete masonry units (dense and light-weight aggregates).*

BS EN 771-4, *Specification for masonry units — Autoclaved aerated concrete masonry units.*

BS EN 771-5, *Specification for masonry units — Manufactured stone masonry units.*

BS EN 771-6, *Specification for masonry units — Natural stone masonry units.*

BS EN 772-1, *Methods of test for masonry units — Determination of compressive strength.*

BS EN 772-2, *Methods of test for masonry units — Determination of percentage area of voids in aggregate concrete masonry units (by paper indentation).*

BS EN 772-11, *Methods of test for masonry units — Determination of water absorption of aggregate concrete, manufactured stone and natural stone masonry units due to capillary action and the initial rate of water absorption of clay masonry units.*

BS EN 772-16, *Methods of test for masonry units — Determination of dimensions.*

BS EN 845-1, *Specification for ancillary components for masonry — Ties, tension straps, hangers and brackets.*

BS EN 845-3, *Specification for ancillary components for masonry — Bed joint reinforcement of steel meshwork.*

BS EN 934-3, *Admixtures for concrete, mortar and grout — Admixtures for masonry mortar — Definitions, requirements, conformity, marking and labelling.*

BS EN 998-2, *Specification for mortar for masonry — Masonry mortar.*

BS EN 1015-2, *Methods of test for mortar for masonry — Bulk sampling of mortars and preparation of test mortars.*

BS EN 1015-11, *Methods of test for mortar for masonry — Determination of flexural and compressive strength of hardened mortar.*

BS EN 1052-1, *Methods of test for masonry — Determination of compressive strength.*

BS EN 1052-4, *Methods of test for masonry — Determination of shear strength including damp proof course.*

BS EN 10025-1, *Hot rolled products of structural steels — General technical delivery conditions.*

BS EN 10088-1, *Stainless steels — List of stainless steels.*

BS EN 12350-2, *Testing fresh concrete — Slump test.*

BS EN 12390-3, *Testing hardened concrete — Compressive strength of test specimens.*

BS EN 12878, *Pigments for the colouring of building materials based on cement and/or lime — Specifications and methods of test.*

BS EN 13877-3, *Concrete pavements — Specification for dowels to be used in concrete pavements.*

BS EN ISO 1461, *Hot dip galvanized coatings on fabricated iron and steel articles — Specifications and test methods.*

DD 86-1, *Damp-proof courses — Methods of test for flexural bond strength and short term shear strength.*

DD 86-2, *Damp-proof courses — Methods of test for creep deformation.*

3 Terms and definitions

For the purpose of this part of BS 5628 the definitions given in BS 5628-1 apply, together with the following.

3.1
masonry
assemblage of masonry units, either laid in situ or constructed in prefabricated panels, in which the masonry units are bonded and solidly put together with concrete and/or mortar so as to act compositely

3.2 Types of masonry

3.2.1
reinforced masonry
masonry in which steel reinforcement is incorporated to enhance resistance to tensile, compressive or shear forces

3.2.2
prestressed masonry
masonry in which pre-tensioned or post-tensioned steel is incorporated to enhance resistance to tensile or shear forces

3.3 Types of reinforced masonry

3.3.1
grouted-cavity
two parallel single-leaf walls spaced at least 50 mm apart, effectively tied together with wall ties. The intervening cavity contains steel reinforcement and is filled with infill concrete so as to result in common action with the masonry under load

3.3.2
pocket-type
masonry reinforced primarily to resist lateral loading where the main reinforcement is concentrated in vertical pockets formed in the tension face of the masonry and is surrounded by in situ concrete [see Figure 5a)]

3.3.3
Quetta bond
masonry at least one and a half masonry units thick in which vertical pockets containing reinforcement and mortar or concrete infill occur at intervals along its length

3.3.4
reinforced hollow blockwork
hollow blockwork that may be reinforced horizontally or vertically and subsequently wholly or partly filled with concrete [see Figure 5b)]

3.4 Types of geometric cross-section wall

3.4.1
diaphragm wall
two single-leaf walls structurally connected together by a series of cross webs of masonry

3.4.2
fin wall
wall with extended piers (fins) at frequent intervals constructed of masonry

3.5
prestressing tendon
high tensile steel wire, strand or bar pre-tensioned or post-tensioned to prestress masonry

3.6
shear tie
bed joint connector used to bond masonry units together in a cross-section in lieu of masonry bonding

3.7
effective depth
depth, in members in bending, from the compression face to the centroid of the longitudinal tensile reinforcement or prestressing tendons

3.8
shear span
ratio of maximum design bending moment to maximum design shear force

4 Symbols

The following symbols are used in this standard.

A_m	cross-sectional area of masonry (mm^2)
A_{ps}	area of prestressing tendons (mm^2)
A_s	cross-sectional area of primary reinforcing steel (mm^2)
A_{s1}	area of compression reinforcement in the most compressed face (mm^2)
A_{s2}	area of reinforcement in the least compressed face (mm^2)
A_{sv}	cross-sectional area of reinforcing steel resisting shear forces (mm^2)
A	shear span (mm)
a_v	distance from face of support to the nearest edge of a principal load (mm)
B	width of section (mm)
b_c	width of compression face midway between restraints (mm)
b_t	width of section at level of the tension reinforcement (mm)
C	lever arm factor
D	effective depth (mm) (see **3.7**)
d_c	depth of masonry in compression (mm)
d_o	overall depth of section (mm)
d_1	the depth from the surface to the reinforcement in the more highly compressed face (mm)
d_2	the depth of the centroid of the reinforcement from the least compressed face (mm)
E_c	modulus of elasticity of concrete (kN/mm^2)
E_m	modulus of elasticity of masonry (kN/mm^2)
E_u	worst credible earth or water load (N) (see **7.3**)
E_s	modulus of elasticity of steel (kN/mm^2)
e_x	resultant eccentricity in plane of bending (mm)
f_b	characteristic anchorage bond strength between mortar or concrete infill and steel (N/mm^2)
f_{ci}	strength of concrete at transfer (N/mm^2)
f_k	characteristic compressive strength of masonry (N/mm^2)
f_{kx}	characteristic flexural strength (tension) of masonry (N/mm^2)
f_p	stress due to prestress at the centroid of the section (N/mm^2)
f_{pb}	stress in tendon at the design moment of resistance of the section (N/mm^2)
f_{pe}	effective prestress in tendon after all losses have occurred (N/mm^2)
f_{pu}	characteristic tensile strength of prestressing tendons (N/mm^2)
f_s	stress in the reinforcement (N/mm^2)
f_{s1}	stress in the reinforcement in the most compressed face (N/mm^2)
f_{s2}	stress in the reinforcement in the least compressed face (N/mm^2)
f_t	diagonal tensile strength of masonry (N/mm^2)
f_v	characteristic shear strength of masonry (N/mm^2)
f_y	characteristic tensile strength of reinforcing steel (N/mm^2)
G_k	characteristic dead load (N)
g_B	design load per unit area due to loads acting at right angles to the bed joints (N/mm^2)
H	clear distance between lateral supports (in mm)
h_{ef}	effective height of wall or column (mm)
J	coefficient derived from Table 13

K_t	coefficient to allow for type of prestressing tendon
L	length of the wall (mm)
L	distance between end anchorages (mm)
l_t	transmission length (mm)
M	bending moment due to design load (N.mm)
M_a	increase in moment due to slenderness (N.mm)
M_d	design moment of resistance (N.mm)
M_x	design moment about the x axis (N.mm)
M'_x	effective uniaxial design moment about the x axis (N.mm)
M_y	design moment about the y axis (N.mm)
M'_y	effective uniaxial design moment about the y axis (N.mm)
N	design vertical load (N)
N_d	design vertical load resistance (N)
P	overall section dimension in a direction perpendicular to the x axis (mm)
Q	moment of resistance factor (N/mm^2)
Q_k	characteristic imposed load (N)
Q	overall section dimension in a direction perpendicular to the y axis (mm)
R	width of shear connector (mm)
S	spacing of shear connectors (mm)
s_v	spacing of shear reinforcement along member (mm)
T	overall thickness of a wall or column (mm)
t_{ef}	effective thickness of a wall or column (mm)
t_f	thickness of a flange in a pocket-type wall (mm)
U	thickness of shear connector (mm)
V	shear force due to design loads (N)
V	shear stress due to design loads (N/mm^2)
W_k	characteristic wind load (N)
Z	section modulus (mm^3)
Z	lever arm (mm)
γ_f	partial safety factor for load
γ_m	partial safety factor for material
γ_{mb}	partial safety factor for bond strength between mortar or concrete infill and steel
γ_{mm}	partial safety factor for compressive strength of masonry
γ_{ms}	partial safety factor for shear strength of masonry
γ_{mt}	partial safety factor for wall ties (including shear ties)
P	A_s/bd
Φt	nominal diameter of tendon

5 Alternative materials and methods of design and construction

The use of materials and methods that are not referred to in this code is acceptable, provided that the materials conform to the appropriate British Standards and that the methods of design and construction are such as to ensure that the strength and durability are at least equal to that recommended in this code.

Alternatively, the materials or methods may be proven by test. The test assembly should be representative as to materials, workmanship and details of the intended design and construction, and should be built under conditions representative of the conditions in the actual building construction.

6 Materials and components

6.1 General

Unless otherwise stated, the materials and components used in the construction of loadbearing walls should conform to the appropriate clause of BS 5628-3.

6.2 Masonry units

Masonry units intended for use in reinforced and prestressed masonry should be selected from the types listed below and should conform to the appropriate clause in the relevant British Standards:

clay masonry units	BS EN 771-1
calcium silicate masonry units	BS EN 771-2
aggregate concrete masonry units	BS EN 771-3
autoclaved aerated concrete masonry units	BS EN 771-4
manufactured stone masonry units	BS EN 771-5
natural stone masonry units	BS EN 771-6
bricks of special shapes and sizes	BS 4729

Selection of masonry units should follow the recommendations contained in BS 5628-3 in respect of durability and other considerations.

The tables and graphs in this part of BS 5628 cover masonry units of compressive strength 7 N/mm^2 or more, when tested in accordance with BS EN 772-1. However, this should not be taken to preclude the use of masonry units of lower strength for certain applications.

Masonry units that have been previously used should not be reused in reinforced and prestressed masonry unless they have been thoroughly cleaned and are generally in accordance with the recommendations of this code in respect of similar new materials.

6.3 Steel

6.3.1 *Reinforcing steel*

Reinforcing steel should conform to the relevant British Standard listed below.

Ribbed weldable steel reinforcing bars	BS 4449
Ribbed and plain round steel wire reinforcement	BS 4482
Steel fabric reinforcement	BS 4483
Plain round steel bars	BS EN 10025-2 or BS EN 13877-3
Stainless steel reinforcing bars	BS 6744, Designations 1.4301, 1.4429 or 1.4436 conforming to BS EN 10088-1
Plain round bars	BS EN 10025
Bedjoint reinforcement of welded wire meshwork	BS EN 845-3, excluding products not intended for structural (i.e. designed) applications

Reinforcement may be galvanized after manufacture in accordance with BS EN ISO 1461.

6.3.2 ***Prestressing steel***

Prestressing wire, strands and bars should conform to BS 4486 or BS 5896.

6.4 Damp-proof courses

Damp-proof courses (d.p.c.s) should conform to one of the British Standards, as appropriate, specified in BS 5628-3:2005, **4.7**.

Designers should pay particular attention to the characteristics of the materials chosen for d.p.c.s. Materials that squeeze out are undesirable in highly stressed walls, and the effect of sliding at the d.p.c. should be considered especially in relation to lateral loading.

Guidance on structural considerations affecting the selection of d.p.c.s is given in BS 8215 and characteristic strengths for some d.p.c. materials are given in DD 86-3. In general, advice on the resistance to compression, tension, sliding and shear should be sought from the manufacturers of the d.p.c.

Tests to determine the shear strength of d.p.c. materials should be in accordance with BS EN 1052-4. Tests to determine the flexural strength should be in accordance with DD 86-1, and tests to determine the deformation due to creep should be in accordance with DD 86-2.

6.5 Wall ties

Wall ties for low-lift grouted-cavity construction (see **11.2.2.2**) should conform to BS EN 845-1 and have a declared tensile load capacity not less than 5 000 N.

Details of a tie for high-lift grouted cavity walls that is suitable for resisting the bursting forces which occur during the cavity filling and compaction operations are given in Annex B. Protection against corrosion should follow the recommendations of **10.1.2.8**.

6.6 Cements

The following cements, or combination of cements, are suitable for use in mortars:

Cements:

Portland cement	BS EN 197-1 Notation CEM I
Limestone cement	BS EN 197-1 Notation CEM II/A-L and CEM II/A-LL
Sulfate-resisting Portland cement	BS 4027
Portland-slag cement	BS EN 197-1 Notation CEM or II/A-S or II/B-S
Portland-fly ash cement	BS EN 197-1 Notation CEM II/A-V or II/B-V

Combinations of cements:

a) Combinations produced in the mortar mixer from Portland cement CEM I conforming to BS EN 197-1 and ground granulated blastfurnace slag conforming to BS 6699 where the proportions and properties conform to CEM II/A-S or CEM II/B-S of BS EN 197-1: 2000, except Clause **9** of that standard.

b) Combinations produced in the mortar mixer from Portland cement CEM I conforming to BS EN 197-1 and limestone fines conforming to BS 7979 where the proportions and properties conform to CEM II/A-L or CEM II/A-LL of BS EN 197-1:2000, except Clause **9** of that standard.

c) Combinations produced in the mortar mixer from Portland cement CEM I conforming to BS EN 197-1 and pulverized fuel ash conforming to BS 3892-1, or to BS EN 450-1, where the proportions and properties conform to CEM II/A-V or CEM II/B-V of BS EN 197-1:2000, except Clause **9** of that standard.

The use of masonry cement and high alumina cement is not permitted.

6.7 Aggregate

Aggregate for mortar should follow the recommendations of BS 5628-3:2005, **4.3**.

Aggregate for concrete should be in accordance with BS 8500.

6.8 Masonry mortars

6.8.1 *General*

Mortars intended for use in reinforced and prestressed masonry should conform to BS EN 998-2. The use of mortars should be in accordance with the recommendations given in BS 5628-3. For site made mortars, the mixing of the mortar should be in accordance with BS 5628-3.

Mortars should be designed or prescribed. For designed mortars, the compressive strength of the mortar provides the control of the hardened mortar quality. When samples are taken from a designed mortar in accordance with BS EN 1015-2, and tested in accordance with BS EN 1015-11, the compressive strength of the mortar should not be less than the declared compressive strength. Table 1 shows the relationship of compressive strength classes to strength.

Table 1 — Masonry mortars

Mortar designation	Compressive strength class	Prescribed mortars (proportion of materials by volume) (see notes 1 and 2)		Compressive strength at 28 days
		cement[a] : lime : sand with or without air entrainment	cement[a] : sand with or without air entrainment	N/mm^2
(i)	M12	1 : 0 to ¼ : 3	—	12
(ii)	M6	1 : ½ : 4 to 4½	1 : 3 to 4	6

[a] Cement or combinations of cement in accordance with Clause 13

NOTE 1 Proportioning by mass will give more accurate batching than proportioning by volume, provided that the bulk densities of the materials are checked on site.

NOTE 2 When the sand portion is given as, for example, 5 to 6, the lower figure should be used with sands containing a higher proportion of fines whilst the higher figure should be used with sands containing a lower proportion of fines.

NOTE 3 Mortar of strength class M4 may only be used in walls incorporating bed joint reinforcement to enhance lateral load resistance (see Annex A). See BS 5628-1:2005, Table 1 for details of the mortar M4.

6.8.2 *Semi-finished factory made and pre-batched mortars*

Semi-finished factory made and pre-batched mortars should conform to the requirements of BS EN 998-2. For designed mortars, the manufacturer should declare the strength that he is offering; for prescribed mortars, the provisions given in **6.8.1** apply.

Where pre-mixed lime-sand mortars are used, the specified addition of cement on site should be gauged.

6.9 Concrete infill and grout

6.9.1 For certain reinforced masonry applications (see **10.1.2.5** and **10.1.2.6**) the concrete infill may comprise a mix consisting of the following proportions by volume of materials:

1 : 0 to ¼ : 3 : 2 cement : lime : sand : 10 mm nominal maximum size aggregate,

otherwise the concrete infill for reinforced masonry, pre-tensioned masonry and post-tensioned masonry should be specified in accordance with BS 8500-1. Specification may be by Designed, Prescribed, Standardized or Designated mix as appropriate to the requirements of use.

The maximum size of aggregate for concrete infill should not exceed the cover to any reinforcement, less 5 mm.

The recommendations for infill concrete, to ensure adequate reinforcement durability, are given in **10.1**.

6.9.2 The workability of all mixes should be appropriate to the size and configuration of the void to be filled and where slumps are specified these should be between 75 mm and 175 mm for unplasticized mixers, when tested in accordance with BS EN 12350-2. In order to ensure that complete filling and compaction is achieved, designers should consider the workability of the infill concrete appropriate to the height and least width of the pour. For small or narrow width sections, the use of plasticized or superplasticized mixes should be considered.

6.9.3 Where tendons are used in narrow ducts which cannot be filled using the appropriate infill concrete described in **6.9.1**, the ducts may be filled with a neat cement grout or a sand : cement grout with a minimum cube strength of 17 N/mm^2 at 7 days, when tested in accordance with BS EN 12390-3. Sand for grout should pass a 1.18 mm sieve conforming to BS 410-1 or -2.

6.10 Colouring agents for mortar

Colouring agents should conform to BS EN 12878 and their content by mass should not exceed 10 % (m/m) of the cement in the mortar. Carbon black colouring agent should be limited to 3 % (m/m) of the cement. The colouring agent should be evenly distributed throughout the mortar.

6.11 Admixtures

6.11.1 *General*

Admixtures should conform to BS EN 943-3.

Calcium chloride should never be added to masonry mortar or infill concrete.

The chloride ion content by mass of admixtures should not exceed 2 % (m/m) of the admixtures or 0.03 % (m/m) of the cement.

6.11.2 *Chlorides*

6.11.2.1 *Chlorides in sands*

The chloride ion content by mass of dry building sand should not exceed 0.15 % (m/m) of the cement.

6.11.2.2 *Chlorides in mixes*

The total chloride ion content of concrete and mortar mixes arising from aggregates and any other sources should not exceed the limits given in Table 2.

Table 2 — Chloride content of mixes

Type or use of concrete or mortar	Maximum total chloride content by mass of cement % (m/m)
Prestressed concrete; heat-cured concrete containing embedded metal	0.1
Concrete or mortar made with cement conforming to BS 4027	0.2
Concrete or mortar containing embedded metal and made with cement conforming to BS EN 197-1	0.4

7 Design objectives and general recommendations

7.1 Basis of design

7.1.1 *Limit state design*

7.1.1.1 The design of reinforced and prestressed masonry should provide an adequate margin of safety against the ultimate limit state. This is achieved by ensuring that the design strength is greater than or equal to the design load.

The design should be such that serviceability limit state criteria are met. Consideration should be given to the limit states of deflection and cracking and others where appropriate, e.g. fatigue.

7.1.1.2 Designers should consider whether the proportion of concrete infill in a given cross-section is such that the recommendations of BS 8110-1 would be more appropriate than the recommendations of this code.

7.1.2 *Limit states*

7.1.2.1 *Ultimate limit state*

The strength of the structure should be sufficient to withstand the design loads, taking due account of the possibility of overturning or buckling. The design loads and the design strengths of materials should be those recommended in **7.3** and **7.4** respectively, modified by the partial safety factors appropriate to the ultimate limit state given in **7.5.2**.

7.1.2.2 *Serviceability limit states*

7.1.2.2.1 *Deflection*

The deflection of the structure or any part of it should not adversely affect the performance of the structure or any applied finishes, particularly in respect of weather resistance.

The design should be such that deflections are not excessive, with regard to the requirements of the particular structure, taking account of the following recommendations.

a) The final deflection (including the effects of temperature, creep and shrinkage) of all elements should not, in general, exceed length/125 for cantilevers or span/250 for all other elements.

b) Consideration should be given to the effect on partitions and finishes of that part of the deflection of the structure taking place after their construction. A limiting deflection of span/500 or 20 mm, whichever is the lesser, is suggested.

c) If finishes are to be applied to prestressed masonry members, the total upward deflection, before the application of finishes, should not exceed span/300 unless uniformity of camber between adjacent masonry units can be ensured.

In any calculation of deflections (see Annex C) the design loads and the design properties of materials should be those recommended for the serviceability limit state in **7.3** to **7.5**. For reinforcement, stresses lower than the characteristic strengths given in Table 3 may need to be used to reduce deflection or control cracking.

7.1.2.2.2 *Cracking*

Fine cracking or opening up of joints can occur in reinforced masonry structures. However, cracking should not be such as to adversely affect the appearance or durability of the structure. The effects of temperature, creep, shrinkage and moisture movement will require the provision of movement joints (see BS 5628-3:2005, **5.4.2**) or other precautions.

7.2 Stability

7.2.1 *General considerations*

The designer responsible for the overall stability of the structure should ensure the compatibility of the design and details of parts and components. There should be no doubt as to who has responsibility for overall stability when some or all of the design and detailing is carried out by more than one designer.

To ensure a robust and stable design it will be necessary to consider the layout of the structure on plan, the interaction of the masonry elements and their interaction with other parts of the structure.

As well as the above general considerations, attention should be given to the following recommendations.

a) Buildings should be designed so that at any level they are capable of resisting a uniformly distributed horizontal load equal to 1.5 % of the total characteristic dead load above that level. This force may be apportioned between the structural elements according to their stiffness.

b) Robust connections should be provided between elements of the structure, particularly at floors and roofs. For guidance, see BS 5628-1:2005, Annex D.

c) Consideration should be given to connections between elements of different materials to ensure that any differences in their structural behaviour do not adversely affect the stability of the elements.

When bed joints are to be raked out for pointing, the designer should allow for the resulting loss of strength.

7.2.2 ***Earth-retaining and foundation structures***

The overall dimensions and stability of earth-retaining and foundations structures, e.g. the area of pad footings, should be determined by appropriate geotechnical procedures which are not considered in this code. However, in order to establish section sizes and reinforcement areas which will give adequate safety and serviceability without undue calculation, it is appropiate in normal design situations to apply values of the partial safety factor for load, γ_f , comparable to other forms of loading. The partial safety factor load, γ_f, should be applied to all earth and water loads unless they derive directly from loads which have already been factored in alternative ways to those described in **7.3** and **7.5.2.1**, in which case the loads should be derived to achieve equilibrium with other design loads. When applying γ_f no distinction is made between adverse and beneficial loads.

7.2.3 ***Accidental forces***

In addition to designing the structure to support loads arising from normal use, there should be a reasonable probability that it will not collapse catastrophically under the effect of misuse or accident. No structure can be expected to be resistant to the excessive loads or forces that could arise due to an extreme cause, but it should not be damaged to an extent disproportionate to the original cause.

Furthermore, owing to the nature of a particular occupancy or use of a structure (e.g. flour mill, chemical plant, etc.), it may be necessary in the design concept or a design appraisal to consider the effect of a particular hazard and to ensure that, in the event of an accident, there is an acceptable probability of the structure remaining after the event, even if in a damaged condition.

Where there is the possibility of vehicles running into and damaging or removing vital loadbearing members of the structure in the ground floor, the provision of bollards, walls, retaining earth banks, etc. should be considered.

All buildings should be robust against misuse and accidental forces that could arise. For Class 1 buildings (see BS 5628-1:2005, Table 11), no additional measures are likely to be necessary other than design to this code of practice and to BS 5628-1 as appropriate.

For Class 2A buildings (see BS 5628-1:2005, Table 11) the recommendations for Class 1 buildings are appropriate with the additional provision that effective horizontal ties, or effective anchorage of suspended floors to walls, should be installed. BS 5628-1 gives guidance on meeting these provisions.

For Class 2B buildings (see BS 5628-1:2005, Table 11) the recommendations for Class 1 buildings are appropriate with the additional provision that either effective horizontal ties and effective vertical ties to supporting walls and columns should be installed, or the notional removal of vertical loadbearing elements of the construction, one at a time, should be demonstrated to be possible without causing collapse. In the event that the notional removal of vertical loadbearing members cannot be accepted, such members should be designed as key elements. BS 5628-1 gives guidance on meeting these provisions.

7.2.4 ***During construction***

The designer should consider whether special precautions or temporary propping are necessary to ensure the overall stability of the structure or of individual elements during construction.

7.3 Loads

Ideally, the characteristic load on a structure should be determined statistically. Since it is not yet possible to express loads in this way the following should be used as characteristic loads.

a) *Characteristic dead load*. The characteristic dead load, G_k, is the weight of the structure complete with finishes, fixtures and partitions and should be taken as equal to the dead load as defined in, and calculated in accordance with, BS 6399-1.

b) *Characteristic imposed load*. The characteristic imposed load, Q_k, should be taken as equal to the imposed load as defined in, and calculated in accordance with, BS 6399-1 and -3.

c) *Characteristic wind load*. The characteristic wind load, W_k, should be taken as equal to the wind load as defined in, and calculated in accordance with BS 6399-2.

For the purposes of this code, worst credible earth and water loads, E_u, should be obtained in accordance with BS 8002. (See also **7.5.2.1**)

7.4 Structural properties and analysis

7.4.1 *Structural properties*

7.4.1.1 *Characteristic compressive strength of masonry,* f_k

7.4.1.1.1 *General*

The characteristic compressive strength of masonry, f_k, used in the design of a member should be that appropriate to the direction of the compressive force in the member.

7.4.1.1.2 *Direct determination of the characteristic compressive strength of masonry,* f_k

The characteristic compressive strength of masonry may be obtained from tests undertaken in accordance with BS EN 1052-1.

7.4.1.1.3 *Value of* f_k *where the compressive force is perpendicular to the bed face of the masonry unit*

Where no specific tests are carried out (see **7.4.1.1.2**), the value of f_k for a given masonry defined in terms of the compressive strength of the masonry units and the mortar strength class may be taken to be the characteristic compressive strength of masonry constructed with masonry units laid in the normal way under laboratory conditions and tested at an age of 28 days under axial compression in such a manner that the effects of slenderness may be ignored (see Table 3).

The value of f_k should be taken from the appropriate section of the table, using the following guidelines.

Table 3a) applies to masonry built with standard format bricks of clay or calcium silicate, conforming to the requirements of BS EN 771-1 or -2, and having no more than 25% formed voids (perforations) or 20% frogs.

The characteristic compressive strength of masonry of walls constructed with bricks of clay or calcium silicate that have more than 25% and less than 55% of formed voids (perforations), may be taken from the values given in Table 3a) multiplied by 0.8, provided that the compressive strength of the units does not exceed 55 N/mm^2.

When brick walls are constructed so that the thickness of the wall or loaded inner leaf of a cavity wall is equal to the width of a standard format brick, the values of f_k obtained from Table 3a) may be multiplied by 1.15.

NOTE This table is intended to cover normal size bricks which have an aspect ratio ≈ 0.63.

Table 3b) applies to masonry built with autoclaved aerated concrete blocks having a ratio of height to least horizontal dimension of 0.6.

Table 3c) applies to masonry built with aggregate concrete blocks and a ratio of height to least horizontal dimension of 0.6.

Table 3d) applies to masonry built with aggregate concrete blocks having not more than 25% of formed voids and a ratio of height to least horizontal dimension of between 2.0 and 4.5.

Table 3e) applies to masonry built with autoclaved aerated concrete blocks having a ratio of height to least horizontal dimension of between 2.0 and 4.5.

Table 3f) applies to masonry built with aggregate concrete blocks having more than 25% but less than 60% of formed voids and a ratio of height to least horizontal dimension of between 2.0 and 4.5.

Table 3g) applies to masonry built with solid aggregate concrete blocks having a block height/wall thickness ratio of between 1.0 and 1.2 as a collar jointed wall

Table 3h) applies to masonry built with solid aggregate concrete blocks laid flat having an as laid height/wall thickness ratio of between 0.4 and less than 0.6

The characteristic compressive strength of masonry bonded with thin layer mortar may be taken as the values given for mortar strength class M12 (mortar designation (i)) in Table 3.

7.4.1.1.4 *Value of f_k where the compressive force is parallel to the bed face of the masonry unit*

The value of f_k for masonry in which the compressive forces act parallel to the bed faces may be taken as follows:

a) for masonry units without formed voids, frogged bricks where the frogs are filled, and blocks with filled formed voids, the strength obtained from the appropriate item of **7.4.1.1.3**;

b) for bricks with formed voids and bricks with perforations, the characteristic compressive strength determined in accordance with **7.4.1.1.2** or, where no test data are available, one-third of the strength obtained from the appropriate item of **7.4.1.1.3**;

c) for blocks with unfilled formed voids, the characteristic compressive strength given in Table 3, using the strength of the block determined in the direction parallel to the bed face of the masonry unit.

7.4.1.1.5 *Value of f_k for masonry units of unusual format or for unusual bonding patterns*

The value of f_k for masonry constructed with masonry units of unusual formats, or with an unusual bonding pattern, may be taken as follows:

a) for brick masonry, the values determined by test in accordance with **7.4.1.1.2**, provided that the value of f_k is not taken to be greater than the appropriate value given in Table 3;

b) for block masonry, the value given in Table 3, using the strength of the block determined in the appropriate aspect.

7.4.1.2 *Characteristic compressive strength of masonry in bending*

For a given masonry construction defined in terms of the compressive strength of the masonry units and mortar designation, the value of f_k derived from **7.4.1.1** may be taken to be the characteristic compressive strength of masonry in bending.

7.4.1.3 *Characteristic shear strength of masonry, f_v*

7.4.1.3.1 *General*

When designing masonry for shear, care should be taken in the use of d.p.c. materials that might reduce the bending and shear strengths of the masonry. Recommended test methods are given in DD 86-1 and BS EN 1052-4.

When it is proposed to use masonry units containing greater than 40 % formed voids, the designer should be satisfied that the required characteristic shear strength can be achieved.

7.4.1.3.2 *Shear in bending (reinforced masonry)*

Characteristic shear strength may be calculated by one of two alternative methods appropriate to whether reinforcement is contained in mortar or in concrete infill.

a) For reinforced sections in which the reinforcement is placed in bed or vertical joints, including Quetta bond and other sections where the reinforcement is wholly surrounded with mortar strength class M12 or M6 (see Table 1), the characteristic shear strength, f_v, may be taken as 0.35 N/mm^2.

No enhancement of characteristic shear strength, f_v, is to be used for the area of primary reinforcing steel provided or for situations where the ratio of the shear span to the effective depth is 2 or greater.

For simply supported beams or cantilevers where the ratio of the shear span (see **3.8**) to the effective depth (see **3.7**) is less than 2, f_v may be increased by the following factor:

$2d/a_v$

where

d is the effective depth;

a_v is the distance from the face of the support to the nearest edge of a principal load;

provided that f_v is not taken to be greater than 0.7 N/mm^2.

At sections in certain laterally loaded walls there may be substantial compressive stresses from vertical loads. In such cases the shear may be adequately resisted by the plain masonry (see BS 5628-1:2005, Clause **22**).

b) For reinforced sections in which the main reinforcement is placed within pockets, cores or cavities filled with concrete infill as defined in **6.9.1**, the characteristic shear strength of the masonry, f_v, may be obtained from the following equation:

$f_v = 0.35 + 17.5\,\rho$

where

$\rho = A_s/bd$

A_s is the cross-sectional area of primary reinforcing steel:

b is the width of section:

d is the effective depth;

provided that f_v is not taken to be greater than 0.7 N/mm^2.

For simply supported reinforced beams or cantilever retaining walls where the ratio of the shear span, a, to the effective depth, d, is six or less, f_v may be increased by a factor $\{2.5 - 0.25\,(a/d)\}$ provided that f_v is not taken to be greater than 1.75 N/mm^2.

7.4.1.3.3 *Racking shear in reinforced masonry shear walls*

When designing reinforced masonry shear walls the characteristic shear strength of masonry, f_v, may be taken to be:

$0.35 + 0.6g_B$, with a maximum of 1.75 N/mm^2

where

g_B is the design load per unit area normal to the bed joint due to the loads calculated for the appropriate loading condition detailed in **7.5**.

Alternatively, for unreinforced sections in which the main reinforcement is placed within pockets, cores or cavities filled with concrete infill as defined in **6.9.1**, the characteristic shear strength of masonry, f_v, may be taken to be 0.7 N/mm^2 provided that the ratio of height to length of the wall does not exceed 1.5.

7.4.1.3.4 *Shear in prestressed sections*

For prestressed sections with bonded or unbonded tendons, the design shear strength of masonry may be determined directly from a consideration of the characteristic diagonal tensile strength of masonry and the prestress (see **9.2.3.1**).

Table 3 — Characteristic compressive strength masonry, f_k, in N/mm^2

a) — Constructed with standard format bricks of clay and calcium silicate having no more than 25% of formed voids, or 20% frogs

Mortar strength Class/Designation	Compressive strength of unit (N/mm^2)[a]										
	7	**10**	**15**	**20**	**30**	**40**	**50**	**75**	**100**	**125**	**150**
M12 / (i)	3.2	4.0	5.3	6.4	8.3	10.0	11.6	15.2	18.3	21.2	23.9
M6 / (ii)	3.1	3.8	4.8	5.6	7.1	8.4	9.5	12.0	14.2	16.1	17.9

b) — Constructed with autoclaved aerated concrete blocks having a ratio of height to least horizontal dimension of 0.6

	Compressive strength of unit (N/mm^2)[a]	
	7.3	**10.4**
M12 / (i)	3.4	4.4
M6 / (ii)	3.2	4.2

c) — Constructed with aggregate concrete blocks having a ratio of height to least horizontal dimension of 0.6

	Compressive strength of unit (N/mm^2)[a]					
	7.3	**10.4**	**17.5**	**22.5**	**30**	**40 or greater**
M12 / (i)	3.4	4.4	6.3	7.5	9.5	11.2
M6 / (ii)	3.2	4.2	5.5	6.5	7.9	9.3

d) — Constructed with aggregate concrete blocks having not more than 25% of formed voids and a ratio of height to least horizontal dimension of between 2.0 and 4.5

	Compressive strength of unit (N/mm^2)[a]					
	7.3	**10.4**	**17.5**	**22.5**	**30**	**40 or greater**
M12 / (i)	6.8	8.8	12.5	15.0	18.7	22.1
M6 / (ii)	6.4	8.4	11.1	13.0	15.9	18.7

e) — Constructed with autoclaved aerated concrete blocks having a ratio of height to least horizontal dimension of between 2.0 and 4.5

	Compressive strength of unit (N/mm^2)[a]	
	7.3	**10.4**
M12 / (i)	6.8	8.8
M6 / (ii)	6.4	8.4

f) — Constructed with aggregate concrete blocks having more than 25% but less than 60% of formed voids and a ratio of height to least horizontal dimension of between 2.0 and 4.5

	Compressive strength of unit (N/mm^2)[a]					
	7.3	**10.4**	**17.5**	**22.5**	**30**	**40 or greater**
M12 / (i)	6.6	8.1	11.2	13.1	16.0	19.4
M6 / (ii)	6.4	7.5	9.9	11.6	14.0	16.7

g) — Constructed with solid aggregate concrete blocks having a block height / wall thickness ratio of between 1.0 and 1.2 as a collar jointed wall

	Compressive strength of unit (N/mm^2)[a]					
	7.3	**10.4**	**17.5**	**22.5**	**30**	**40 or greater**
M12 / (i)	5.5	7.0	9.7	11.6	14.2	16.5
M6 / (ii)	4.5	5.7	7.9	9.4	11.6	13.5

h) — Constructed with solid aggregate concrete blocks laid flat having an as laid height/ wall thickness ratio of between 0.4 and less than 0.6

	Compressive strength of unit (N/mm^2)[a]					
	7.3	**10.4**	**17.5**	**22.5**	**30**	**40 or greater**
M12 / (i)	5.6	7.2	9.7	11.3	13.5	15.4
M6 / (ii)	4.6	5.9	7.9	9.1	10.8	12.3

[a] Measured in normal direction of test for units

7.4.1.4 *Characteristic strength of reinforcing steel, f_y*

The characteristic tensile yield strength of reinforcement, f_y, is given in Table 4. To obtain the corresponding compressive strength, the given value should be multiplied by a factor 0.83.

Table 4 — Characteristic tensile yield strength of reinforcing steel, f_y

Designation	Grade	Nominal size	Characteristic tensile yield strength, f_y MPa
Steel reinforcement, conforming to BS 4449	500	All	500
Steel wire reinforcement, conforming to BS 4482	250	≤ 12 mm	250
Steel wire reinforcement, conforming to BS 4482	500	≤ 12 mm	500
Steel reinforcement used in fabric, conforming to BS 4483	500	All	500
Plain dowel bars, conforming to BS EN 10025-2 or BS EN 13877-3	—	>12 mm	235
Plain stainless steel bars, conforming to BS 6744	200	All	200
Ribbed stainless steel bars, conforming to BS 6744	500	All	500
Bedjoint reinforcement of welded wire mesh, conforming to BS EN 845-3			Value of characteristic yield strength of longitudinal wires as declared, but ≤ 500
NOTE If stainless steel bars are welded, the characteristic strength in the heat affected zone should be reduced to 180 MPa, or to a value that can be justified by test data, whichever is the greater.			

7.4.1.5 *Characteristic breaking load of prestressing steel*

The characteristic breaking load of prestressing wire, strand and bar should be that specified in BS 4486 or BS 5896, as appropriate.

7.4.1.6 *Characteristic anchorage bond strength, f_b*

The characteristic anchorage bond strength, f_b, between the reinforcement and the mortar or concrete infill should be taken from Table 5. The values given apply to ribbed bars as defined in BS 4449.

In forms of construction not covered in Table 5 or where stainless steel reinforcement or reinforcement other than ribbed bars as defined in BS 4449 is used, tests described in BS 4449:2005, Annex A should be undertaken.

Table 5 — Characteristic anchorage bond strength

Form of construction	Bar type	Concrete infill or mortar strength class / designation	Recommended bond strength N/mm²	
Quetta bond	Plain and ribbed	M12 / (i) and M6 / (ii)	1.5	
Reinforced[c] bed joints	Plain	M12 / (i) and M6 / (ii)	0.7[a]	
	Ribbed	M12 / (i) and M6 / (ii)	2.0[b]	
Grouted cavity construction			Bars ≤ 12 mm	Bars > 12 mm
	Plain	1 : ¼ : 3 : 2[d]	1.8	1.4
	Plain	C25/30 (or stronger)	1.8	1.4
	Ribbed	1 : ¼ : 3 : 2[d]	3.4	2.5
	Ribbed	C25/30 (or stronger)	4.1	3.4
Pocket type construction	Plain	C25/30 (or stronger)	1.4	
	Ribbed	C25/30 (or stronger)	1.4	
Reinforced hollow blockwork	Plain	C25/30 (or stronger)	3.4	
	Ribbed	C25/30 (or stronger)	4.1	

[a] 0.5 N/mm² can be used in strength class M4 mortar.
[b] Value also applies to strength class M4 mortar.
[c] Where ribbed bars no greater than 12 mm in diameter are built into horizontal voids no greater than 60 mm deep or 120 mm wide, formed in beams and filled with mortar, the values for reinforced bed joints may be used.
[d] See **6.9.1**.

7.4.1.7 *Elastic moduli*

Where elastic methods of analysis are adopted, the following elastic moduli may be used in the absence of relevant test data:

a) for clay, calcium silicate and concrete masonry, including reinforced masonry with infill concrete, the short term elastic modulus, $E_m = 0.9\ f_k$ kN/mm²;

b) for concrete infill used in prestressed masonry, the appropriate value of the elastic modulus E_c as given in Table 6;

c) for all steel reinforcement and all types of loading, the elastic modulus, E_s = 200 kN/mm²;

d) for prestressing tendons, the appropriate value of E_s as given in Figure 4.

Table 6 — Elastic modulus for concrete infill, E_c

28 day cube strength N/mm²	E_c kN/mm²
20	24
25	25
30	26
40	28
50	30
60	32

7.4.2 *Analysis of structure*

When analysing any cross-section within the structure, the properties of the materials should be assumed to be those associated with their design strengths appropriate to the limit state being considered. Due allowance should be made when materials with different properties are used in combination. Where the member to be designed forms part of an indeterminate structure, the method of analysis employed to determine the forces in the member should be based on as accurate a representation of the behaviour of the structure as is practicable.

When elastic analysis is used to determine the force distribution throughout the structure, the relative stiffnesses of the members may be based throughout on any one of the following cross-sections:

a) the entire masonry section, ignoring the reinforcement;

b) the entire masonry section including the reinforcement on the basis of the modular ratio derived from the appropriate values of modulus of elasticity given in **7.4.1.7**;

c) the compression area of the masonry cross-section combined with the reinforcement on the basis of the modular ratio as derived in b).

7.5 Partial safety factors

7.5.1 *General*

The partial safety factors for materials (γ_{mm}, etc) make allowance for the variation in the quality of the materials and for the possible difference between the strength of masonry constructed under site conditions and that of specimens built in the laboratory for the purpose of establishing its physical properties. The values used in this code assume that the special category of construction control (see **11.3.1**) will be specified by the designer. If this is considered to be impracticable, higher partial safety factors should be used.

The values of partial safety factor for loads, γ_f, used in this code are based on those adopted in BS 5628-1.

The factor γ_f is introduced to take account of:

a) possible unusual increases in load beyond those considered in deriving the characteristic load;

b) inaccurate assessment of effects of loading and unforeseen stress redistribution within the structure;

c) the variations in dimensional accuracy achieved in construction.

7.5.2 *Ultimate limit state*

7.5.2.1 *Loads*

When using the design relationships for the ultimate limit state given in Clause **8** and Clause **9**, the design load should be taken as the sum of the products of the component characteristic loads, or for earth loads, the nominal load, multiplied by the appropriate partial safety factor, as shown below. Where alternative values are shown, the case producing the more severe conditions should be selected, except for earth and water loads as described in **7.2.2**.

a) *Dead and imposed load*

Design dead load	$= 0.9\ G_k$ or $1.4\ G_k$
Design imposed load	$= 1.6\ Q_k$
Design earth and water load	$= 1.2\ E_u$

b) *Dead and wind load*

Design dead load	$= 0.9\ G_k$ or $1.4\ G_k$
Design wind load	$= 1.4\ W_k$
Design earth and water load	$= 1.2\ E_u$

In the particular case of freestanding walls and laterally loaded wall panels, the removal of which would in no way affect the stability of the remaining structure, γ_f applied on the wind load may be taken as 1.2.

c) *Dead, imposed and wind load*

Design dead load = 1.2 G_k

Design imposed load = 1.2 Q_k

Design wind load = 1.2 W_k

Design earth and water load = 1.2 E_u

d) *Accidental forces* (see **7.2.3**). For this load case, reference should be made to BS 5628-1:2005, Clause **33**).

For all these cases:

G_k is the characteristic dead load;

Q_k is the characteristic imposed load;

W_k is the characteristic wind load;

E_u is the worst credible earth or water load (see **3.3**);

and the numeral values are the appropriate γ_f factors.

Where other than worst credible earth and water loads are used, such as nominal loads determined in accordance with Civil Engineering Code of Practice No. 2 1951, the appropriate partial safety, γ_f, for design earth and water load determination is 1.4.

In design, each of the load combinations a) to d) should be considered and that giving the most severe conditions should be adopted.

When considering the overall stability of a structure other than a retaining wall, the design horizontal load should be taken to be the design wind load, for the case being considered, or 0.015 G_k, for conformity to **7.2.1**a), whichever is the greater.

In certain circumstances other values of γ_f may be appropriate, e.g. in farm buildings. Reference should be made to the relevant British Standards, e.g. BS 5502-22.

Where a detailed investigation of soil conditions has been made and account has been taken of possible soil-structure interaction in the assessment of earth loads, it may be appropriate to derive design values for earth and water loads by different procedures. In such cases, additional consideration should be given to conditions in the structure under serviceability loads.

7.5.2.2 *Materials*

The design strength of a material or ancillary component is the characteristic strength divided by the appropriate partial safety factor, i.e. γ_{mm} for the compressive strength of masonry (see Table 7); γ_{mv} for the shear strength of masonry (see Table 8); γ_{mb} for the bond strength between infill concrete or mortar and steel (see Table 8) and γ_{ms} for the strength of steel including bed joint reinforcement (see Table 8); and γ_{mt} for the strength of wall ties (see Table 8).

The appropriate partial safety factor given in Table 7 should be applied, it having been established by manufacturer's declaration whether the units are Category I or Category II (with reference to BS EN 771-1 to -6) in respect of the manufacturing control of the units.

The values given in Table 7 and Table 8 assume that all the recommendations in Clause **11** for the special quality of construction control will be followed. If any of the recommendations of Clause **7** cannot be followed, e.g. in masonry incorporating bed joint reinforcement (see Annex A), higher partial safety factors for material strength should be used.

Table 7 — Partial safety factors, γ_{mm}, for strength of reinforced masonry in direct compression and bending: ultimate limit state

Category of manufacturing control of masonry units	Value of γ_{mm}
Category I	2.0
Category II	2.3

Table 8 — Partial safety factors γ_{mv}, γ_{mb} and γ_{ms}: ultimate limit state

Partial safety factor	Value
Shear strength of masonry, γ_{mv}	2.0
Bond strength between concrete infill or mortar and steel, γ_{mb}	1.5
Strength of steel, γ_{ms}	1.15
Wall ties (including shear ties) conforming to BS EN 845-1, γ_{mt}	2.5

When considering the effects of accidental loads or localized damage, the values of γ_{mm} , γ_{mv} and γ_{mt} may be halved. The values of γ_{mb} and γ_{ms} should then be taken as 1.0.

7.5.3 *Serviceability limit state*

7.5.3.1 *Loads*

The design loads for a serviceability limit state should be taken as follows:

a) *Dead and imposed load*

Design dead load = 1.0 G_k

Design imposed load = 1.0 Q_k

Design earth and water load = 1.0 E_u

b) *Dead and wind load*

Design dead load = 1.0 G_k

Design wind load = 1.0 W_k

Design earth and water load = 1.0 E_u

c) *Dead, imposed and wind load*

Design dead load = 1.0 G_k

Design imposed load = 0.8 Q_k

Design wind load = 0.8 W_k

Design earth and water load = 1.0 E_u

where

G_k is the characteristic dead load;

Q_k is the characteristic imposed load;

W_k is the characteristic wind load;

E_u is the worst credible earth or water load (see **3.3**).

In assessing short-term deflections, each of the load combinations a) to c) should be considered and that giving the most severe conditions should be adopted.

It may also be necessary to examine additional time-dependent deflections due to creep, moisture movements and temperature, and their effect on the structure as a whole, with particular reference to cracking and other forms of local damage (see **8.3.5**).

7.5.3.2 *Materials*

The value of γ_{mm} for masonry should be taken as 1.5 and that of γ_{ms} for steel as 1.0, for deflection calculations and for assessing the stresses or crack widths at any section within a structure.

7.5.4 ***Moments and forces in continuous members***

In the analysis of continuous members it will be sufficient to consider the following arrangements of load:

a) alternate spans loaded with the design load (1.4 G_k + 1.6 Q_k) and all other spans loaded with the minimum design dead load (0.9 G_k);

b) all spans loaded with the design load (1.4 G_k + 1.6 Q_k)

where

G_k is the characteristic dead load;

Q_k is the characteristic imposed load.

8 Design of reinforced masonry

8.1 General

This clause covers the design of reinforced masonry. It assumes that for reinforced masonry structures the ultimate limit state will be critical. The design, therefore, is carried out using the partial safety factors appropriate to the ultimate limit state. If the recommendations given here are followed, the serviceability limit states of deflection and cracking will not normally need to be checked. As an alternative, the designer may calculate deflections and crack widths, using partial safety factors appropriate to the serviceability limit state (see Annex C).

8.2 Reinforced masonry subjected to bending

8.2.1 ***General***

Clause **8.2** covers the design of elements subjected only to bending. These elements include beams, slabs, retaining walls, buttresses and piers. Panel and free-standing (cantilever) walls reinforced, either vertically or horizontally, primarily to resist wind forces or other horizontal loads, may also be designed in accordance with this clause.

Where the form of a reinforced masonry element and its support conditions permit, it may be designed as a two-way spanning slab using conventional yield line analysis or other appropriate theory.

8.2.2 ***Effective span of elements***

The effective span of simply supported or continuous members should normally be taken as the smaller of:

a) the distance between centres of supports;

b) the clear distance between supports plus the effective depth.

The effective span of a cantilever should be taken as the smaller of:

1) the distance between the end of the cantilever and the centre of its support;

2) the distance between the end of the cantilever and the face of the support plus half its effective depth.

8.2.3 ***Limiting dimensions***

8.2.3.1 *General*

To avoid detailed calculations to check that the limit states of deflection and cracking are not reached, the limiting ratios given in Table 9 and Table 10 may be used, except when the serviceability requirements are more stringent than the recommendations in **7.1.2.2**.

8.2.3.2 *Walls subjected to lateral loading*

When walls are reinforced to resist lateral loading, the ratio of span to effective depth of the wall may be taken from Table 9.

For free-standing walls not forming part of a building and subjected predominantly to wind loads, the ratios given in Table 10 may be increased by 30 %, provided such walls have no applied finish that can be damaged by deflection or cracking.

Table 9 — Limiting ratios of span to effective depth for laterally-loaded walls

End condition	Ratio
Simply supported	35
Continuous or spanning in two directions	45
Cantilever with percentage of reinforcement up to and including 0.005	18

8.2.3.3 *Beams*

The limiting ratios of span to effective depth for beams with various end conditions may be taken from Table 10.

Table 10 — Limiting ratios of span to effective depth for beams

End condition	Ratio
Simply supported	20
Continuous	26
Cantilever	7

To ensure lateral stability of a simply supported or continuous beam, it should be proportioned so that the clear distance between lateral restraints does not exceed:

$60\ b_c$ or $250\ b_c^2/d$, whichever is the lesser

where

d is the effective depth;

b_c is the width of the compression face midway between restraints.

For a cantilever with lateral restraint provided only at the support, the clear distance from the end of the cantilever to the face of the support should not exceed:

$25\ b_c$ or $100\ b_c^2/d$, whichever is the lesser.

8.2.4 ***Resistance moments of elements***

8.2.4.1 *Analysis of sections*

When analysing a cross-section to determine its design moment of resistance, the following assumptions should be made:

a) plane sections remain plane when considering the strain distribution in the masonry in compression and the strains in the reinforcement, whether in tension or compression;

b) the compressive stress distribution in the masonry is represented by an equivalent rectangle with an intensity taken over the whole compression zone of f_k/γ_{mm} where f_k is obtained from **7.4.1.2** and γ_{mm} is given the value appropriate to the limit state being considered (see **7.5**);

c) the maximum strain in the outermost compression fibre at failure is 0.0035;

d) the tensile strength of the masonry is ignored;

e) the characteristic strength of the reinforcing steel is taken from Table 4, and the stress-strain relationship is taken from Figure 1;

f) the span to effective depth ratio of the member is not less than 1.5.

In the analysis of a cross-section that has to resist a small axial thrust, the effect of the design axial force may be ignored if it does not exceed $0.1\ f_k\ A_m$, where A_m is the cross-sectional area of the masonry, i.e. the member may be designed for bending only.

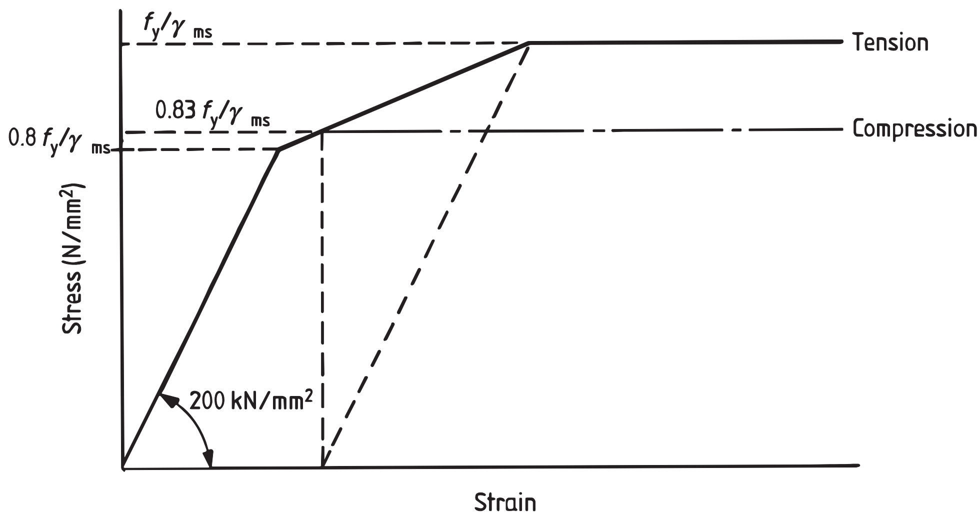


Figure 1 — Short-term design stress-strain curve for reinforcement

8.2.4.2 *Design formulae for singly reinforced rectangular members*

8.2.4.2.1 Based on the assumptions discussed in **8.2.4.1**, the design moment of resistance, M_d, of a single reinforced rectangular member may be obtained from the equation:

$$M_d = \frac{A_s f_y z}{\gamma_{ms}}$$

provided that M_d is not taken to be greater than:

$$0.4\frac{f_k b d^2}{\gamma_{mm}}$$

where

z is the length of the lever arm given by:

$$z = d\left(1 - \frac{0.5 A_s f_y \gamma_{mm}}{b d f_k \gamma_{ms}}\right)$$

provided that

z is not taken to be greater than $0.95d$;
A_s is the cross-sectional area of primary reinforcing steel:
b is the width of the section;
d is the effective depth;
f_k is the characteristic compressive strength of masonry;
f_y is the characteristic tensile strength of reinforcing steel given in Table 3;
γ_{mm} is the partial safety factor for strength of masonry given in **7.5**;
γ_{ms} is the partial safety factor for strength of steel given in **7.5**.

8.2.4.2.2 The expression for the lever arm given in **8.2.4.2.1** cannot be used directly to calculate the area of reinforcement, A_s. It is more convenient to express the design moment of resistance, M_d, in terms of a moment of resistance factor, Q, such that:

$M_d = Q_{bd}^2$

where

b is the width section;

d is the effective depth;

Q is the moment of resistance factor given by:

$Q = 2c\,(1–c)\,f_k/\gamma_{mm}$

where

f_k is the characteristic strength of masonry;

γ_{mm} is the partial safety factor for strength of masonry given in **7.5**;

c is the lever arm factor = z/d.

The relationship between Q, c and f_k/γ_{mm} is shown in Table 11 and Figure 2.

Where the ratio of the span to the depth of a beam is less than 1.5, it should be treated as a wall beam. Tension reinforcement should be provided to take the whole of the tensile force, calculated on the basis of a moment arm equal to two-thirds of the depth, with a maximum value equal to 0.7 times the span.

8.2.4.3 *Design formulae for walls with the reinforcement concentrated locally*

8.2.4.3.1 *Flanged members*

Where the reinforcement in a section is concentrated locally such that the section can act as a flanged beam, the thickness of the flange, t_f, should be taken as the thickness of the masonry but in no case greater than 0.5d, where d is the effective depth.

The width of the flange should be taken as the least of:

a) for pocket-type walls, the width of the pocket or rib plus 12 times the thickness of the flanges;

b) the spacing of the pockets or ribs;

c) one-third the height of the wall.

The design moment of resistance, M_d, may be obtained from the equation given in **8.2.4.2.1**, provided that it is not taken to be greater than the value given by the following equation:

$$M_d = \frac{f_k}{\gamma_{mm}} b t_f (d - 0.5 t_f)$$

where

b is the width of the section;

d is the effective depth;

f_k is the characteristic compressive strength of masonry given in **7.4.1.2**;

t_f is the thickness of the flange;

γ_{mm} is the partial safety factor for strength of masonry given in **7.5**.

Where the spacing of the pocket or ribs exceeds 1 m, the ability of the masonry to span horizontally between the ribs should be checked.

8.2.4.3.2 *Locally reinforced hollow blockwork*

When the reinforcement in a section is concentrated locally such that the section cannot act as a flanged member, the reinforced section should be considered as having a width of three times the thickness of the blockwork.

8.2.5 *Shear resistance of elements*

8.2.5.1 *Shear stresses and reinforcement in members in bending*

The shear stress, v, due to design loads at any cross-section in a member in bending should be calculated from the equation:

$$v = \frac{V}{bd}$$

where

- b is the width of the section;
- d is the effective depth (or for a flanged member, the actual thickness of the masonry between the ribs, if this is less than the effective depth as defined in **3.7**);
- V is the shear force due to design loads.

Where the shear stress calculated from this equation is less than the characteristic shear strength of masonry, f_v, divided by the partial safety factor, γ_{mv}, shear reinforcement is not generally needed. In beams, however, the designer should consider the use of nominal links, bearing in mind the sudden nature of shear failure. If required, they should be provided in accordance with **8.6.5.2**.

Where the shear stress, v, exceeds f_v/γ_{mv}, shear reinforcement should be provided. The following recommendation should be observed:

$$\frac{A_{sv}}{s_v} \text{«} \frac{b(v - f_v/\gamma_{mv})\gamma_{ms}}{f_y}$$

where

- A_{sv} is the cross-sectional area of reinforcing steel resisting shear forces;
- b is the width of the section;
- f_v is the characteristic tensile strength of masonry obtained from **7.4.1.3**;
- f_y is the characteristic tensile strength of the reinforcing steel resisting shear forces obtained from Table 4;
- s_v is the spacing of shear reinforcement along the member, provided that it is not taken to be greater than $0.75d$ (see **8.6.4**);
- v is the shear stress due to design loads, provided that it is not taken to be greater than $2.0/\gamma_{mv}$ N/m^2;
- γ_{ms} is the partial safety factor for strength of steel given in **7.5.2.2**;
- γ_{mv} is the partial safety factor for shear strength of masonry given in **7.5.2.2**;

NOTE This part of BS 5628 does not give guidance on the use of bent up bars as shear reinforcement in masonry, and the principles given in BS 8110 may be used.

Table 11 — Values of the moment of resistance factor, *Q*, for various values of f_k/γ_{mm} and lever arm factor, *c*

c	f_k/γ_{mm}													
	1	2	3	4	5	6	7	8	9	10	11	12	13	15	20
0.95	0.095	0.190	0.285	0.380	0.475	0.570	0.665	0.760	0.855	0.950	1.045	1.140	1.235	1.425	1.900
0.94	0.113	0.226	0.338	0.451	0.564	0.677	0.790	0.902	1.015	1.128	1.241	1.354	1.466	1.692	2.256
0.93	0.130	0.260	0.391	0.521	0.651	0.781	0.911	1.042	1.172	1.302	1.432	1.562	1.693	1.953	2.604
0.92	0.147	0.294	0.442	0.589	0.736	0.883	1.030	1.178	1.325	1.472	1.619	1.766	1.914	2.208	2.944
0.91	0.164	0.328	0.491	0.655	0.819	0.983	1.147	1.310	1.474	1.638	1.802	1.966	2.129	2.457	3.276
0.90	0.180	0.360	0.540	0.720	0.900	1.080	1.260	1.440	1.620	1.800	1.980	2.160	2.340	2.700	3.600
0.89	0.196	0.392	0.587	0.783	0.979	1.175	1.371	1.566	1.762	1.958	2.154	2.350	2.545	2.937	3.916
0.88	0.211	0.422	0.634	0.845	1.056	1.267	1.478	1.690	1.901	2.112	2.323	2.534	2.746	3.168	4.224
0.87	0.226	0.452	0.679	0.905	1.131	1.357	1.583	1.810	2.036	2.262	2.488	2.714	2.941	3.393	4.524
0.86	0.241	0.482	0.722	0.963	1.204	1.445	1.686	1.926	2.167	2.408	2.649	2.890	3.130	3.612	4.816
0.85	0.255	0.510	0.765	1.020	1.275	1.530	1.785	2.040	2.295	2.550	2.805	3.060	3.315	3.825	5.100
0.84	0.269	0.538	0.806	1.075	1.344	1.613	1.882	2.150	2.419	2.688	2.957	3.226	3.494	4.032	5.376
0.83	0.282	0.564	0.847	1.129	1.411	1.693	1.975	2.258	2.540	2.822	3.104	3.386	3669	4.233	5.644
0.82	0.295	0.590	0.886	1.181	1.476	1.771	2.066	2.362	2.657	2.952	3.247	3.542	3.838	4.428	5.904
0.81	0.308	0.616	0.923	1.231	1.539	1.847	2.155	2.462	2.770	3.078	3.386	3.694	4.001	4.617	6.156
0.80	0.320	0.640	0.960	1.280	1.600	1.920	2.240	2.560	2.880	3.200	3.520	3.840	4.160	4.800	6.400
0.79	0.332	0.664	0.995	1.327	1.659	1.991	2.323	2.654	2.986	3.318	3.650	3.982	4.313	4.977	6.636
0.78	0.343	0.686	1.030	1.373	1.716	2.059	2.402	2.746	3.089	3.432	3.775	4.118	4.462	5.148	6.684
0.77	0.354	0.708	1.063	1.417	1.771	2.125	2.479	2.834	3.188	3.542	3.896	4.250	4.605	5.313	7.084
0.76	0.365	0.730	1.094	1.459	1.824	2.189	2.554	2.918	3.283	3.648	4.013	4.378	4.742	5.472	7.296
0.75	0.375	0.750	1.125	1.500	1.875	2.250	2.625	3.000	3.375	3.750	4.125	4.500	4.875	5.625	7.500
0.74	0.385	0.770	1.154	1.539	1.924	2.309	2.694	3.078	3.463	3.848	4.233	4.618	5.002	5.772	7.696
0.73	0.394	0.788	1.183	1.577	1.971	2.365	2.759	3.154	3.548	3.942	4.336	4.730	5.125	5.193	7.884
0.72	0.403	0.806	1.210	1.613	2.016	2.419	2.822	3.226	3.629	4.032	4.435	4.838	5.242	6.048	8.064

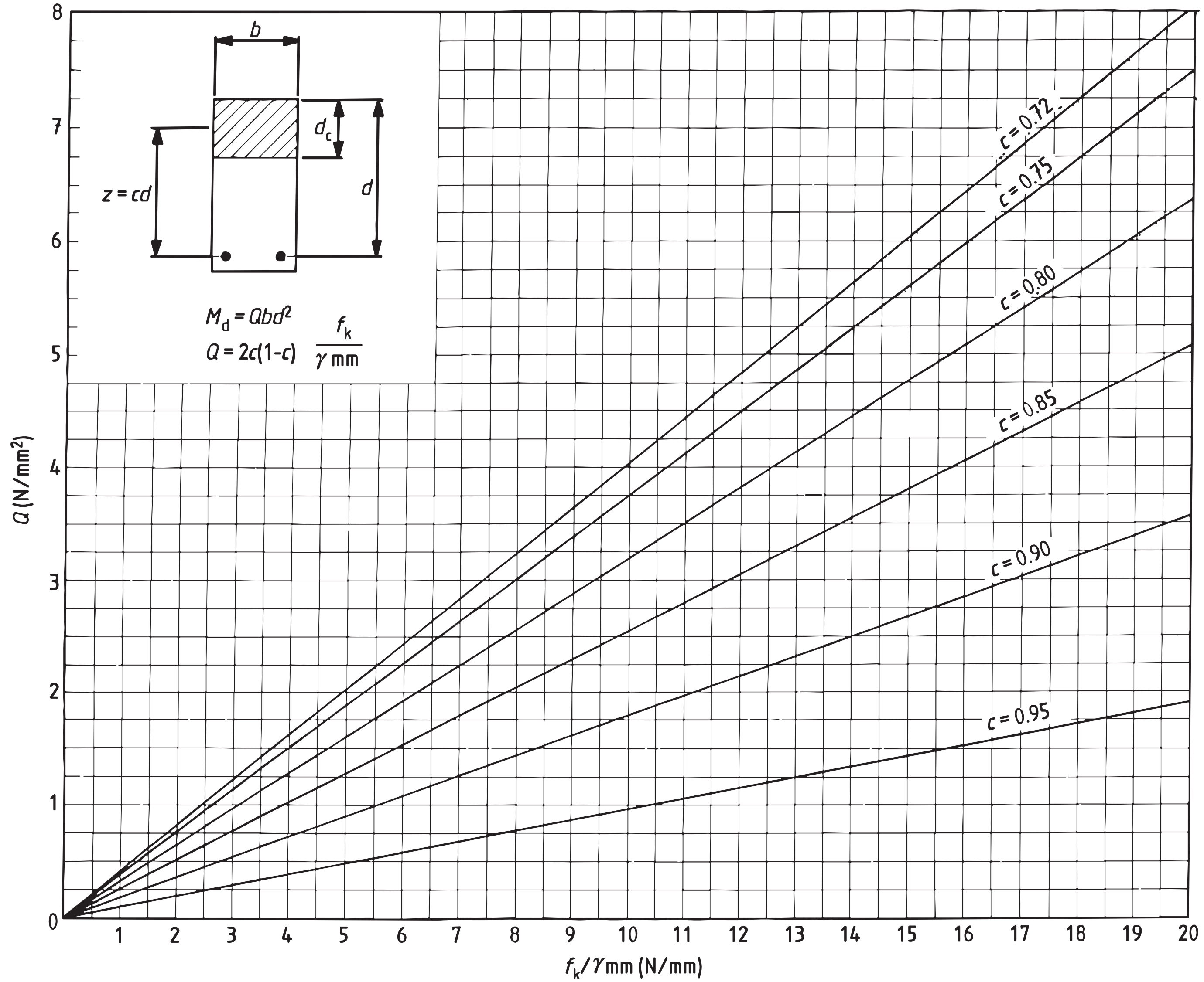


Figure 2 — Moment of resistance factor, Q

8.2.5.2 *Shear stress in retaining walls*

In vertical retaining walls, the design shear stress may be reduced by the horizontal component of force in any tension bars inclined to the vertical so as to increase the resistance to the applied shear force. The reduction available is:

$$\frac{M}{BD^2}\sin\phi$$

where

ϕ is the angle of inclination to the vertical.

If this reduction is used, f_v, which is taken from **7.4.1.3.1**b), should be based on the reduced steel area $A_s - A_{si}$, where A_{si} is the sectional area of the inclined bars.

Where the main reinforcement is not lapped at the same effective depth, as, for example, in the case of stepped pocket type retaining walls, sufficient shear reinforcement (e.g. links) should be provided to transfer the shear force.

8.2.5.3 *Concentrated loads near supports*

Where the distance from the face of a support to the nearest edge of the principal load, a_v, is less than twice the effective depth, d, the main reinforcement should be provided with an anchorage in accordance with **8.6.9**. Any concentrated load (or loads) should be treated as a principal load when it contributes more than 70 % of the total shear force at a support.

8.2.6 ***Deflection***

Deflection of members may be calculated (see Annex C) and compared with the recommendations for serviceability given in **7.1.2.2.1**, but in all normal cases the deflection will not be excessive if the member has a span/depth ratio within the limits given in **8.2.3**.

8.2.7 ***Cracking***

In most cases the recommendations for detailing reinforcement given in **8.6** will ensure that cracking in members is not excessive.

8.3 Reinforced masonry subjected to a combination of vertical loading and bending

8.3.1 ***General***

Clause **8.3** gives recommendations for the design of members subjected simultaneously to substantial vertical and horizontal loading or to eccentric vertical loads where the resultant eccentricity exceeds 0.05 times the thickness of the member in the direction of the eccentricity.

8.3.2 ***Slenderness ratios of walls and columns***

8.3.2.1 *Limiting slenderness ratios*

The slenderness ratio of walls and columns should not exceed 27, except in the case of cantilever walls and columns, when it should not exceed 18. Special consideration should be given to deflection where the percentage of reinforcement in cantilever walls or columns exceeds 0.5 % of the cross-sectional area obtained by multiplying the effective depth by the breadth of the section.

8.3.2.2 *Lateral support*

A lateral support should be capable of transmitting to the elements of construction that provide lateral stability to the structure as a whole, the sum of the following design lateral forces:

a) the simple static reactions to the total applied design horizontal forces at the line of lateral support; and

b) 2.5 % of the total design vertical load that the wall or column is designed to carry at the line of lateral support. The elements of construction that provide lateral stability to the structure as a whole need not be designed to support this force.

However, the designer should satisfy himself that loads applied to lateral supports will be transmitted to the elements of construction providing stability, e.g. by the floors or roofs acting as horizontal girders.

Simple resistance to lateral movement may be assumed for a lateral support if the forces defined in a) and b) can be transmitted.

Enhanced resistance to lateral movement for walls may be assumed where:

— floors or roofs of any form of construction span on to the wall from both sides at the same level; or,

— an in situ concrete or reinforced masonry floor or roof, or a precast concrete floor or roof giving equivalent restraint, irrespective of its direction of span, has a bearing of at least one-half the thickness of the wall on to which it spans but in no case less than 90 mm.

Further information on lateral supports is given in Section 4 of BS 5628-1:2005.

8.3.2.3 *Effective height*

The effective height, h_{ef}, of a wall, panel or column should preferably be assessed by structural analysis. Alternatively, the values given in Table 12 may be adopted, where h is the clear distance between lateral supports.

Table 12 — Effective height of walls and columns

End condition	Effective height, h_{ef}
Wall with lateral supports at top and bottom which provide enhanced resistance to lateral movement (see BS 5628-1:2005, **24.2.4.1**)	0.75 h
Wall with lateral supports at top and bottom which provide simple resistance to lateral movement (see BS 5628-1:2005, **24.2.4.2**)	H
Column with lateral supports restricting movement in both directions at top and bottom	h in respect of both directions
Column with lateral supports restricting movement in one direction only at top and bottom	h in respect of restrained direction
	$2h$ in respect of unrestrained direction

8.3.2.4 *Effective thickness*

For single-leaf walls and columns, the effective thickness, t_{ef}, should be taken as the actual thickness.

For cavity walls and for columns with only one leaf reinforced, the effective thickness should be taken as two-thirds the sum of the actual thicknesses of the two leaves or the actual thickness of the thicker leaf, whichever is the greater.

The effective thickness of a grouted-cavity wall should be taken as the overall thickness of the wall, provided that the cavity does not exceed 100 mm. If the cavity width exceeds 100 mm, the effective thickness should be calculated as the total thickness of the two leaves plus 100 mm.

8.3.3 ***Design***

8.3.3.1 *Columns subjected to a combination of vertical loading and bending*

8.3.3.1.1 *Short columns*

Where the slenderness ratio of a column does not exceed 12, only single axis bending generally requires consideration. Even where it is possible for significant moments to occur simultaneously about both axes, it is usually sufficient to design for the maximum moment about the critical axis only. However, where biaxial bending has to be considered reference should be made to **8.3.3.1.2**.

Either the cross-section of the column may be analysed to determine the design moment of resistance and the design vertical load resistance, using assumptions a), c), d) and e) given in **8.2.4.1**, or the following design method may be used.

a) Where the design vertical load, N, does not exceed the value of the design vertical load resistance, N_d, given in the following equation, only the minimum reinforcement given in **8.6.1** or **8.6.3** is required:

$$N_d = \frac{f_k}{\gamma_{mm}} b(t - 2e_x)$$

where

- b is the width of the section;
- e_x is the resultant eccentricity;
- f_k is the characteristic compressive strength of the masonry;
- t is the overall thickness of the section in the plane of bending;
- γ_{mm} is the partial safety factor for strength of masonry.

NOTE This formula does not cover cases where the resultant eccentricity:

$$N_d = \frac{f_k}{\gamma_{mm}} b(t - 2e_x)$$

exceeds 0.5 t, where M is the bending moment due to design load.

b) Where the design vertical load, N, is greater than that given by the equation in a) the strength of the section may be assessed by using the following equations and the relation $f_{s1} = 0.83f_y$.

$$N_d = \frac{f_k}{\gamma_{mm}} bd_c + \frac{f_{s1}\ A_{s1}}{\gamma_{ms}} - \frac{f_{s2}\ A_{s2}}{\gamma_{ms}}$$

$$M_d = \frac{0.5f_k}{\gamma_{mm}} bd_c(t - d_c) + \frac{0.83f_y}{\gamma_{ms}} A_{s1}(0.5t - d_1) + \frac{f_{s2}}{\gamma_{ms}} A_{s2}(0.5t - d_2)$$

where

- A_{s1} is the area of compression reinforcement in the more highly compressed face;
- A_{s2} is the area of the reinforcement nearer the least compressed face; this may be considered as being in compression, inactive or in tension, depending on the resultant eccentricity of the load;
- b is the width of the section;
- d_1 is the depth from the surface to the reinforcement in the more highly compressed face;
- d_c is the depth of masonry in compression;
- d_2 is the depth to the reinforcement from the least compressed face;
- f_k is the characteristic compressive strength of the masonry;
- f_{s1} is the stress in the reinforcement in the most compressed face;
- f_{s2} is the stress in the reinforcement in the least compressed face, equal to –0.83 f_y in compression or $+f_y$ in tension;
- f_y is the characteristic tensile strength of the reinforcement nearer the least compressed face;
- M_d is the design moment of resistance;
- N_d is the design vertical load resistance;
- t is the overall thickness of the section in the plane of bending;
- γ_{mm} is the partial safety factor for strength of masonry given in **7.5**;.
- γ_{ms} is the partial safety factor for strength of steel given in **7.5**.

The designer should choose a value of d_c which ensures that both the design vertical load resistance, N_d, and the moment of resistance, M_d, obtained from these equations exceed the design vertical load, N, and the design bending moment, M. The choice of d_c establishes the assumed strain distribution in the section. Appropriate values for the stresses in the reinforcement may be determined from the stress-strain relationship given in Figure 2 or as follows:

1) where d_c is chosen as t, then f_{s2} varies linearly between 0 and –0.83 f_y;

2) where d_c is chosen between $(t\text{-}d_2)$ and t, then $f_{s2}= 0$;

3) where d_c is chosen between $(t\text{-}d_2)$ and $t/2$, and f_{s2} varies linearly between 0 and f_y;

4) where d_c is chosen between $t/2$ and $2d_1$, f_{s2} may be taken as + f_y;

5) d_c should not be chosen as less than $2d_1$.

c) As an alternative to b) when the resultant eccentricity is greater than $(t/2 - d_1)$, the vertical load may be ignored and the section designed to resist an increased moment, M_a, given by:

$$M_a = M + N\,(t/2 - d_1)$$

The area of tension reinforcement necessary to provide resistance to this increased moment may be reduced by:

$$N\gamma_{ms} / f_y$$

8.3.3.1.2 *Short columns: biaxial bending*

Where it is necessary to consider biaxial bending in a short column, a symmetrically reinforced section may be designed to withstand an increased moment about one axis given by the following equations:

$$M'_x = M_x + j\left(\frac{p}{q}\right)M_y \text{ for } \frac{M_x}{p} \geq \frac{M_y}{q}$$

or

$$M'_y = M_y + j\left(\frac{q}{p}\right)M_x \text{ for } \frac{M_x}{p} < \frac{M_y}{q}$$

where

M_x is the design moment about the x axis;
M_y is the design moment about the y axis;
M'_x is the effective uniaxial design moment about the x axis;
M'_y is the effective uniaxial design moment about the y axis;
p is the overall section dimension in a direction perpendicular to the x axis;
q is the overall section dimension in a direction perpendicular to the y axis;
j is a coefficient derived from Table 13.

Table 13 — Values of the coefficient *j*

Value of $N/A_m f_k$	Value of j
0	1.00
0.1	0.88
0.2	0.77
0.3	0.65
0.4	0.53
0.5	0.42
$\geq$ 0.6	0.30
NOTE N is the design vertical load; A_m is the cross-sectional area; f_k is the characteristic compressive strength of masonry.	

8.3.3.1.3 *Slender columns*

In a slender column with a slenderness ratio greater than 12 it is essential to take account of biaxial bending where appropriate, and also of the additional moment induced by the vertical load, due to lateral deflection, M_a, which may be obtained from the equation:

$$M_a = \frac{N\left(h_{ef}\right)^2}{2000t}$$

where

t is the width of the column in the plane of bending;

h_{ef} is the effective height of the column;

N is the design vertical load.

The cross-section may be analysed using the assumptions given in **8.2.4.1** to determine its design moment of resistance and design vertical load resistance. As an alternative, slender columns subjected to bending about one axis only may be designed using the equations given in **8.3.3.1.1** but including the additional bending moment, M_a, determined by the equation given in this subclause in the design bending moment.

8.3.3.2 *Walls subjected to a combination of vertical loading and bending*

8.3.3.2.1 *Short walls*

When the slenderness ratio of a wall does not exceed 12, the wall may be analysed to determine the design moment of resistance and design vertical load resistance, using the assumptions given in **8.2.4.1**.

If the resultant eccentricity, e_x, is greater than 0.5 t, the member may be designed as a member in bending in accordance with **8.2**, discounting the vertical load.

8.3.3.2.2 *Slender walls*

When the slenderness ratio of a wall exceeds 12, the wall should be designed in accordance with **8.3.3.2.1**, including in the design bending moment the additional bending moment, M_a, determined in accordance with **8.3.3.1.3**.

8.3.4 ***Deflection***

Within the limiting dimensions given in **8.2**, it may be assumed that the lateral deflection of a wall is acceptable.

8.3.5 *Cracking*

Unacceptable cracking due to bending is unlikely to occur in a wall or column where the design vertical load exceeds:

$A_m f_k/2$

where

A_m is the cross-sectional area of masonry;

f_k is the characteristic compressive strength of masonry.

A more lightly loaded column should be treated as a beam for the purposes of crack control and reinforced in accordance with the recommendations of **8.6**.

8.4 Reinforced masonry subjected to axial compressive loading

Reinforced masonry walls or columns subjected to axial loading or vertical loading having a resultant eccentricity not exceeding 0.05 times the thickness of the member in the direction of the eccentricity, may be designed as either described in BS 5628-1:2005, Clause **25**, i.e. taking no account of the reinforcement, or using the methods given in **8.3** of this code. In the latter case the following should be used:

a) design axial load resistance, N_d, determined in accordance with **8.3.3.1.1**b), in conjunction with;

b) the design moment of resistance, M_d, determined in accordance with **8.3.3.1.1**b) and;

c) the increase in moment due to slenderness, M_a, determined in accordance with **8.3.3.1.3**, where the slenderness ratio of the element exceeds 12.

Walls subjected to concentrated loads should be designed following the recommendations of BS 5628-1:2005, Clause **27**.

8.5 Reinforced masonry subjected to horizontal forces in the plane of the element

8.5.1 *Racking shear*

8.5.1.1 Where a vertically reinforced wall resists horizontal forces acting in its plane, adequate provision against the ultimate limit state in shear being reached may be assumed if the following relationship is satisfied:

$$v < \frac{f_v}{\gamma_{mv}}$$

where

f_v is the characteristic shear strength of masonry (see **7.4.1.3.2**);

γ_{mv} is the partial safety factor for shear strength of masonry given in **7.5.2.2**;

v is the shear stress due to design loads given by:

$$v = \frac{V}{tL}$$

where

t is the thickness of the wall;

L is the length of the wall;

V is the horizontal shear force due to design loads.

8.5.1.2 Where the relationship given in **8.5.1.1** is not satisfied, horizontal shear reinforcement should be provided but in no case should v exceed 2.0 $/\gamma_{mv}$ N/mm^2.

Where horizontal reinforcement is provided, the following requirement should be satisfied:

$$A_{sv} \geq \frac{Lt(v - f_v/\gamma_{mv})}{f_y/\gamma_{ms}}$$

where

A_{sv} is the cross-sectional area of reinforcing steel resisting shear forces;

L is the length of the wall;

t is the thickness of the wall;

f_v is the characteristic shear strength of masonry obtained from **7.4.1.3.2**;

f_y is the characteristic tensile strength of the reinforcing steel resisting shear forces obtained from Table 4;

γ_{mv} is the partial safety factor for shear strength of masonry given in **7.5.2.2**;

γ_{ms} is the partial safety factor for strength of steel given in **7.5**.

8.5.2 *Bending*

When the bending is in the plane of the wall, the analysis and design of the wall should follow the recommendations for beams given in **8.2**. Where the slenderness ratio exceeds 12 in any direction, it is essential also to take account of the slenderness at right angles to the plane of the wall by calculating the maximum compressive stress in the wall and checking that the recommendations for slender columns described in **8.3.3.1.3** are satisfied.

8.6 Detailing reinforced masonry

8.6.1 *Area of main reinforcement*

Designers should consider whether the area of main reinforcement is such that the recommendations for unreinforced masonry given in BS 5628-1 would be more appropriate than the recommendations given in this part of BS 5628.

8.6.2 *Maximum size of reinforcement*

The size of reinforcing bars used in reinforced masonry should not exceed 6 mm when placed in joints or 25 mm elsewhere, except in the case of pocket-type walls, where bar sizes up to 32 mm may be used.

8.6.3 *Minimum area of secondary reinforcement in walls and slabs*

In all walls and slabs designed to span in one direction only, the area of secondary reinforcement provided should not be less than 0.05%, based on the effective depth times the breadth of the section.

Secondary reinforcement may be omitted from pocket-type walls except where specifically required to tie the masonry to the infill concrete.

Some or all of the secondary reinforcement may be used to help control cracking due to shrinkage or expansion, thermal and moisture movements.

8.6.4 *Spacing of main and secondary reinforcement*

The minimum clear horizontal or vertical distance between individual parallel bars should be equal to the maximum size of aggregate plus 5 mm or the bar diameter, whichever is greater, but in no case less than 10 mm.

The maximum spacing of main secondary tension reinforcement should not exceed 500 mm.

Where the main reinforcement is concentrated in cores or pockets, e.g. in pocket-type walls, the maximum spacing centre-to-centre between the concentrations of main reinforcement may exceed these recommendations.

In vertical pockets or cores less than 125 mm × 125 mm, only one reinforcing bar should be used, except at laps.

Where shear reinforcement is provided, the spacing of the bars in the direction of the span should not exceed 0.75*d*, where *d* is the effective depth.

8.6.5 *Anchorage, minimum area, size and spacing of links*

8.6.5.1 *Anchorage of links*

A link may be considered to be fully anchored if it passes round another bar of at least its own diameter through an angle of 90° and continues beyond for a minimum length of eight times its own diameter, or through 180° and continues for a minimum length of four times its own diameter. In no case should the radius of any bend in a link be less than twice the radius of the test bend guaranteed by the manufacturer of the reinforcement.

8.6.5.2 *Beam links*

Where nominal shear reinforcement is required (see **8.2.5.1**) it should be provided throughout the span such that:

$$\frac{A_{sv}}{s_v} = 0.002\ b_t \text{ for mild steel; or}$$

$$\frac{A_{sv}}{s_v} = 0.0012\ b_t \text{ for high yield steel,}$$

where

A_{sv} is the cross-sectional area of reinforcing steel resisting shear forces;

b_t is the width of beam at the level of the tension reinforcement;

s_v is the spacing of shear reinforcement, which should not exceed 0.75 *d*, where *d* is the effective depth.

8.6.5.3 *Column links*

In columns where the area of steel, A_s, is greater than 0.25 % of the area of the masonry, A_m, links should be provided if more than 25% of the design axial load resistance is to be used. In columns where A_s is not greater than 0.25% A_m, links need not be provided.

Where links are required, they should be not less than 6 mm in diameter. The spacing of these links should not exceed the least of:

a) the smallest dimension of the column;

b) 50 × link diameter;

c) 20 × main bar diameter.

Where links are provided, they should surround the main vertical steel. Every vertical corner bar should be supported by an internal angle at every link spacing and this angle should not exceed 135°. Internal vertical bars need only be supported by the internal angles at alternate link spacings.

8.6.6 *Anchorage bond*

To prevent bond failure, the following recommendations should be observed:

a) the tension or compression in any bar due to design loads should be developed on each side of the section by the appropriate anchorage bond strength given in **7.4.1.6** divided by the partial safety factor for bond, γ_{mb}, from Table 7; and

b) the cover of concrete infill or mortar should not be less than the bar diameter.

For bed joint reinforcement conforming to BS EN 845-3, the anchorage length to maintain full effectiveness of the reinforcement may be taken as the declared value of lap length relevant to the type and strength of mortar and masonry units being used. Alternatively, the declared value of the anchorage bond strength may be used to calculate an anchorage length which is shorter than the lap length provided the number of welded intersections per length in the anchorage is not less than the number for a representative length of reinforcement.

NOTE The test method given in BS EN 846-2 for the anchorage bond of bed joint reinforcement determines the characteristic value of load capacity required to pull out a representative length of reinforcement. For a ladder type this is a length containing two welded intersections (i.e. the pitch of the welded cross wires) and for a truss type it is 60% of a length containing two welded intersections (i.e. 60% of the length of the diagonal wires).

8.6.7 *Laps and joints*

Connections transferring stress may be lapped or jointed with a mechanical device. They should where practicable occur away from points of high stress and should be staggered.

Where the stress in the bar at the joint is entirely compressive, the load may be transferred by end bearing of square sawn-cut ends held in concentric contact by a suitable sleeve or mechanical device, e.g. a threaded coupler.

When bars are lapped, the length of the lap should be at least equal to the anchorage length (see **8.6.6**) required to develop the stress in the smaller of the two bars lapped. The length of lap provided, however, should not be less than 25 times the bar size plus 150 mm in tension reinforcement nor less than 20 times the bar size plus 150 mm in compression reinforcement.

Where bed joint reinforcement is lapped, the length of lap should be at least equal to the declared value of lap length relevant to the type and strength of mortar and masonry units being used.

8.6.8 *Hooks and bends*

Hooks, bends and other reinforcement anchorages should be of such form, dimension and arrangement as to avoid overstressing the concrete or mortar. Hooks, which should be used only to meet specific design requirements, should be of U or L-type, as specified in BS 8666.

The effective anchorage length of a hook or bend should be measured from the start of the bend to a point four times the bar size beyond the end of the bend (see Figure 4), and may be taken as the greater of the actual length and the following:

a) for a hook, eight times the internal radius of the hook, but not greater than 24 times the bar size;

b) for a 90 degree bend, four times the internal radius of the bend, but not greater than 12 times the bar size.

In no case should the radius of any bend be less than twice the radius of the test bend guaranteed by the manufacturer of the bar.

When a hooked bar is used at a support, the beginning of the hook should be at least four times the bar size inside the face of the support (see Figure 3).

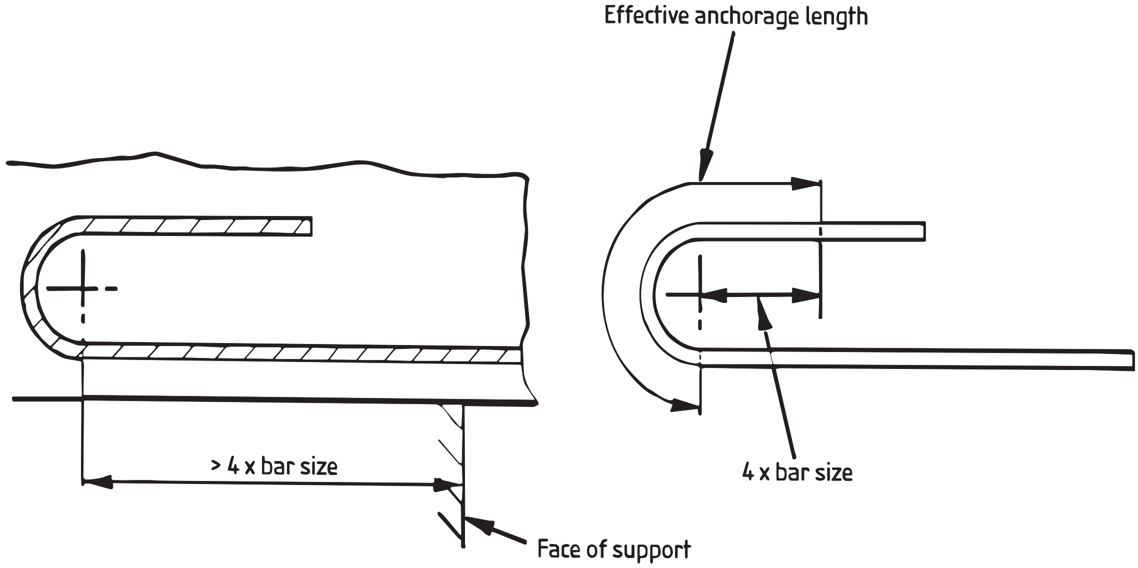


Figure 3 — Hooks and bends

8.6.9 *Curtailment and anchorage*

In any member subjected to bending, every bar should extend, except at end supports, beyond the point at which it is no longer needed, for a distance equal to the effective depth of the member or 12 times the size of the bar, whichever is the greater. The point at which reinforcement is no longer needed is where the resistance moment of the section, considering only the continuing bars, is equal to the necessary moment. In addition, reinforcement should not be stopped in a tension zone unless one of the following conditions is satisfied for all arrangements of design load considered.

a) The bars extend at least the anchorage length appropriate to their design strength, f_y/γ_{ms}, from the point at which they are no longer required to resist bending,

where

f_y is the characteristic tensile strength of reinforcing steel;

γ_{ms} is the partial safety factor for strength of steel.

b) The design shear capacity at the section where the reinforcement stops is greater than twice the shear force due to design loads, at that section.

c) The continuing bars at the section where the reinforcement stops provide double the area required to resist the moment at that section.

At a simply supported end of a member, each tension bar should be anchored by one of the following:

1) an effective anchorage equivalent to 12 times the bar size beyond the centre line of the support, where no bend or hook begins before the centre of the support;

2) an effective anchorage equivalent to 12 times the bar size plus $d/2$ from the face of the support, where d is the effective depth of the member, and no bend begins before $d/2$ inside the face of the support.

Where the distance, a_v, from the face of a support to the nearest edge of a principal load (see **8.2.5.3**) is less than twice the effective depth, d, all the main reinforcement should continue to the support and be provided with an anchorage equivalent to 20 times the bar diameter.

9 Design of prestressed masonry

9.1 General

There are two methods of prestressing masonry:

a) Post-tensioning

The tendons are tensioned against the masonry when it has achieved sufficient strength using mechanical anchorages. The tendons may be:

i) unrestrained against lateral movement in cavities or voids in the masonry;

ii) restrained against lateral movement at discrete points by projecting masonry units or continuously by ducts built into the masonry;

iii) bonded to the surrounding masonry by grout or concrete infill.

b) Pre-tensioning

The tendons are tensioned against an independent anchorage and released only when the masonry and/or infill concrete has achieved sufficient strength. The transfer of the prestress force to the masonry is provided by bond alone.

Pre-tensioning is usually only appropriate for prefabricated products, for which type testing will be necessary.

Post-tensioning is most frequently used in vertically spanning walls and in columns. The structural performance of such masonry can be enhanced by constructing the masonry member with a geometric cross-section, such as found in a diaphragm wall or a fin wall. The geometric cross-section is proportioned to have high section modulus (for enhanced performance at the ultimate limit state) and a high radius of gyration (for enhanced performance under axial compression), as appropriate. The designer needs to ensure that the cross-section has satisfactory shear strength.

It is recommended that tendons and anchorages for prestressed, post tensioned masonry are inspectable unless corrosion resistant materials are used. A written statement for inspection and remedial action should be given to the owner.

Where tendons and anchorages are made from materials that could degrade and are not inspectable and replaceable, they should be robustly and reliably protected from the agents that cause that degradation. Such protection should be resistant to disruption from subsequent operations.

Clause **9** covers the design of all prestressed masonry. As it is not possible to assume that a particular limit state will always be critical, design methods are given to allow the recommendations for both the ultimate and the serviceability limit states to be observed.

9.2 Design for the ultimate limit state

9.2.1 *Bending*

When analysing a section, the following assumptions should be made:

a) plane sections remain plane when considering strain distribution in the masonry;

b) the distribution of stress is uniform over the whole compression zone and does not exceed:

f_k/γ_{mm}

where

f_k is the characteristic compressive strength of masonry;

γ_{mm} is the partial safety factor for compressive strength of masonry;

c) the maximum strain at the outermost compression fibre is 0.0035;

d) the tensile strength of masonry is ignored;

e) plane sections remain plane when considering the strains in bonded tendons and any other bonded reinforcement, whether in tension or in compression;

f) stresses in bonded tendons, whether initially tensioned or untensioned, and in any other reinforcement are derived from the appropriate stress-strain curves shown in Figure 1 and Figure 4;

g) stresses in unbonded tendons in post-tensioned members are limited to 70 % of their characteristic strength;

h) the effective depth, d, to unbonded tendons is determined by taking full account of the freedom of the tendons to move.

The resistance moment, M_u, of members containing bonded or unbonded tendons, all of which are located in the tension zone, may be taken as:

$$M_u = f_{pb} A_{ps} z$$

where

f_{pb} is the tensile stress in tendon at ultimate limit state;

z is the lever arm;

A_{ps} is the area of prestressing tendons.

In members with unbonded tendons, the strain induced in the tendons by the applied moment is not the same as that in the adjacent masonry. For such members with rectangular compression zones, and with $\gamma_{mm} = 2$, values of f_{pb} and x, the neutral axis depth, may be obtained from the following equations:

$$f_{pb} = f_{pe} + 700 \times \{d/l\} \times \{1 - 1.4\ (f_{pu}/f_k) \times (A_{ps}/bd)\}$$

$$x = 2\ (f_{pu}/f_k) \times (f_{pb}/f_{pu}) \times (A_{ps}/bd)\ d$$

where

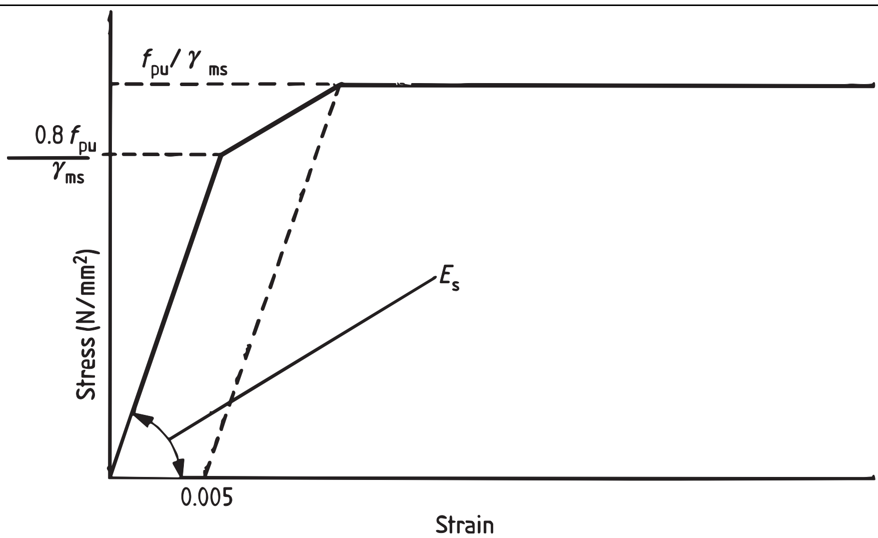
f_{pe} is the effective prestress after losses;

d is the effective depth to centroid of tendons;

l is the distance between end anchorages;

f_{pu} is the characteristic strength of tendons;

b is the breadth of masonry compression zone.



E_s 205 kN/mm² for cold drawn wire conforming to BS 5896;
195 kN/mm² for strand conforming to BS 5896;
165 kN/mm² for rolled and stretched bars conforming to BS 4486;

f_{pu} is the characteristic tensile strength of prestressed tendons;

γ_{ms} is the partial safety factor for strength of steel

Figure 4 — Typical short-term design stress-strain curves for normal and low relaxation tendons

9.2.2 ***Loading parallel to principal axis***

The strength of a slender prestressed member subjected to loading parallel to a principal axis may be assessed by the method given in BS 5628-1:2005, Clause **25** for solid walls, except that, if the cross-section of the member is not solid rectangular in plan, the capacity reduction factor which allows for the effects of slenderness and the eccentricity of the applied load may need to be calculated in accordance with the design assumptions of BS 5628-1:2005, Annex B.

When a member is post-tensioned, the prestress may have to be limited to take account of the slenderness, and possible buckling failure, of the member due to the prestress alone.

9.2.3 ***Shear resistance***

NOTE Members built with full masonry bonding rely for their shear strength on the masonry, while members that use metal shear ties in the bed joints for bonding rely on the strength of the shear ties.

9.2.3.1 *Shear strength of masonry*

The shear stress, v, due to design loads at the section being considered may be determined from:

$$V = \frac{V}{d_o b}$$

and, for prestressed sections with bonded or unbonded tendons and which are uncracked in flexure, the design shear strength may be taken as:

$\{(f_t/\gamma_{mv})^2 + 0.9\, f_p\, (f_t/\gamma_{mv})\}^{0.5}$

where

- V is the shear force due to design loads at the section being considered;
- d_o is the overall depth of the section;
- b is the width of the section resisting shear;
- f_t is the diagonal tensile strength of masonry;
- f_p is the stress due to prestress at the centroid of the section;
- γ_{mv} is the partial safety factor for shear strength of masonry.

The characteristic diagonal tensile strength of masonry, f_t, may be taken as:

$f_t = 1.3 - 0.275M/Vd_o$

where

M is the bending moment due to design loads at the section being considered with:

$0.2 < f_t < 0.75$ N/mm^2 for dense aggregate solid concrete blockwork masonry; and

$0.2 < f_t < 1.60$ N/mm^2 for brickwork masonry.

For members which are cracked in flexure, the above relationships for determining design shear strength may be used with the additional beneficial effects of the increase in the prestressing force due to the flexural cracking also being taken into account by using an enhanced value of f_p.

The shear stress, v, should not exceed the design shear strength.

9.2.3.2 *Shear ties*

The design shear resistance of shear ties conforming to BS EN 845-1 should be taken as the declared shear load capacity, relevant for the type of mortar and masonry units, divided by the divided by the value of γ_{mt} for use with ancillary components (see **7.5.2.2**).

Alternatively, the size and spacing of flat metal shear connectors may be calculated using the following formula:

$$ru = 12t_w sv/(0.87f_y)$$

where

r	is the width of the tie;
u	is the thickness of the tie;
t_w	is the width of the masonry section in vertical shear;
s	is the spacing of the ties;
v	is the design vertical shear stress on the masonry section;
f_y	is the yield strength of the tie.

The shear connectors should be of flat metal section and should also conform to the recommendations for wall ties in respect of anchorage and embedment length and they should be embedded to a depth of at least 50 mm at each end.

9.3 Design for the serviceability limit state

9.3.1 When analysing a section, the following assumptions should be made:

a) plane sections remain plane when considering strain distribution in the masonry;

b) stress is proportional to strain;

c) no tensile stresses are allowed in the masonry;

d) after losses, the effective prestressing force does not change.

In general there are two serviceability conditions which need to be examined: at transfer of prestress, and under the design loads after losses, but there may be some intermediate stages when the load is applied incrementally.

9.3.2 The compressive stress should be limited to one third of the characteristic compressive strength of the masonry, f_k, under the design loads, and to $0.4f_{kt}$ at transfer, where f_{kt} is the compressive strength of the masonry at transfer.

Designers should assess the value of f_{kt} either by masonry tests in accordance with Annex D, or from the known behaviour of the materials being used. If compression tests on mortar samples, stored under the same conditions as the masonry, show that the specified 28 day strength has been achieved, f_{kt} may be taken to be equal to f_k.

9.3.3 Where the area of concrete infill represents more than 10 % of the section under consideration, elastic analysis should be undertaken using the transformed area calculated from the values of elastic modulus given in **7.4.1.7** (see also **7.1.1.2**).

9.3.4 The deflection of members should be calculated following the recommendations of **7.1.2.2.1**.

9.4 Design criteria for prestressing tendons

9.4.1 ***Maximum initial prestress***

The jacking force should not exceed 70 % of the characteristic breaking load of the tendon.

9.4.2 *Loss of prestress*

9.4.2.1 *General*

When calculating the forces in the tendons at the various stages considered in design, allowance should be made for the appropriate losses of prestress resulting from:

a) relaxation of the tendons (see **9.4.2.2**);

b) elastic deformation of masonry (see **9.4.2.3**);

c) moisture movement of masonry (see **9.4.2.4**);

d) creep of masonry (see **9.4.2.5**);

e) "draw-in" of the tendons during anchoring (see **9.4.2.6**);

f) friction (see **9.4.2.7**);

g) thermal effects (see **9.4.2.8**).

Where low levels of strain are induced in the prestressing tendon, the accumulation of losses may cancel the effects of prestress.

9.4.2.2 *Relaxation of tendons*

The loss of prestress should be taken to be the maximum relaxation of the tendon after 1 000 h duration given in the manufacturer's UK Certificate of Approval. In the absence of such a certificate, the values appropriate to the jacking force at transfer should be taken from BS 4486 or BS 5896, as appropriate. These standards give values corresponding to a maximum initial prestress of 60 % and 70 % of the breaking load. For initial loads of less than 60 % of the breaking load, the 1 000 h relaxation value may be assumed to decrease from the value given for 60 % to zero at 30 % of the breaking load.

When a load equal to or greater than the relevant jacking force has been applied to a tendon for a short time prior to the anchoring, no reduction in the value of the relaxation should be made.

Where stainless steel is used at more than 67 % of the 0.2 % proof stress, the 1 000 h relaxation might not give a true indication of relaxation and specialist advice should be sought.

9.4.2.3 *Elastic deformation of masonry*

Calculation of the immediate loss of force in the tendons due to elastic deformation in the masonry at transfer may be based directly on the values of the short-term elastic moduli, E_c, E_m and E_s, obtained from **7.4.1.7**, and the appropriate strength of the masonry (see **9.3**).

In post-tensioned masonry, when the tendons are stressed simultaneously, elastic deformation occurs during tensioning and thus there is no loss in prestress due to elastic deformation at transfer. With tendons that are not stressed simultaneously, there is a progressive loss during transfer, and the resulting total loss should be taken as being equal to half the product of the modular ratio and the stress in the masonry adjacent to the centroid of the tendons, unless the tendons are restressed.

9.4.2.4 *Moisture movement of masonry*

Where the moisture movement of masonry results in an eventual shrinkage, this will lead to a loss of prestress in the tendons, which may be calculated assuming that the maximum shrinkage strain is 500×10^{-6} for concrete and calcium silicate masonry. The effect of moisture expansion of fired-clay masonry on the force in the tendons should be disregarded in design.

9.4.2.5 *Creep of masonry*

The loss of force in the tendons due to the effects of creep in fired-clay or calcium silicate brick masonry and dense aggregate concrete block masonry may be calculated by assuming that the creep is numerically equal to 1.5 and 3.0 times the elastic deformation of the masonry, respectively. The elastic deformation should be based on the appropriate value of the elastic modulus, E_m, obtained from **7.4.1.7**.

NOTE Some information is available in research reports about the estimation of creep values. However, at the time of publication, no information is available for masonry built with lightweight concrete blocks.

9.4.2.6 *Anchorage draw-in*

In post-tensioning systems, and particularly for short members, allowance should be made for any movement of the tendon at the anchorage when the prestressing force is transferred from the tensioning equipment to the anchorage.

9.4.2.7 *Friction*

In post-tensioning systems with tendons in ducts, there will be movement of the greater part of the tendon relative to the surrounding duct during the tensioning operation and, if the tendon is in contact with the duct or any spacers provided, friction will cause a reduction in the prestressing force. In the absence of other information, the stress variation likely to be expected should be assessed following the recommendations of BS 8110-1:1997, **4.9**.

9.4.2.8 *Thermal effects*

Consideration should be given to differential thermal movement between the masonry and the prestressing tendon, especially where tendon stresses are low.

9.4.3 ***Transmission length in pre-tensioned members***

The length of member required to transmit the initial prestressing force in a tendon to the concrete or grout surrounding it depends upon a number of variables, the most important being the strength and homogeneity of the concrete or grout and the size, type and deformation, e.g. crimp, of the tendon.

The transmission length should, where possible, be based on experimental evidence from known site or factory conditions. In the absence of such evidence, the following equation for the transmission length, l_t, may be used for initial prestressing forces up to 75 % of the characteristic strength of the tendon when the ends of the units are fully compacted:

$$I_t = \frac{K_t \phi_t}{\sqrt{f_{ci}}}$$

where

f_{ci} is the concrete strength at transfer;

φ_t is the nominal diameter of the tendon;

K_t is a coefficient for the type of tendon and is selected from the following:

a) plain or indented wire (including crimped wire with a small wave height): $K_t = 600$;

b) crimped wire with a total wave height not less than 0.15φ: $K_t = 400$;

c) 7-wire standard or super strand: $K_t = 240$;

d) 7-wire drawn strand: $K_t = 360$.

9.5 Detailing prestressed masonry

9.5.1 ***Tendons***

For vertically spanning post-tensioned members, bar tendons should be sufficiently stiff to stand vertically without lateral support so that the masonry can be built around them. Where strands or wires are used they should be suspended from scaffolding or placed in vertically standing ducts. Tendon anchorages should be correctly positioned.

For pretensioned masonry, where tendons or groups of tendons are surrounded by concrete, the distance between individual tendons or groups of tendons should be not less than the maximum aggregate size plus 5 mm to allow for adequate compaction of the concrete.

To prevent overstressing of the masonry, it is essential for the designer to specify the correct tensioning sequence for the tendons and the compressive strength of the masonry at transfer.

9.5.2 ***Anchorage in reinforced concrete***

Where tendon anchorages are embedded in reinforced concrete, this may form part of a foundation, wall coping or column capital. In such circumstances, the reinforced concrete needs to be designed to sustain all the forces it is subjected to, both internal (from the anchorages) and external (from, for example, floors, roof and ground). Where bending stresses arise from the reinforced concrete spanning over voids or spaces in the masonry cross-section, to avoid the development of splitting cracks in the masonry near the anchorage, the design tensile stress in the reinforcement should be limited to 85 N/mm^2. The reinforced concrete should be designed to distribute the prestress from the anchorages to the wall to prevent local overstressing of the masonry.

9.5.3 ***Detailing prestressed masonry***

The local bearing stress on the masonry immediately beneath a prestressing anchorage, after locking off the tendon, should not exceed:

a) $1.5f_k/\gamma_{mm}$, following the recommendations of BS 5628-1:2005, Clause **27** where the prestressing loads are perpendicular to the bed joints; or

b) $0.65f_k/\gamma_{mm}$, where the prestressing loads are parallel to the bed joints

where

f_k is the characteristic compressive strength of masonry;

γ_{mm} is the partial safety factor for compressive strength of masonry.

9.5.4 ***Tendons***

To prevent overstressing of the masonry, it is essential for the designer to specify the correct tensioning sequence for the tendons and the compressive strength of the masonry at transfer.

Where tendons or groups of tendons are surrounded by concrete, the distance between individual tendons or groups of tendons should not be less than the maximum aggregate size plus 5 mm to allow for adequate compaction of the concrete.

9.5.5 ***Links***

Where links are required, they should be provided in accordance with **8.6.5**.

10 Other design considerations

10.1 Durability

10.1.1 ***Masonry units and mortars***

Guidance on the durability of masonry units and mortars is given in BS 5628-3:2005, **5.6**.

10.1.2 ***Resistance to corrosion of metal components***

10.1.2.1 *General*

Adequate durability may be ensured either by selecting appropriately protected reinforcement, or by providing sufficient concrete cover of the appropriate quality.

The type of reinforcement and the minimum level of protective coating for reinforcement which should be used in various types of construction and site exposures is given in Table 14. This table applies to low carbon steel, high yield steel, galvanized steel, with or without a resin coating, and austenitic stainless steel. In all cases, concrete infill to cavities should be in accordance with **10.1.2.5** and **10.1.2.6**.

As an alternative to the requirements of Table 14, carbon steel reinforcement may be used provided that the concrete cover is in accordance with Table 15.

Annex E summarizes the durability recommendations for a number of construction types.

10.1.2.2 *Classification of exposure situations*

Exposure situations are classified into the following four situations.

Exposure situation E1. Internal work and the inner skin of ungrouted external cavity walls and behind surfaces protected by an impervious coating that can readily be inspected or external parts built where the exposure category given in BS 5628-3:2005, Table 10 is Sheltered or Very Sheltered.

Exposure situation E2. Buried masonry and masonry continually submerged in fresh water or external parts built where the exposure category given in of BS 5628-3:2005, Table 10 is Sheltered/Moderate or Moderate/Severe.

Exposure situation E3. Masonry exposed to freezing whilst wet, subjected to heavy condensation or exposed to cycles of wetting by fresh water and drying out or external parts built where the exposure category given in BS 5628-3:2005, Table 10 is Severe or Very Severe.

Exposure situation E4. Masonry exposed to salt or moorland water, corrosive fumes, abrasion or the salt used for de-icing.

10.1.2.3 *Exposure situations requiring special attention*

Special consideration should be given to any feature that is likely to be subjected to more severe exposure than the remainder of the building or structure. In particular, parapets, sills, chimneys and the details around openings in external walls should be examined. Normally such situations should be considered equivalent to exposure situation E3.

Where stressed reinforcement and tendons may be directly exposed to chlorinated atmosphere, e.g. in swimming pools, stainless steel should be used only following specialist advice.

10.1.2.4 *Effect of different masonry units*

The protection against corrosion provided by brickwork tends to be improved if high strength low water absorption bricks are used in strong mortar. Where bricks that have a greater water absorption than 10 %, when determined in accordance with BS EN 772-2, or concrete blocks having a net density less than 1 500 kg/m^3 are used, the steel recommended for the next most severe exposure situation or, where appropriate, stainless steel should be used, unless protection to the reinforcement is to be provided by concrete cover in accordance with **10.1.2.6**.

10.1.2.5 *Concrete infill*

Concrete infill for reinforced masonry should be of minimum grade C30 or equivalent and be specified in accordance with BS 8500-1, taking into account minimum cement content, maximum free water/cement ratio and cover as given in Table 15.

For grouted cavity and Quetta bond reinforced masonry construction the concrete infill may consist, at the option of the designer, of a 1 : 0 to ¼ : 3 : 2 cement : lime : sand : 10 mm nominal maximum size aggregate mix, or a mortar infill, as appropriate to the exposure situation and reinforcement type, in accordance with the requirements of Table 14 and **10.1.2.6**. Where high lift grouted cavity construction (see **11.2.2.3**) or Quetta bond is employed, the infill concrete mix, if required to provide durability protection to the reinforcement, should contain an expanding agent or other suitable measures to avoid early age shrinkage.

Concrete infill for pre-tensioned masonry should be of minimum grade C40, minimum cement content, maximum free water/cement ratio and cover as given in Table 15.

Concrete infill for post-tensioned masonry should be of minimum grade C30 or equivalent and be specified in accordance with BS 8500-1, taking into account minimum cement content and maximum free water/cement ratio as given in Table 15. This recommendation is nominal as the durability of post-tensioned masonry is usually assured by direct protection of the tendons.

10.1.2.6 *Cover*

Where austenitic stainless steel, or steel coated with at least 1 mm of austenitic stainless steel, is used, there is no minimum cover required to ensure durability. However, some cover will be required for the full development of bond stress (see **8.6.6**).

Where reinforcement is placed in bed joints, the minimum depth of mortar cover to the exposed face of the masonry should be 15 mm.

For grouted-cavity or Quetta bond construction, the minimum cover for reinforcement selected using Table 14 should be as follows:

a) carbon steel reinforcement used in internal walls and exposure situation E1: 20 mm mortar or concrete;

b) carbon steel reinforcement used in exposure situation E2: 20 mm concrete;

c) galvanized steel reinforcement: 20 mm mortar or concrete;

d) stainless steel reinforcement: not required for durability.

Table 14 — Selection of reinforcing steel for durability

Exposure class	Minimum level of protection for reinforcement, excluding cover	
	Located in bed joints or special clay masonry units	Located in grouted cavity or Quetta bond construction
E1	Carbon steel galvanized in accordance with BS EN ISO 1461. Minimum mass of zinc coating 940 g/m^2 or for bed joint reinforcement material/coating reference R1 or R3.[a]	Carbon steel
E2	Carbon steel galvanized in accordance with BS EN ISO 1461. Minimum mass of zinc coating 940 g/m^2 or for bed joint reinforcement material/coating reference R1 or R3.	Carbon steel or, where mortar is used to fill the voids, carbon steel galvanized in accordance with BS EN ISO 1461 to give a minimum mass of zinc coating of 940 g/m^2
E3	Austenitic stainless steel in accordance with BS EN 10088 or carbon steel coated with at least 1 mm of stainless steel or for bed joint reinforcement material/coating reference R1 or R3.	Carbon steel galvanized in accordance with BS EN ISO 1461. Minimum mass of zinc coating 940 g/m^2
E4	Austenitic stainless steel in accordance with BS EN 10088 or carbon steel coated with at least 1 mm of stainless steel or for bed joint reinforcement material/coating reference R1 or R3.[b]	Austenitic stainless steel in accordance with BS EN 10088 or carbon steel coated with at least 1 mm of stainless steel.[b]

[a] In internal masonry other than the inner leaves of external cavity walls, carbon steel reinforcement or bed joint reinforcement with any material/coating reference may be used.

[b] Austenitic stainless steel grades should be selected according to the exposure and environmental aggression applicable. Not all grades will necessarily be suitable for the most aggressive environments, particularly those environments where regular salts application is used as in highways de-icing situations.

Table 15 — Minimum concrete cover for carbon steel reinforcement

Exposure situations	*Concrete grade in BS EN 206-1 and BS 8500*				
	C25/30	C28/35	C32/40	C35/45	C40/50
	Minimum cement content (kg/m³)[a]				
	275	300	325	350	400
	Maximum free water/cement ratio				
	0.65	0.6	0.55	0.50	0.45
	Thickness of concrete cover				
	mm	mm	mm	mm	mm
E1[b]	20	20	20[c]	20[c]	20[c]
E2	—	35	30	25	20
E3	—	—	40	30	25
E4	—	—	—	60[d]	50

[a] All mixes are based on the use of normal-weight aggregate of 20 mm nominal maximum size (but see **6.9**). Where other sized aggregates are used, cement contents should be adjusted in accordance with Table 16.

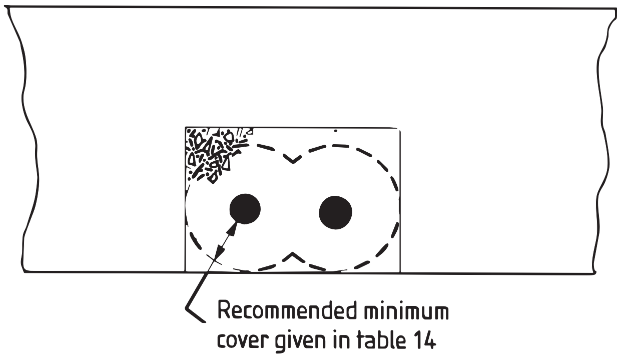
[b] Alternatively, 1 : 0 to ¼ : 3 : 2 cement : lime : sand : 10 mm nominal aggregate mix may be used to meet exposure situation E1, when the cover to reinforcement is 15 mm minimum.

[c] These covers may be reduced to 15 mm minimum provided that the nominal maximum size of aggregate does not exceed 10 mm.

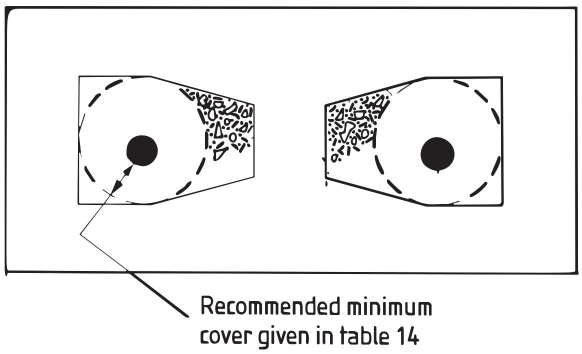
[d] Where the concrete infill may be subjected to freezing whilst wet, air entrainment should be used.

Table 16 — Adjustments to minimum cement contents for aggregates other than 20 mm nominal maximum size

Nominal maximum aggregate size mm	Adjustments to minimum cement contents in Table 15 kg/m³
10	+40
14	+20
20	0



a) Pocket-type wall



b) Reinforced hollow blockwork wall

Figure 5 — Minimum concrete cover in pocket-type walls and in reinforced hollow blockwork walls

Figure 5 shows the minimum concrete cover recommended for carbon steel reinforcement in pocket-type walls and in reinforced hollow blockwork walls.

The cut ends of all bars, except those of solid stainless steel, should have the same cover as that appropriate to carbon steel in the exposure situation being considered, unless alternative means of protection are used.

10.1.2.7 *Prestressing tendons*

Where tendons are placed in pockets, cores or cavities that are filled with concrete or mortar, the recommendations given in **10.1.2.1**, **10.1.2.5** and **10.1.2.6** should be followed.

Where carbon steel tendons or bars are installed in open cavities, pocket or ducts they should be suitably protected.

NOTE Under certain circumstances, galvanizing may lead to hydrogen enbrittlement and should therefore be avoided.

Ducts for unbonded tendons should be suitably drained.

NOTE Where carbon steel is protected in grouted ducts, it is essential to ensure that the protection is complete. Generally this type of protection should be avoided unless this is carried out in accordance with BS 8110-1:1997, **8.9**.

10.1.2.8 *Wall ties*

Wall ties should be protected against corrosion appropriate to the exposure conditions in accordance with BS 5628-3:2005.

In some cases the material of the wall ties will differ from that of the reinforcement. In such cases the two dissimilar metals should not be allowed to come into contact.

10.2 Fire resistance

The recommendations for fire resistance of reinforced and prestressed concrete elements given in Section 4 of BS 8110-2:1985 should be followed, but taking masonry as part of the cover.

10.3 Accommodation of movement

Precautions should be taken against cracking due to movement in walls, following the recommendations of BS 5628-3:2005, Clause **5.4**. Where contraction joints are not designed to act as expansion joints, separate expansion joints should be provided at intervals of 30 m in concrete block, concrete brick or calcium silicate brick free-standing or retaining walls. In earth-retaining walls, where the temperature and moisture content of the masonry do not vary greatly, joint spacings of up to 20 m may be justified.

In addition, debonded dowels may be provided to restrict lateral movement between adjacent panels whilst permitting movement within the plane of the wall. Where appropriate, dowels should also be incorporated at the joint between a panel wall and its frame.

10.4 Spacing of wall ties

In ungrouted cavity walls and low-lift grouted-cavity walls, the spacing of ties should follow the recommendations of BS 5628-1.

In high-lift grouted-cavity walls, the wall ties should be spaced at not greater than 900 mm centres horizontally and 300 mm centres vertically, with each layer staggered by 450 mm. Additional ties should be provided at openings, spaced at not greater than 300 mm centres vertically.

10.5 Drainage and waterproofing

When retaining walls support earth, other than freely draining granular material, a drainage layer of rubble or coarse aggregate of 200 mm thickness, or 100 mm thick porous blocks, should be placed behind the wall for the full height of the earth retained. Preferably, perforated land drains should be laid behind retaining walls and be provided with a positive outfall to suitable drainage positions. Weepholes can be used as drainage condition indicators, but their positioning should be carefully considered in order to avoid uncontrolled overflow onto hard areas where there is pedestrian or vehicular public access.

To minimize salts ingress and other staining of the exposed masonry face in service, all walls retaining earth should be painted with a waterproofing compound on the face in contact with the earth. Where practicable, a layer of self-adhesive bituminous sheet, with all joints lapped, may be applied in place of the waterproofing compound. Waterproofing compounds and sheeting should be protected before backfilling.

Where it is not practicable to provide retaining walls with weepholes or land drains, e.g. in basement walls, or where the wall is designed to resist a permanent water pressure, asphalt tanking or a similar positive waterproofing layer should be applied and protected before backfilling.

At vertical movement joints where anything other than minor movement is anticipated, a water bar may be used.

NOTE BS 8102 gives guidance on the waterproofing of structures below ground level.

10.6 D.P.C.s and copings

The provision of d.p.c.s and copings should follow the recommendations of BS 5628-3:2005, **5.5** having regard to the material of the d.p.c. and its effect on the bending and shear strength of the member.

11 Work on site

11.1 Materials

All materials used in reinforced and prestressed masonry should follow the recommendations of Clause **6**.

11.2 Construction

11.2.1 *General*

Work should generally conform with BS 5628-3.

Storage and handling of masonry units, and storage and mixing of materials for mortars should follow the recommendations of Section 1 of BS 5628-1:2005. Storage and mixing of materials for concrete, and storage, handling, fixing, and positioning of reinforcement and prestressing tendons should follow the recommendations of BS 8110-1:1985, Sections 6, 7 and 8.

For laying of masonry units in reinforced and prestressed masonry, plumbness and alignment of the masonry and precautions to protect the work in adverse weather conditions and when the work is temporarily stopped, reference should be made to BS 5628-3:2005, Clause **6**.

The maximum height of masonry that should normally be built in a day is 1.5 m.

Infill concrete should be in accordance with **6.9**. Special consideration should be given to the workability of the infill concrete and the height of pour when filling small sections, to ensure that complete filling is achieved.

Reinforcement should be in accordance with **6.3** and fixed as shown on the detail drawings. Care should be taken to ensure that the specified cover to the reinforcement is maintained, e.g. by using spacers. Where spacers are used or where bed joint reinforcement crosses voids or pockets that contain reinforcement and are to be filled with concrete, the spacers should be of such a type and the reinforcement so positioned that compaction of the infill concrete is not prevented.

Reinforcement should be free from mud, oil, paint, retarders, loose rust, loose mill scale, snow, ice, grease or any other substance that may adversely affect the steel or concrete chemically, or reduce the bond. Normal handling prior to embedment is usually sufficient for the removal of loose rust and scale from reinforcement. Bed joint reinforcement should be completely surrounded with mortar.

Anchorages, bar couplings and other supports to prestressing tendons should be positioned and aligned so that the tendons are not kinked or bent beyond the manufacturer's recommendations.

11.2.2 *Grouted-cavity construction*

11.2.2.1 *General*

It is essential that mortar droppings or scrapings should not be permitted to remain in the cavity (see BS 5628-3:2005, **A.5.4**). Ties between the leaves of grouted-cavity walls should be provided in accordance with **10.4**.

11.2.2.2 *Low-lift*

In low-lift grouted-cavity construction, the concrete infill should be placed as part of the process of laying the masonry units at maximum vertical intervals of 450 mm. Any excess mortar in the cavity should be removed before infilling. The infill concrete should be placed in layers to within 50 mm of the level of the last course laid and should be placed using receptacles with spouts to avoid staining and splashing of face work. It is important that the concrete infill should be compacted immediately after pouring.

Care should be taken to avoid raising the walls too rapidly, causing disruption due to excessive lateral pressure from the infill concrete before the masonry has had time to gain sufficient strength. If the wall should move at any level due to these forces, it is essential to take it down and rebuild it.

11.2.2.3 *High-lift*

In the high-lift technique, walls should be built up to a maximum 3 m high and clean-out holes left along the base of one leaf. These holes should be of minimum size 150 mm × 200 mm and spaced at approximately 500 mm centres. Prior to infilling with concrete, and preferably soon after laying, debris should be removed from the cavity and the clean-out holes should then be blocked off. The concrete infill should be placed not sooner than 3 days after building.

The infill should be placed and compacted, usually in two lifts. Recompaction of the concrete in each lift may be necessary after initial settlement, due to water absorption by the masonry, but before setting.

Wall ties (see **6.5**, **10.4** and Annex B) should be used to hold the leaves together against the lateral pressure exerted by the concrete infill.

11.2.3 ***Reinforced hollow blockwork***

11.2.3.1 *General*

All hollow blocks should be laid on a full bed of mortar and any excess mortar in the core should be removed before placing of the infill.

11.2.3.2 *Low-lift*

The procedure for low-lift filled hollow blockwork should in general follow the corresponding recommendations for low-lift grouted-cavity construction, except that the maximum vertical intervals at which concrete infill is placed may be increased to 900 mm.

11.2.3.3 *High-lift*

In the high-lift technique, walls should be built up to a maximum 3 m high and clean-out holes left along the base of the wall. These holes should occur at every core which is to be filled and should be of minimum size 100 mm × 100 mm. Alternatively, particularly where every core is to be filled, the base course may consist of bricks spaced to suit the size of block in order to achieve a clear opening at each core.

High-lift grouting should not be used for walls the overall thickness of which is less than 190 mm.

Prior to infilling with concrete, and preferably soon after laying, debris should be removed from the core and the clean-out holes blocked off. Infilling should not be carried out sooner than one day after building; a longer time should be allowed in cold weather. Concrete infill should be placed and compacted, usually in two lifts. Recompaction of the concrete in each lift may be necessary after initial settlement, due to water absorption by the masonry, but before setting.

11.2.4 ***Quetta bond and similar bond walls***

Main reinforcement should be fixed sufficiently in advance of the masonry construction so that other work can proceed without hindrance. The cavities formed around the reinforcement by the bonding pattern should be filled with mortar or concrete infill as the work proceeds. Alternatively, if the cavities are sufficiently large, they may be filled by the low- or high-lift techniques described in **11.2.3.2** and **11.2.3.3** respectively. Secondary reinforcement, where required, should be incorporated in the bed joints, in accordance with Clause **6**, as the work proceeds.

11.2.5 ***Pocket-type walls***

In pocket-type wall construction, the walls are generally built to full height before the infill concrete is placed. Main reinforcement should preferably be fixed in advance of wall construction, especially where it is necessary to incorporate reinforcement in the bed joints. Care should be taken to ensure that the formwork to the back face of the pocket is adequately tied to the wall or propped to prevent disturbance of the formwork during placing and compaction of the infill concrete, and to avoid grout loss.

11.2.6 ***Tensioning of prestressing tendons***

Tensioning of tendons should be carried out following the recommendations of BS 8110-1:1997, **8.7**. It is essential to ensure that the specified value for the masonry strength at transfer is achieved. In walls where it is not possible simultaneously to prestress all the tendons, prestressing should follow a pre-arranged programme, loading the tendons in stages until the required level of prestress is applied along the wall.

11.2.7 ***Forming chases and holes, and provision of fixings***

The protection of prestressing tendons should follow the recommendations of BS 8110-1:1997, **8.8**. Protection against corrosion may be provided by the use of stainless steel tendons.

11.2.8 ***Replacement of unbonded tendons***

If tendons have to be replaced, the defective tendons should be de-stressed gradually under competent supervision to safeguard persons from injury and to avoid damage to equipment and the works. The strength, stability and serviceability of the structure, of which the prestressed masonry may only be a part, should be maintained during de-stressing.

11.2.9 ***Forming chases and holes and provisions of fixings***

Chasing of completed walls, the formation of holes and the inclusion of fixings should be carried out only when approved by the designer and then following the recommendations of BS 5628-3:2005, **A.5.6.10**.

11.2.10 ***Jointing and pointing***

Joints should only be raked-out or pointed when approved by the designer.

11.3 Quality control

11.3.1 ***Workmanship***

Workmanship should generally conform to BS 5628-3.

The designers should specify, supervise and control the construction of reinforced and prestressed masonry to ensure that the construction is compatible with the special category of construction control as defined in BS 5628-1.

Preliminary and site testing should be carried out (see **11.3.2**).

11.3.2 ***Material***

11.3.2.1 *General*

All sampling and testing of materials should be carried out in accordance with the appropriate British Standard.

11.3.2.2 *Masonry units*

If clay masonry units are used that have an initial rate of water absorption, when tested in accordance with BS EN 772-11, greater than 1.5 kg/($m^2 \cdot$min), they may need wetting before laying (see BS 5628-3:2005, **A.4.2.2.2**).

11.3.2.3 *Mortar*

The procedures for trial mixes and site control of mortar should follow the recommendations of BS 5628-1.

11.3.2.4 *Infill concrete*

All sampling and testing of fresh and hardened infill concrete should be carried out in accordance with BS 1881. A prescribed mix should, unless otherwise specified, be judged on the basis of the specified mix proportions and required workability.

A designed mix should be assessed according to the strength of the hardened concrete.

Annex A (normative) Design method for walls incorporating bed joint reinforcement to enhance lateral load resistance

A.1 General

Recommendations for the design of unreinforced walls subjected to lateral loads are given in BS 5628-1:2005, Clause **29**. The use of bedjoint reinforcement enhances the capacity of walls to resist lateral loading. This Annex is based on the restricted amount of research available and includes four alternative approaches to design that may be used (see **A.3** to **A.6**).

The proposed design methods may be applied to walls made from masonry units described in BS 5628-1:2005, Clause **7** and mortar of strength class M4 may be used. The characteristic compressive strength of masonry constructed using types of masonry unit and mortar strength classes not given in the tables in this part of BS 5628 is given in BS 5628-1:2005, Clause **19**. Partial safety factors should be chosen for the appropriate level of quality control from BS 5628-1:2005, Clause **23**.

NOTE The recommendations of **11.3.1** of this part of BS 5628 apply only to the special category of construction control as defined in BS 5628-1.

Special care is required to ensure that adequate provision is made to protect bed joint reinforcement against corrosion. The designer should follow the recommendations of **10.1**.

The values for characteristic anchorage bond strength given in **7.4.1.6** should be used with caution. It is advisable in all cases to consult the reinforcement manufacturer and this is particularly important where some form of coating against corrosion has been specified for use on the steel.

A.2 Design recommendations

A.2.1 *General*

The experimental evidence available suggests that for walls reinforced with the percentage of steel that is common for bed joint reinforcement, the load at which the wall first cracks is comparable to the ultimate load for a similar unreinforced wall, although the cracking patterns may differ.

A.2.2 *Support conditions and continuity*

The degree of restraint provided by different types of support should be assessed as described in BS 5628-1:2005, Clause **29**.

A.2.3 *Limiting dimensions*

The limiting dimensions of panels should be as follows:

a) panel supported on three edges:

1) two or more sides continuous:
height × length equal to 1 800 t_{ef}^2 or less;

2) all other cases:
height × length equal to 1 600 t_{ef}^2 or less.

b) panel supported on four edges:

1) three or more sides continuous:
height × length equal to 2 700 t_{ef}^2 or less;

2) all other cases:
height × length equal to 2 400 t_{ef}^2 or less.

No dimension should exceed 60 t_{ef} where t_{ef} is the effective thickness as defined in **8.3.2.4**.

A.2.4 *Minimum amount of reinforcement*

It may be assumed that the wall will have enhanced lateral load resistance compared with an unreinforced wall if reinforcement with a minimum cross-sectional area of 14 mm^2 is placed at vertical intervals not exceeding 450 mm.

A.2.5 *Compressive strength of masonry*

In general there is little likelihood of the compressive strength of the masonry in bending being exceeded in walls that are reinforced with bed joint reinforcement. However, when using masonry units of low compressive strength or highly perforated masonry units and frequent reinforcement of the bed joints, the designer should check that this is the case by using the appropriate formula (see **8.2.4**) and values of f_k appropriate to the direction of the compressive force.

A.2.6 *Partial safety factors*

Where reference is made to the use of the design formulae in **8.2.4**, the appropriate partial safety factor for the compressive strength of masonry, γ_m, should be taken from BS 5628-1:2005, Clause **20**.

A.3 Method 1: design as horizontal spanning wall

Single-leaf walls and reinforced leaves of cavity walls may be designed as spanning horizontally between supports, following the recommendations of **8.2.4.2** and considering steel which is in tension. For a cavity wall where both leaves are reinforced, the design lateral strength may be considered to be the sum of the design strengths of the two leaves.

It is essential to ensure that the wall ties are capable of transmitting the required forces. Recommendations for the use of wall ties as panel supports are given in BS 5628-1:2005, Clause **29**.

The maximum enhancement of lateral load resistance above that for the equivalent unreinforced wall, which may include some element of two-way spanning, should be taken to be 50 % unless a serviceability and deflection check is carried out in accordance with **A.6**.

A.4 Method 2: design with reinforced section carrying extra load only

Single-leaf walls may be designed to span horizontally between supports on the basis that the enhancement in lateral load resistance above that for the unreinforced wall is derived from the reinforced section.

The reinforced section should be designed using the equation in **8.2.4.2.1**. The maximum enhancement of load capacity above that for the unreinforced wall should be limited to 30 % unless a serviceability and deflection check is carried out in accordance with **A.6** (see note).

NOTE This approach to design cannot be rigorously justified in theoretical terms as it combines the flexural resistance of the uncracked unreinforced section spanning two ways with the design resistance of the reinforced section, which may be cracked, spanning one way.

A.5 Method 3: design using modified orthogonal ratio

Single-leaf walls and cavity walls may be designed following the appropriate recommendations of BS 5628-1:2005, **32.4** but using a modified orthogonal ratio.

For leaves which contain bed joint reinforcement, the orthogonal ratio is defined as the ratio of the moment of resistance about a horizontal axis (i.e. when the plane of failure is parallel to a bed joint), to the moment of resistance about a vertical axis (i.e. when the plane of failure is perpendicular to a bed joint). The moment of resistance about the horizontal axis is given by:

$$\frac{f_{kx}Z}{\gamma_m}$$

where

- f_{kx} is the characteristic flexural strength of the masonry, when the plane of failure is parallel to the bed joints, given in BS 5628-1:2005, Clause **20**;
- γ_m is the partial safety factor for strength of masonry given in BS 5628-1:2005, Clause **23**;
- Z is the section modulus per unit length of the bed joint.

The design moment of resistance about the vertical axis is as given in **8.2.4.2**.

The design moment in the panel is found using the appropriate bending moment coefficient in BS 5628-1:2005, Table 8. The design moment of resistance of the panel is determined from **8.2.4.2**.

For cavity walls, the recommendations of BS 5628-1:2005, **32.4.5** should be followed.

The maximum enhancement of lateral load resistance above that for the equivalent unreinforced wall should be taken to be 50 %, unless a serviceability and deflection check is carried out in accordance with **A.6**.

A.6 Method 4: design based on cracking load

Since the load causing cracking of a single-leaf wall containing bed joint reinforcement is at least as large as the ultimate load of a similar unreinforced wall, the cracking load may be used to assess whether the wall conforms to the serviceability requirements, up to the design strength of the reinforced section.

The failure strength of the wall, excluding reinforcement, should be calculated in accordance with BS 5628-1:2005, **32.4**, taking the value of γ_m as 1.0. The service strength is then determined by dividing this strength by the partial safety factor for masonry for the serviceabililty limit state taken from **7.5.3.2**.

To ensure that there is an adequate margin of safety against reaching the ultimate limit state, the wall should be designed as described in **A.3**, **A.4** or **A.5** but with no limitation on the load enhancement. The appropriate partial safety factor, γ_f, should be obtained from **7.5.2.1**, bearing in mind the recommendations of **A.2.6**. However, the designer should ensure that in service the deflection will not be excessive; the deflection at service load may be calculated assuming that the wall acts as an elastic plate.

A.7 Cavity walls

Where cavity walls have both leaves reinforced to increase lateral load capacity, the enhancement in design lateral strength of each leaf should be limited to the values given in **A.3** to **A.6**. The total load capacity of the wall may be taken as the sum of the design lateral strengths of the leaves.

Where only one leaf of a cavity wall is reinforced, the maximum enhancement of the design lateral strength, appropriate to the method, relates to that leaf.

Annex B (informative) Wall tie for high-lift cavity walls

Figure B.1 illustrates a wall-tie that may be used in the construction of high-lift grouted walls. The ties should be provided at the spacings given in **10.4**, and should be of 6 mm diameter galvanized low carbon steel, resin coated galvanized low carbon steel or austenitic stainless steel (see **10.1.2.8**) bent to the shape and nominal size shown in Figure B.1. For galvanized ties, the minimum mass of zinc should be as given in Table 13. The cover should be that recommended for carbon steel reinforcement in Table 14.

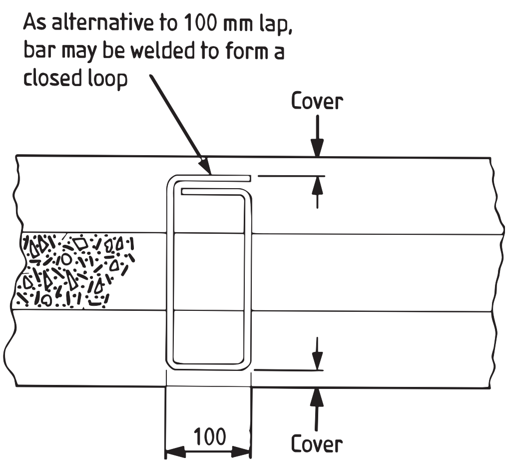


Figure B.1 — Wall tie for high-lift grouted-cavity wall

Annex C (informative) Estimation of deflection

When deflection of reinforced members is calculated, it should be realized that there are a number of factors that may be difficult to allow for in the calculation but which can have a considerable effect on its reliability, examples of which are as follows:

a) Estimates of the restraints provided by supports are based on simplified and often inaccurate assumptions.

b) The precise loading, or that part of it which is of long duration, is unknown.

c) Considerable differences will occur in the deflections, depending on whether the member has or has not cracked.

An elastic analysis should be used to estimate deflections. The following assumptions may be made:

1) The section to be used for the calculation of stiffness is the gross cross-section of the masonry, no allowance being made for the reinforcement.

2) Plane sections remain plane.

3) The reinforcement, whether in tension or compression, is elastic.

4) The masonry in compression is elastic. Under short term loading, the moduli of elasticity may be taken as the appropriate value given in **7.4.1.7**. The long term elastic modulus, E_m, allowing for creep and shrinkage where appropriate, may be taken as:

— for clay and dense aggregate concrete masonry, $E_m = 0.45 f_k$ kN/mm^2;

— for calcium silicate, aircrete and lightweight concrete masonry, $E_m = 0.3 f_k$ kN/mm^2

where

f_k is the characteristic compressive strength of masonry obtained from **7.4.1.2**.

The deflection at the appropriate applied bending moment may be estimated directly or from the estimated curvature.

Annex D (normative) Method for determination of characteristic strength of masonry, f_k

D.1 General

The characteristic compressive strength of masonry perpendicular to the bed joints should be determined in accordance with BS EN 1052-1.

Where the bonding pattern of the masonry units and the direction of the force in the compression zone of elements is such that the specimen format in BS EN 1052-1 is inappropriate, a specimen type should be used which is representative of, and built in the same attitude to, that on site. The preparation of the specimens, the conditioning required before testing, the testing machine, the method of calculation and the test report should follow the guidance in BS EN 1052-1. The test specimens should be loaded in such a direction as to represent the compressive force in the element on site. Typical brickwork specimens are shown in Figure D.1.

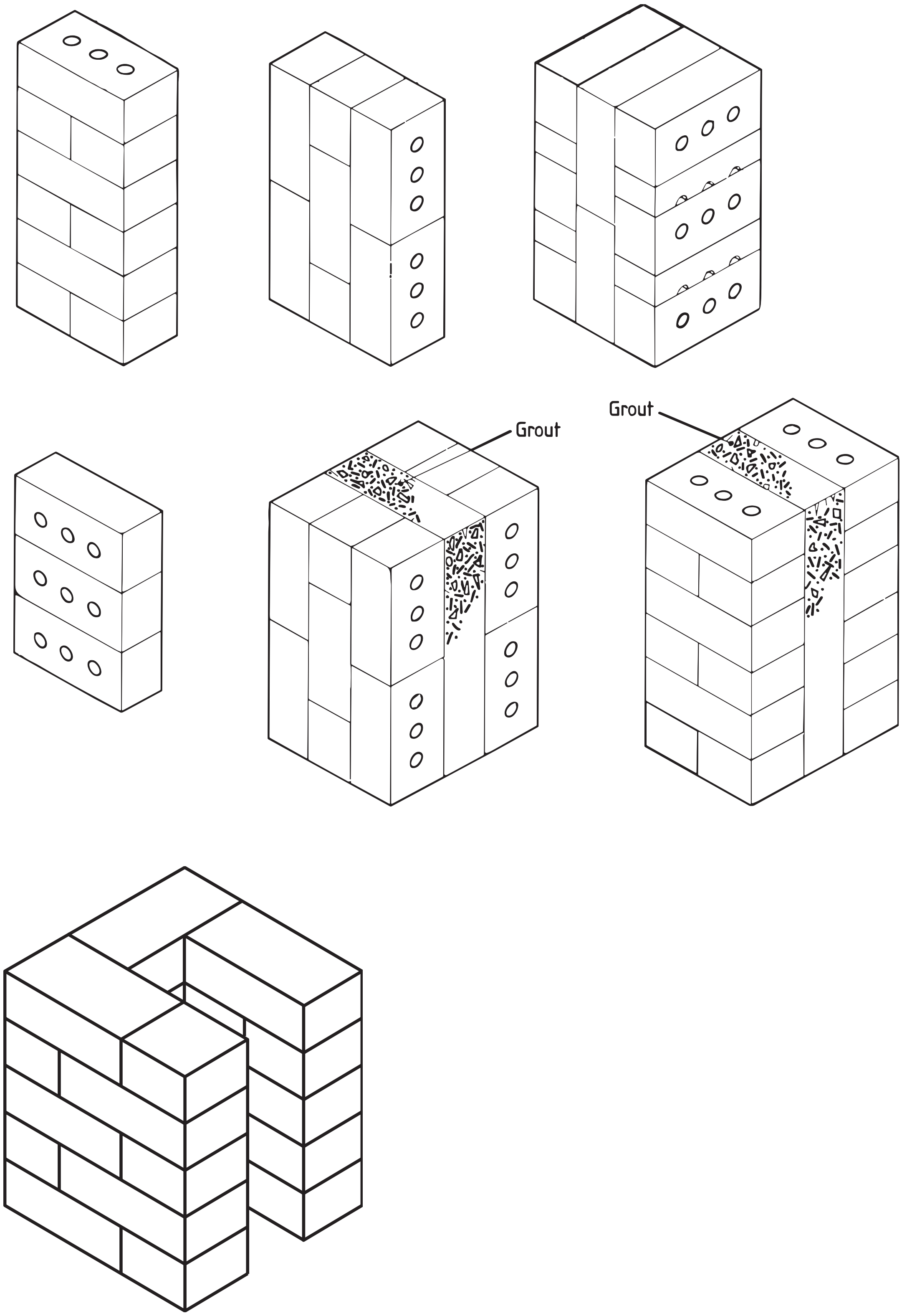


Figure D.1 — Typical prisms for determination of f_k

Annex E (informative)
Durability recommendations for various construction types

Table E.1 gives the recommendations for durability for various construction types.

Table E.1 — Durability recommendations for various construction types

Construction type	Exposure situation	Steel reinforcement type	Concrete or mortar infill cover to reinforcement	Concrete or mortar infill specification
Reinforced cavity or Quetta bond walls	E1, E2, E3, E4	As Table 14 appropriate to exposure situation	20 mm minimum (see Note 1)	1 : 0 to ¼ : 3 : 2 or Concrete C30 grade or equivalent or mortar as appropriate to reinforcement type
	E3, E4	Carbon steel	Concrete specified in accordance with Table 15	
Pocket type walls, reinforced hollow blockwork walls and other reinforced masonry construction	E1, E2, E3, E4	Carbon steel	Concrete specified in accordance with Table 15	
Pre-tensioned masonry	E1, E2, E3, E4	Carbon steel	Concrete specified in accordance with Table 14 (see Note 2)	
Post-tensioned masonry	E1, E2, E3, E4	—	Concrete C30 grade or equivalent (see Note 3)	

NOTE 1 Where austenitic stainless steel reinforcement is used, there is no recommendation for minimum infill cover except that needed to develop bond.

NOTE 2 The minimum concrete suitable is C35 grade or equivalent.

NOTE 3 This specification for concrete infill is nominal as the durability of reinforcement in post-tensioned masonry will usually be provided by direct protection of the reinforcement itself.

NOTE 4 Where concrete infill may be subjected to aggressive environments (e.g. sulfate attack), the recommendations given in BS 8110 regarding minimum grade specifications should be followed. Mortars should follow the recommendations of BS 5628-3.

Bibliography

Standards publications

BS 8102:1990, *Code of practice for protection of structures against water from the ground.*

BS EN 846-2:2000, *Methods of test for ancillary components for masonry. Determination of bond strength of prefabricated bed joint reinforcement in mortar joints.*

DD 86-3:1990, *Damp-proof courses — Part 3: Guide to characteristic strengths of damp-proof course material used in masonry.*

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