HANDBOOK TO BS 5628: PART 2

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A Viewpoint Publication

HANDBOOK TO BS 5628: PART 2

BS 5628:1985: British Standard Code of Practice for use of masonry
Part 2: Structural use of reinforced and prestressed masonry
by

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Figures 4.5, 7.1 and 7.2	Cement & Concrete Association

Preface

The design of masonry is governed by the fact that masonry (brick, block or stone) is strong in compression but weak in tension. Traditionally, masonry buildings have been designed so as to prevent tension developing in any part of the structure. This approach was used on a grand scale in the great cathedrals of the Middle Ages, with their arches, vaults, pinnacles and flying buttresses. However, there is another familiar design concept which has been used extensively for concrete throughout this century but, so far, rarely for masonry, namely the use of reinforcement to carry the tensile loads.

Reinforced masonry was first tried in 1825 and has since been used mainly in areas subjected to siesmic loads. The fact that its use has not been more widespread may be attributed to concern about durability, lack of design guidance and the need for higher standards of workmanship.

The new Code is intended to give designers the necessary guidelines. The design principles outlined are similar to the well tried methods in BS 8110 [CP 110] and a short section on prestressing has been included. The section on durability is based on the latest research by the Building Research Establishment. The level of workmanship is specifically defined and the main construction techniques are described.

This Handbook provides a great deal of useful background information which will assist designers firstly to use the Code and secondly to create buildings in reinforced masonry which will rival the more well known concrete structures.

C E Phillips
British Standards Institution
Secretary of the Code Committee

The development of reinforced and prestressed masonry

The reinforcement of masonry is not a new concept. In the 18th Century external iron straps were commonly used in stonework. It was not until 1825 that the first use of reinforced brickwork was recorded. Sir Marc Brunel used the technique in the construction of two caissons, one either side of the River Thames for the Wapping-Rotherhithe Tunnel[1]. The diameter of each caisson was 50 ft. and they were 42 ft. and 70 ft. deep respectively. The walls consisted of two leaves of 9 in. brickwork reinforced horizontally by iron hoops 9 in. wide and $\frac{1}{2}$ in. thick and vertically by 1 in. diameter wrought iron bars. Brunel was impressed by the structural performance of reinforced masonry and during the period 1836–1838 he carried out experiments on reinforced brickwork beams and cantilevers. The most important of these tests was the “Nine Elms” beam which had a clear span on 21 ft. 4 in.[2], which is shown in Figure 1. Tensile failure of the reinforcement occurred at a load of approximately 30 ton f. Further tests were carried out by Colonel Pasley in 1837[3]. It is interesting to note that this work predates the development of both Portland cement and reinforced concrete. There were few other significant uses of reinforced masonry in the 19th Century, with the exception of a 100 ft. diameter 35 ft. high reservoir built in Georgetown, USA, in 1853, which is shown in Figure 2. This was used until 1897 and was eventually demolished in 1932[4].

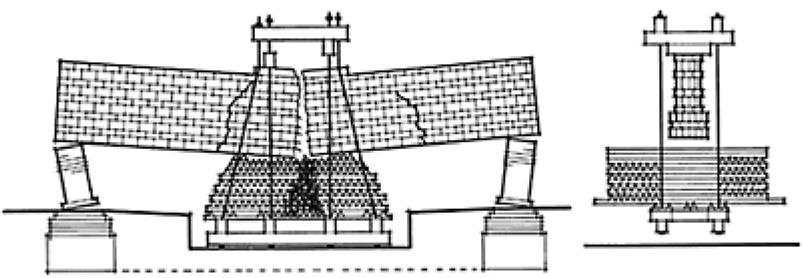


Figure 1 Nine Elms beam test, 1838

At the turn of the Century, a number of reinforced brickwork buildings were built by a French structural engineer, Paul Cottancin. Cottancin had patented a method for reinforcing concrete in 1889, which consisted of using mesh placed in thin (50 mm) slabs. These slabs were supported by a triangulated system of ribs or, as they were known “spinal stiffeners”. His ideas for reinforced concrete soon developed and he also began to reinforce brickwork walls and columns using the same principle as for his slabs and ribs. Buildings constructed in this way include

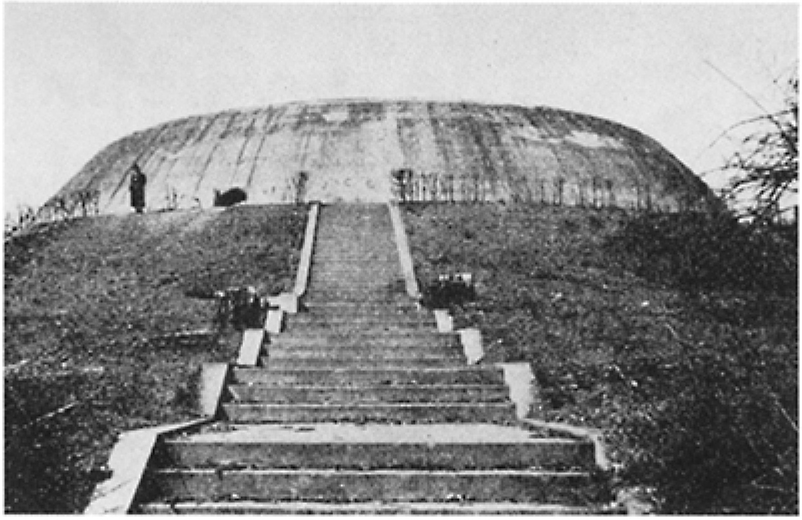


Figure 2 Reservoir built in Georgetown, USA, in 1853

the San Merino Pavillion for the 1900 Paris Exhibition, the Church of St Jean de Montmarre and a fashionable house in the Avenue Rapp, Paris. Figure 3 illusstrates a cross section through the Sidwell Street Methodist Church in Exeter. The walls are of cavity construction, the cavity being 530 mm wide; the bricks are 215 mm long×73 mm deep×75 mm thick, each containing four perforations. Vertical wires pass through each of the perforations and horizontal wires pass through each bed joint, the latter being interwoven with the verticals. The external walls are joined in places by cross ribs as indicated in Figure 4, and at these positions a larger steel flat was used as vertical reinforcement. The walls support a dome which consists of an inner dome of reinforced brickwork and an outer dome of 50 mm thick reinforced concrete. The dome supports a lantern tower and an ornate ventilator turret. The gallery consists of two 50 mm thick reinforced concrete slabs interconnected by ribs; this cantilevers some 4 m off the walls, the only other support coming from the staircases at either end. Without doubt, Cottancin was a pioneer and his buildings include numerous interesting features, some of which are illustrated in Figure 5.

In the 1920's a great deal of reinforced brickwork was built in Bihar and Orissa in India which was reported by Sir Alexandar Brebner[5]. Figure 6 shows a beam being subjected to a "live" load, whilst Figure 7 shows an attractive application. At Quetta reinforced brickwork was built in a special bond (Quetta bond), as shown in Figure 8, to increase resistance to seismic loads. This same technique was considered in the UK during the Second World War for the construction of air raid shelters[6].

More recent developments include the widespread use of reinforced hollow block masonry, particularly in seismic areas, as shown in Figure 9. Other typical applications for vertical reinforced masonry include increasing the resistance of walls to wind loading, and Figure 10 shows the reinforced Chevron walls of a museum.

The post-tensioning of structures (and particularly of masonry structures) has been available as a technique for a long time, for example, in the tying together of ageing buildings with iron rods, the force in which instance is generated by the cooling of the rods which were clamped whilst hot. It is only within the last 40 years, and particularly in the last 15 years in the UK, however, that much attention has been given to the technique. A great many floors in Europe have been constructed using prestressed, pretensioned ceramic or concrete units[7] with other units spanning between them as infill blocks. This type of construction is shown in Figure 11. In this case the units prestressed were not what would be described in the UK as bricks or blocks; the idea was seriously considered in the USA, where the infill blocks would more correctly be described as "tiles". In 1957 a USA patent[8] was issued for a method of constructing partition walls by using tiles prestressed

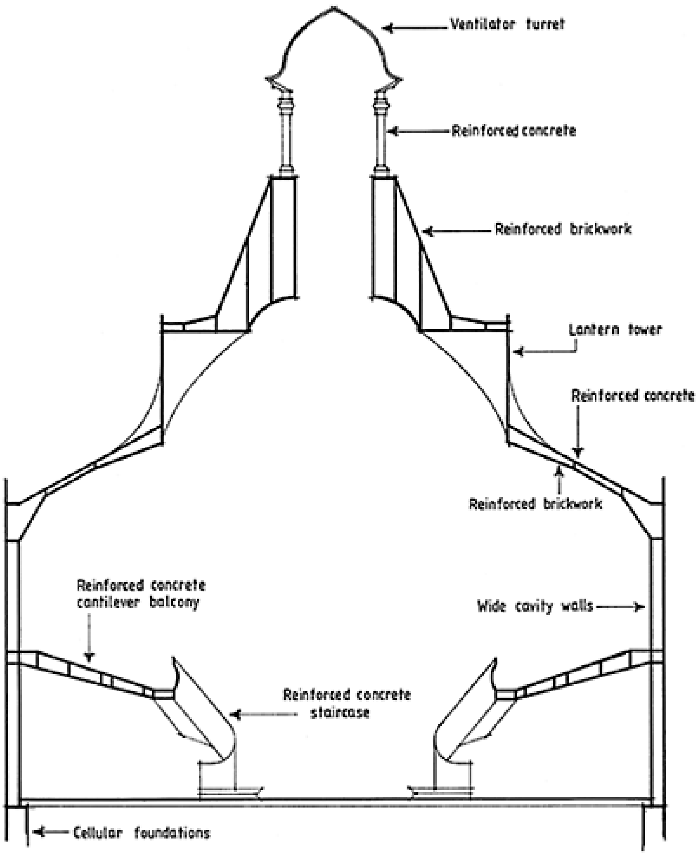


Figure 3 Sidwell Street Methodist Church, Exeter (overall height 25 m) completed 1906

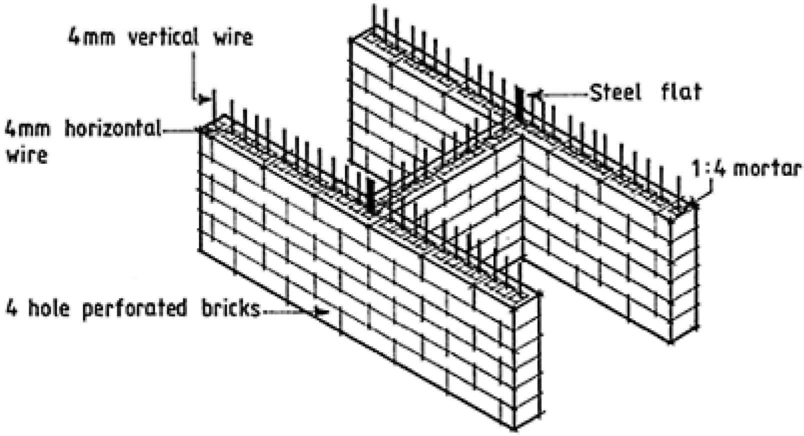
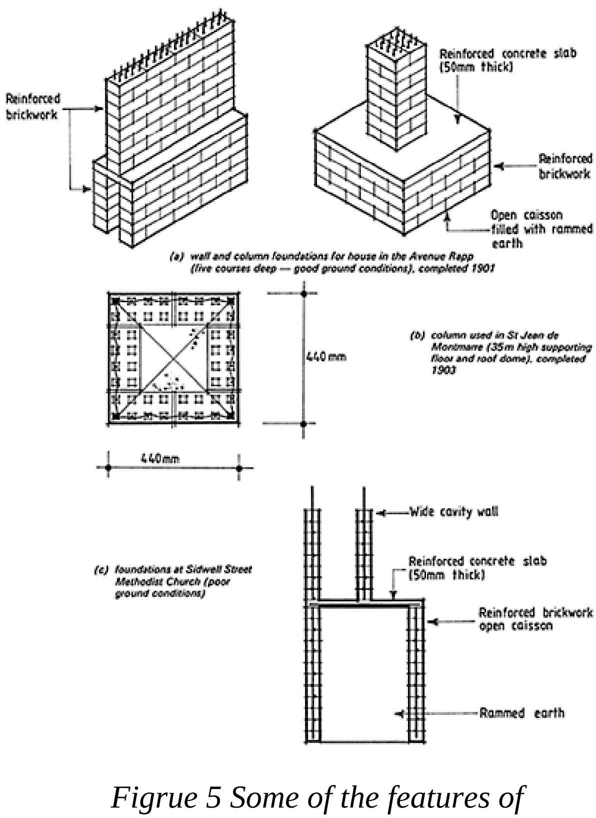


Figure 4 Detail of wall and cross rib of Sidwell Street Methodist Church



Figrue 5 Some of the features of Cottancins buildings:

together using external steel banding to prefabricate storey height units, as shown in Figure 12. These were subsequently built into the wall and plastered.

A great deal of attention has been given to the possibility of producing prestressed brickwork[9,10,11] and bonding arrangements have been devised which permit the introduction of both prestressing tendons and shear reinforcement, as shown in Figure 13. As yet, in spite of a lot of laboratory testing, however, there have been no practical applications of this type of element. The most common use of prestressing in building construction is the vertical post-tensionsing of walls to resist lateral loading from either wind, stored material or retained earth[12,13,14,15]. Figures 14 and 15 show examples of the use of steel rods which have been post-tensioned to increase the lateral load resistance of cavity and diaphragm walls.

Figure 16 shows the Triumfator Church in the Hague where slender brickwork columns have been post-tensioned from ringbeams at their top and bottom. Posttensioned diaphragm walls were also used by W G Curtin and Partners[16] for the

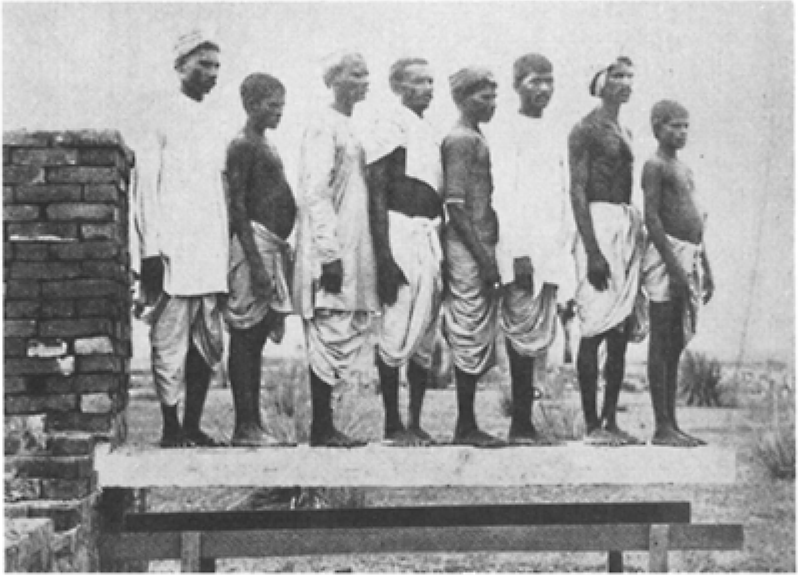


Figure 6 Reinforced brickwork cantilever demonstration, India, c. 1920s

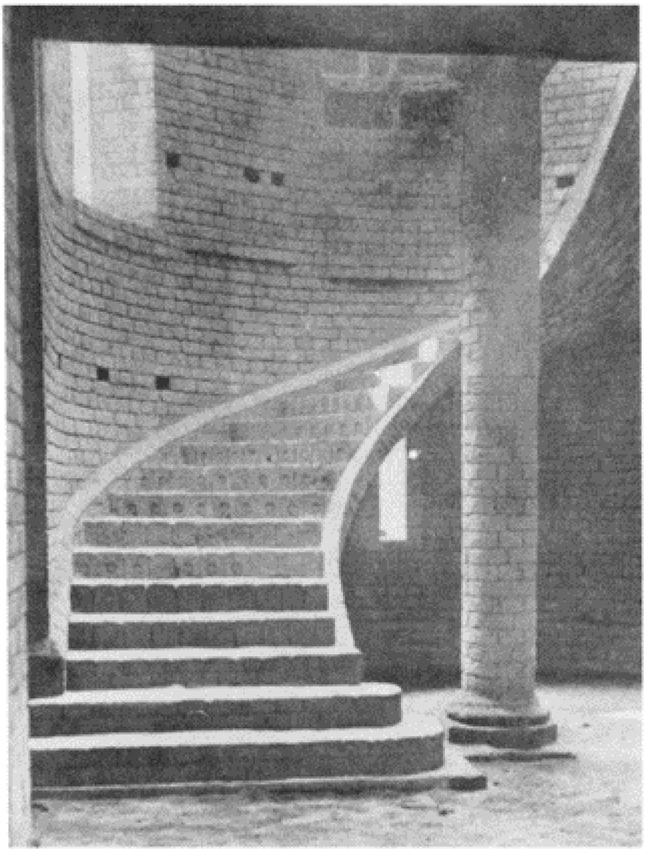


Figure 7 Reinforced brickwork staircase, India, c. 1920

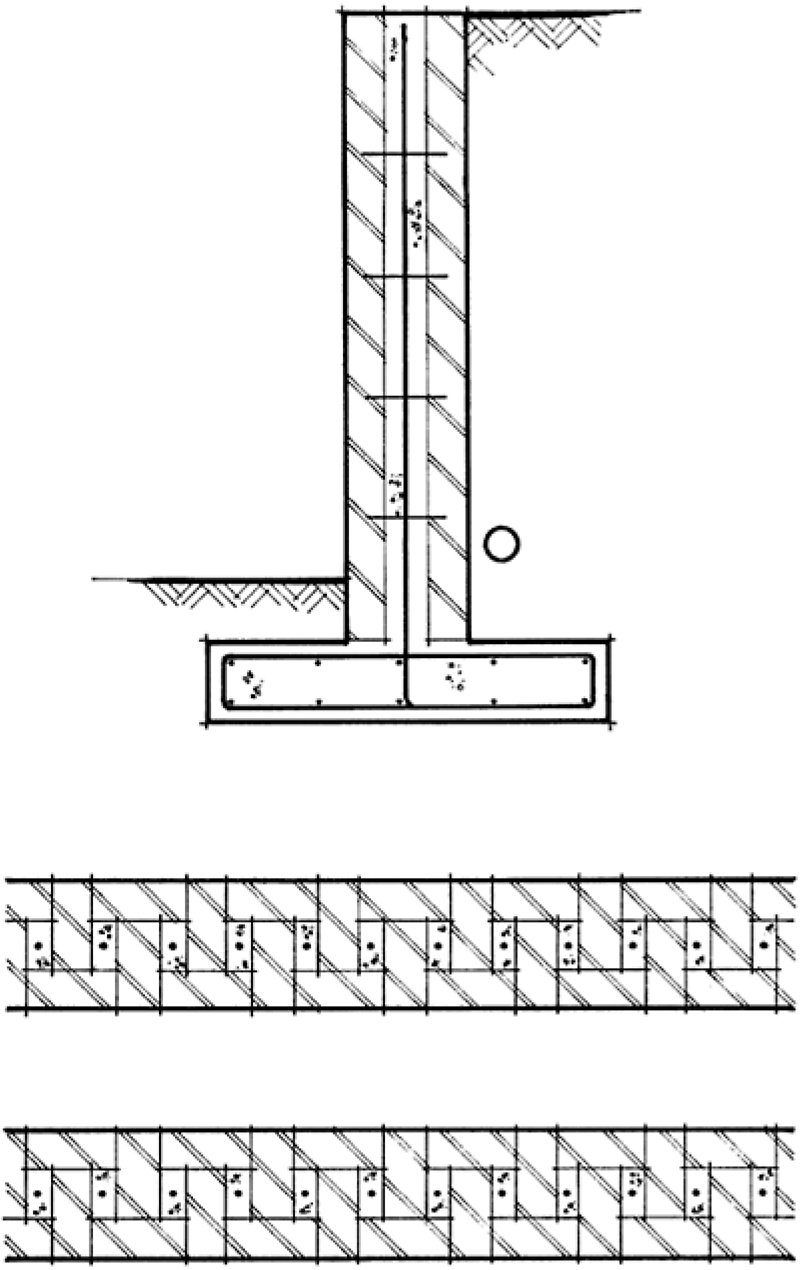


Figure 8 Quetta Bond retaining wall showing details of alternate courses in plan

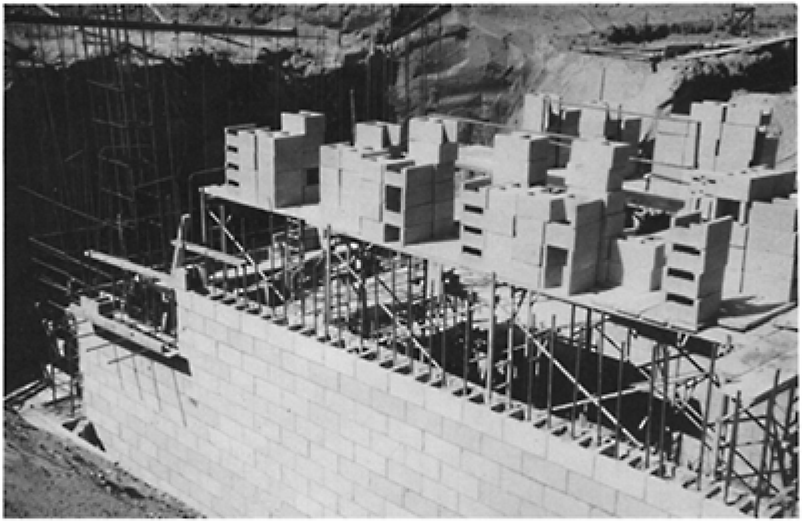


Figure 9 Reinforced hollow-block masonry in the USA

Figure 10 Reinforced chevron walls of Beaulieu Motor Museum

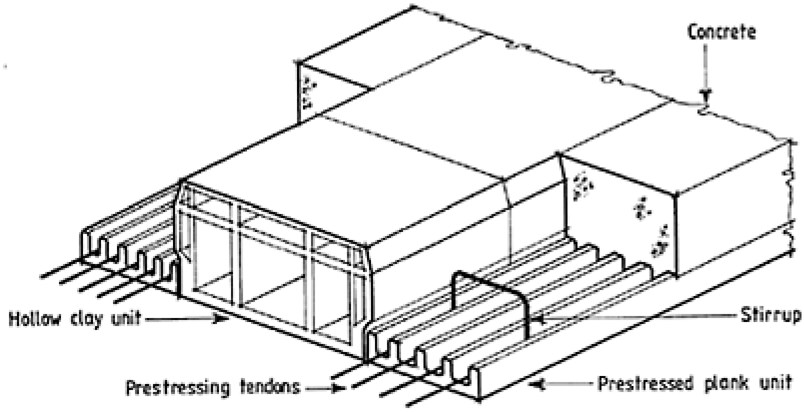


Figure 11 "Stahlton" floor (developed in Switzerland)

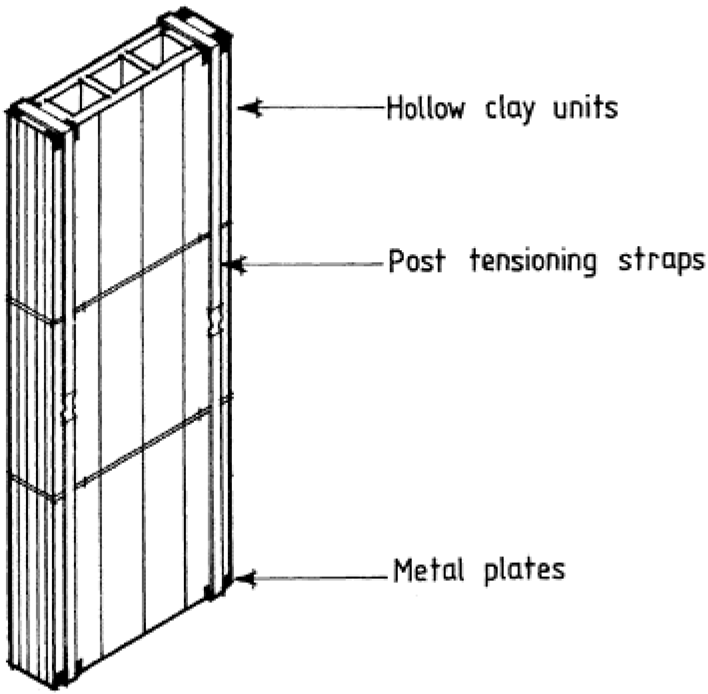


Figure 12 Prestressed hollow clay unit partition panel. Patented by Robert B.Taylor in 1957

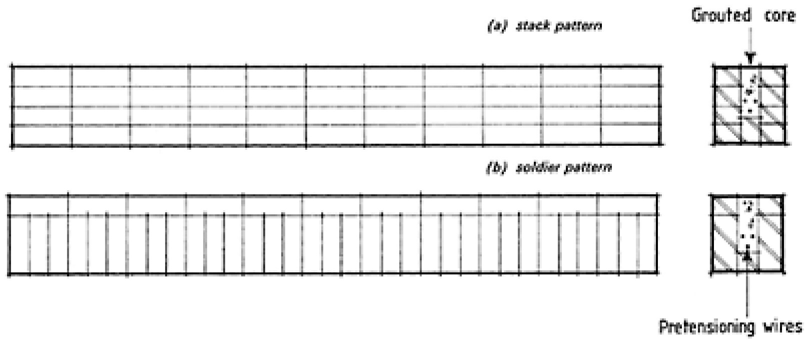


Figure 13 Alternative designs of pretensioned beams:

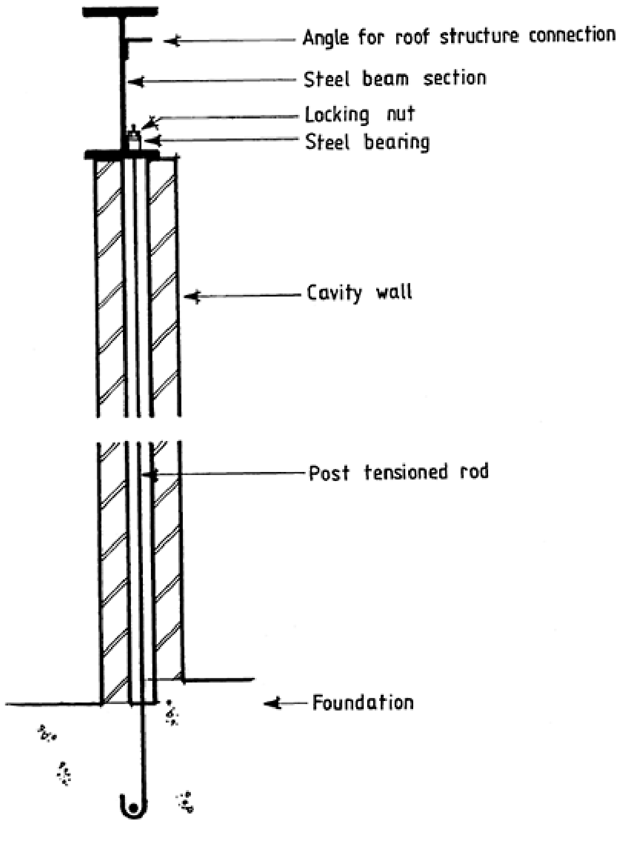


Figure 14 Post-tensioned factory wall[22] *(Darlington, UK)*

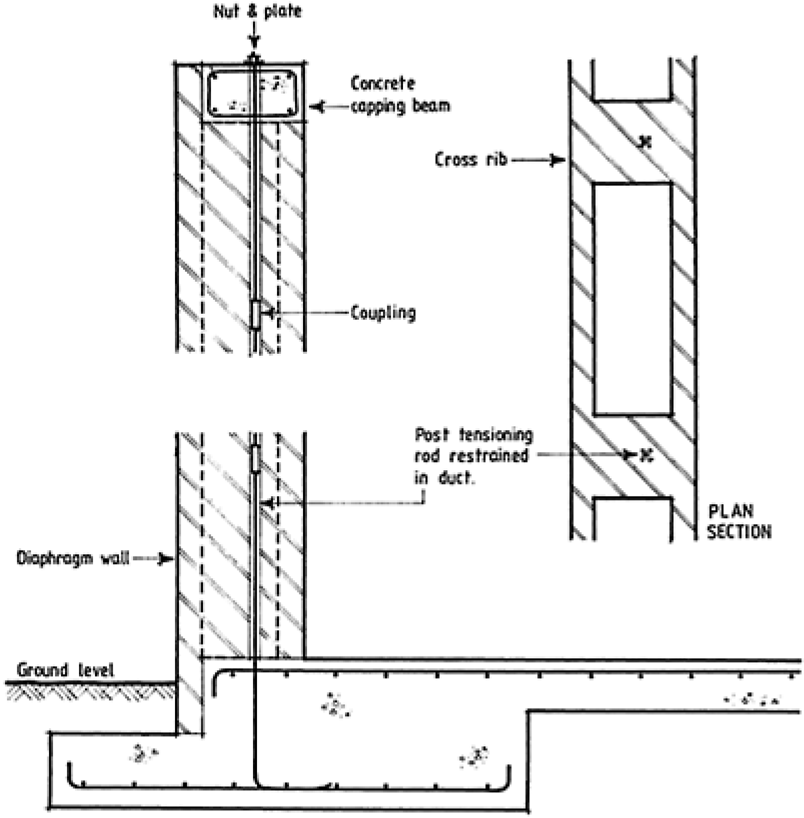


Figure 15 Post-tensioned diaphragm wall

(a) rods restrained in ducts in cross ribs

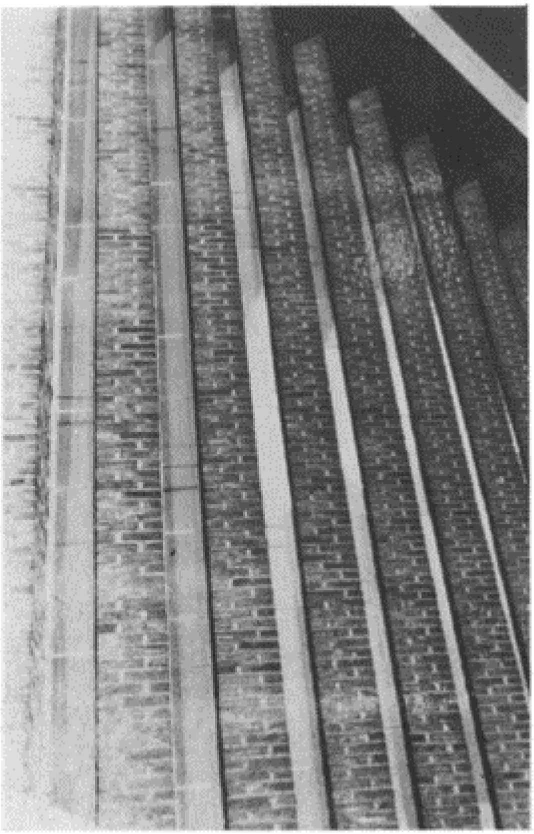


Figure 16 The Triumfator Church, The Hague

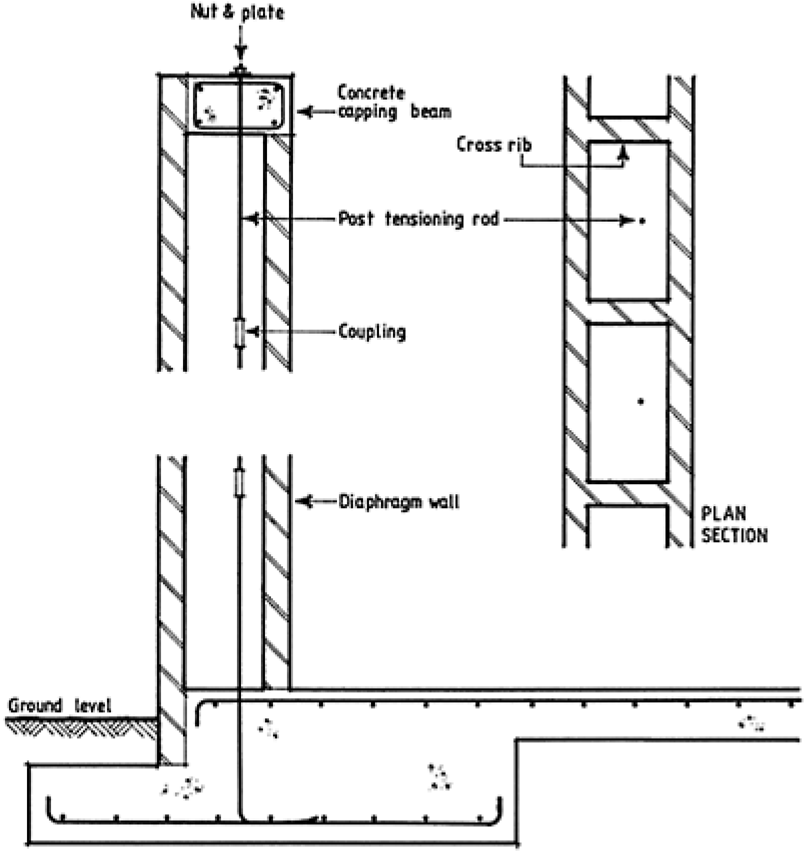


Figure 15 Post-tensioned diaphragm wall

(b) alternative arrangement, rods not restrained

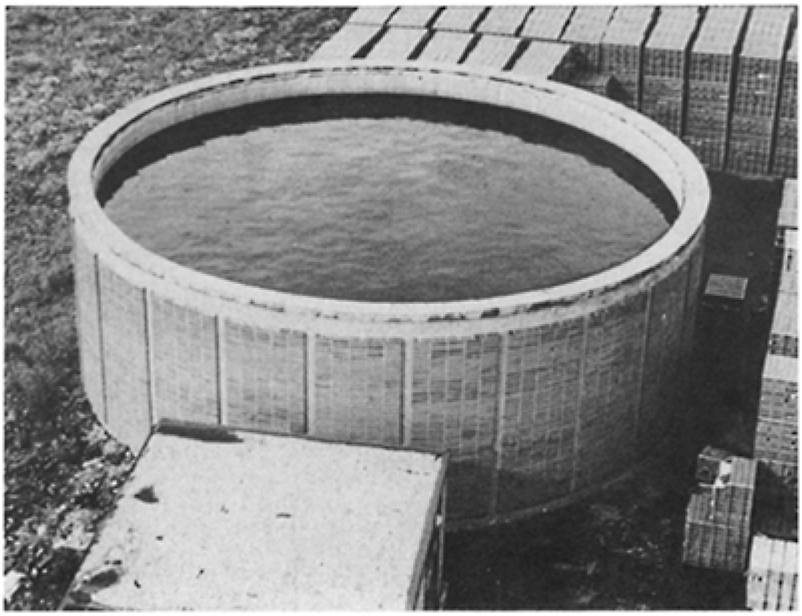


Figure 17 A prestressed brickwork water tank

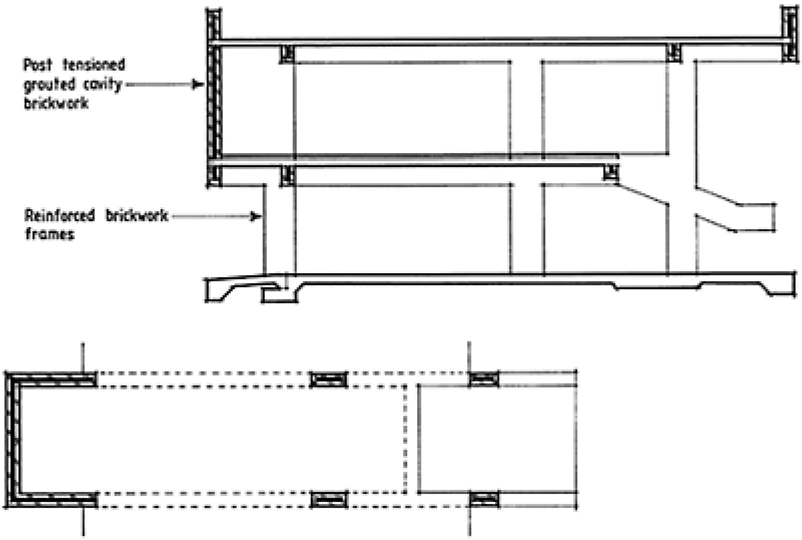


Figure 18 Part elevation and plan of the central area of the George Armitage office block, Wakefield

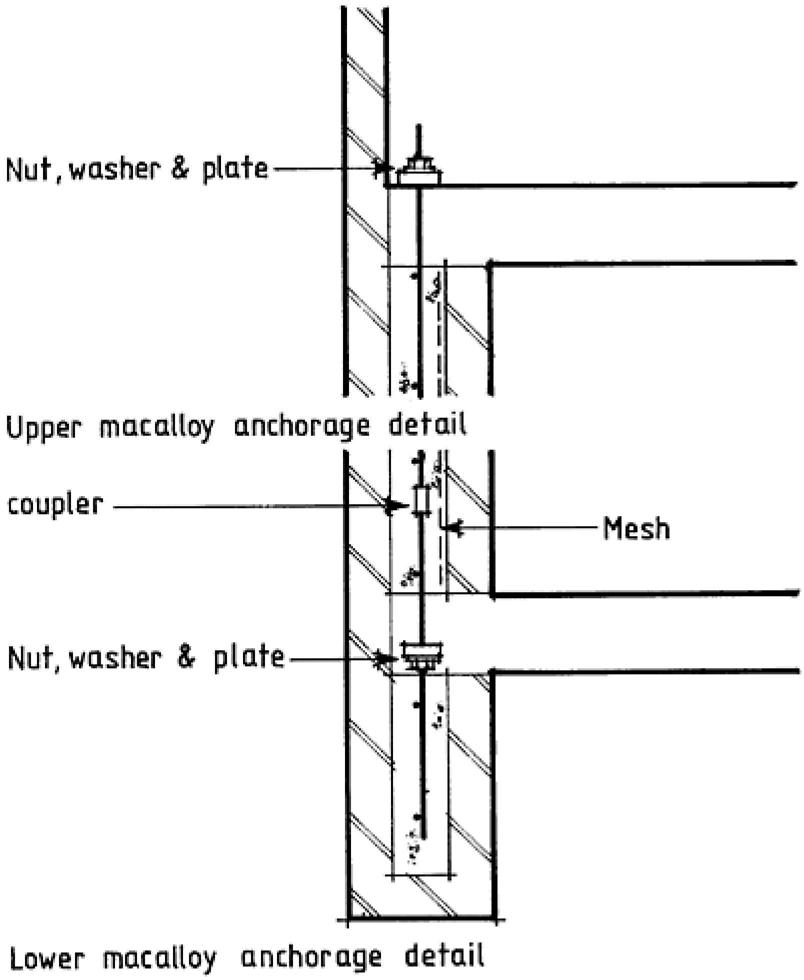


Figure 19 Post-tensioning rod detail in the storey height beams of the George Armitage office block

Oak Tree Lane Community Centre, Mansfield, to provide a building which would resist the massive settlement expected (1 m) due to mining activity. The building did, in fact, suffer some superficial damage due to this settlement which produced differential settlements of 125 mm. Reinforced brickwork has been used in a number of instances in water storage tanks. In one case post-tensioned brickwork was used to build a 540000 litre water storage tank[17], which is shown in Figure 17. Vertically prestressed walls which act compositely with connected floors have been laboratory tested[18] and also used in the George Armitage office block to build storey height box section cantilevers[19] and shown in Figures 18, 19 and 20. Clearly there is no reason why hollow blockwork should not be prestressed, however, there has been relatively little use of this form of construction except in New Zealand where seismic considerations are important and post-tensioned blockwork has been used[20], and in Ireland where silos have been constructed using post-tensioned external hoops[21] as shown in Figure 21.

Over its long history there have been a number of interesting and spectacular uses of reinforced and prestressed masonry[22]. There is currently a growing awareness of the potential of the medium and the existence of a modern Code of Practice will enable designers to use the techniques of both reinforced and prestressed masonry with confidence.

Figure 20 Head office block of George Armitage & Sons plc

Figure 21 20000 ton prestressed concrete rock phosphate silo (before steelwork and cladding fixed)

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Introduction to the Code

Preparation of the first design guidance for reinforced brickwork commenced in 1937. It was not, in fact, ready for issue until after the establishment of the Codes of Practice Committee for Civil Engineering, Public Works, Building and Constructional Work. Since the guidance had not been prepared in the form of a Code, it was issued in 1943 as an interim measure in the form of a British Standard, BS 1146[1]. The definition and scope from this document are produced below:

DEFINITION

1 Reinforced brickwork consists of loadbearing brickwork masonry in which adequate amounts of suitable reinforcement are so embedded and bonded that the two materials act together in resisting forces

SCOPE

2 This Specification defines the materials, factors governing design and the methods of assembly of reinforced brickwork

Steel is only considered in the Specification for the purpose of reinforcement

Interestingly, only one grade of mortar was permitted, a $1:\frac{1}{8}-\frac{1}{4}:3\frac{1}{2}$ (max) cement: flaked lime: fine aggregate. The material used to fill pockets or cavities containing steel consisted of the same mix with sufficient water added to make it pourable. Design was based on elastic methods with permissible stresses and modular ratios provided. A minimum coating of grout over a bar of $\frac{1}{8}$ in was required with the cover from the exterior face of the masonry ranging from 2 in. to 3 in.

Although the scope of BS 5628: Part 2 also considers only the use of steel as reinforcement, it is otherwise much wider than that of BS 1146. All types of bricks, blocks and square dressed natural stones covered by British Standards are included. Prestressed masonry, a relatively new development other than in situations where massive self-weight is used structurally (for example in a flying buttress), has also been included. Experience with the use of reinforced concrete and the application of research work has lead to a wholly different approach to the protection of the reinforcing steel against corrosion.

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Section One:
General

1. Scope

BS 5628: Part 2 was prepared to bring together UK design experience and practice of the use of reinforced and prestressed masonry. Where appropriate, overseas experience was introduced to supplement that available in the UK.

The document gives recommendations for the structural design of reinforced and prestressed masonry constructed of brick or block masonry, and masonry of square dressed natural stone. Far more experience was available in the use of reinforced masonry than in prestressed masonry and this is apparent in both the scope and content of these respective parts of the document. Included in the document, in Appendix A, is guidance on design methods for walls containing bed joint reinforcement to enhance their resistance to lateral load.

Since this Code is essentially structural in content, attention is drawn to the need to satisfy other than structural requirements (for example, requirements such as fire resistance, thermal insulation and acoustic performance) in the sizing of members and elements.

The Code also assumes that the design of reinforced and prestressed masonry is entrusted to "appropriately qualified and experienced people" and that "the execution of the work is carried out under the direction of appropriately qualified supervisors". This latter requirement is highlighted by the fact that BS 5628: Part 2, unlike Part 1[1], only recognises the special category of construction control.

2. Definitions

The definition of masonry permits units to either be laid in situ or as prefabricated panels. In both cases the units must be bonded and solidly put together with concrete and/or mortar so as to act compositely. The use of prefabricated panels is not new and has been established on a limited scale for a number of years. Very often, however, such panels are concrete elements to which masonry slips are bonded during the manufacturing process. The design procedures contained in BS 5628: Part 2 should enable efficient reinforced masonry panels to be produced.

There are a number of forms in which units of different types may be bonded together to leave clear channels or cavities which may be reinforced or prestressed. The Code defines the four types of construction most likely to be employed, but the many other possibilities are equally valid. The types defined are:

(a) grouted cavity
(b) pocket type

(c) Quetta bond
(d) reinforced hollow blockwork.

It is interesting to observe that the general definition of reinforced brickwork in BS 1146[2] has now been omitted in favour of a definition of reinforced masonry which includes all types of masonry unit put together in any form. The first three types of construction previously listed are, however, more commonly constructed of brickwork.

2.3.1. Grouted cavity masonry

Grouted cavity construction is probably the construction method with the widest application and may employ virtually any type of masonry unit. Essentially two parallel leaves of units are built with a cavity at least 50 mm wide between them. The two leaves must be fully tied together with wall ties. Reinforcing steel is placed in the cavity which is filled with high slump concrete. The word “grout” in this context is derived from United States practice. In the UK Code “infilling concrete” is the term corresponding to the USA term “grout”. The word grout is reserved for the material used to fill ducts in prestressed concrete and prestressed masonry. A typical grouted cavity construction is illustrated in Figure 1.1.

Earlier guidance on reinforced brickwork[3] did not include the concrete or mortar in the cavity as contributing to the compressive stength of the wall. The reason for this conservative approach was the fear that in the long term, differential movement would lead to a loss of composite action. The Code committee accepted that this approach was unnecessarily cautious but included a restriction on the effective thickness of a grouted cavity wall section. For cavities up to 100 mm the effective thickness may be taken as the total thickness of the two leaves plus the width of the cavity, but for greater cavity widths the effective thickness is the thickness of the two leaves plus 100 mm. Attention should be paid to Clause 32 which specifies the type of steel and cover necessary for a given condition of exposure. In some cases mortar may be used to fill the cavity rather than concrete and, because this reduces the protection offered to the reinforcing steel, steel which has some additional form of resistance to corrosion may need to be specified. Regardless of the type of infill, the minimum permitted cover of concrete or mortar to the steel is 20 mm, except where stainless steel is used.

2.3.2. Pocket type masonry

This type of construction is so named because the main reinforcement is concentrated in vertical pockets formed in the masonry[4]. This type of wall is primarily used to resist lateral forces in retaining or wind loading situations. It is the most efficient of the brickwork solutions if the load is from one side only and the wall section may be increased in thickness towards the base. An example is shown in Figure 1.2.

A particular advantage of the simplest and most common form of the pocket type wall is that the “pocket” may be closed by a piece of temporary formwork propped or nailed to the masonry. After the infilling concrete has gained sufficient strength, this formwork may be removed and the quality of the concrete and workmanship inspected directly.

Clause 32 specifies the cover, grade of concrete and the minimum cement content to ensure the durability of the steel in a pocket type wall; low carbon steel (mild or high yield) without any surface coating would normally be used.

2.3.3. Quetta bond masonry

The Quetta bond traces its origin to the early use of reinforced brickwork in the civil reconstruction of the town of Quetta in India following earthquake damage[5]. The section produced by this bond is at least one and a half units thick, as shown in Figure 1.3, and the vertical pocket formed may be reinforced with steel and filled with concrete or mortar. The face of the wall has the appearance of Flemish bond. There is also a modified form of Quetta bond in which the face of the wall has the appearance of Flemish garden wall bond and is illustrated in Figure 1.4. In thicker walls the steel may be placed nearer to the faces to resist lateral loading more efficiently.

When Quetta bond and grouted cavity construction are employed using similar materials they are treated similarly from the viewpoint of durability and in certain exposure conditions protected reinforcement may be necessary.

2.3.4. Reinforced hollow blockwork

In this form of construction the cores of hollow blocks are reinforced with steel and filled with in situ concrete[6]. The work size of the most common blocks is 440× 215×215 mm, although 390×190×190 mm blocks are also widely available. Although other sizes of blocks may be available, they are not nearly so common in the UK. In addition to the standard two core hollow blocks, specials such as lintel and bond beam blocks are available and are illustrated in Figure 1.5. For retaining walls up to about 2.5 m high, a single leaf of reinforced hollow blockwork is usually all that is required. It is, therefore, a very cost effective way of building small retaining walls.

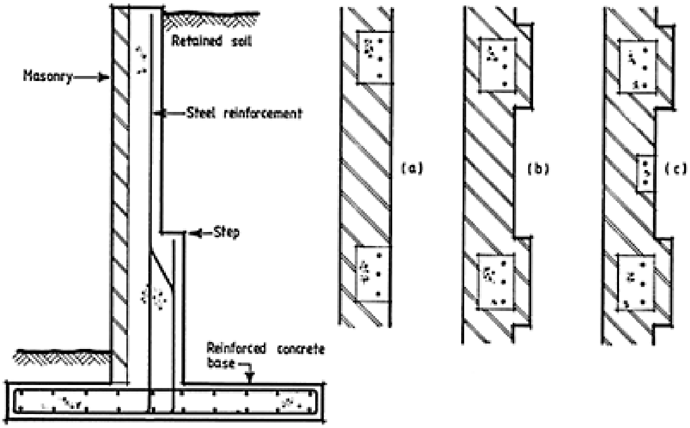

Figure 1.1 Typical grouted cavity construction

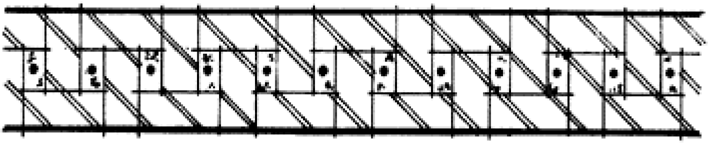

Figure 1.2 Pocket type example

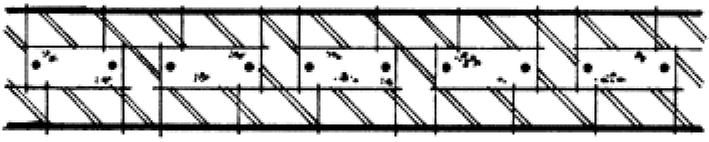

Figure 1.3 Section produced by Quetta Bond

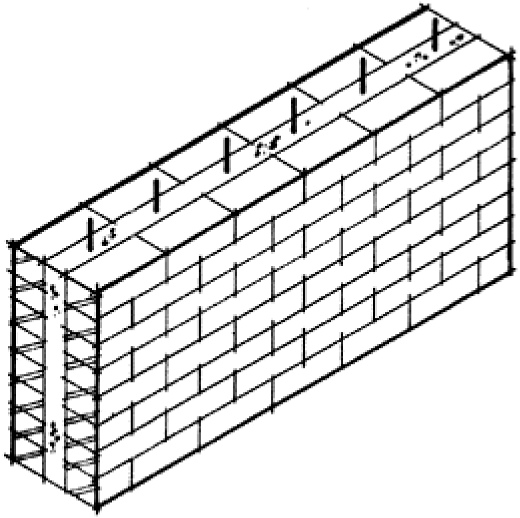

Figure 1.4 Flemish garden wall bond

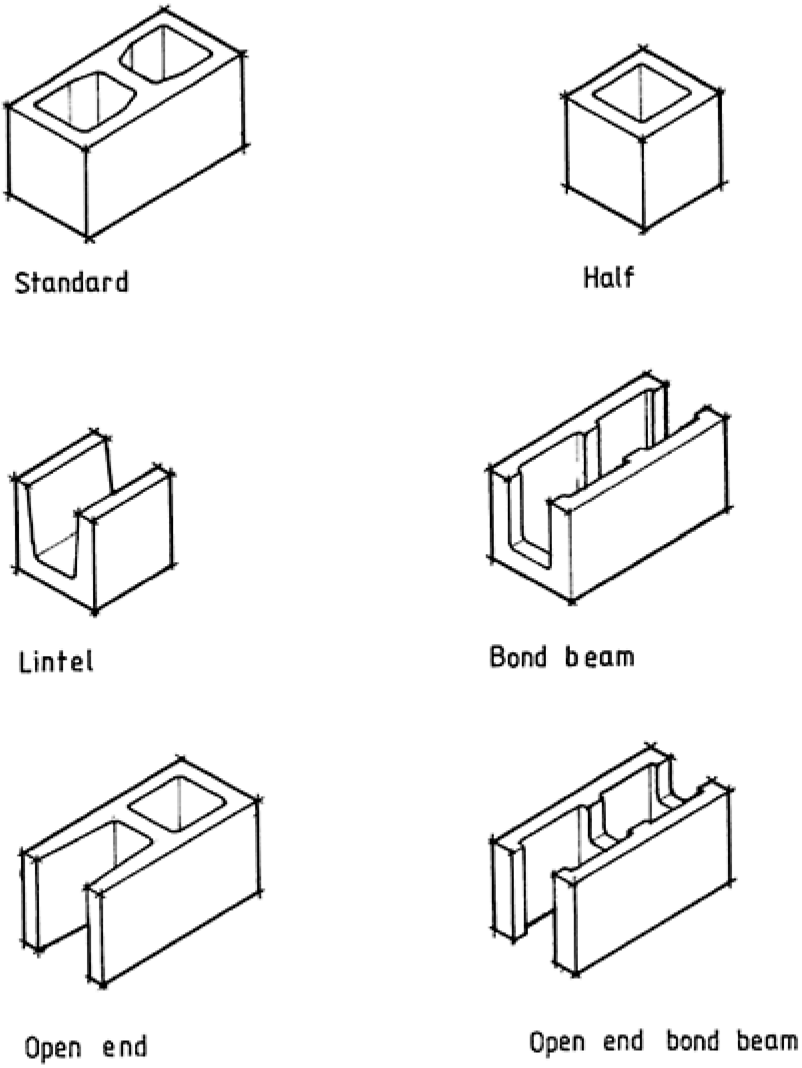

Figure 1.5 Types of hollow concrete block

3. Symbols

The following symbols are used in the Code:

A_m	cross sectional area of masonry
A_s	cross sectional area of primary reinforcing steel
A_{s_1}	the area of compression reinforcement in the most compressed face
A_{s_2}	the area of compression reinforcement in the least compressed face
A_{sv}	cross sectional area of reinforcing steel resisting shear force
a	shear span
a_d	deflection
a_v	distance from face of support to the nearest edge of a principal load
b	width of section
b_c	width of compression face midway between restraints
b_t	width of section at level of the tension reinforcement
c	lever arm factor
d	effective depth [see Clause 2.4]
d_c	depth of masonry in compression
d_1	the depth from the surface to the reinforcement in the more highly compressed face
d_2	depth of the centroid of the reinforcement from the least highly compressed face
E_c	modulus of elasticity of concrete
E_m	modulus of elasticity of masonry
E_{mi}	initial or short term modulus of elasticity
E_{m1}	long term modulus of elasticity taking account of creep and shrinkage
E_n	nominal earth or water load
E_s	modulus of elasticity of steel
e	base of Napierian logarithms [2.718]
e_x	resultant eccentricity in plane of bending
F_{bst}	tensile bursting force
F_c	compressive force
f_b	characteristic anchorage bond strength between mortar or

	concrete infill and steel
f_{ci}	strength of concrete at transfer
f_k	characteristic compressive strength of masonry
f_{kx}	characteristic flexural strength [tension] of masonry
f_{pb}	stress in tension at the design moment of resistance of the section
f_{pe}	effective prestress in tendon after all losses have occurred
f_{pu}	characteristic tensile strength of prestressed tendons
f_s	stress in the reinforcement
f_{s1}	stress in the reinforcement in the most compressed face
f_{s2}	stress in the reinforcement in the least compressed face
f_v	characteristic shear strength of masonry
f_y	characteristic tensile strength of reinforcing steel
G_k	characteristic dead load
g_b	design load per unit area due to loads acting at right angles to the bed joints
h	clear distance between lateral supports
h_{agg}	maximum size of aggregate
h_{ef}	effective height of wall or column
I	moment of inertia of the section
K_t	coefficient to allow for type of prestressing tendon
k	constant
L	length of the wall
l	effective span of the member
l_t	transmission length
M	bending moment due to design load
M_a	increase in moment due to slenderness
M_d	design moment of resistance
M_p	permanent load moment
M_t	total design bending moment
M_x	design moment about the x axis
$M_{x'}$	effective uniaxial design moment about the x axis
M_y	design moment about the y axis
$M_{y'}$	effective uniaxial design moment about the y axis
N	design axial load
N_d	design axial load resistance

N_{dz}	design axial load resistance of column, ignoring all bending
p	overall section dimension in direction perpendicular to the x axis
Q	moment of resistance factor
Q_k	characteristic imposed load
q	overall section dimension in a direction perpendicular to the y axis
r_{ip}	reciprocal of instantaneous curvature due to permanent load
r_{it}	reciprocal of instantaneous curvature
r_{1p}	reciprocal of long term curvature due to the permanent loads
r_{qn}	reciprocal of overall long term curvature
s_v	spacing of shear reinforcement along member
t	overall thickness of a wall or column
t_{ef}	effective thickness of a wall or column
t_f	thickness of a flange in a pocket type wall
V	shear force due to design loads
v	shear stress due to design loads
W_k	characteristic wind load
Z	section modulus
z	lever arm
α	coefficient
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 strength of steel
γ_{mt}	partial safety factor for strength of tie connections used to restrain the perimeter of a panel
γ_{mv}	partial safety factor for shear strength of masonry
θ	rotation
μ	coefficient of friction due to curvature in a prestressing duct
ϱ	$\dfrac{A_s}{bd}$

ϕ nominal diameter of tendon

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4. Alternative materials and methods of design and construction

This section requires no further detailed comment.

Section Two: Materials and components

5. General

The materials and components employed to produce reinforced masonry should generally comply with BS 5628: Part 3[1] or BS 5390[2]. If materials not covered by these documents are to be used, they should be carefully specified. Reinforced masonry may require the use of special units, unusual wall ties, and so on, which may not be commonly available and these will need to be carefully described in any specification (see *Section 10*).

6. Structural units

Units to be used for reinforced and prestressed masonry should comply with the appropriate British Standard. In the case of clay bricks and blocks this is BS 3921[3], whilst concrete masonry units are covered by BS 6073: Part 1[4]. Calcium silicate bricks should comply with BS 187[5]. It is also possible to reinforce cast stone and stone masonry, and these are covered by BS 1217[6] and BS 5390 respectively. If units have been used previously they should not be re-used in reinforced and prestressed masonry without thorough cleaning and inspection. A check should be made to ensure that re-used materials comply with current recommendations. In addition to complying with the relevant Standards, the units should meet the minimum strength requirements and follow the recommendations of BS 5628: Part 3 or BS 5390 (for stone masonry only) in respect of durability and such like.

Minimum strength requirement for masonry units
This part of the Code of Practice includes values for the characteristic strength of masonry units whose compressive strength is at least 7 N/mm^2. Ideally the elasticity of the masonry and infilling concrete should be matched, but in practice a wide variation in constituent properties does not appear to have caused significant problems. There are a number of reasons why properties are not directly comparable. For example, different characteristic strengths are necessary for bricks and blocks of a given unit strength because smaller and squatter units give a greater apparent strength when tested between the platens of a testing machine—this effect can be clearly demonstrated by comparing the characteristic compressive strength of masonry constructed from 20 N/mm^2 bricks with that constructed from 7 N/mm^2 blocks. Both mortar and infilling concrete are normally tested in the form of cubes, the effect of which is that the apparent mortar or concrete strength may be different to the in situ strength. A further factor which can affect the in situ strength of mortar and infilling concrete is the amount of water absorbed by the units. The unit may absorb a considerable proportion of the water from the mortar

or the concrete, thereby reducing the water/cement ratio and increasing the strength. Standard cubes made in metal moulds will have a higher water/cement ratio and indicate a lower strength. In practice the strength of the infill concrete may well be determined by the minimum cement content necessary for adequate protection of the reinforcement against corrosion.

There may be certain circumstances where the specification of a minimum strength for the units is not appropriate, for example in a relatively lightly loaded post-tensioned diaphragm wall. The Code does not preclude the use of lower strength units in these circumstances but the designer should consider this carefully. This relaxation is also particularly appropriate for situations where local reinforcement is provided within a building. It is possible to reinforce locally around openings, to provide an in situ lintel, to provide an alternative path for structural support or to improve lateral load resistance even when low strength units are employed. The use of a low strength unit will, however, mean that only a low characteristic masonry strength may be used even though the infilling concrete is significantly stronger. It may be appropriate, in exceptional circumstances, to consider the brick or block element as permanent non-loadbearing formwork and design the element as a reinforced concrete section based on the area of the infilling concrete, and using CP 110[7]. A final point which should be noted is that the block strength is normally measured and quoted on the gross area of the unit. In the case of hollow or cellular blocks it may be necessary to convert the gross strengths to nett strengths (see BS 6073[4]) to check compliance with any minimum strength requirement.

Durability of masonry units

Detailed information on the suitability of different types of unit for various conditions of exposure is provided in BS 5628: Part 3. Further information is given in *Section 32*.

7. Steel

7.1 Reinforcing steel

The steel to be used for the reinforcement of masonry will generally be bar, wire or fabric conforming to the requirements of BS 4449[8] or BS 4461[9], BS 4482[10] or BS 4483[11] respectively. However, in certain circumstances, for example for reasons of corrosion resistance, it will be necessary to use steel other than those covered by the above standards. Some guidance on the main alternatives is given below and data on sizes, weights, and so on, for bar and fabric are given in Tables 2.1, 2.2 and 2.3.

Table 2.1: Reinforcing bars

Bar size (mm)	6	8	10	12	16	20	25	32
Area (mm^2)	28.3	50.3	78.5	113	201	314	491	804
Weight (kg/m)	0.22	0.40	0.62	0.89	1.50	2.47	3.85	6.31
No. metres per tonne	4500	2530	1620	1130	633	406	259	158

Stainless steel

Three types of stainless reinforcing steel are available as a direct substitute for conventional ribbed high yield steel reinforcing bars.

Currently, a solid stainless steel reinforcing bar would cost six to seven times as much as high yield steel, depending upon the type of stainless steel and the bar size. One type consists of solid 18–8 stainless steel and another, Type 316 (18% chromium, 10% nickel, $2\frac{1}{2}\%$ molybdenum), stainless steel. Stainless steel cold twisted bar is also produced from 18–9 Type 302/304 austenitic stainless steel. Bars are

Table 2.2: Sectional area in mm^2 per m width

Bar spacing (mm)	Bar size (mm)							
	6	8	10	12	16	20	25	32
75	377	671	1047	1510	2680	4190	6550	10700
100	283	503	785	1130	2010	3140	4910	8040
125	226	402	628	905	1610	2510	3930	6430
150	189	335	523	754	1340	2096	3270	5360
175	162	287	449	646	1150	1800	2810	4600
200	142	252	392	566	1010	1570	2450	4020
225	126	224	338	503	894	1400	2180	3570
250	113	201	314	452	804	1260	1960	3220
300	94.3	168	262	377	670	1050	1640	2680
350	80.9	144	224	323	575	898	1400	2300
400	70.8	126	196	283	503	786	1230	2010
450	62.7	113	169	251	447	598	1090	1790
500	56.6	100	157	226	402	628	982	1610

available made from both hot rolled 18–8 and cold twisted 18–9 austenitic stainless steels. Even higher standards of corrosion resistance are achieved when bars made from warm worked Type 316 are used.

A relatively new development is a bar which consists of an outer skin of at least 1 mm thickness of 18–8 (18% chromium, 8% nickel) and a core of high yield steel. The bar has a similar profile to a ribbed high yield bar. The relative cost of this type of bar varies with size but currently 16 mm bars are some 12% cheaper than solid stainless steel bars.

Electrostatically epoxy resin coated reinforcing bar

A number of epoxy coatings have been developed in the USA as a method of affording additional protection to reinforcing steel. An attraction in this approach is the cost, which is only approximately 50% greater than that of conventional uncoated reinforcing steel in the USA. Several companies supply this material in North America and there is now a Fusion Bonded Coaters Association.

A number of factors should be considered when evaluating the possible use of coated reinforcing bars for reinforced masonry in the UK. For example, coated bars need to be carefully handled to avoid impact damage and it may not be possible to bend the bars to standard radii.

In the absence of a UK source of supply or of wide experience of use in this country, it was not possible to make specific recommendations for these products in the Code*. It is not intended, however, to preclude their use once further assessments have been made (Clause 4).

Types of bed joint reinforcement available in the UK

A number of types of bed joint reinforcement are available in the UK. In Figure 2.1, type 1 consists of two parallel longitudinal rods welded to a continuous zig-zag cross rod to form a lattice truss. The yield strength of the steel is 500 N/mm^2. This type of bed joint reinforcement is available galvanised with the addition of an epoxy polyester powder coat† (applied after fabrication) or in stainless steel. Table 2.4 gives an indication of the sizes available and the effective cross sectional areas of the bars. A much lighter form of bed joint reinforcement is made from 1.25 mm high tensile steel main wires and 0.71 mm mild steel bonding wires, illustrated as type 2 in Figure 2.1. This wire may be obtained galvanised or in stainless steel. The following table, Table 2.5, gives the sizes available together with the effective cross sectional areas. The minimum cross sectional area recommended in Appendix A means that this percentage of reinforcement is considered to be too low to give an enhancement in lateral load performance which can be relied upon for design purposes.

*Provided no chlorides are present, it would appear that galvanised steel is as good as fusion bonded coated steel.

†For both epoxy coated and galvanised steel it is the thickness of the coating which is important. Work in the USA suggests a coating thickness of approximately 0.2 mm is necessary for the epoxy systems.

Table 2.3: Fabric reinforcement

Standard metric sizes Specifications — Standard sheets Sizes and shipping dimensions

BS 4483 ref.	Mesh sizes nominal pitch of wires		Diameter of wire		Cross-sectional area per m width		Nominal mass per m^2 kg	Number of sheets per t.	Approximate dimensions of bundle for shipment				
	Main mm	Cross mm	Main mm	Cross mm	Main mm^2	Cross mm^2			Contents of bundle Sheets	Length cm	Width cm	Depth cm	Weight kg
A393	200	200	10	10	393	393	6.16	15	12	480	240	13.0	852
A252	200	200	8	8	252	252	3.95	22	15	480	240	13.0	683

A193	200	200	7	7	193	193	3.02	29	18	480	240	13.0	626
A142	200	200	6	6	142	142	2.22	40	21	480	240	13.0	537
A 98	200	200	5	5	98	98	1.54	57	25	480	240	13.0	444
B1131	100	200	12	8	1130	252	10.9	8	10	480	240	13.0	1256
B785	100	200	10	8	785	252	8.14	11	12	480	240	13.0	1125
B503	100	200	8	8	503	252	5.93	15	15	480	240	13.0	1025
B385	100	200	7	7	385	193	4.53	20	18	480	240	13.0	939
B283	100	200	6	7	283	193	3.73	24	21	480	240	13.0	902
B196	100	200	5	7	196	193	3.05	29	25	480	240	13.0	878
C785	100	400	10	6	785	70.8	6.72	13	12	480	240	13.0	929
C636	100	400	9	6	636	70.8	5.55	16	14	480	240	13.0	895
C503	100	400	8	5	503	49.0	4.34	21	16	480	240	13.0	800
C385	100	400	7	5	385	49.0	3.41	26	18	480	240	13.0	707
C283	100	400	6	5	283	49.0	2.61	34	21	480	240	13.0	631

Note: Fabric to BS 4483[11], f_y=485 N/mm^2

Table 2.4: Type 1 bed joint reinforcement

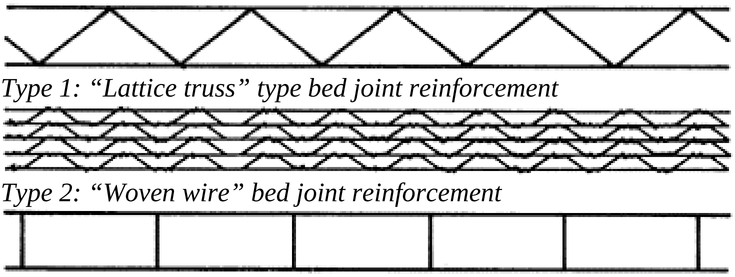
Total width of section (mm)	Diameter of each main parallel bar (mm)	Total effective csa of main bars (mm^2)
60	4	25
100	4	25
150	4	25
200	4.75	35
250	4.75	35
280	4.75	35

Table 2.5: Type 2 bed joint reinforcement

Total width of section (mm)	Number of main wires	Total effective csa of main bars (mm^2)
50	3	3.7
75	4	4.9
100	5	6.1
125	6	7.4
150	7	8.6
175	8	9.8

200	9	11.0
225	10	12.3
250	11	13.5
275	12	14.7
300	13	18.4

Figure 2.1



Type 1: "Lattice truss" type bed joint reinforcement

Type 2: "Woven wire" bed joint reinforcement

Type 3: "Ladder type" bed joint reinforcement

A third type of reinforcement consists of parallel drawn steel wires, 3.58 mm in diameter with orthogonal cross wires 2.5 mm in diameter as shown in Figure 2.1 (type 3). The following table, Table 2.6, gives details of this type of reinforcement:

Table 2.6: Type 3 bed joint reinforcement

Total width of section (mm)	Diameter of each main parallel bar (mm)	Total effective csa of main bars (mm^2)
40	3.58	20
60	3.58	20
100	3.58	20
160	3.58	20

7.2 Prestressing steel

A range of high strength tendons is available for prestressing masonry, including stainless steel tendons. These are typically from 6−39 mm in diameter with nominal tensile strengths of over 1000 N/mm^2 available and should comply with BS 4486[30] or BS 5896[31].

8. Damp-proof courses

Reference should be made to BS 5628: Part 3[1] to ensure that the damp-proof course is suitable. In reinforced masonry a damp-proof course may present a particular problem since in some applications it will not be possible to introduce a membrane which will not interfere with the structural behaviour of the wall. Even in more conventional applications, materials which might squeeze out in a highly loaded element should not be used. Care should also be taken to consider the effect of sliding at the damp-proof course as well as adhesion to the mortar when the masonry is acting in flexure[12].

The absence of a damp-proof course in applications such as retaining walls may result in appearance and durability problems with certain facing units, and manufacturer's advice should be sought. Materials such as engineering bricks can be employed as a dpc in some situations. In other applications, such as prestressed diaphragm walls, it will generally be possible to employ one of the more conventional dpcs[13,14].

It may be necessary to provide a vertical membrane between the cross rib and outer face of a diaphragm wall. In this instance it is usual to employ a liquid dpc, either painted directly onto the outer leaf masonry or on the perpend of the cross rib. Further guidance is given in *Section 37*.

9. Wall ties

When the low lift grouting technique is employed in conjunction with cavity construction, the vertical twist type of tie complying with BS 1243[15] may be used. The requirements regarding length of tie in this Standard are not applicable to reinforced masonry but the designer should ensure that adequate embedment is possible. It is recommended that in situations where the masonry is likely to be wetted for prolonged periods, such as retaining walls, stainless steel ties be employed.

Where the high lift grouting technique is to be used with cavity construction then a more substantial tie should be used to resist the pressure exerted by the infilling concrete during placing. A suitable tie is described in Appendix B to the Code and, again care should be taken to ensure adequate protection against corrosion. Other forms of tie may be used providing they give adequate restraint against the pressure exerted by the concrete.

Whatever type of tie is employed it is clearly necessary to avoid filling the cavity until the leaves have achieved sufficient strength and sufficient bond strength has developed between the mortar and the tie. A minimum of three days is recommended in normal ambient conditions.

Wall ties for prestressed diaphragm wall construction where the cross ribs are not bonded into the outer leaf of the masonry will usually need to be obtained from a specialist supplier. A tie of substantial cross section is required to provide adequate shear resistance.

The spacing of ties is covered in *Section 35*.

10. Cements

The types of cement which may be used with reinforced masonry are as follows:

1. Ordinary and rapid-hardening Portland cement (BS 12[16])
2. Portland blast-furnace cement (BS 146[17])

3. Sulphate-resisting Portland cement (BS 4027[18])

Neither masonry cement nor high alumina cement are permitted. BS 5628: Part 3 still permits the use of supersulphated cement to BS 4248[19] but this does not seem to have been used in conjunction with reinforced masonry in the UK and has, therefore, been excluded.

Lime

Limes which may be non-hydraulic (calcium), semi-hydraulic (calcium) and magnesium, should meet the requirements of BS 890[20].

11. Aggregates

The recommendations of BS 5628: Part 3 should be followed when considering the suitability of aggregates for mortar. Essentially this means that the fine aggregate should be free from deleterious substances and comply with BS 1200[21]. Marine sands should be washed to remove chlorides. Sands for mortar should be well graded. Single size sands or those with an excess of fines should be avoided if possible, but where their use is unavoidable, trial mixes should be assessed for suitability. Sands to grade M of BS 882 may well be found to be suitable.

Aggregates for infill concrete should meet the requirements of CP 110, which are generally that they comply with BS 882 and 1201[22], BS 877[23], BS 1047[24] or BS 3797[25].

Good mix design practice indicates in general that the largest possible maximum size of aggregate should be used in concrete. In the particular case of reinforced masonry, however, the need to produce a flowing concrete able to fill comparatively small sections without segregation will dictate the maximum size of aggregate which may be employed. In any case the maximum size of aggregate should not be greater than the cover to the steel less 5 mm. The making of trial mixes is recommended to produce the best concrete from the materials available.

Attention is drawn to the limits on chlorides discussed in *Section 15*.

12. Mortar

12.1 General

The recommendations given in BS 5628: Part 3 and BS 5390 should be followed for the mixing and use of mortars. The mix proportions and mean compressive strengths at 28 days are provided in Table 2 of BS 5628: Part 2, which is reproduced here as Table 2.7. The testing of mortars should be carried out in accordance with Appendix A1 of BS 5628: Part 1, which gives information on preliminary tests, the interpretation of test results and site tests. It should be noted that the compressive

Table 2.7: Recommendations for mortar

Mortar designation (see Note 1)	Type of mortar (proportions by volume) (see Note 2)		Mean compressive strength at 28 days	
	Cement: lime: sand	Cement: sand with plasticiser	Preliminary (laboratory) tests	Site test
(i)	$1:0-\frac{1}{4}:3$	—	16.0 N/mm^2	11.0 N/mm^2
(ii)	$1:\frac{1}{2}:4-4\frac{1}{2}$ (see Note 3)	1:3–4 (see Note 3)	6.5 N/mm^2	4.5 N/mm^2

Note 1: Designation (iii) mortar may be used in walls incorporating bed joint reinforcement to enhance lateral load resistance (see Appendix A)
Note 2: Proportioning by mass will give more accurate batching than by volume, provided that the bulk densities of the materials are checked on site
Note 3: In general, the lower proportion of sand applies to Grade G in BS 1200 whilst the higher proportion applies to Grade S in BS 1200[21]

strength values given in the Table are fairly low and many sands will yield higher strength mortars.

The batching of mortars should be carried out by weight or by the use of gauge boxes. It is not acceptable for reinforced masonry purposes to batch mortar using a shovel since this invariably results in less cement being added than the specification requires.

12.2 Readymixed mortars

Readymixed lime: sand for mortars is now widely established and should comply with BS 4721[26]. Care should be taken to ensure that the correct proportion of cement is added on site.

The Code indicates that readymixed retarded mortars should only be used with the written permission of the designer. However, their use is likely to spread because of the convenience factor. Readymixed retarded mortars are delivered to site and placed in small skips which may be mechanically handled near to the point of use. Typically these mortars have a working life of three days, but once the mortar is used in the wall it sets and gains strength in a similar manner to conventional mortars.

13. Concrete infill and grout

The minimum grade of concrete infill which may be employed in reinforced masonry is a prescribed or designed mix, as described in BS 5328[27], Grade 25. As an alternative to the Grade 25 prescribed mix, a mix of the following proportions by volume of the dry materials may be used:

$1:0-\frac{1}{4}:3:2$ cement: lime: sand: 10 mm maximum size aggregate

It is intended that these mixes should be used with slumps between 75 mm and 175 mm for mixes without plasticisers. The slump should be adjusted to suit the particular size, configuration and type of masonry to be filled.

It is considered important to use a wet mix to ensure that the units or cavities are completely filled and the concrete properly compacted, but clearly the masonry may absorb a considerable amount of water, thereby effectively reducing the water/cement ratio. One method of keeping the water/cement ratio low whilst still producing a flowing mix is to employ a plasticiser or superplasticiser. The mix has to be produced with a carefully controlled slump, typically of 60 mm, before the admixture is added to give a collapse slump. The concrete then needs to be placed within 20–30 minutes.

To improve the protection offered to the reinforcing steel by the concrete cover, a range of options for a particular exposure condition is given in Table 14 of the Code. In some situations a concrete of a Grade better than 25, up to a Grade 40, may be required and this is discussed more fully in *Section 32*.

It is important to realise the difference between a *prescribed* mix and a *designed* mix. The two prescribed mixes applicable to BS 5628: Part 2 are the C 25 P and the C 30 P, and to comply with BS 5328 these mixes must be weigh batched. The C stands for compressive strength, the number indicates the characteristic crushing strength in N/mm^2 which the concrete can be expected to achieve at an age of 28 days, and the P indicates a prescribed rather than designed mix. Compressive strength as such is not part of the specification and whilst the designated strength can be expected to be achieved with a high degree of confidence, strength testing must not be used to prove compliance. A certificate produced by the contractor or the readymix supplier stating the contents of the mix, or the checking on site of the materials batched at the mixer, are both means of checking that the concrete is of the specified quality. Methods are also available to analyse the concrete to determine the mix proportions.

Information on suitable proportions for prescribed mixes is provided in Table 2.8. This is based on the recommendations of BS 5328 in which more detailed information can be found.

Table 2.8: Weights of dry aggregate to be used with 100 kg of cement [extracted from BS 5328] and percentage by weight of fine aggregate to total aggregate

Grade of concrete	Nominal maximum size of aggregate [mm]	20		14		10	
	Workability	Medium	High	Medium	High	Medium	High
	Range of slump [mm]	25–75	65–135	5–55	50–100	0–45	15–65
	Total aggregate	kg	kg	kg	kg	kg	kg
C 25 P		510	460	490	410	450	370
C 30 P		460	400	410	360	380	320
	% by weight of fine						

C 25 P and C 30 P	aggregate						
	Coarse[22] [Zone 1]	40	45	45	50	50	55
	Medium [Zone 2]	35	40	40	45	45	50
	Medium [Zone 3]	30	35	35	40	40	45
	Fine [Zone 4]	25	30	30	35	35	40

Grade 35 and 40 concretes are designed mixes and strength testing should be carried out in accordance with BS 1881[28] to check compliance. It is, of course, equally valid to design the 25 or 30 Grade mix rather than use the equivalent prescribed mixes. On a large job it may well be more cost effective to design the mix using the locally available materials.

It will be necessary to specify the maximum size of aggregate in situations where the space to be filled is less than 100 mm×100 mm. As a rough guide the maximum size of aggregate should not exceed $\frac{2}{5}$ of the space to be filled. In any situation the maximum size of aggregate should not be greater than 5 mm smaller than the cover to the steel.

In the case of prestressed masonry a Grade 40 concrete is required and this will of necessity be a designed mix.

Information on plasticised concretes is provided in *Section 15*.

For the grouting of prestressing ducts, reference should be made to specialist literature.

14. Colouring agents for mortar

By choosing the mortar colour with care, a range of effects can be achieved to match or contrast with the units.

A very light coloured mortar may be produced by using a light sand together with white cement and lime. Even where a coloured mortar is required, white cement will be necessary for some of the lighter mortar colours. White cement is, however, more expensive than ordinary Portland cement.

Pigments can be used to produce a coloured mortar. The final colour will depend not only upon the pigment, but also the cement, lime, sand, and the water/cement ratios. The final colour may also be affected by the water absorbtion of the unit and whether the mortar has been re-tempered.

There is a very wide range of pigments available and these should comply with BS 1014[29] and should be used in accordance with the manufacturer’s instructions.

Under no circumstances should the amount of pigment used exceed 10% by weight of the cement in the mortar. In the case of carbon black, the total pigment content should be limited to 3% by weight of the cement.

15. Admixtures

15.1 General

The term admixtures is taken to include plasticisers for mortar and superplasticisers for infill concrete. The Code indicates that admixtures should only be used with the written permission of the designer. Clearly the manufacturer's requirements should be carefully followed, and if it is intended to use more than one admixture in a mix, then their compatability should be checked. It is also important to recognise that the effect of an admixture will vary with different types of cement.

Care should be taken to check that any admixture to be used with reinforced masonry does not affect the durability of the units, mortar or concrete, nor should it increase the risk of corrosion of the reinforcement.

To avoid potential corrosion problems the chloride ion content of admixtures should not exceed 2% by mass of the admixture or 0.03% by mass of the cement. In addition the requirements of Table 2 of the Code should be met to limit the total chloride ion content of the mix.

15.2 Chlorides

Limits are placed in Table 2 on both the percentage of chloride ion present in sands and in concrete and mortar mixes. The limits are based on the approach taken in the draft revision of CP 110 (BS 8110). The intention is to prevent sufficient chloride ion being present in reinforced masonry to lead to problems caused by the corrosion of the reinforcing steel.

Plasticisers for concrete

There are five types of admixture specified in BS 5075: Part 1:1974, namely:

1. accelerating
2. retarding
3. normal water-reducing
4. accelerating water-reducing
5. retarding water-reducing

Only those of particular relevance to reinforced masonry are considered below in detail:

Normal water-reducing admixtures

Water-reducing admixtures (plasticisers, workability aids) increase the fluidity of the cement paste and, for a given mix, will either increase the workability without increasing the water/cement ratio or will maintain the same workability with reduced water/cement ratio.

Most proprietary admixtures of this type are based on lignosulphonates or solutions of hydroxylated carboxylic acid salts. These work by improving the dispersion of the cement particles. For infilling concrete mixes for reinforced masonry, they offer the following potential benefits:

1. increasing the cohesion and reducing segregation of high workability mixes by lowering the water content whilst maintaining the same workability
2. reducing the water content and hence increasing the strength whilst maintaining the workability

The dosage is usually quite small (0.1–0.25 litres/50 kg cement) and trial mixes are recommended. Over-dosage can lead to retardation.

Superplasticisers
A flowing concrete may be produced by using a superplasticiser as a workability agent. Concrete produced in this way can be expected to have a slump of 200 mm or greater and should not exhibit excessive bleeding or segregation. Slumps in excess of 175 mm are generally considered as *collapse* slump. Superplasticisers may be based on one of the following chemicals:

1. sulphonated melamine formaldehyde condensates
2. sulphonated napthalene formaldehyde condensates
3. modified lignosulphonates
4. polyhydroxylated polymers
5. mixtures of acid amides and polysaccharides

They differ from other commonly used admixtures such as those based on lignosulphonates or carboxylic acid in greatly increasing the workability which may be achieved—the penalty being the greater cost.

Mix design of superplasticised concrete
The basic approach to use a superplasticiser is to design a concrete to have an initial slump of 60–75 mm, which is then dosed with between 1–6 litres per cubic metre (depending on type) of superplasticiser, thereby increasing the slump to collapse.

The extent to which a fluid concrete is produced will depend upon the aggregate type, shape and overall grading.

The first stage in the design is to use conventional mix design procedures to determine the water/cement ratio and mix proportions needed to give the specified strength with a slump of 75 mm. The proportions of cement, sand and aggregate now need to be checked and adjusted to avoid segregation. There are two methods of doing this:

1. add 4–5% extra sand
2. provide a combined fines content as shown in the following table:

Table 2.9

Maximum aggregate size (mm)	Minimum proportion of combined fines* (kg/m^3)
38	400
20	450

Although most information is available for the use of superplasticisers with OPC cements, rapid-hardening and sulphate-resisting cements may also be used. It would be prudent, however, to check both the time-dependent bulk fluidity and the ultimate strength.

For a cement content of 270 kg/m^3 or more, 24–35% of 0–1.18 mm sand (as a percentage of the total aggregate) should be used. If the cement content is less than 270 kg/m^3, the percentage of sand passing the 1.18 mm sieve must be increased above 35%.

Using a superplasticiser

The superplasticiser needs to be added to the concrete at the point of use and the concrete mixed for a further 2–5 minutes. The concrete should be used immediately since maximum workability is retained for only 30–60 minutes. The period during which high workability will be retained is, to some extent, dependent upon the type of mixer and the rate of mixing. The faster the mixing action, the quicker the fall off in high workability.

[*]Cement and sand having a particle size of less than 300 μm

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Section Three: Design objectives and general recommendations

16. Basis of design

16.1 Limit state design

CP 110:1972[1] states that the purpose of design is to ensure that all the criteria relevant to safety and serviceability are considered in the design process, these criteria being associated with limit states. This was the first UK Code to adopt limit state design, a philosophy which was applied to BS 5628: Part 1[2] which was published in 1978.

The adoption of limit state design was only possible against a background of a better understanding of performance requirements. Essentially the design process is one of balancing all the factors involved. For example, a wall could be strong enough to withstand a high wind load, but not without deflecting excessively and incurring unacceptable cracking in applied finishes. Conversely, the wall could be designed to minimise deflection but not possess adequate lateral strength to provide an acceptable factor of safety against collapse.

There is insufficient data available to be able to confidently calculate every limit state for every potential reinforced masonry element. From a designer's point of view it is often convenient to be able to use simple sizing rules to ensure that the limit states of deflection and cracking will not be reached and then to carry out a detailed structural analysis of the element for the ultimate limit state. This approach has been adopted in BS 5628: Part 2, although it is possible to exceed the sizing requirements provided that checks are made to ensure that deflection and cracking are not likely to be excessive. In an ideal situation, the probability of reaching a particular limit state should be determined from a full statistical analysis of the behaviour of masonry appropriate to that limit state. In the absence of such comprehensive information, however, BS 5628: Part 2 employs a partial safety factor approach, using characteristic values of strengths.

The characteristic strength of masonry, for example, is defined as the value of the strength of masonry below which the probability of test results falling is not more than 5%. This characteristic strength value is modified by a partial safety factor to give the value (e.g., strength), to be used in design—the design value.

$$\text{i.e. design strength} = \frac{\text{characteristic strength of masonry}}{\text{partial safety factor}}$$

There are two types of partial safety factor employed in BS 5628: γ_f, which is applied to loads, and γ_m, which is applied to materials.

The partial safety factor for loads (γ_f) is intended to take account of:

1. possible unusual increases in load beyond those considered in deriving the characteristic load
2. inaccurate assessment of effects of loading and unforeseen stress redistribution within the structure
3. variations in dimensional accuracy achieved in construction

The partial safety factor for materials (γ_m) takes account of:

1. differences between site and laboratory constructed masonry
2. variations in the quality of materials in the structure

It should be noted that BS 5628: Part 2 allows the designer to design in accordance with CP 110 if the cross section of the infill concrete is substantial. This would involve disregarding the effect of the masonry units, considering them solely as permanent formwork making no contribution to the strength of the element. If the designer chooses to exercise this option, he should ensure that the mix design, method of placing and detailing are also in accordance with CP 110.

16.2 Limit states

16.2.1 Ultimate limit state

BS 5628: Part 2 indicates that "The strength of the structure should be sufficient to withstand the design loads taking due account of the possibility of overturning or buckling". It is thus necessary to show that the strength of the structure is such that there is an acceptable probability that it will not collapse under the load described above. The calculations must take account not only of primary and secondary effects in members, but also in the structure as a whole.

16.2.2 Serviceability limit state

16.2.2.1 The deflection of a reinforced masonry element may affect not just the element itself in terms of appearance and durability, but also lead to the cracking or loss of bond of any applied finishes. The cracking of a render, for example, might lead to an excess of water entering into a wall and, in the case of some types of clay brickwork, could lead to problems of sulphate attack if the bricks have a high sulphate content.

The Code makes three recommendations to ensure that, within the limitations of the calculation procedures, deflections are not excessive. These may be summarised as:

1. final deflection not to exceed $\frac{\text{length}}{125}$ for cantilevers or $\frac{\text{span}}{250}$ for all other elements
2. limiting deflection of $\frac{\text{span}}{500}$ or 20 mm, whichever is the lesser, after partitions and finishes are completed

3. total upward deflection of prestressed elements not to exceed $\frac{\text{span}}{300}$ if finishes are to be applied, unless uniformity of camber between adjacent units can be achieved

16.2.2.2 Little guidance is given in the Code on the subject of cracking. Fine cracking is to be expected in reinforced masonry but the crack width should be limited to avoid possible durability problems. The Code also recommends that the effects of temperature, creep, shrinkage and moisture movement be considered and allowed for with appropriate movement joints.

Although the Code does not give any further guidance, the authors have tried to provide indications of crack widths where this is available. The maximum crack width which the authors consider likely to occur in reinforced masonry designed to the Code is 0.3 mm.

17. Stability

17.1 General recommendations

The purpose of this note in the Code is to make clear the need for one person to be responsible for the overall design of the structure. This designer has to coordinate the work of other members of the design team to ensure that the stability of the structure is adequate and that the design and detailing of individual elements and components does not impair this stability.

Clearly the layout and interaction of the elements will significantly affect the stability and robustness of the overall design. As a matter of course the inclusion of a significant proportion of reinforced masonry within the structure will tend to improve the overall tieing together of the structure if adequate connections are provided.

It is necessary, as is the case with unreinforced masonry, that the building be designed to resist at any level a uniformly distributed horizontal load equal to 1.5% of the characteristic dead load above that level. In addition, robust connections need to be provided between elements of the construction as detailed in Appendix C of Part 1. Finally, of course, compatibility between elements of different materials should be considered when making connections between them.

An example of the latter situation can occur in prestressed diaphragm wall construction. The outer leaf of the wall may well be constructed of clay brickwork with the inner leaf and cross webs of concrete or calcium silicate masonry. Not only do the general expansion of clay brickwork and the shrinkage of the concrete or calcium silicate masonry need to be considered, but also the creep movements and the way in which the prestressing loads are distributed. In the case of a reinforced concrete building the minimum effective vertical tie requirements in columns and walls for buildings of five or more storeys are specified in CP 110 together with minimum reinforcement requirements. Although few reinforced masonry buildings have been constructed wholly of reinforced masonry in the UK, there is no doubt that a similar approach could be adopted. The Code recommends that buildings of five storeys and above should follow the additional requirements of Clause 37 of BS 5628: Part 1. In general the consequences of collapse are more significant for taller buildings and it is usual to increase the horizontal tieing required in relation to the number of storeys in the building. For a reinforced masonry building it may well be found that the steel incorporated to resist the usual load cases is sufficient to ensure adequate tieing together of the elements.

17.2 Earth-retaining and foundation structures

There will, in these situations, be a number of factors to be taken into consideration to ensure the overall stability of structures. For example, although the stem of a retaining wall may be designed according to this Code, there are other considerations to ensure adequate resistance against sliding, overturning and so on. These are essentially geotechnical considerations although, for example, the location, thickness and weight of the wall may be of relevance.

The partial safety factor, γ_f, to be applied to earth and water loads is as for other types of load whether the load is beneficial, e.g., passive pressure on a retaining wall, or not. The designer should only consider revising the value of γ_f if the loads, due to their method of derivation, have already been factored.

17.3 Accidental forces

This clause of the Code requires the design to consider the consequences of misuse or accident. It is not expected that the building should be capable of resisting the forces which would result in an extreme case. It is expected, however, that damage resulting from any particular accident should not be disproportionate to the cause of the accident.

The general recommendations of Clause 20.3 of Part 1 are applicable to all building types. In passing it is worth considering the fact that the adoption of a (uniformly distributed) lateral load expressed as a percentage of the total characteristic dead load is a common requirement in seismic regions. In a zone where a significant earthquake risk exists, this percentage should be greater than 1.5%, but the latter should be adequate in the UK where there is only the risk of a relatively minor tremor.

In the case of buildings of five storeys and above (Category 2 in Part 1), it is recommended that either:

1. an assessment is made of the resultant stability and extent of damage following the removal of a loadbearing element *or*
2. sufficient horizontal and/or vertical tieing is provided within the structure

The first approach involves a detailed examination of the structure to calculate the effect of the loss within each “compartment” of a loadbearing element unless they are designed as protected members. The latter involves (depending on whether option [2] or [3] is taken from Table 12 of Part 1) either analysis for vertical elements only or no further assessment because of the extent of tieing. A more general appreciation of the background of the requirements contained in Part 1 can be found in the handbooks by Haseltine and Moore[3] and Roberts, Tovey, *et al*.[4]

17.4 During construction

This note is intended as a warning to ensure that consideration is given to the need for temporary support during the construction phase. For example, reinforced masonry beams can be readily built in situ off of a horizontal shutter which will need to be propped until the masonry has developed sufficient strength to allow the shutter to be removed.

18. Loads

In principle, limit state design requires that the characteristic load on any structure is statistically determined. Regrettably, insufficient data is available as yet to express loads in this way. It is assumed that the characteristic dead, imposed and wind loads may be taken from BS 6399[5]*. Nominal earth load (E_n) may be obtained in accordance with current practice, for example, as described in CP 2004[6].

19. Structural properties and analysis

19.1 Structural properties

19.1.1 Characteristic compressive strength of masonry, f_k

19.1.1.1 General

The purpose of this warning in the Code is to draw attention to the fact that in a reinforced or prestressed element, the units may be loaded in a direction other than that which would normally occur in unreinforced masonry.

The compressive strength of masonry units is determined by applying loads through the platens of a testing machine normal to the bed faces of the unit. The strength so obtained is unique to that direction of loading. Even allowing for the adjustment necessary for the effect of changing the aspect ratio when the unit is tested in a different direction (for example, load normal to the header faces), the strength of the unit is still likely to be different, depending upon the type of unit.

*which replaces CP 3: Chapter V

In the case of solid aggregate blocks, variations in strength with unit orientation will be introduced by the method of manufacture, although these will generally be small. In many cases, vertical compaction and vibration during manufacture could lead to a variation in strength over the height of the unit, whereas a few machines mould blocks on end which could lead to variation in properties along the length of the unit. Autoclaved aerated blocks are cut to size from “cakes” of foamed concrete and here the properties of the units may depend on the orientation in which the units are cut from the “cake”.

For design purposes solid concrete units and hollow and cellular concrete units filled with concrete are assumed to have the same characteristic strength regardless of the direction of loading, even on end. When unfilled cellular or hollow blocks are employed loaded in directions other than “normal” the characteristic strength must be determined by test as discussed later.

In the case of some extruded wire cut bricks which have a number of perforations (20–25% of bed area), the strength when loaded through the header faces may be of the order of 10–15% of that obtained when loaded through the bed faces. This is clearly related to the geometrical form of the unit, since when on end the brick is more slender than on bed and platen restraint is reduced. In addition, the perforations act as stress raisers and superimposed on these effects are any directional properties due to the extrusion process. Although this reduction in strength is dramatic, the available test results indicate that when built into an element the strength of the reinforced clay brickwork when loaded parallel to the bed faces is at least 40% of that when loaded normal to the bed faces[7]. Brickwork made from some pressed bricks is stronger when loaded parallel to the bed faces than when normal to them.

The compressive strength of the unit is not, of course, the characteristic strength of the masonry, but the above hopefully illustrates how variations in performance with direction of loading are likely to occur in practice. In the following section the determination of characteristic compressive strength of masonry is discussed.

19.1.1.2 Direct determination of the characteristic compressive strength of masonry, f_k

The “characteristic” masonry strengths presented in Table 3 of the Code are based on those presented in BS 5628: Part 1. Although these are termed characteristic they have not been determined statistically but are in general agreed lower bounds to the masonry strength based substantially on updated information from the permissible stress Code CP 111[8]. The designer may wish to directly determine a value of the characteristic compressive strength of a particular combination of units and mortar. This may be done by deriving a value statistically from test results (see Appendix D).

19.1.1.3 Value of f_k where the compressive force is perpendicular to the bed face of the unit

This section essentially reflects the information provided in Part 1 except that only mortar designations (i) and (ii) are considered. A new table, Table 3(B) and accompanying figure 1(b), have been added which cater for the use of units with a height to thickness ratio of 1.0. This information is useful for reinforced hollow block masonry with filled cores (remembering to use the nett unit strength unless the infill concrete is less strong than the compressive strength of the units, in which case the cube strength of the infill should be used to determine the characteristic compressive strength of the masonry).

19.1.1.4 Value of f_k where the compressive force is parallel to the bed face of the unit

This section requires no further detailed comment. Note that filled hollow blocks are treated as solid units and are not covered by this section.

19.1.1.5 Value of f_k for units of unusual format or for unusual bonding patterns

This section requires no further detailed comment.

19.1.2 Characteristic compressive strength of masonry in bending

This clause indicates that the value derived for the characteristic compressive strength of masonry should be used for both direct and flexural compression. The reason for the statement is that designers familiar with CP 111[8,9] or indeed other Codes based on permissible stress design, will be used to enhance the maximum permissible compressive stress when this is due to flexural compression. Such enhancements compensate for the inaccurate assumption that the stress distribution is linear across the section and are not necessary for the different assumptions made with limit state design.

19.1.3 Characteristic shear strength of masonry

Further information on the provision for shear is given in Clause 22.5.

19.1.3.1 Shear in bending (reinforced masonry)

19.1.3.1.1 The value of the characteristic shear strength of masonry, f_v, in which the reinforcement is placed in bed or vertical joints (including Quetta bond) or is surrounded by mortar and not concrete is 0.35 N/mm^2. No enhancement in shear strength is given for the amount of tensile reinforcement since this type of section has been shown experimentally[10] not to warrant such an enhancement when mortar is the embedment medium. It is not entirely clear why this should be so but is likely to be due to a reduction in the amount of dowel action which can be utilised in such reinforcement. Consequently, there is a reduction in the contribution by dowel action to the average shear strength across the section. It may be noted that 0.35 N/mm^2 is also the characteristic shear strength assumed for unreinforced masonry.

For simply supported beams or cantilevers an enhancement factor of $2\dfrac{d}{a_v}$ (with a limiting factor of 2) can be applied when a principal load (usually accepted as one contributing to 70% or more of the shear force as a support) is at a distance a_v from the support. This is again demonstrated in the work of Suter and Hendry[11]. The maximum factor of 2 implies a cut off in the shear strength at a ratio $\dfrac{a_v}{d} = 1.0$.

The Code suggests that in certain walls where substantial precompression can arise, for example, in loadbearing walls reinforced to enhance lateral load resistance, it is often more advisable to treat the wall as plain masonry, i.e., unreinforced, and design to BS 5628: Part 1[2].

19.1.3.1.2 For sections in which the main reinforcement is enclosed by concrete infill, an enhancement to f_v is given depending upon the amount of tensile reinforcement, by the formula:

$$f_v = 0.35 + 17.5\varrho$$

where

$$\varrho = \frac{A_s}{bd}$$

with an upper limit of 0.7 N/mm^2.

19.1.3.1.3 For simply supported beams or cantilever retaining walls an enhancement in the shear strength as derived above is given by the formula:

$$\left[2.5 - 0.25\left(\frac{a}{d}\right)\right]$$

Here the shear span is defined as the ratio of the maximum design bending movement to the maximum design shear force, i.e., $\frac{d_c}{t}$. This enhancement is similar to that in 19.1.3.1.1, but has been derived on a more rational basis reflecting the greater amount of more specific data on this subject. An upper limit of 1.75 N/mm^2 is applied, i.e., a maximum enhancement of 2.5 when $\frac{a}{d} = 0$; the enhancement factor equals 1.0 when $\frac{a}{d} = 6$. Much below $\frac{a}{d} = 2$, the masonry would act as a corbel not a beam, above $\frac{a}{d} = 6$, the failure mode would be flexural, shear failure being most unlikely. Between these values a "transition" occurs from shear to flexural failure. This behaviour in shear is analogous to that of reinforced concrete upon which much has been written. Values of f_v for various percentages of reinforcement and $\frac{a}{d}$ ratios are given in Table 3.1.

Table 3.1: Characteristic shear strength of masonry (f_v) N/mm^2

Proportion ϱ	% steel (100 ϱ)	$\frac{a}{d}$				
		⩾6	5	4	3	⩽2
0.002	0.2	0.385	0.48	0.58	0.67	0.77

0.004	0.4	0.42	0.52	0.63	0.74	0.84
0.006	0.6	0.455	0.57	0.68	0.80	0.91
0.008	0.8	0.49	0.61	0.74	0.86	0.98
0.010	1.0	0.525	0.66	0.79	0.92	1.05
0.012	1.2	0.56	0.70	0.84	0.98	1.12
0.014	1.4	0.595	0.74	0.89	1.04	1.19
0.016	1.6	0.63	0.79	0.94	1.10	1.26
0.018	1.8	0.665	0.83	1.00	1.16	1.33
0.020	2.0	0.70	0.88	1.05	1.22	1.4

19.1.3.2 Racking shear in reinforced masonry shear walls

The first part of this clause deals with walls subjected to racking shear as if they were unreinforced (see BS 5628: Part 1). The increase of 0.6 g_B due to vertical loads both here and in 19.1.3.1.1 is due to an increased "friction effect" preventing sliding.

A note is given in the Code relating to the effect on shear resistance of dampproof courses. Some information exists[12], and some general guidance is given in *Sections 2.8* and *6.37* of this handbook.

19.1.3.3 Shear in prestressed sections

The formulae for shear given here is similar to that in 19.1.3.2, the enhancement factor of 0.6 g_B is applied for similar reasons to those given in 19.1.3.2 with two additional points worth noting:

1. the prestressing load (when applied across the bed joints) is treated in the same manner as a vertical imposed load since its effect is the same
2. in certain walls subjected to bending the enhancement reflects the increased contribution to average shear provided by the compression block

It is noted in the Code that where the prestressing force is parallel to the bed joints, g_B=0. A similar enhancement to that given in 19.1.3.1.3 for reinforced masonry is given here for prestressed masonry, its value depending in the same way on the $\frac{a}{d}$ ratio.

19.1.4 Characteristic strength of reinforcing steel, f_y

The characteristic tensile strength of reinforcing steel is given in the Code as Table 4. The appropriate compressive strength may be obtained by multiplying these values by 0.83.

19.1.5 Characteristic breaking load of prestressing steel

This section requires no further detailed comment.

19.1.6 Characteristic anchorage bond strength, f_b

Reinforcement exhibits better bond strength in concrete than in mortar and this is reflected in the values given here. Unlike CP 110[1], the same value is given for bars in compression or tension and any increase due to increase in strength of the concrete is not permitted. This approach is likely to be conservative, but it was felt by the Code Committee that insufficient evidence existed to extend the given values further.

Characteristic anchorage bond strength (N/mm^2) for tension or compression reinforcement embedded in:

	Plain bars	Deformed bars
Mortar	1.5	2.0
Concrete	1.8	2.5

The Code contains a note to the effect that these values may not be applicable to reinforcement used solely to enhance lateral load resistance of walls. This is for two reasons:

1. the shape, type and size of certain (proprietary) reinforcement will differ from the bars normally used as reinforcement
2. normal detailing rules do not generally apply in this situation

The values of f_b apply to austenitic stainless steel for deformed bars only and in other cases values will need to be established by test in accordance with Appendix E of CP 110.

19.1.7 Elastic moduli

For all types of reinforced masonry the short term elastic modulus, E_m, may be taken as 0.9 f_k kN/mm^2. Although the accuracy of this estimate does vary with different types of masonry, it is reasonably well substantiated by experimental work and is consistent with overseas data[13]. It must be noted that this is the “gross” elastic modulus of reinforced masonry including the concrete infill; an “effective” modulus should not be calculated based on a transformed section incorporating different values of modulus for the concrete infill and masonry separately. This approach is likely to be somewhat conservative, particularly where relatively high strength concrete is used with relatively low strength units and particularly for blockwork.

The elastic modulus of concrete infill used in prestressed masonry is given in Table 5 of the Code, thus effectively allowing the use of transformed sections.

The long term moduli appropriate to various types of reinforced masonry are given in Appendix C.

The elastic modulus of all steel reinforcement is given as 200 kN/mm^2 and that for prestressing steel may be taken from the appropriate British Standard with due allowance made for relaxation under sustained loading conditions.

20. Partial safety factors

20.1 General

In the comment on *Section 3.16.1,* the role of the partial safety factor is indicated. The partial safety factor for loads, γ_f, is used to take account of possible unusual increases in load beyond those considered in deriving the characteristic load, inaccurate assessment of effects of loading, unforeseen stress redistribution within the structure and the variations in dimensional accuracy achieved in construction. The partial safety factor for materials, γ_m, makes 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.

20.2 Ultimate limit state

20.2.1 Loads

The four load cases (a) to (d) in this section indicate the appropriate combinations of design dead load, design imposed load, design wind load, i.e. their corresponding characteristic loads which, together with their attendant values of γ_f need to be considered. These values were selected to produce acceptable global factors of safety.

It will be apparent that load case (a) will be the one which governs the design of many buildings. Case (b) will dominate in the situation where wind load is the primary load. Case (c) considers the combination of all three loads with reduced values of γ_f applied to each due to the fact that it is unlikely that extreme values for all three will occur simultaneously. To those used to designing to CP 110 it will be apparent that the load cases to be considered when designing reinforced and prestressed masonry are virtually identical to those used for reinforced concrete. One difference occurs in load case (a) where CP 110 uses 1.0 G_k, whereas BS 5628: Parts 1 and 2, use 0.9 G_k.

There are cases when it may be appropriate to either use different partial safety factors to those recommended or in fact derive design loads in a completely different way. The Code refers to two areas in particular. In the case of farm buildings[14] the design loads are determined on a basis which allows for likely levels of human occupation and is incorporated in a partial safety factor which is described as the classification factor.

The partial safety factors to be used for the various load combinations are given in Table 3.2.

20.2.2 Materials

When considering the adequacy of a structure or an element to resist design loads, the design strength is considered to be the characteristic strength divided by a partial safety factor. The values to be used have all been recommended on the assumption that the quality of construction on site is what has been described in BS 5628: Part 1 as *special*. Essentially this means that the designer ensures that the construction is in accordance with the Code and any other Specification and that preliminary and site testing of materials is carried out. Quality control is discussed more fully in *Section 4.40*.

The values to be used for the partial safety factors are given in Table 3.3. The value used for γ_{mm} depends on whether the masonry units are supplied to the normal or special category of construction control. Special category construction control may be claimed by a manufacturer who agrees to provide units which meet or exceed an agreed compressive strength described as the “acceptance limit” with a specified degree of confidence. To do this the manufacturer must operate a quality control scheme, the results of which may be examined by the purchaser. The scheme must be such that it can be demonstrated to the purchaser that the likelihood of the mean compressive strength of a sample taken from any consignment of units being below the acceptance limit is less than $2\frac{1}{2}\%$. The procedure for special category control is also described in BS 6073: Part 2[15]. If the manufacturer cannot make the above claim and substantiate it, the designer should choose the slightly larger partial safety factor (γ_{mm}) corresponding to the normal category of manufacturing control.

20.3 Serviceability limit state

20.3.1 Loads

The values of the partial safety factors to be used for the various load combinations are given in Table 3.2. As when considering the adequacy from a strength point of view, the worst combination of loads should be used when assessing deflections, and the effects of creep, thermal movement, and so on, may need to be considered.

20.3.2 Materials

When considering deflections, stresses or cracking, the values of γ_{mm} should be chosen as 1.5 and that of γ_{ms} as 1.0.

20.4 Moments and forces in continuous members

In continuous members and their supports it is necessary to consider the effects of

Table 3.2: Partial safety factors for loads

Limit state	Load combinations													
	a			b			c				d			
	Dead+Imposed			Dead+Wind			Dead+Imposed+Wind				Dead+Imposed+Wind			
	G_k	Q_k	E_n	G_k	W_k	E_n	G_k	Q_k	E_n	W_k	G_k	Q_k	E_n	W_k
Ultimate	0.9 or 1.4	1.6	1.4	0.9 or 1.4	1.4	1.4	1.2	1.2	1.2	1.2	0.95 or 1.05	0.35	0.35	0.35
Serviceability	1.0	1.0	1.0	1.0	1.0	1.0	1.0	0.8	0.8	0.8	Non-applicable			

Table 3.3: Partial safety factors for material strengths

Loading combination	Strength				
	Direct compression and bending		Shear strength of masonry	Bond of steel to mortar/concrete	Strength of steel
	Category of manufacturing control				
	Special	Normal			
Dead+Imposed Dead+Wind Dead, Imposed+Wind	γ_{mm}	γ_{mm}	γ_{mv}	γ_{mb}	γ_{ms}
	2.0	2.3	2.0	1.5	1.15
Accidental forces	1.0	1.15	1.0	1.0	1.0

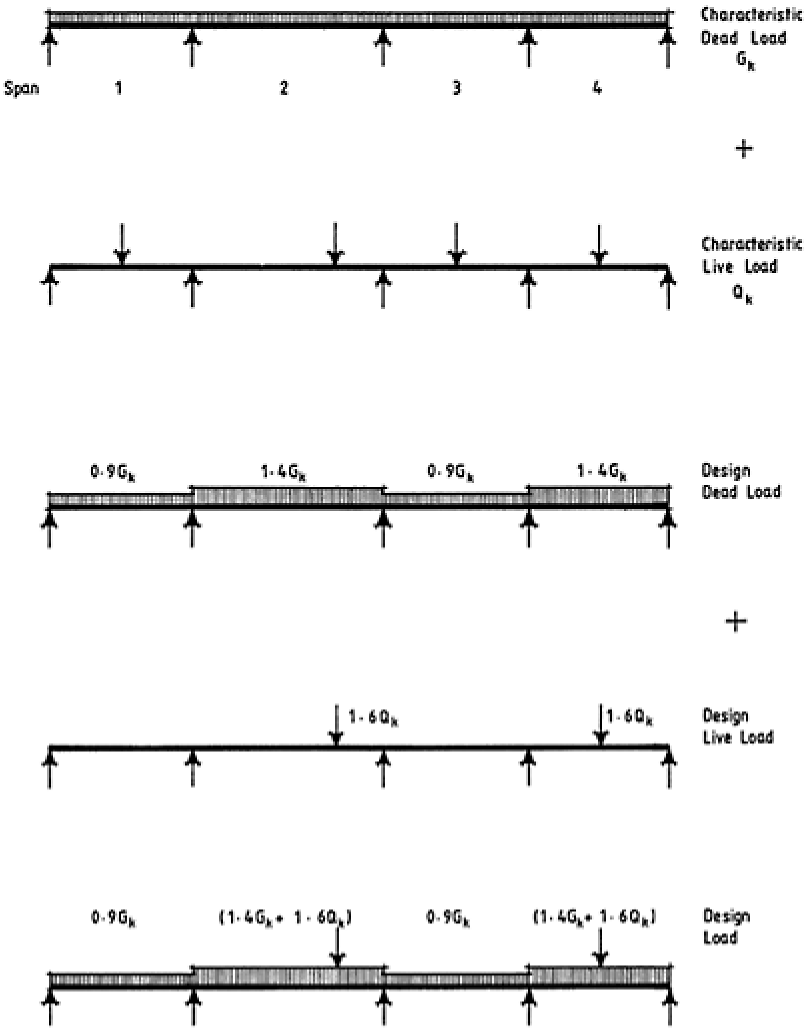
pattern loading. It is considered that an adequate assessment will be made of the structure at the ultimate limit state if the two conditions below are considered:

1. alternate spans loaded with maximum combination of dead+imposed load (1.4 G_k+1.6 G_k) and minimum dead load (0.9 G_k)
2. all spans loaded with maximum combination of dead and imposed load

Combination 1. above is illustrated in Figure 3.1.

Figure 3.1 Load combination for continuous members

Note: Also to be considered—0.9G_k spans 2 and 4 with (1.4G_k+1.6G_k) on spans 1 and 3



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Section Four:
Design of reinforced masonry

21. General

This section indicates that the subsequent recommendations are based on the assumption that design against reaching the ultimate limit state is critical. As mentioned elsewhere, the serviceability considerations are met using either simple sizing rules or by the detailed calculation of, for example, deflection, at service loads.

22. Reinforced masonry subjected to bending

22.1 General

This section of the Code deals with the design of elements subjected only to bending. Clearly this applies to a wide range of elements including beams, slabs, retaining walls, buttresses and piers. The design approach may also be applied to panel or cantilever walls reinforced primarily to resist wind forces. Walls containing bed joint reinforcement to enhance lateral load resistance should be designed following the recommendations of Appendix A, which are described in *Section 8*. In a few situations it may be appropriate to design a reinforced masonry element as a two-way spanning slab using conventional yield-line analysis.

The approach which has been adopted in the Code to the design of members subjected to bending has been developed from the simplified approach presented in CP 110: Part 1:1972[1]. A consequence of employing similar procedures to the structural concrete Code is that designers should find it comparatively easy to switch from reinforced concrete to reinforced masonry design.

The designer may calculate deflections using the procedure described in Appendix C to check that a member will not deflect excessively under service loads. In many situations, however, it will be sufficient to limit the ratio of the span to the effective depth. The same limiting values should also ensure that cracking in service conditions will not be excessive, although little research evidence is available on this topic. By designing elements within the limiting ratios imposed by the simple sizing rules, it is only necessary to determine that the design resistances exceed the design forces or moments to ensure that there is an adequate factor of safety against reaching the ultimate limit state.

22.2 Effective span of elements

The effective span of either simply supported or continuous members may be taken as the lesser of:

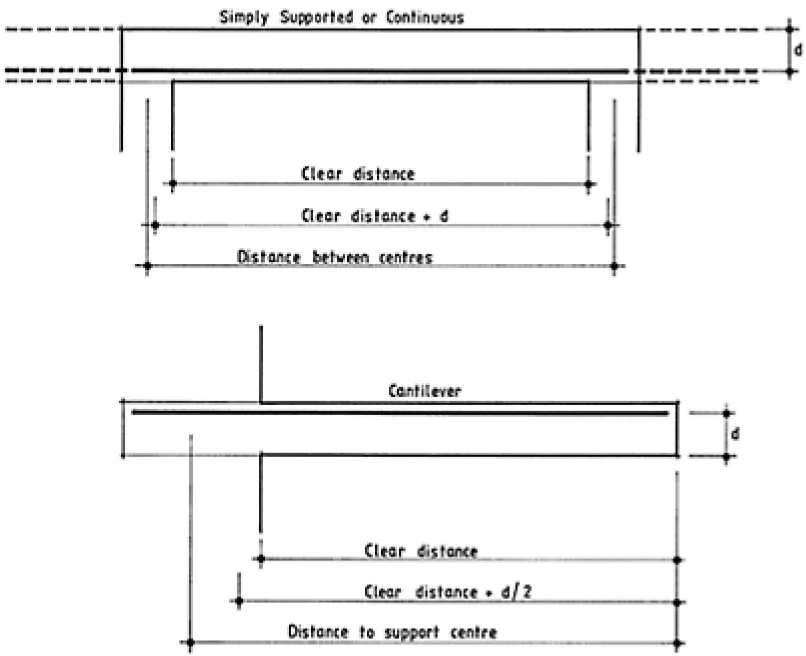
1. the distance between the centres of supports
2. the clear distance between the faces of the supports plus the effective depth

The effective span of a cantilever may be taken as the lesser 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 the effective depth

These definitions are illustrated in Figure 4.1.

Figure 4.1 Effective span of elements



22.3 Limiting dimensions

22.3.1 General

Attention is drawn to the fact that the limiting ratios given in Tables 8 and 9 of the Code should not be used when the serviceability requirements are more stringent than those given in *Section 16.2.2,* i.e., when more stringent limitations on deflection and/or cracking are required.

22.3.2 Walls subjected to lateral loading

Limiting values of the ratio span to effective depth for walls subjected to lateral loads are given in Table 4.1 (Table 8 of the Code). In the case of cavity walls, the effective depth of the reinforced leaf should be used. In the case of freestanding walls that do not form part of a building and are subjected primarily to wind loading, the limiting ratios may be enhanced by 30% provided that increased deflections and cracking are not likely to cause damage to applied finishes.

22.3.3 Beams

In the case of beams, relatively little data exists to indicate what might be reasonable limiting ratios of span to effective depth. As a result, the same limiting ratios as are incorporated in CP 110 for reinforced concrete have been adopted, although as yet no enhancement based on the level of working stress has been introduced, as it has in the case of reinforced concrete. Further data is required before this can be done, but the evidence available suggests that the recommended values which are given in Table 4.2 (Table 9 of the Code) are fairly conservative.

For simply supported or continuous beams the distance should not exceed the lesser of 60 b_c and 250 $\frac{b_c^2}{d}$. For a cantilever the clear distance from the end to the face of the support should not exceed the lesser of 25 b_c and 100 $\frac{b_c^2}{d}$. In the case of simply supported or continuous beams, b_c is the breadth of the compression block midway between restraints, in the case of a cantilever it is suggested that b_c be taken as the breadth of the compression zone at the support.

Table 4.1: Limiting span/effective depth ratios for laterally loaded walls

Wall type	Limiting ratio
Simply supported	35
Continuous or two-way spanning	45
Cantilevers with less than 0.5% reinforcement	18

Table 4.2: Limiting span/effective depth ratios for beams

Beam type	Limiting ratio
Simply supported	20
Continuous	26
Cantilever	7

22.4 Resistance moments of elements

For any singly reinforced masonry section there is a unique amount of reinforcement which would fail in tension at the same bending moment as that at which the masonry would crush. This section is described as balanced and if lower amounts of reinforcement were incorporated the section would be described as underreinforced. If an under-reinforced section were tested to destruction in flexure the failure would be due solely to that of the steel in tension. In laboratory tests tensile failure often leads to massive deflections and subsequent compressive failure in the masonry. When large amounts of reinforcement are provided, greater than that required for a balanced section, the failures in test beams are due solely to the masonry in the compression zone having inadequate strength. These failures can be sudden, are sometimes explosive and the aim of the Code recommendations is to ensure that all the sections designed using them are under-reinforced.

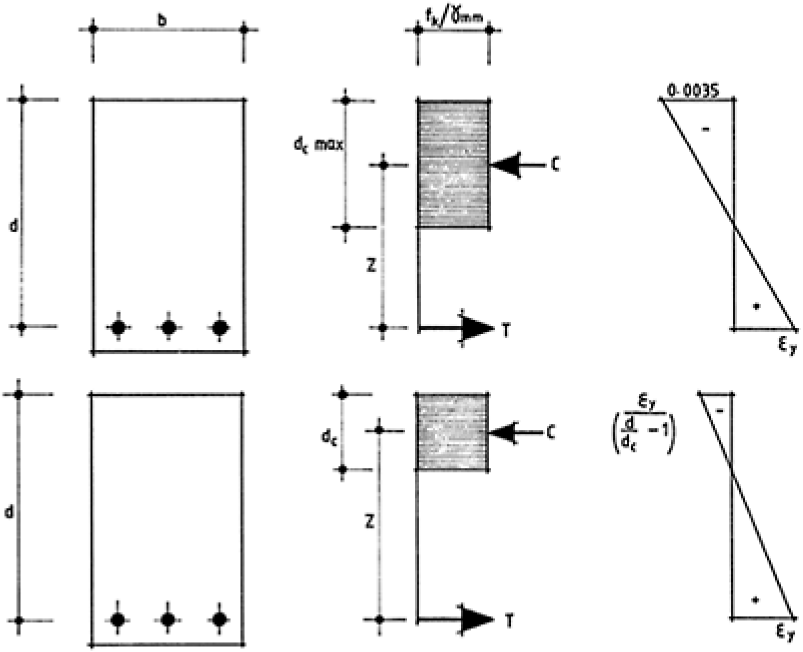
Some relatively simple assumptions have been made which enable the design moment of resistance of any under-reinforced section to be determined. An upper limit to the design moment of resistance has been set, which is that of the balanced section.

22.4.1 Analysis of sections

The idealised distribution of stress and strain in those singly reinforced sections which fail as balanced sections or due to tensile failure of the reinforcement are illustrated in Figure 4.2.

The mean stress at failure of the masonry in compression is assumed to be $\frac{f_k}{\gamma_{mm}}$, where f_k is the characteristic compressive strength of masonry and γ_{mm} is the partial safety factor for the compressive strength of masonry. This partial safety factor is intended to allow for the possibility that the masonry in the structural element on site may be weaker than similar masonry constructed in the laboratory. An allowance for other factors which affect the capacity of the section (rather than the masonry in the compression zone) is also included in this partial safety factor and consequently these influences are treated as being equivalent to a reduction in the strength of the masonry. This formulation does not necessarily attribute the various causes of uncertainty in the bending moment capacity to the most appropriate parameters because further evidence of the likely magnitude of the various influences is needed before this can be done. The current recommendations are conservative.

The maximum strain in the outermost compression fibre is assumed to be 0.0035 and is reached when the masonry fails in compression. For a balanced section the compression block is considered to have its greatest depth, $d_{c\ max}$ and plane sections are considered to remain plane. This depth is defined by the tensile strain in the steel at failure. This is found from the assumed stress-strain relationship for steel given in the Code (see Figure 4.3).



(a) balanced section
(b) under-reinforced section

Figure 4.2 Idealised stress/ strain distributions

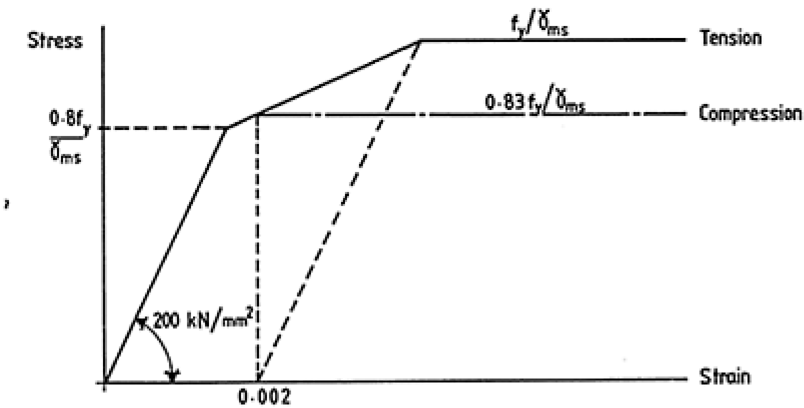


Figure 4.3 Short term design stress/strain curve for reinforcement

The strain at ultimate tensile stress (ε_y) is 0.0031 for mild steel and 0.004 for high yield steel. For these extreme values of the strain at tensile failure the maximum depth of compression block would be 0.53 and 0.47 times the effective depth respectively.

In sections which fail solely in tension the tensile force is accurately defined by the area of tensile strength of the steel. The assumption that plane sections remain plane is used to define the internal lever arm, as shown in Figure 4.2. The use of $\frac{f_k}{\gamma_{mm}}$ as a uniform compressive stress leads, in this case, to insignificant error as the only factor affected is the lever arm which has relatively little influence over the Moment of Resistance. The maximum value of the lever arm to be used in design is taken as 0.95 times the effective depth.

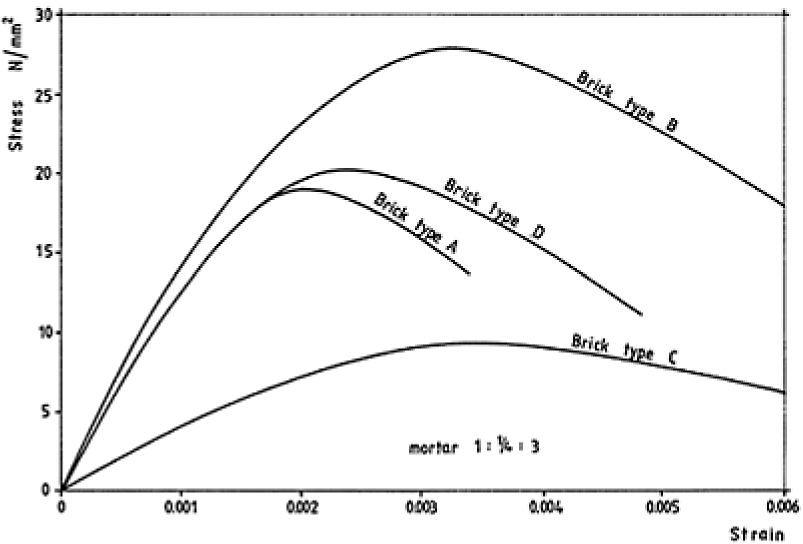


Figure 4.4 Stress/strain curves for clay brickwork

The short term stress-strain relationship for stocky specimens of brickwork has been established as a curve which may be represented by a parabola with a falling branch. Figure 4.4 shows a typical family of curves based upon the work of Powell and Hodgkinson[2]. Although less research has been conducted, it is apparent that the stress strain curve for reinforced hollow concrete blockwork is either parabolic or rectangular-parabolic (i.e., of the same form as that for reinforced concrete given in CP 110: Part 1:1972).

Figure 4.5 shows typical curves from the work of Newson[3]. If the assumption is made that plane sections remain plane, a logical form for the stress block is parabolic. The advantages of the simplicity and familiarity of the rectangular stress block approach are, however, substantial and there is considerable merit for design purposes in replacing the parabola by a statically equivalent rectangle. This is the approach adopted in the Code with the exception that the mean height of the stress block is f_k and not, as may be derived from theory, 0.75 f_k[4]. The accuracy of the simplified approach adopted in the Code has, however, been demonstrated by experimental work[5].

For those sections which are acting primarily in flexure, but which are also subjected to a small axial thrust, it is considered reasonable to ignore the thrust for design purposes because the flexural stress will dominate. The limiting stress due to the axial thrust which may be ignored in this way is 10% of the characteristic compressive strength of the masonry.

22.4.2 Design formulae for singly reinforced rectangular members

This section deals with the design of singly reinforced rectangular members which are sufficiently long (i.e., the ratio of span to effective depth is greater than 1.5) to be acting primarily in flexure. The designer must ensure that the design Moment of Resistance of the section (which is determined on the basis that it is an under reinforced section) is greater than the bending moment due to the design loads. The design formula is:

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

and this must not exceed

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

where:

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

and: M_d=design moment of resistance

b=width of the section

d=effective depth

f_y=characteristic tensile strength of reinforcing steel given in Table 4 of the Code

f_k=characteristic compressive strength of masonry

z=lever arm, which should not exceed 0.95 d

γ_{mm}=partial safety factor for strength of masonry

γ_{ms}=partial safety factor for strength of steel

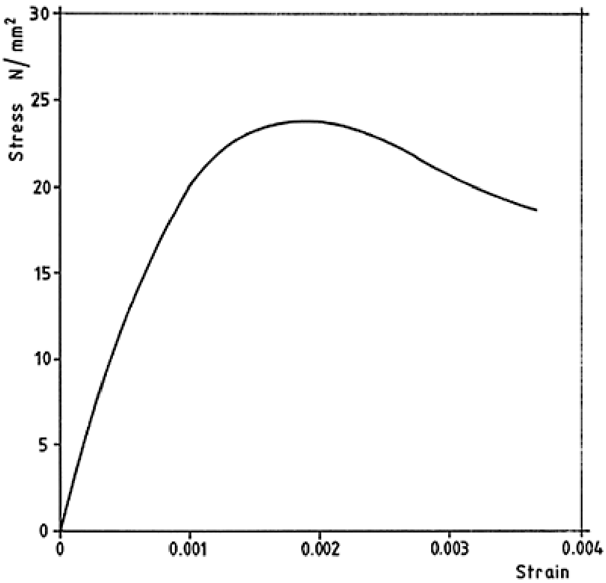
For the compression block depths derived for balanced sections on page 35 i.e., 0.53 d and 0.47 d, the corresponding design moments of resistance are:

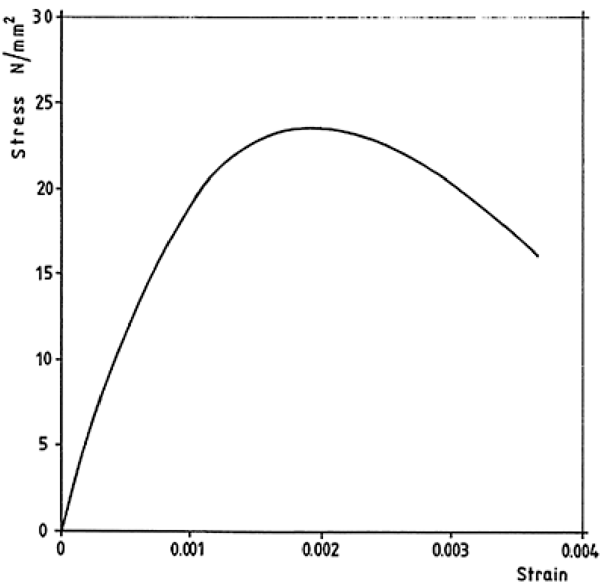
$$0.39 \frac{f_k b d^2}{\gamma_{mm}} \text{ and } 0.36 \frac{f_k b d^2}{\gamma_{mm}}$$

Figure 4.5

(a) strees vs surface strain for type 2 prisms—filled

(b) strees vs overall strain for type 2 prisms—filled





the value of $0.4\dfrac{f_k b d^2}{\gamma_{mm}}$ is consequently a reasonable approximation for design purposes for the specified types of reinforcing steel.

In the case of a beam where the width and effective depth have been fixed, possibly by other than structural considerations or by a simple sizing rule, then, if the bending moment due to the design loads is *M*, the designer must ensure that $M \leqslant M_d$.

As a first approximation, if

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

where f_k and γ_{mm} have been determined by the choice of masonry units and the mortar grade (Clause 19), the design formula may be used to estimate the area of steel required by setting z equal to 0.75 d, so:

$$A_s = \frac{4}{3}\frac{M}{d}\frac{\gamma_{ms}}{f_y}$$

A better estimate of the lever arm may then be made by substituting the value obtained for A_s into the lever arm expression. Further estimates of A_s and z may then be made using the design formula and the lever arm expression until successive estimates are judged to be sufficiently close. As an alternative to this iterative solutions, provided that

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

the proportion of tensile steel required may be found from the smaller root of the equation:

$$\left(\frac{A_s}{bd}\right)^2 - 2\left(\frac{f_k}{f_y}\right)\left(\frac{\gamma_{ms}}{\gamma_{mm}}\right)\left(\frac{A_s}{bd}\right) + 2\left(\frac{f_k}{f_y}\right)\left(\frac{\gamma_{ms}}{\gamma_{mm}}\right)\left(\frac{\gamma_{ms}}{f_y}\right)\left(\frac{M}{bd^2}\right) = 0$$

which is given by:

$$\frac{A_s}{bd} = \frac{f_k}{f_y}\frac{\gamma_{ms}}{\gamma_{mm}}\left(1 - \sqrt{1 - \frac{2M}{bd^2}\frac{\gamma_{mm}}{f_k}}\right)$$

The designer may, if he wishes, use the design charts shown in Figure 4.6, which also ensure that the basic assumptions have been satisfied. The charts show, for various steel strengths, the relationship between the non-dimensional bending moment $\dfrac{M}{bd^2 f_k}$ and the ratio of the steel proportion to the masonry strength, $\dfrac{\varrho}{f_k}$.

If the required bending moment is *M,* and the sizes of the sections have been chosen, the required steel and masonry strengths may be fixed by trial and error using the charts.

A further alternative which enables the designer to more readily adjust the value of $\frac{f_k}{\gamma_{mm}}$ when considering a tentative design is to use the design chart in Figure 4.7. In this instance a moment coefficient, *Q*, is defined such that:

$$M_d = Qbd^2$$

Q is a function of the design masonry strength, $\frac{f_k}{\gamma_{mm}}$, and the ratio of the lever arm, *z*, to the effective depth, *d*. The value of $\frac{z}{d}$ (referred to in the Code as *c*) is not permitted to be greater than 0.95 and cannot be less than 0.72, a limit which is defined by the balanced section. For any required value of *Q* then *c*, which is consistent with any assumed $\frac{f_k}{\gamma_{mm}}$, may be interpolated from Figure 4.7. The The required steel area is found by rearranging the design formula so that:

$$A_s = M_d \frac{\gamma_{ms}}{f_y} \frac{1}{z}$$

The Code acknowledges that when the span/depth ratio is small (<1.5) a beam does not act in the same simple way as more slender beams. In this situation it is recommended that the area of reinforcement should be calculated to resist a tensile force defined by dividing the design bending moment by a lever arm equal to two thirds of the overall beam depth. This simple approach considers the beam to behave in a similar way to a wall beam. The lever arm should not be taken as greater than 0.7 of the span.

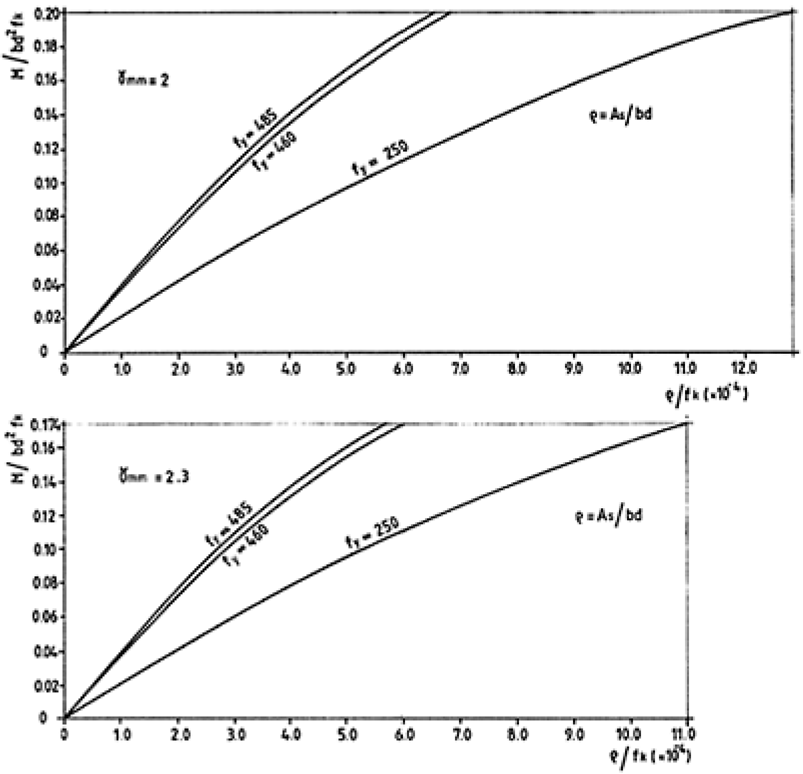


Figure 4.6 Design charts for singly reinforced rectangular members subjected to flexure (a) γ_{mm}=2.0 (b) γ_{mm}=2.3

22.4.3 Design formulae for walls with the reinforcement concentrated locally

22.4.3.1 Flanged members

There are a number of situations where reinforced masonry elements may be considered to act as flanged members and the Code includes recommendations for the more usual cases, which are in walls. Naturally, the same principles apply in other cases also.

The width of the masonry which is considered to act as a flange is limited in an arbitrary way so that the design is not extended to cases where the stability of the flanges is critical. Nevertheless, it is important that, when the spacing between concentrations of reinforcement exceeds 1 m, the capacity of the masonry to span between them should be checked.

The thickness of the flange, t_f, is taken as the masonry thickness provided that this value does not exceed half the effective depth. The width of the flange is then taken as the least of:

1. for pocket-type walls, the width of the pocket or rib plus 12×the thickness of the flange
2. the spacing of the pocket or ribs
3. one third of the height of the wall

In the case of pocket type walls where the pocket is contained wholly within the thickness of the wall, it acts as a homogeneous cantilever. For design purposes, however, it is convenient to group pocket type walls with other walls in which the reinforcement is placed in local concentrations. Examples of the recommendations are illustrated in Figure 4.8.

The design moment of resistance for under-reinforced sections is the same as that for singly reinforced rectangular sections, i.e., given by the design formula. The upper limit for the balanced section is given below:

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

When checking the capacity of the masonry to span between the concentrations of reinforcement, it may be considered to be arching horizontally and justified using Clause 36 of Part 1 to the Code[6]. It is important for the designer to ensure that, at the end of a wall, there is sufficient resistance to the component of the arch thrust that acts in the plane of the wall. The necessary force may be provided by part of an adjacent structure. Alternatively, the end of the wall may be restrained by the provision of additional reinforcement. Similarly the design should not rely on the action of arching forces across movement joints and these are generally located at positions where an additional reinforced rib, pocket or core, have been included in the wall. An example is shown in Figure 4.9.

22.4.3.2 Locally reinforced hollow blockwork

It is possible, particularly in the case of hollow blockwork, that reinforcement is concentrated locally. For example, a hollow blockwork wall may have a few cores

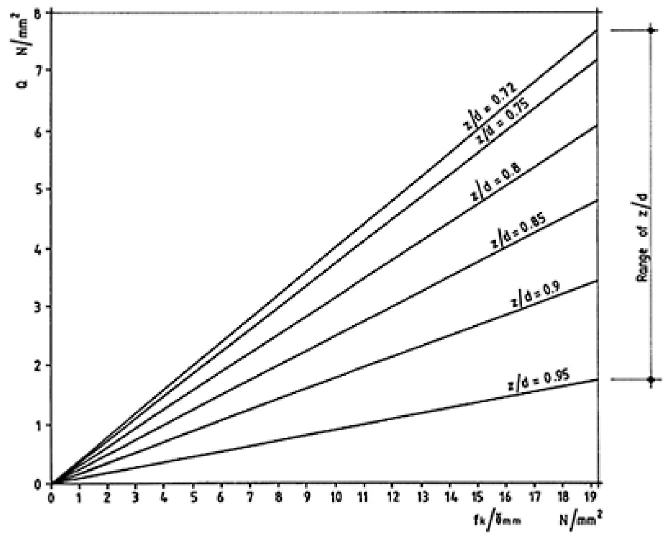


Figure 4.7 Design chart for singly reinforced rectangular section subjected to flexure

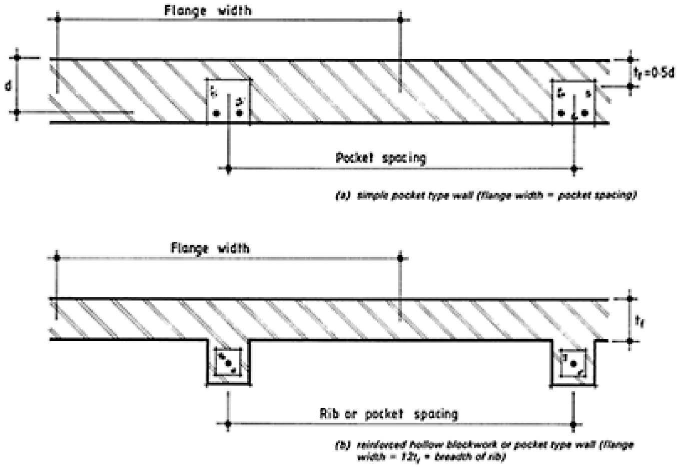


Figure 4.8 Examples of recommended flange widths

Flange width=12t_f+breadth of rib or pocket, pocket or rib spacing or one third of the wall height, whichever is the lesser

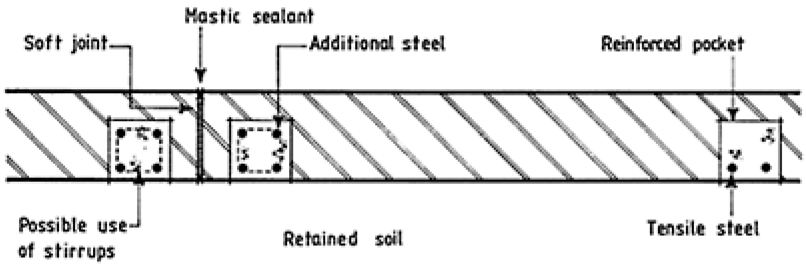


Figure 4.9 Additional pocket at movement joint

reinforced vertically at the centre of a length of walling to divide the horizontal span. In this case the reinforced element is considered to be limited in width to 3×the thickness of the block.

22.5 Shear resistance of elements

This clause deals with the shear requirements of elements in pure bending, although the recommendations are equally applicable to elements subjected to a combination of vertical load and bending where the effect of the moment is much greater than the axial load (i.e., resultant eccentricity, $\frac{M}{N}$, is greater than $\frac{t}{2}$).The design for shear in this case would tend to be conservative as there is no method of taking account of the enhanced resistance to shear afforded by the precompression. Shear due to in-plane forces, i.e., racking shear, is dealt with under Clause 25.

22.5.1 Shear stresses and reinforcement in members in bending

Behaviour in shear

The shear stress at any cross section, v, is calculated from the equation

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

where: b=the width of the section

d=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 *2.4*)

V=the shear force due to design loads

This equation treats the shear stress as if it were uniformly distributed over the whole cross section as far as the tensile reinforcement, viz:

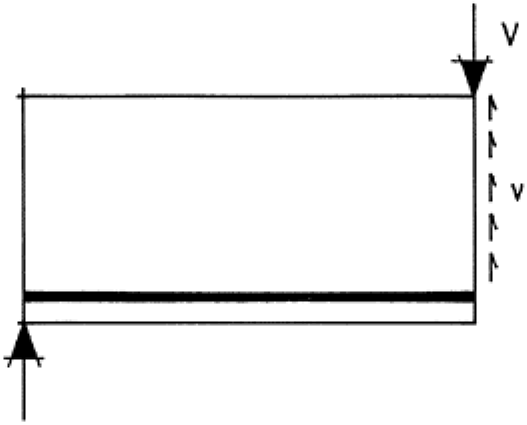


Figure 4.10 Idealised uniform distribution of shear stress

This is not strictly true and many researchers have found that, for reinforced concrete without shear reinforcement, the shear resistance is made up of a number of component forces. The situation has been found to be similar for reinforced masonry. These component forces can be idealised as:

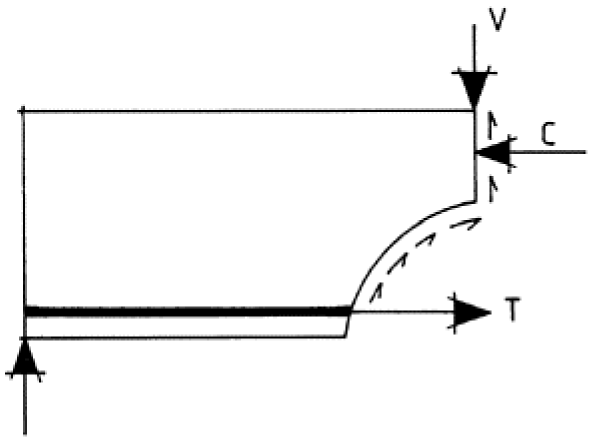


Figure 4.11 Components of shear resistance of section

The shear resistance of the section includes contributions from the uncracked part of the section which is primarily in compression, dowel action of the tensile reinforcement and any interlock along the tensile cracks. In reinforced concrete design the shear resistance is increased with an increase in the compressive strength of the concrete and also the amount, but not the grade, of tensile reinforcement. There is no recognised method of allowing for interlock which, in the case of reinforced concrete, is due to aggregates. Also, as dowel action depends for its effectiveness on the tensile strength of the concrete in that the cover must not burst, it should not in general be relied upon. As in practice, however, the figures for shear resistance are derived from tests, there will be a contribution based on both interlock and dowel action included in the design.

Enhancement due to masonry strength

Masonry research in references[7,8] relating to UK work and reference[9] referring to work from Canada, has shown that the shear resistance of masonry depends to some extent on the compressive strength of the masonry and the percentage of reinforcement when the reinforcement is located in bed joints or bond beam units. The former, however, has not been included in *19.1.4.1* since there is insufficient information available. The various types of masonry unit and methods of construction will perform differently in this context and the enhancement is relatively small. The increase in shear strength when the amount of tensile reinforcement is increased is not great and no enhancement is permitted for design purposes. An enhancement due to the percentage of reinforcing steel is included in the formula to be used for reinforced sections in which the main reinforcement is placed within pockets, cores or cavities filled with concrete.

$$f_v = 0.35 + 17.5\,\varrho$$

Additional enhancement factors for simply supported beams and cantilever retaining walls include an additional multiplier to allow for the fact that the shear strength of sections increases as the shear span/effective depth ratio decreases, hence:

$$f_v = [0.35 + 17.5\,\varrho]\left[2.5 - 0.25\,\frac{a}{d}\right]$$

where $\varrho = \dfrac{A_s}{bd}$

$\dfrac{a}{d}$ = the shear span/effective depth ratio with a being taken as the ratio of the maximum design bending moment to the maximum design shear force, $\dfrac{M}{V}$

No such enhancement is permitted when the reinforcement is surrounded by mortar instead of concrete due to lack of evidence. The value of f_v can also be enhanced in relation to any precompression which exists (see *19.1.3.3*).

From the above it is clear that some evaluation has to be made to decide which value of f_v is appropriate. This value is then divided by the partial safety factor for masonry in shear, γ_{mv}, of 2.0 and compared with the value of v obtained from equation (Clause 22.5.1).

Flanged members

The calculation of shear force is simple enough for plane elements but for flanged members *b* is taken as the width of the flange and as a result *d* has to be modified. Many other Codes and design guides (CP 110, for example), take *b* as the width of the rib and not the width of the flange. It was felt by the Code Committee that since the shear resistance of the masonry itself (neglecting the contribution from the reinforcement) comes principally from the compression block, then the width should be the full width of the flange. This in turn has an effect on the depth of masonry resisting shear since this depth cannot be the effective depth as is the case for rectangular beams. Tests on pocket type retaining walls have demonstrated that shear failures are extremely difficult to produce[10] and that the Code approach is reasonable, even if not reflecting the true mechanism by which the resistance is mobilised.

Consider a flanged member in which the thickness of the masonry between the ribs is less than *d*, say 0.5 *d:*

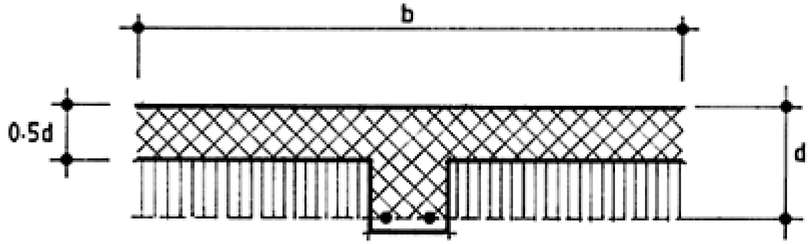


Figure 4.12 Area considered for shear resistance of a flanged member

If *d* was used in the equation (23.5.1) for flanged members, the area of masonry resisting shear would include the vertically shaded area, which does not exist. The true area resisting shear is the cross-hatched area illustrated above (Figure 4.12). Thus, the use of the thickness of masonry between leaves will be slightly conservative since it ignores any contribution of the remaining area of rib.

Provision of shear reinforcement

If $\frac{f_v}{\gamma_{mv}}$ is $\geqslant v$, then for many structures (for example, retaining walls) shear reinforcement is not generally needed. For beams in which v is $< \frac{1}{2}\frac{f_v}{\gamma_{mv}}$, for short span lintels supporting masonry and for shallow depth beams (<225 mm), shear reinforcement can be safely omitted. Masonry above a lintel will tend to arch over the opening whilst for a shallow beam flexure will generally be the critical design parameter. Shear failure of beams is very rare and even for long spans or deep beams, nominal shear reinforcement may not be required. When the designer has specified the use of nominal links, they should be provided in accordance with Clause 26.5.2.

If the value of v is too large, the designer is faced with a number of alternatives. The mean shear stress could be reduced by increasing the depth of the section and in some cases this is a reasonable solution. For example, in the case of a retaining wall, the thickness can be increased in steps towards the base. In this situation a further advantage is gained since the shear span/effective depth ratio will decrease. In the case of a brickwork beam containing only bed joint reinforcement, increasing the size of the section may well be the only cost-effective solution. A further option for some sections will be to increase the diameter of the main steel since this may enable a higher characteristic shear strength to be used. Where, however, $\frac{f_v}{\gamma_{mv}} < v$, and it is not possible to adjust the section as previously described, shear reinforcement should be provided according to the requirement:

$$\frac{A_{sv}}{s_v} \geqslant \frac{b\left(v - \frac{f_v}{\gamma_{mv}}\right)\gamma_{ms}}{f_y}$$

where: A_{sv}=cross sectional area of reinforcing steel resisting shear forces

b=the width of the section or the rib width in the case of a flanged beam

f_y=the characteristic strength of the reinforcing steel

s_v=spacing of shear reinforcement along the member $\leqslant 0.75\ d$

v=shear stress due to design loads $\leqslant \frac{2.0}{\gamma_{mv}}$ N/mm^2

This formula has been developed from the truss analogy and has been shown experimentally[11] to be conservative. In the first application of the truss analogy to reinforced concrete it was assumed that the reinforcement and concrete could be considered to behave in a similar way to an N type truss. The tension forces in the truss are carried by the longitudinal and stirrup reinforcement whilst the concrete carries the thrust in the compression zone and the diagonal thrust across the web*. Experimental observations of cracking indicated that the inclined compression struts can be taken at 45° to the longitudinal axis of the beam. Thus, to ensure that any crack is crossed by at least one stirrup, their spacing is limited to 0.75 *d* (see Figure 4.13). Bent-up bars are not included in masonry design since no experimental evidence exists as to their effectiveness and since they are unlikely to be suitable without accompanying stirrups (c.f. reinforced concrete design to CP 110).

It may be noted than nominal links of high yield steel or mild steel will provide a contribution to the total shear resistance of not less than 0.43 N/mm^2. Thus if

*When large shear forces are being supported it is possible that the diagonal compressive force could cause failure of the masonry, thus the maximum average shear stress *(v)* has been limited to $\leqslant \frac{2.0}{\gamma_{mv}}$ N/mm^2.

$v > \frac{f_v}{\gamma_{mv}}$ by no more than 0.43 N/mm^2, then nominal links will suffice. On the other hand, where $v > \frac{f_v}{\gamma_{mv}}$ by more than 0.43 N/mm^2 links will need to be provided to the formula:

$$\frac{A_{sv}}{s_v} \geqslant \frac{b\left(v - \frac{f_v}{\gamma_{mv}}\right)\gamma_{ms}}{f_y}$$

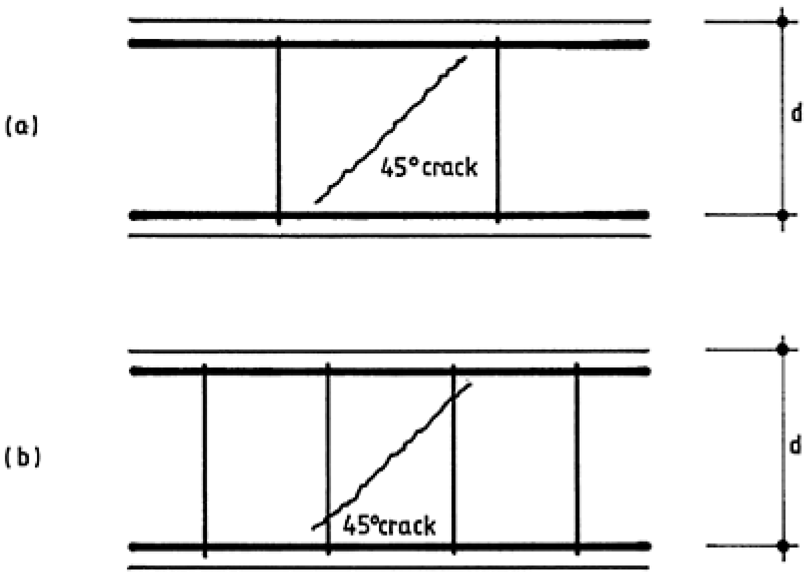


Figure 4.13 Stirrup spacing rule:

(a) s_v>d. stirrups may be ineffective

(b) s_v<0.75d. stirrups always effective

22.5.2 Concentrated loads near supports

When the ratio of shear span to effective depth of a beam is reduced below 2 the shear capacity is considerably increased[11]. Figure 4.14 shows the relationship between shear strength and shear span to effective depth ratio for reinforced hollow concrete blockwork[8]. Superimposed on this Figure is the recommendation that where $\frac{a_v}{d}$ is less than 2, the term f_v may be replaced by 2 $\left(\frac{d}{a_v}\right)$ with a

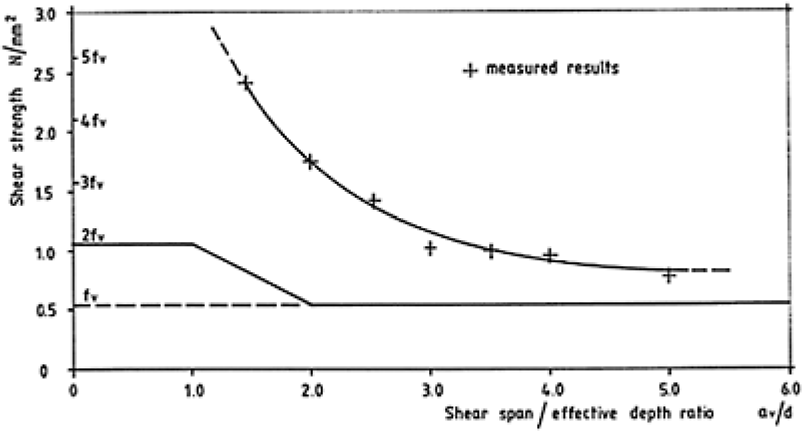


Figure 4.14 Comparison of allowable increase in f_v for concentrated loads near supports

maximum of 2 f_v. When $\frac{a_v}{d}$ is much less than 1, say 0.6, then corbel action takes place and vertical stirrups are not very effective. In this situation horizontal stirrups parallel to the main tension reinforcement become necessary. No information exists on the performance of reinforced masonry corbels.

The clause requires that when a principal load (which is defined as one contributing more than 70% of the total shear force at a support) acts at a distance a_v from the support (where a_v is less than twice the effective depth) then shear reinforcement should be placed over the distance a_v. Generally, however, with simply supported beams and cantilever retaining walls, the designer may wish to take advantage of the enhancement in the shear resistance of the section because of its low shear span/effective depth ratio as described in the previous paragraph, up to a maximum of 2 f_v. The two enhancements must not be applied simultaneously.

22.6 Deflection

The calculation of deflections in reinforced masonry is not usual and deflections will generally be acceptable if the sizing rules are obeyed. The values in Table 8 of the Code have been in use for some time and appear to be reasonable, there is no experience of the use of Table 9 but, based as it is on the most conservative values in CP 110, it is likely to be conservative. Should it be considered, in special circumstances, to be necessary to calculate deflections and compare them with the serviceability requirements, some guidance is given in Appendix C. Care is needed in the assessment of deflections calculated according to the Appendix as there are a number of uncertainties in the calculation and the result should be considered an estimate only.

22.7 Cracking

Less precise information is available about cracking than deflection and consequently calculations regarding crack widths are not to be recommended. The designer is advised to use the sizing rules in Tables 8 and 9 of the Code.

23. Reinforced masonry subjected to a combination of vertical loading and bending

Research into this aspect of reinforced masonry is somewhat limited. The design methods given in the Code are, therefore, something of a compromise following in part BS 5628: Part 1 and CP 110. An eccentricity of 0.05 times the depth of the section in the plane of bending is a common reference point for both documents. In the former it is a limiting eccentricity up to which the effect of the eccentricity is so small that the loading may be considered to be axial, the sense in which it is used in BS 5628: Part 2. In CP 110, however, it is used as an initial eccentricity which allows for maximum erection tolerances. The difference between the two is small and lies in the way in which the partial safety factors are interpreted.

23.1 General

The members covered by Clause 23 are those which, in accordance with the above, have resultant eccentricities due to simultaneously applied substantial vertical and horizontal or eccentric (eccentricity greater than 0.05 times the depth of the section in the plane of bending) vertical loading.

23.2 Slenderness ratios of walls and columns

23.2.1 Limiting slenderness ratios

Slenderness ratios for reinforced masonry walls and columns have been limited to the same values as those given for unreinforced masonry in BS 5628: Part 1. These had been based on CP 111: Part 2:1970[12] values, and those in turn upon good judgement. These limiting ratios have been adopted because, in the absence of adequate experimental data, they are known to produce satisfactory results. An arbitrary limit, taken from SP 91[13], for cantilever columns of up to 0.5% reinforcement (based upon breadth of section×effective depth) is proposed beyond which special consideration should be given to deflection.

The slenderness ratio is defined, as in BS 5628: Part 1, as the ratio of the effective height to the effective thickness and for rectangular solid sections, is thus, the same as for reinforced concrete columns designed to CP 110. A more fundamental approach could be used for other than rectangular sections using a slenderness ratio of effective length to radius to gyration, providing the limits given in Clause 23.2.1 are modified accordingly.

The sub-clauses dealing with lateral support, effective height and thickness, again give similar guidance to that given in BS 5628: Part 1.

23.2.2 Lateral support

The lateral support requirements are intended to ensure that consideration is given to the overall stability of the structure and to the satisfactory interaction of its elements. Simple and enhanced resistance to lateral movement are described.

23.2.3 Effective height

The slenderness of masonry walls and columns is important as it determines their susceptibility to buckling failure based upon the Euler buckling theory. The assessment of effective height by structural analysis referred to in the Code means the analysis of the deflected shape of the member under load and the comparison of it with the idealised deflected profile of a pin ended strut. As an alternative to the more rigorous approach, Table 10 in the Code gives values which may be adopted by the designer. These are the same as given in BS 5628: Part 1 and reflect semi-empirical but conservative assumptions based, in the case of reinforced masonry, largely on theoretical studies. As with unreinforced masonry, it is assumed that a reinforced masonry column will exhibit a somewhat lower strength for a given height than will a reinforced masonry wall.

23.2.4 Effective thickness

The effective thickness of a reinforced masonry wall depends upon its form. For single leaf walls and columns, the actual thickness is used. Where one leaf of a cavity wall is reinforced, the effective thickness may be taken as $\frac{2}{3}$ of the sum of the actual thickness of the two leaves, or as the actual thickness of the thicker leaf, whichever is the greater. In the case of the cavity wall, for reasons of practicality, the reinforced leaf will usually be the thicker and its actual thickness will probably be used as the effective thickness, thus avoiding the need to share the load between the leaves and check that the shear between them can be accommodated. For grouted cavity walls the effective thickness is taken as the actual overall thickness with the limitation that the width of the cavity shall be taken as not thicker than 100 mm. This is an arbitrary limitation to prevent the excessive thickening of the concrete infill merely to reduce the slenderness ratio of the wall. The limitation also ensures that the masonry can interact with the concrete infill. If a very wide cavity was desirable it would generally be more economic to design on the concrete section only, regarding the masonry as permanent formwork.

23.3 Design

23.3.1 Columns subjected to a combination of vertical loading and bending

This clause deals with the design of short columns (slenderness ratio less than 12) subjected to both single axis and biaxial bending and slender columns (slenderness ratio greater than 12) subjected to single axis bending.

23.3.1.1 Short columns

As in CP 110 it is usually considered sufficient to design short columns for the maximum moment about the critical axis only, even where it is possible for significant moments to occur simultaneously about the axes.

Two methods are given for the design of short columns. The first is based upon first principles in which the cross section of the column is analysed using strain compatability to determine the design moment of resistance and the design axial load capacity, and the second is to use the design formulae given. The former method entails the use of assumptions (a) to (e) given in Clause 22.4.1 for the stress and strain distributions in the section being analysed.

The three sets of formulae given in 23.3.3.1 are also based on the assumptions (a) to (e) referred to previously, including the assumption of a simplified rectangular stress block with an intensity of $\frac{f_k}{\gamma_{mm}}$. These formulae are similar to the corresponding formulae in CP 110, and are illustrated for three cases.

The formula in case (a) is used when the design axial load, N, is less than the capacity, N_d, in the stress diagram (Figure 4.15). The column is then reinforced with a nominal area of reinforcement (see *Section 26.1*). No allowance is made for this reinforcement as the column has been designed as effectively unreinforced (c.f. Appendix B, BS 5628: Part 1). The obvious point that e_x cannot exceed 0.5 t is made in the Code.

The formulae in case (b) are used when the design axial load, N, exceeds N_d in (a) above. These formulae provide a simple method of design avoiding the complications involved in using the rigorous application of the beam bending assumptions given in 22.4.1. To assist in the use of method (b), guide values for f_{s2} are given in the formulae which vary with the chosen depth of the masonry compression block. These formulae can be used for non-symmetrical arrangements of reinforcement.

In Figure 4.16 the depth of compression block, d_c, is plotted against the stress in the reinforcement in the least compressed face of the column. d_c should not be chosen as less than 2 d' to avoid the possibility of the occurrence of a narrow band of masonry forming the compression zone, leading to a local crushing failure.

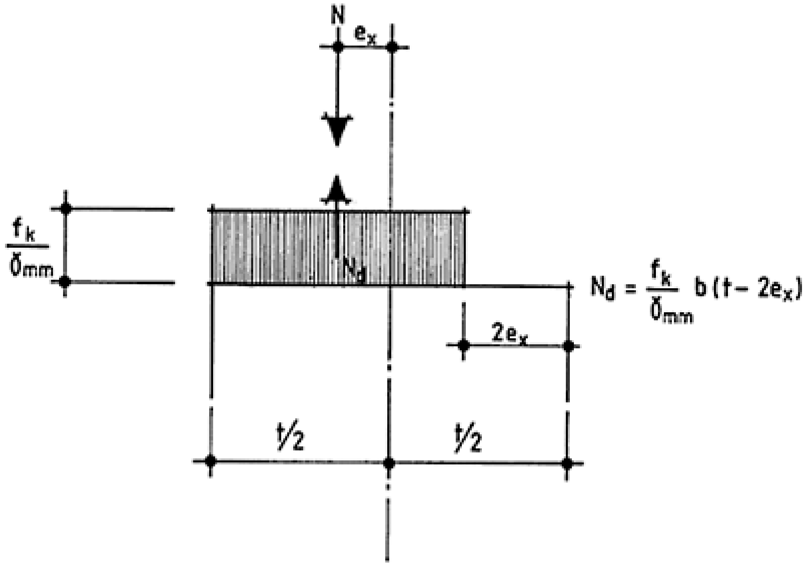


Figure 4.15 Design of short columns ignoring reinforcement

Case (c) is offered as an alternative to case (b) and permits the design axial load to be ignored when the resultant eccentricity exceeds $\left(\frac{t}{2} - d'\right)$, provided the section is designed to resist an extra bending moment equal to the design axial load acting at an eccentricity of $\left(\frac{t}{2} - d'\right)$. The method permits the area of tension reinforcement resisting this increased moment to be reduced by *N*. This method may be useful when the direction of bending is irreversible and it is considered necessary to use a symmetrical arrangement of reinforcement.

Unlike CP 110, no design charts have been included in the Code as an alternative to the design formulae and design from first principle. However, charts are a very useful design tool and are, therefore, included in this handbook. The design charts, in the form of interaction curves, given at the end of this Chapter, are generally based upon the same assumption as the beam and column design formulae referred to above. Thus, unlike CP 110, these design charts are based upon the assumption of a rectangular stress block rather than a rectangular parabolic stress block. This has a considerable practical advantage in that each chart can deal with any value of unit characteristic strength, f_k, so that fewer charts are necessary and interpolation between charts is not required. The charts do, however, assume that the reinforced masonry columns being designed have equal amounts of reinforcement positioned at equal depths from the column faces.

In general the assumption of a rectangular stress block will result in a somewhat greater area of reinforcement than would the more complex stress blocks. This is of greatest significance when columns or walls carry predominantly vertical loads, i.e., when the depth of compression zone to depth of section ratio, $\frac{d_c}{t}$, is high. This is apparent because even when $d_c=t$, the rectangular parabolic and the parabolic stress blocks have some moment of resistance by virtue of their non-symmetrical shape.

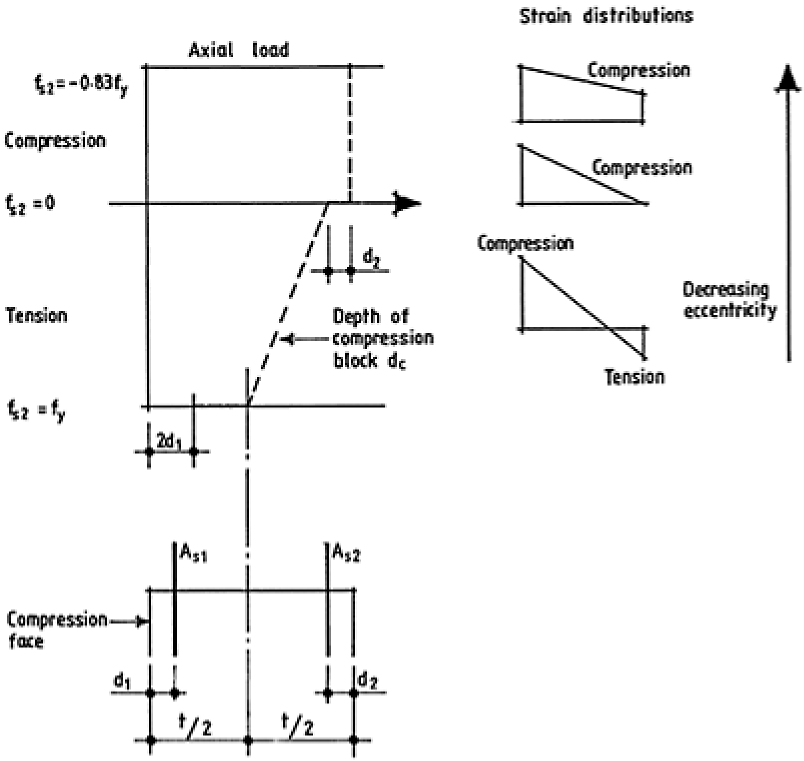


Figure 4.16 Relationship between depth of compression zone and strees in steel in least compressed face of column

A series of tests on eccentrically loaded reinforced brickwork columns by Anderson and Hoffman[14] has shown good agreement with a calculated interaction curve, and as the use of such curves is well established for reinforced concrete design, they should prove to be equally useful for reinforced masonry design.

23.3.1.2 Short columns: biaxial bending

This sub-clause deals with short columns which are subjected to biaxial bending and is only applicable to symmetrically reinforced rectangular sections, which may be designed to withstand an increased moment about one axis given by the following relationships:

(a) for $\frac{M_x}{p} \geqslant \frac{M_y}{q}$ $M'_x = M_x + \alpha\left(\frac{p}{q}\right) M_y$

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

where: M_x=the design moment about the *x* axis

M_y=the design moment about the *y* axis

M'_x=the effective uniaxial design moment about the *x* axis

M'_y=the effective uniaxial design moment about the *y* axis

p=the overall section dimension in a direction perpendicular to the x axis (see Figure 4.17)

q=the overall section dimension in a direction perpendicular to the *y* axis (see Figure 4.17)

α=a coefficient derived from the following table, Table 4.3.

Table 4.3: Values of the coefficient, *α*

$\frac{N}{N_{dz}}$	α
0	1.00
0.1	0.88
0.2	0.77
0.3	0.65
0.4	0.53
0.5	0.42
>0.6	0.30

where: *N*=the design axial load

N_{dz}=the design axial load resistance of the column, ignoring all bending which, for a section of area A_m with symmetrically disposed reinforcement, may be calculated from the expression: $N_{dz}=f_k A_m$

f_k=characteristic compressive strength of masonry

The initial published version of the Code contains a number of errors in this section. Firstly, the term $\frac{q}{p}$ in the equation for M'_y has been inverted. The correct equations are given above. Secondly, the expression for N_{dz} contains an expression for the steel, which it should not, and the partial safety factor for the compressive strength of masonry, which should also be omitted: i.e.,

$$N_{dz} = f_k A_m$$

This empirical method is based upon the CEB Bulletin D'Information No 141[15] which presents an approximate formulae for symmetrically reinforced sections. Extensive comparisons were made with more rigorous computer based analysis in the drafting of BS 8110[16] which justified the more favourable values of α given in Table 4.3. Table 12 of the Code contains unmodified values comparable to those contained in CEB Bulletin No 141.

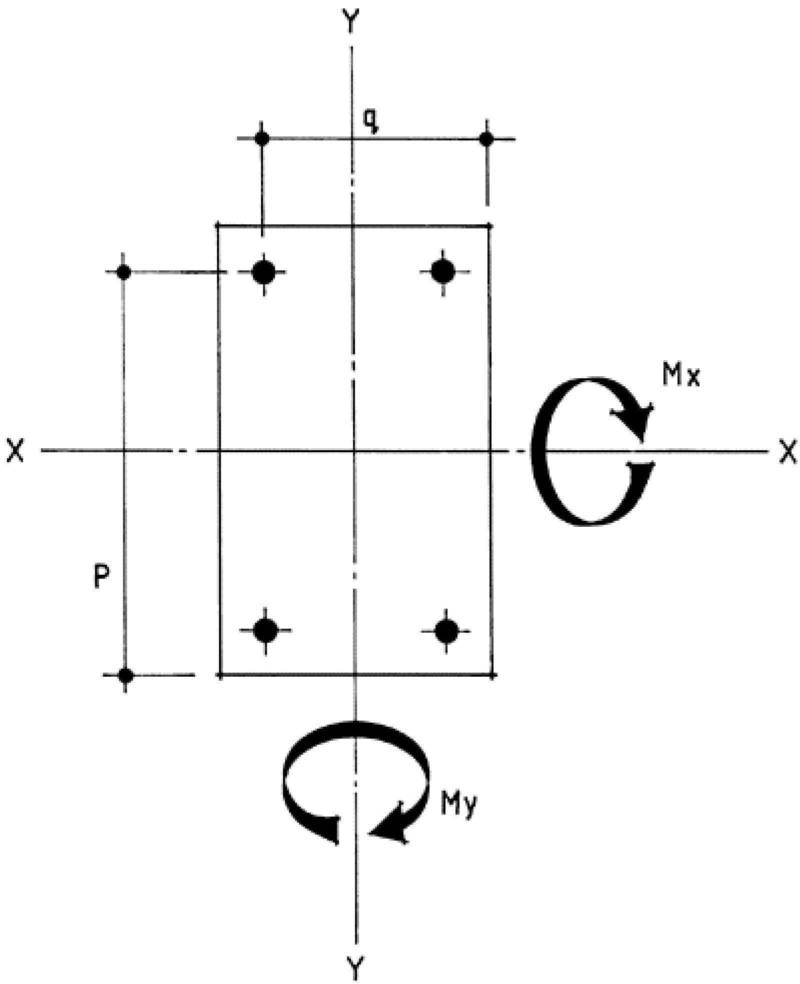


Figure 4.17

23.3.1.3 Slender columns

Slender columns are those defined as having a slenderness ratio, $\frac{h_{ef}}{t}$, greater than 12. This clause emphasises that account of biaxial bending should be taken where appropriate when designing slender columns, and also of the additional moment, M_a, induced by the vertical load and lateral deflection of the column. The additional moment concept was developed for reinforced concrete design and the Code equation for M_a is similar to that given in CP 110 and is based both upon CP 110 and the additional eccentricity which would be derived for unreinforced masonry using BS 5628: Part 1. It should be noted that although no reference is made in BS 5628: Part 1 to the braced/unbraced column concept adapted in CP 110, the reference to simple and enhanced lateral support effectively refer to columns which are “braced”, i.e., which do not contribute to the overall lateral stability of the building. Columns without simple or enhanced resistance to lateral movement must be considered as cantilevers.

The design of slender columns can be carried out either by analysis of the cross section from first principles to ensure that the design bending moment, including the additional moment, and the design axial load are exceeded by the design moment of resistance and the design axial load capacity respectively, or by use of the Code equations. Alternatively, the design charts may be used, the design moment being modified to include the additional moment. It will be noted that unlike CP 110 the Code does not differentiate between bending about a minor and a major axis. Consideration of the additional eccentricity of the axial load due to slenderness effects calculated from various slenderness ratios is instructive, the values being given in the following table (Table 4.4). These are derived by rearranging the equation in the Code.

It may be noted that e_{add} for columns at the limit of slenderness ratio for short columns (i.e., 12) correlates with the eccentricity 0.05t which may be ignored when designing short columns.

Figure 4.18 indicates the basis of the additional moment concept for slender columns.

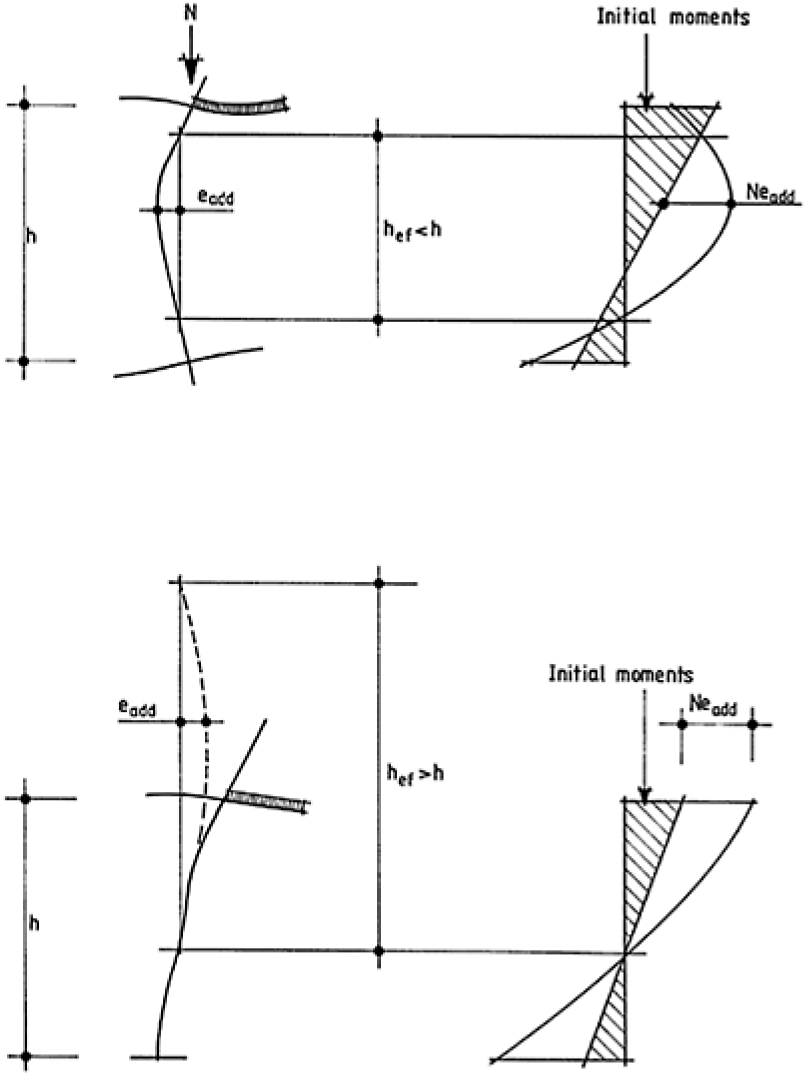
Table 4.4: Relationship between slenderness ratio and eccentricity of additional moment

$\frac{h_{ef}}{t}$	e_{add}	
12	0.07t	
18	0.16t	limit for cantilever columns
24	0.29t	
27	0.36t	limit for columns

Figure 4.18 Relationship between effective height of columns and additional moment diagram

(a) restrained columns

(b) cantilever columns



23.3.2 Walls subjected to a combination of vertical loading and bending

These walls both short and slender as defined for columns, may be designed in the same way as columns subjected to combined loading. Although the sub-clause dealing with short walls refers only to analysis of the section using the assumptions given in 22.4.1, there is no reason why the design formulae given in 23.3.1.1 should not be used, or indeed the design charts, taking the width of the section, *b*, as the unit length of the wall. Often, however, walls in reinforced masonry will be singly reinforced with the reinforcement placed approximately centrally in the section, and it is then necessary to amend the formulae.

23.3.2.1 The short wall sub-clause (23.3.2.1) refers specifically to the situation where the resultant eccentricity, e_x, is greater than 0.5t. In this case the axial load may be neglected and the wall designed as a member in bending in accordance with Clause 22.

23.3.2.2 Slender walls are treated in the same way as short walls with the exception that the additional moment derived in the same way as for columns is included.

23.4 Deflection

This clause refers the designer to the limiting dimensions given in Clause 22.3, but does not make it clear whether Table 8 or Table 9 should be used. Some degree of judgement is required in this matter. Since Table 8 refers to walls, its use seems appropriate, but consideration should be given to the situation where a column of primary structural importance is subjected to a predominantly substantial bending moment, i.e., if the member is designed as a beam in accordance with 23.3.1.1, Table 9 should also be used.

23.5 Cracking

If the design vertical load of a wall or column exceeds $\frac{A_m f_k}{2}$, then the eccentricity of the load at a critical cross section is not likely to be great enough to cause cracking due to flexural tension. In more lightly loaded columns reinforcement may be provided to control cracking and this should be provided in the same way as for beams. The recommendations are given in Clause 26.

24. Reinforced masonry subjected to axial compressive loading

This clause deals with walls and columns which carry a design vertical load, the resultant eccentricity of which does not exceed 5% of the thickness of the member in the direction of the eccentricity.

As mentioned in *Section 23* in this respect, BS 5628: Part 2 differs from CP 110. The CP 110 design equations for short reinforced concrete columns include an allowance for an additional moment due to erection tolerances based on an eccentricity of 5% of the depth of the section. Thus reinforced concrete column designs automatically assume a minimum eccentricity of 5% for columns with a nominal axial load. In BS 5628: Part 2, the designer is referred either to the equations appropriate for columns subjected to combined loading, or to the design method given in BS 5628: Part 1, making no allowance for the reinforcement. Recourse to Part 1 is also recommended for the design of walls subjected to concentrated loads, the implication being that the provision of special reinforcement is impractical.

25. Reinforced masonry subjected to horizontal forces in the plane of the element

Where walls are used to provide overall stability to a structure, significant horizontal loads can be applied in the plane of the walls. The capability of the element to resist these forces should be checked in respect of both the resistance to racking shear and the resistance to bending.

25.1 Racking shear

Walls which are subjected to in-plane horizontal forces and loaded to failure, crack typically in the manner illustrated in Figure 4.19. The cracks are caused by diagonal tension and, although there has been some research into the strength of brickwork when subjected to biaxial loading[19], it is usual to treat the design of walls on the basis of the average stress over the plan area. Thus, if the total design horizontal force is *V*, the shear stress due to design loads is considered to be *v*, where:

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

and where *t* and *L* are the thickness and length of the wall respectively.

The Code states that adequate provision against the ultimate limit state being reached must be assumed if the average shear stress is less than the design shear strength, i.e.:

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

f_v is the characteristic racking shear strength taken from 19.1.3.2, i.e., 0.35+0.6 g_B N/mm^2, where g_B is the design vertical load per unit area of wall cross section due to the vertical dead and imposed loads calculated from the appropriate loading condition. (The maximum value to be taken for f_v is 1.75 N/mm^2.)

Alternatively research[20] has shown that for walls which are reinforced with the main reinforcement in pockets, cores or cavities, a lower bound for the shear resistance is 0.7 N/mm^2 and this may be used as a characteristic value instead of 0.35+0.6 g_B. The value of 0.7 N/mm^2 was derived from tests on walls with a limited range of shapes and so the use of the value is limited to walls where the height/length ratio is not greater than 1.5.

Where v is greater than $\frac{f_v}{\gamma_{mv}}$, horizontal shear reinforcement should be provided (but in no case should v exceed $\frac{2.0}{\gamma_{mv}}$ N/mm^2). This reinforcement should be provided according to Code equation:

$$\frac{A_{sv}}{s_v} \geqslant \frac{t\left[v - \frac{f_v}{\gamma_{mv}}\right]}{\frac{f_y}{\gamma_{ms}}}$$

Part of the applied shear force, $V=vtL$, is considered to be resisted by a com ponent of force in the masonry, $\frac{f_v}{\gamma_{mv}}tL$, and the remainder by the total area of horizontal steel acting in tension across any incipient crack. If the crack is assumed to be at 45°, the number of points at which horizontal steel crosses the crack is then $\frac{L}{s_v}$. The formula can then be written:

$$V = vtL \leqslant \frac{f_v}{\gamma_{mv}} tL + \frac{A_{sv} f_y}{\gamma_{ms}} \frac{L}{s_v}$$

which, rearranged, gives:

$$\frac{\left(v - \frac{f_v}{\gamma_{mv}}\right)t}{\frac{f_y}{\gamma_{ms}}} \leqslant \frac{A_{sv}}{s_v}$$

Any vertical reinforcement will also help resist shear in racking by dowel action. This is not as effective as the horizontal reinforcement in tension, and so has been ignored. In any event, many shear walls will not require any horizontal steel specifically for shear resistance, particularly where some light horizontal distribution steel is already provided.

In any case of reinforced or unreinforced masonry where the designer is considering the use of shear walls, particular consideration must be given if any type of damp-proof course has been introduced which is likely to produce a plane at the base of the wall along which sliding could occur[21].

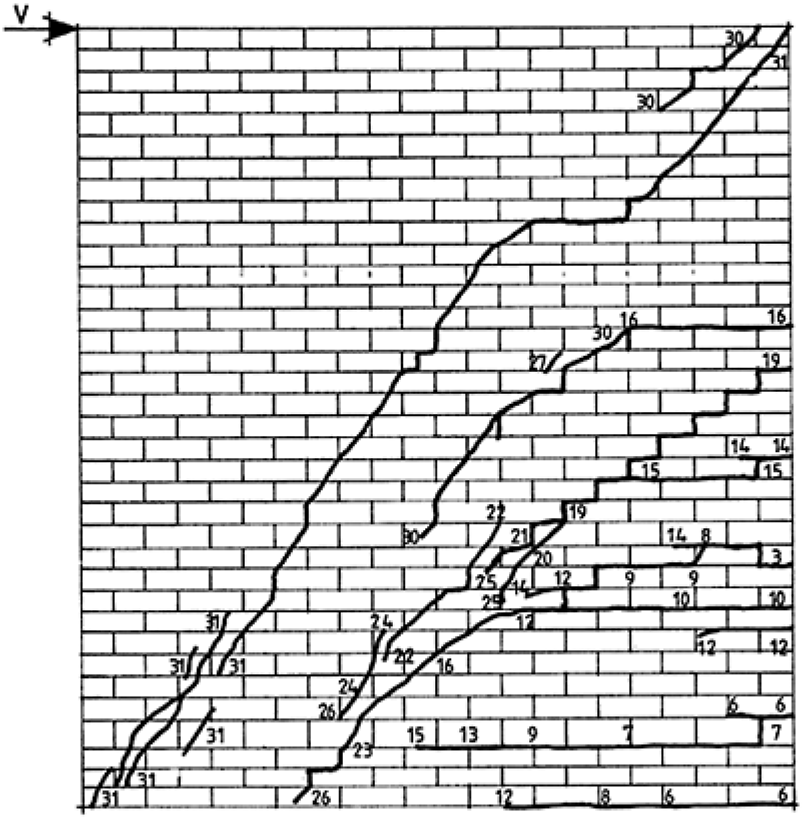


Figure 4.19 Diagonal cracking due to racking load

25.2 Bending

When bending is in the plane of the wall, the analysis and design of the wall should follow the recommendations for flexural members given in Clause 24. The designer should satisfy himself that in designing the wall as an in-plane cantilever, the fixity at the base is adequate. Assumption (f) in Clause 22.4.1 may be ignored.

It is unlikely that bending due to the horizontal forces will be critical, shear is more likely to be so. However, where the slenderness ratio of the wall in either direction exceeds 12, then additional moments may be set up in the wall. It is then necessary 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 with Clause 23.3.1.3. This essentially checks bending at right angles to the length of the wall where shear walls support no vertical loads. This approach is likely to be very conservative.

26. Detailing reinforced masonry

The previous clauses have covered the basis of design and the analytical procedures to be followed to arrive at the area of reinforcement required to give an adequate margin of safety against failure. As with reinforced concrete, it is the detailing of the reinforcement which is paramount if the calculated design performance is to be achieved in practice. This section explains the requirements and gives guidance on how reinforcement may be incorporated in masonry so that the main steel is effective, any secondary steel economically provided and any cracking controlled.

26.1 Area of main reinforcement

The area of main reinforcement that is provided is usually expressed as a proportion of the area defined as the effective depth×the breadth of the section. There are no minimum recommendations in the Code, although many of the early drafts included the following limitation:

$$A_s \geqslant 0.002 \ bd \text{ for mild steel}$$

and

$$A_s \geqslant 0.0015\,bd \text{ for high yield steel}$$

It would be unusual for reinforced sections to include areas of main reinforcement which are much below these values. However, there are a number of situations where the size of the element may be fixed for other than structural reasons and the area of steel supplied does not need to meet such requirements. For example, low grouted cavity retaining walls have an effective depth dictated by the thickness of the units used and the cavity width but may be adequately reinforced using mesh which does not provide an area in excess of the appropriate value above. Another example is where a wall beam is designed according to Clause 22.4.2 where the application of a restriction on the percentage of steel could lead, in the case of hollow blockwork, to extraordinary amounts of steel being required. In this case the reason is the large overall depth of the element.

The omission does lead to certain difficulties and the Code draws the designer's attention to the fact that in some cases a design in accordance with BS 5628: Part 1, i.e., ignoring the reinforcement, may be appropriate.

It should be noted that when considering the percentage of reinforcement in an element, this may well relate to a locally reinforced section, for example, if some cores of an otherwise unreinforced hollow blockwork wall are reinforced, then the locally reinforced section should be considered for calculating the proportion of reinforcement when designing for flexure or shear.

26.2 Maximum size of reinforcement

The limiting sizes given are based on practical considerations. Most mortar joints are designed as 10 mm thick and, therefore, to maintain some cover above and below joint reinforcement, the 6 mm maximum is specified. In most cores and cavities a 25 mm bar is the largest which can be incorporated, particularly if the bars are to be lapped. In pocket type walls, where the pockets can be made large enough, a 32 mm bar can be used. These limitations are based on experience in the UK. In the USA and Canada larger bars are commonly used, but are incorporated in very wide cavities or cores (such as 300 mm wide concrete blocks) and reinforcement is often spliced rather than lapped. Such a wide range of units is not available in the UK.

26.3 Minimum area of secondary reinforcement in walls and slabs

Secondary reinforcement is required in walls and slabs to ensure monolithic action. The minimum required is 0.05% of *bd* and can be provided in any of the following ways:

1. proprietary bed joint reinforcement
2. light reinforcement (6 mm) in bed joints
3. reinforcement in bond beams in reinforced hollow blockwork
4. within the cavity of grouted cavity construction

(Note: in pocket type walls secondary reinforcement is usually omitted)

Such reinforcement can also perform a secondary function of controlling movements in the masonry (see Clause 34). Particular attention should be paid to the durability requirements of a section especially with respect to steel embedded in mortar.

26.4 Spacing of main and secondary reinforcement

The minimum bar spacings are aimed primarily at allowing adequate room for the concrete to flow around the bars and at obtaining adequate compaction. The particular requirements are illustrated in Figure 4.20. Bars can be grouped in pairs either horizontally or vertically.

Bundling of bars is unlikely to be necessary since the percentage of steel required is comparatively low and this is not generally recommended for reinforced masonry because of the limited size of sections available. Where an internal vibrator is to be used, room should be left between any top bars in beams for its insertion. It is also for this reason that only one bar should be incorporated in pockets or cores whose size is less than 125×125 mm. This does not apply at laps of course, but consideration should be given to the use of splices and connectors.

Generally, spacings wider than the minimum should be aimed at, particularly between top bars, to allow the concrete to pass through easily.

The maximum bar spacing of 500 mm is specified for two reasons:

1. to control crack widths
2. to enable walls and slabs to act monolithically

In reinforced hollow blockwork this spacing would typically mean one bar every alternate core. This maximum spacing may be exceeded when the element is designed as a flanged member, but care must be taken to ensure that the masonry between concentrations of reinforcement, where no flange action can occur or where the allowable flange width is exceeded, can span unreinforced between these concentrations. In pocket type retaining walls the spacing between concentrations of reinforcement is likely to be within the range 1.2–1.5 m. The maximum spacing of shear links is 0.75 *d* (see also Clause 22.5).

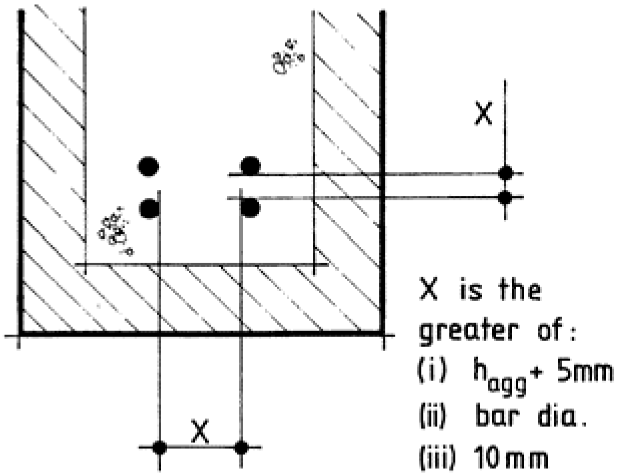


Figure 4.20 Minimum bar spacing—h_{agg} is the maximum size of aggregate. Note that the distances are clear distances between bars, not centre-to-centre dimensions

26.5 Anchorage, minimum area, size and spacing of links

26.5.1 Anchorage of links

All links must be anchored to perform their function. Bearing stresses within the bends are not likely to be excessive, but due to the confined nature of certain reinforced masonry elements, it is suggested that mild steel links be given preference over high tensile steel links.

26.5.2 Beam links

The minimum area and spacing of links should be provided such that:

$$\frac{A_{sv}}{s_v} \geqslant 0.002\, b_t \text{ for mild steel}$$

and

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

The equations differ such that the tensile force provided by the links is approximately equivalent for both mild steel and high yield steel.

Providing the nominal number of links will give a ultimate shear strength over and above that carried by the masonry (i.e., $v - \frac{f_v}{\gamma_{mv}}$) of 0.435 N/mm^2 and 0.480 N/mm^2 for mild steel and high yield steel respectively. The maximum permitted value of v is $\frac{2.0}{\gamma_{mv}}$, i.e., 1.0 N/mm^2 and thus, nominal links will often be adequate even where the masonry shear strength, $\frac{f_v}{\gamma_{mv}}$, is substantially exceeded.

26.5.3 Column links

Links are required in reinforced concrete columns to prevent buckling failure of the reinforcement and bursting of the concrete cover. In most reinforced masonry columns the loads will be sufficiently low and the confinement provided by the masonry sufficiently high to prevent such a failure. Where the area of steel is greater than 0.25% of the area of masonry, A_m, and more than 25% of the axial load capacity of the column is to be mobilised, then this type of failure is considered to be possible. Links are thus required in this situation and the size and spacing of links are specified as follows:

Minimum size of 6 mm at a maximum c/c distance of:

1. the least lateral dimension of the column
2. 50×link diameter
3. 20×main bar diameter

whichever is the smallest (3 will usually be the limiting dimension). Rules are given to ensure that every bar is adequately restrained.

26.6 Anchorage bond

The aim of this clause is to ensure that the forces assumed to be present at the reinforcement level can be safely transmitted to the bars without bond failure occurring. This is achieved by providing a length of bar far enough beyond the point being considered for the calculated stresses in that bar to be developed. The phrase “design loads” refers to the ultimate limit state and commonly the force requird to be “locked-off” will be $\frac{f_y}{1.15}$, for example, over the supports in continuous beams. The lengths of bar required to develop this stress have been provided as Tables 4.5 and 4.6 for different types, sizes and strengths of reinforcement in concrete infill or mortar. Note that for bars in compression, these lengths should be multiplied by 0.83 and that the values have been generated from the general formula:

$$\text{Length of lap} = 0.326\frac{f_s}{f_b} \times \text{bar diameter}$$

where f_s for the tabular values was taken as f_y, the yield stress of steel, but can take any value depending on the force in the bar at the point considered.

Table 4.5: Anchorage bond lengths (mm)* f_y=250 N/mm^2

Bar size (mm)	In mortar		In concrete	
	Plain	Deformed	Plain	Deformed
8	435	325	360	260
10	540	410	455	325
12	650	490	545	390
16	870	650	725	520
20	1090	815	905	650
25	1360	1020	1130	815
32	1740	1300	1450	1040

*based on bar attaining full yield stress (÷ γ_{ms})

Table 4.6: Anchorage bond lengths (mm)* f_y=460 N/mm^2

Bar size (mm)	In mortar		In concrete	
	Plain	Deformed	Plain	Deformed
8	800	600	665	480
10	1000	750	835	600
12	1200	900	1000	720
16	1600	1200	1330	960
20	2000	1500	1670	1200
25	2500	1875	2080	1500
32	3200	2400	2670	1920

*based on bar attaining full yield stress (÷ γ_{ms})

26.7 Laps and joints

26.8 Hooks and bends

26.9 Curtailment and anchorage

It is necessary for a number of reasons to continue bars beyond the point where they are no longer required to resist bending:

1. to allow for variations in load distribution in which case the shape of the bending moment diagram will be different to that calculated
2. to allow for tolerances in the placement of reinforcement
3. if the presence of stirrups would cause stress in the reinforcement at that point to increase to that corresponding to the moment at a section roughly an effective depth (for 45° cracks) away from that point
4. cracks of above average size may well occur at the points where the bars stop, which may locally reduce the shear strength

The minimum extension beyond the theoretical cut-off point is the greater of the effective depth or 12×the bars size to cater for points 1 to 3, whilst the extra provisions (a) to (c) deal with 4. Provisions (a) and (c) control the size of the crack at the cut-off point and (b) ensures that there is a reserve of shear strength. Provision (c) will be the easiest to apply and is recommended for general use. Provision (b) will often apply where low shear stresses are present and any nominal links provided automatically supply excess shear strength. Extra links can be added to comply with (b) but this is not recommended since extra shear calculation will be necessary and the amount and complexity of the reinforcement involved will generally be more than if the main reinforcement is extended to comply with (a) or (c). It should be noted that Clause 26.6 requires that no bars be cut-off less than an appropriate anchorage length from the last point at which it is assumed to be fully stressed. This will, on occasion, override the requirements discussed above.

No guidance is given in respect of curtailment of compression reinforcement or tension reinforcement which extend into compression zones beyond points of contraflexure. Provisions (a) to (c) can safely be ignored here but bars should extend the greater of 12 bar diameters or an effective depth beyond the point at which they are theoretically no longer required. For anchoring bars at simple supports using hooks or bends, reference should be made to Clause 26.8.

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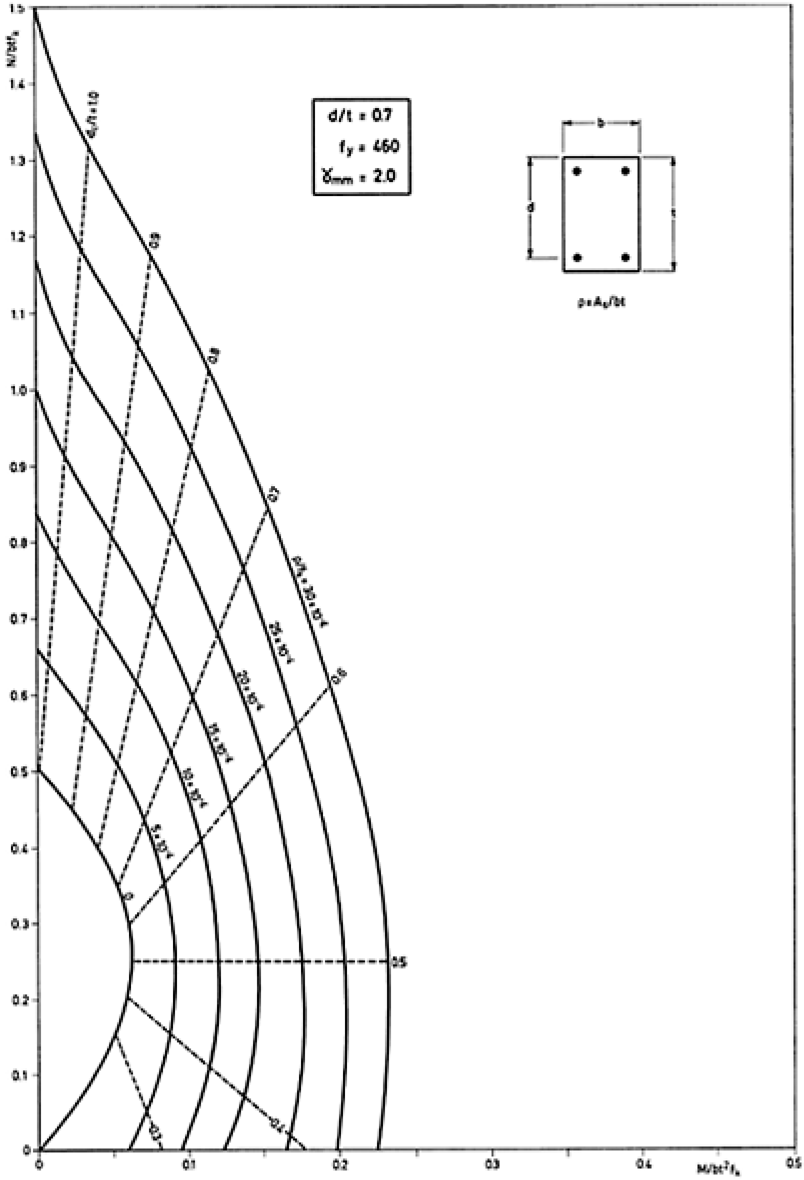
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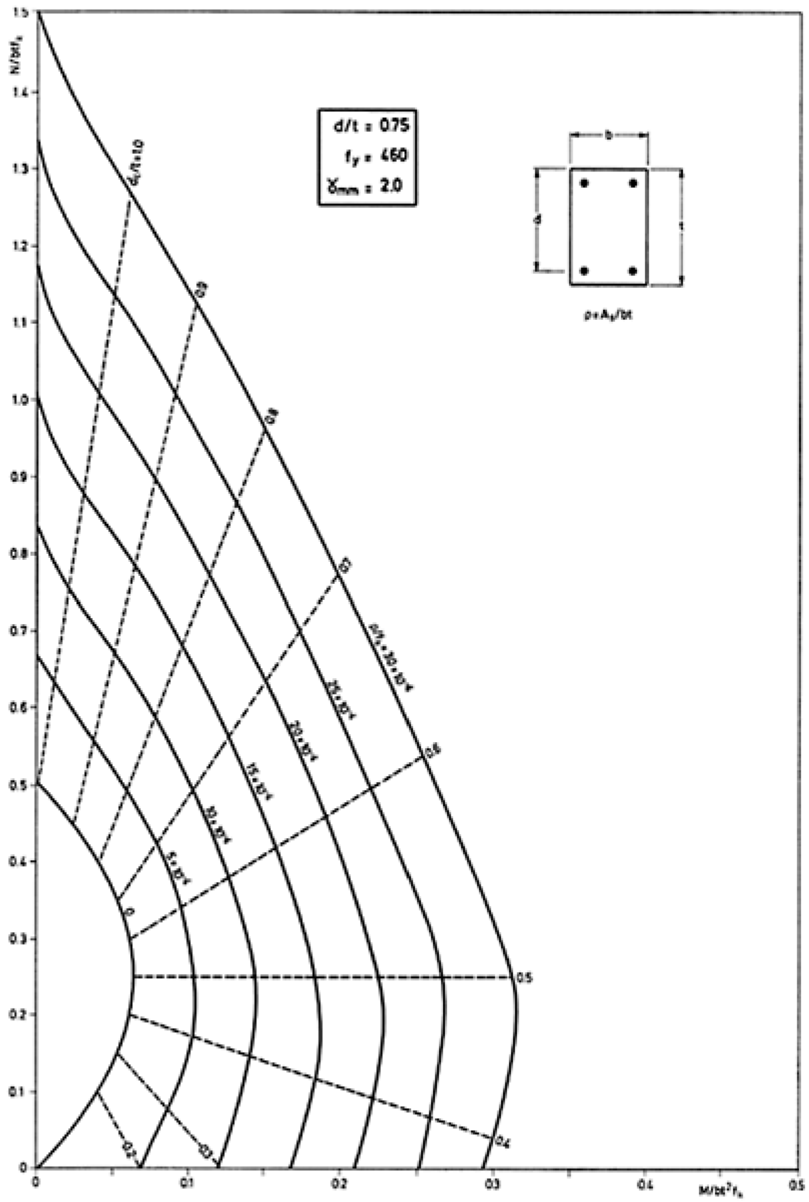
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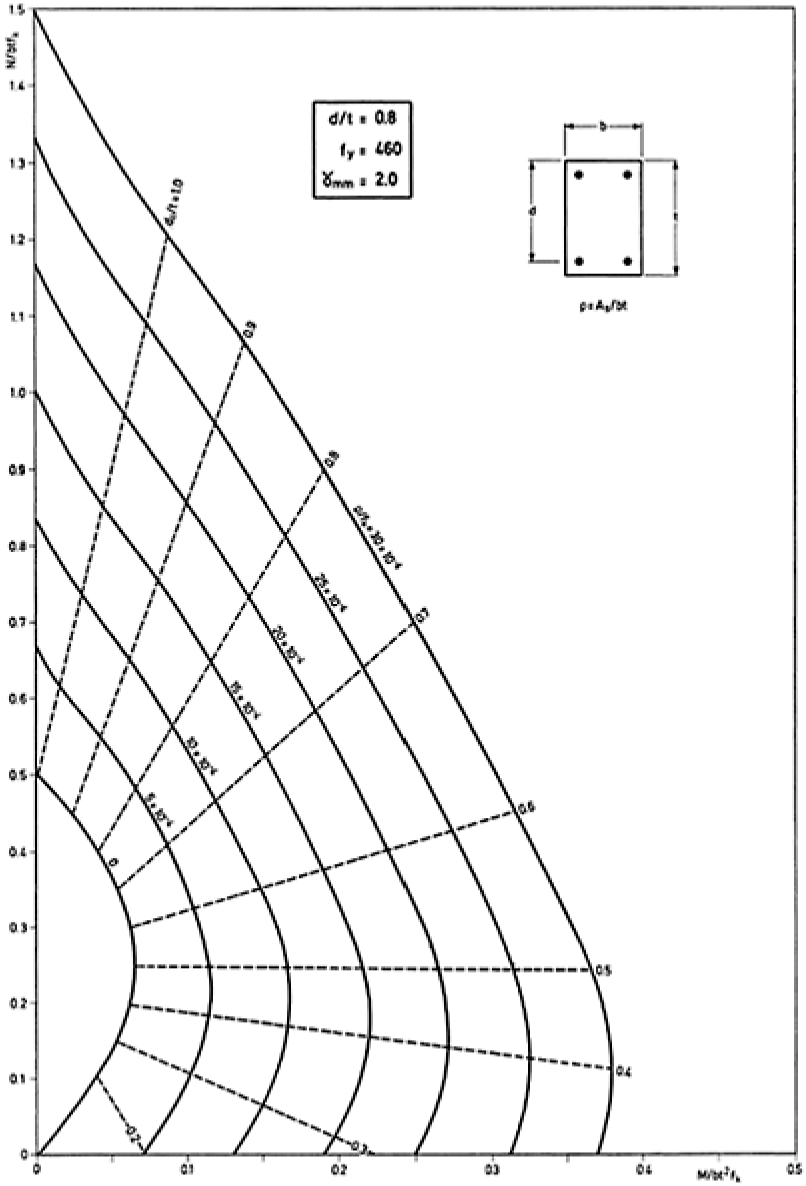
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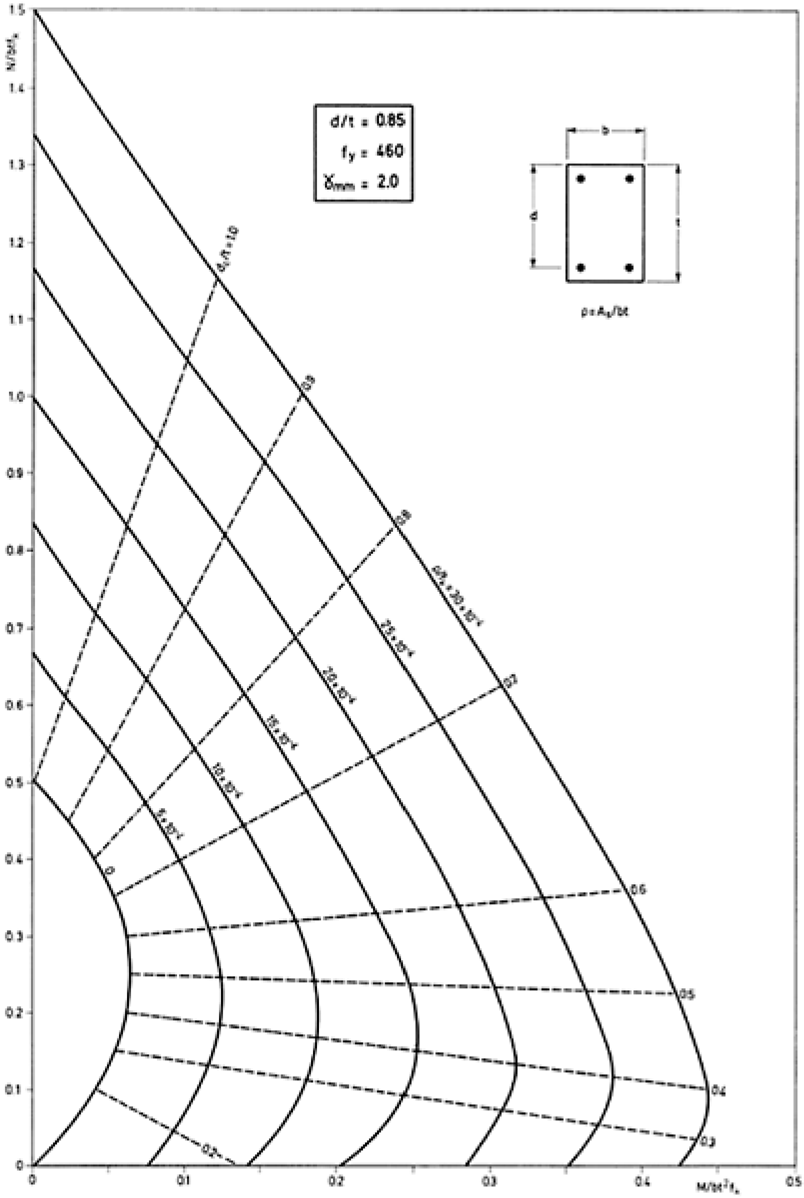
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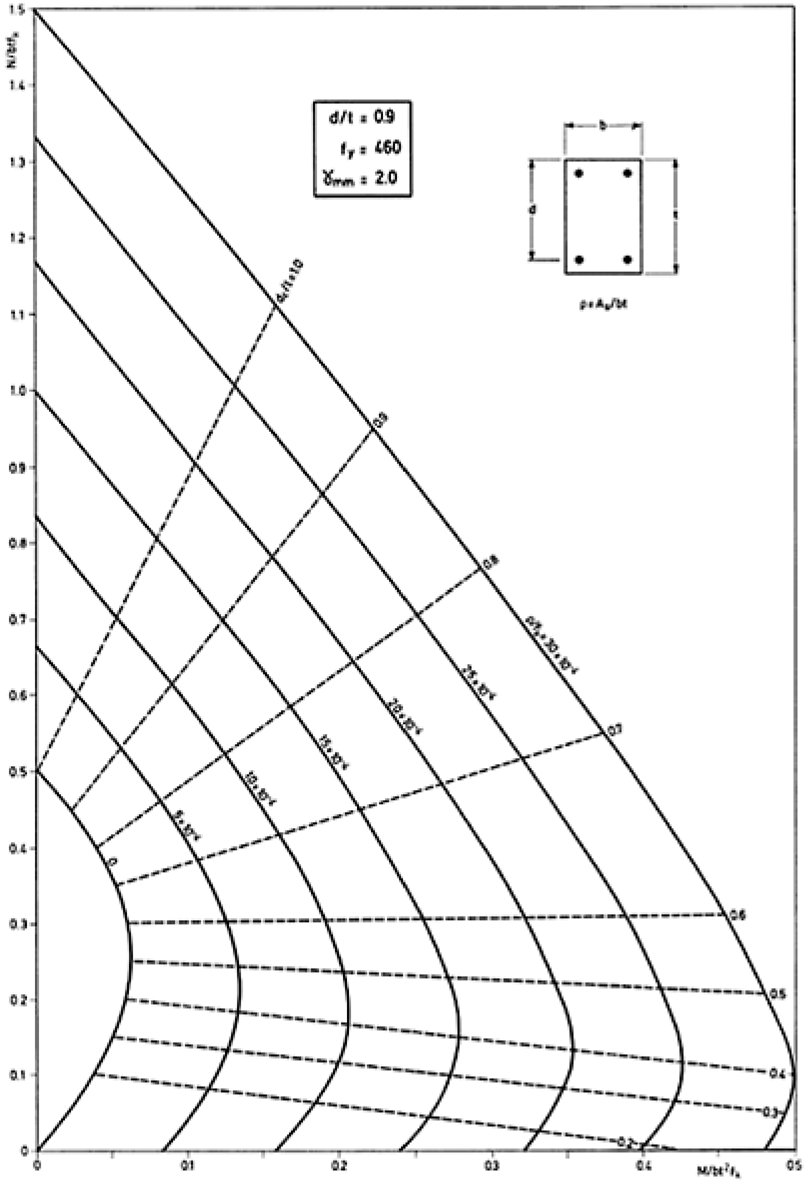
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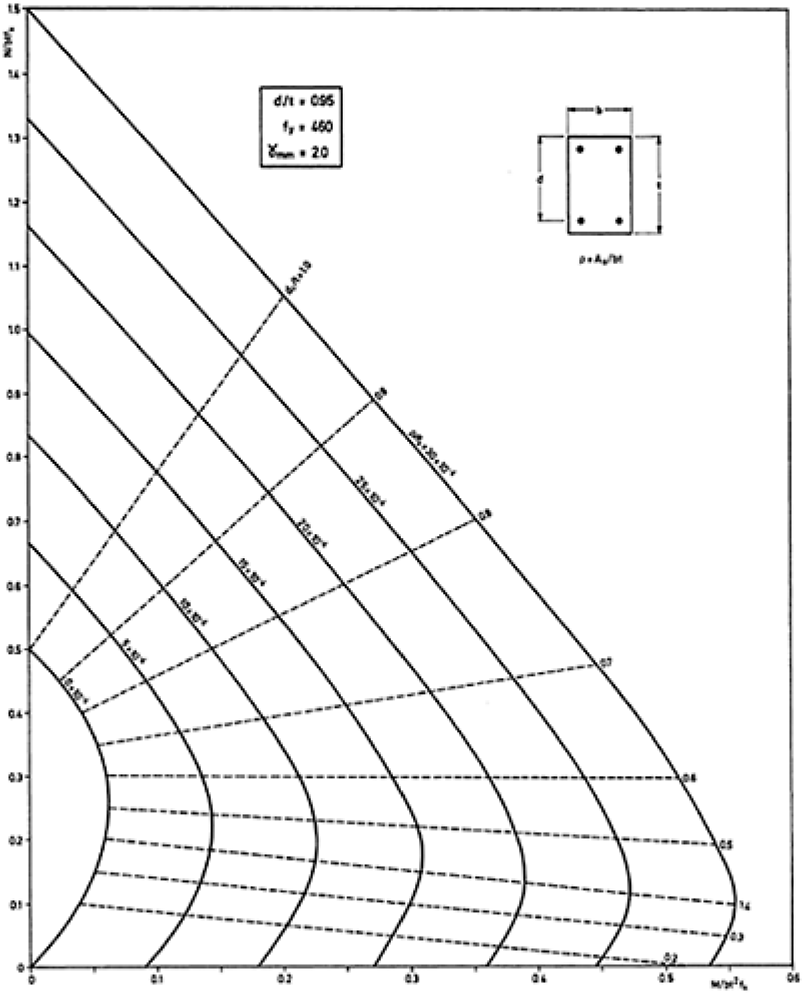
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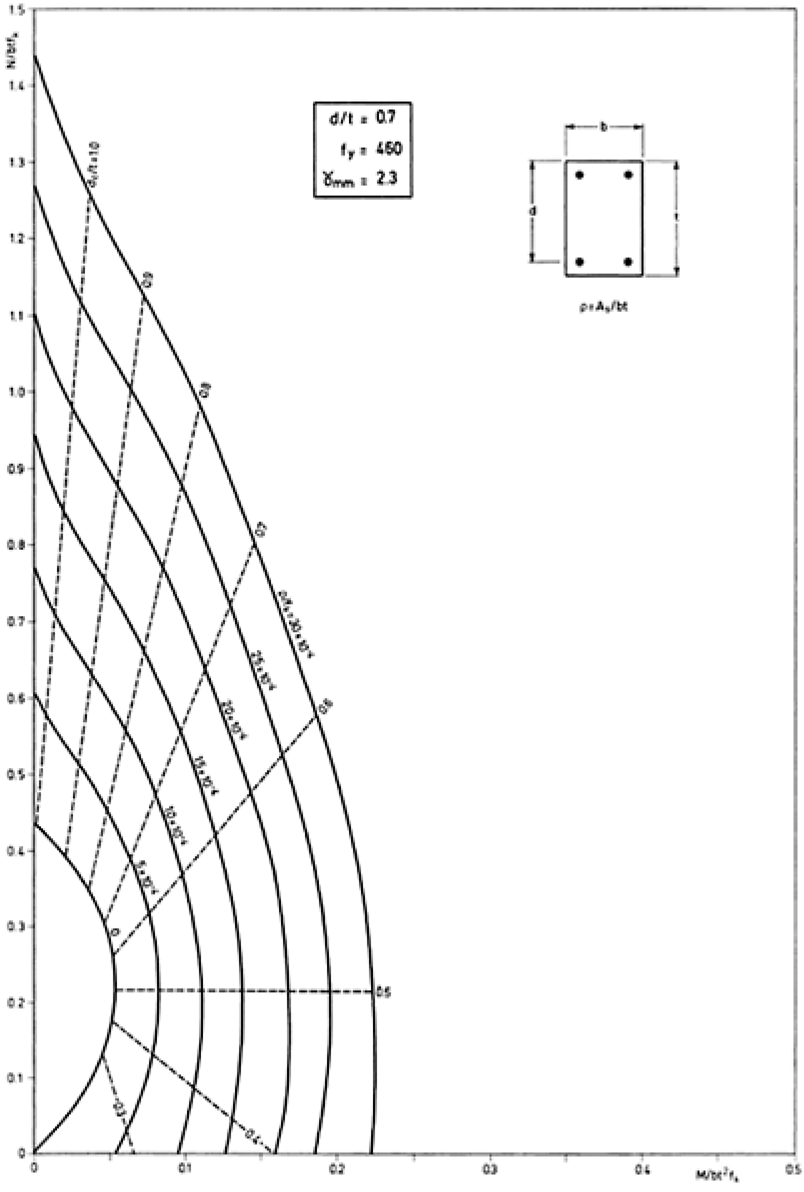
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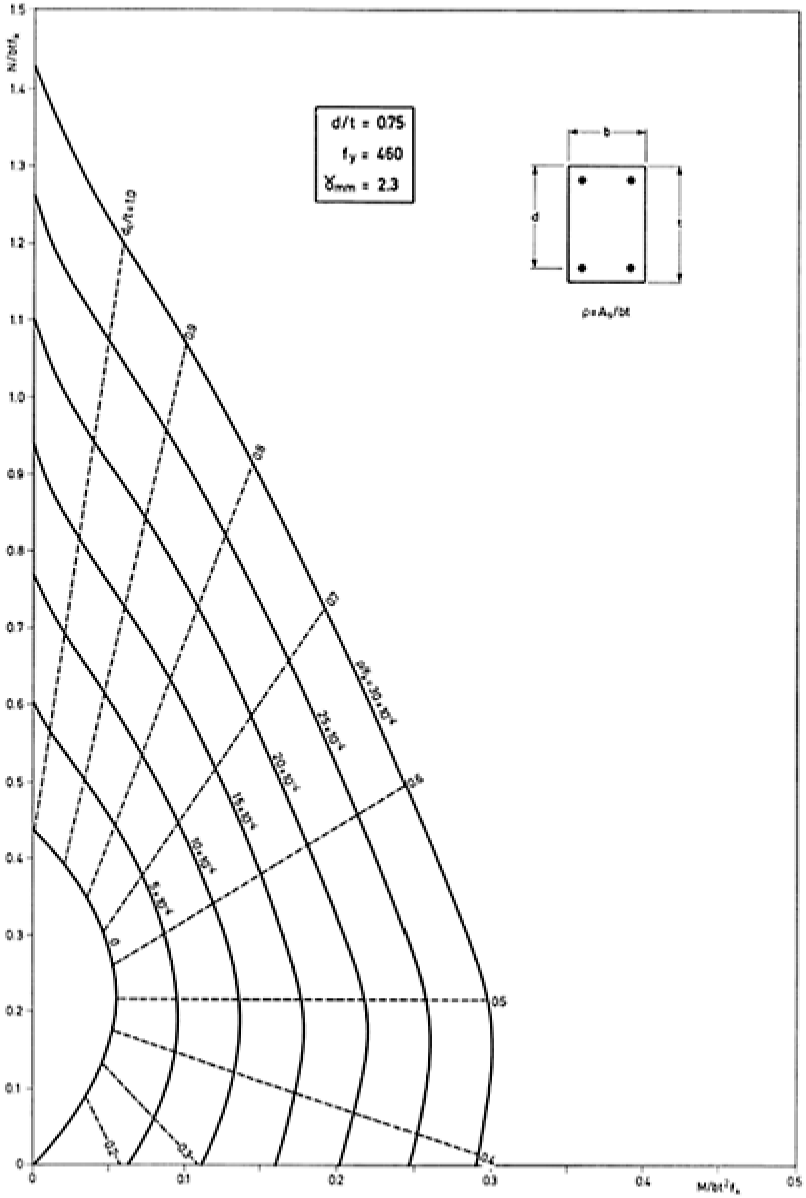
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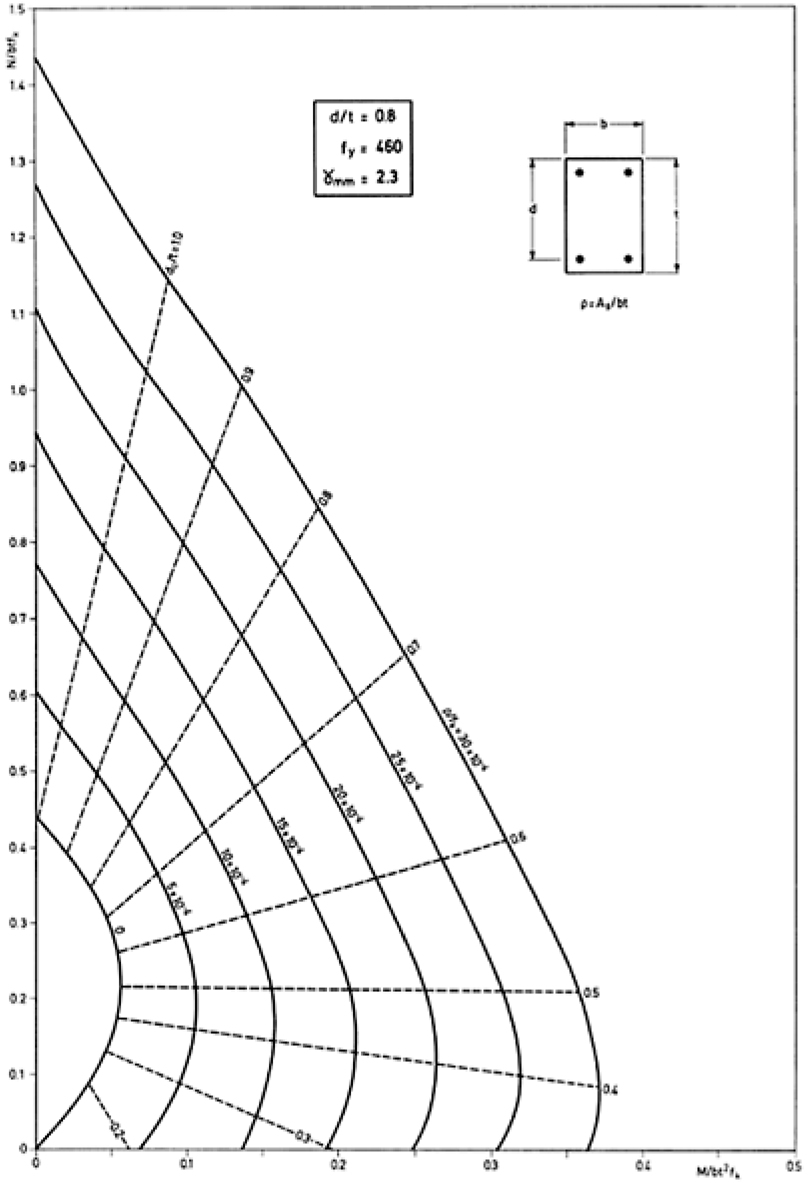
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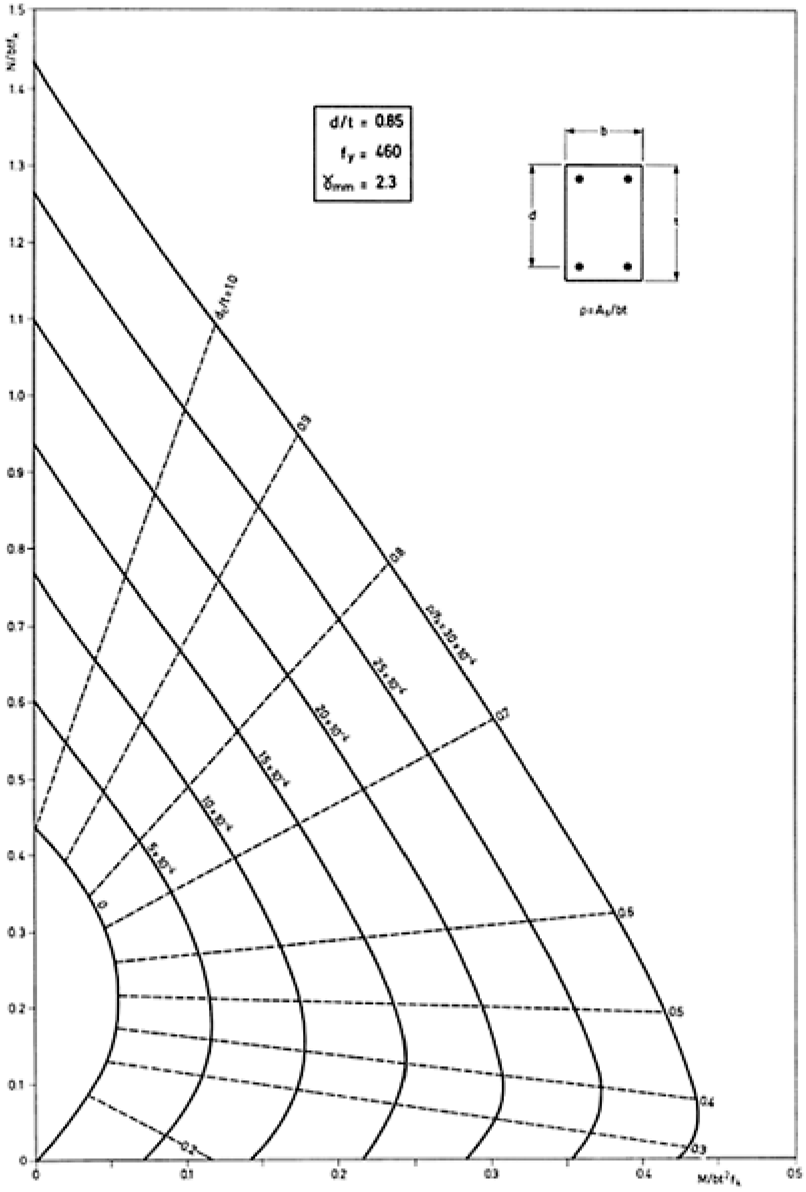
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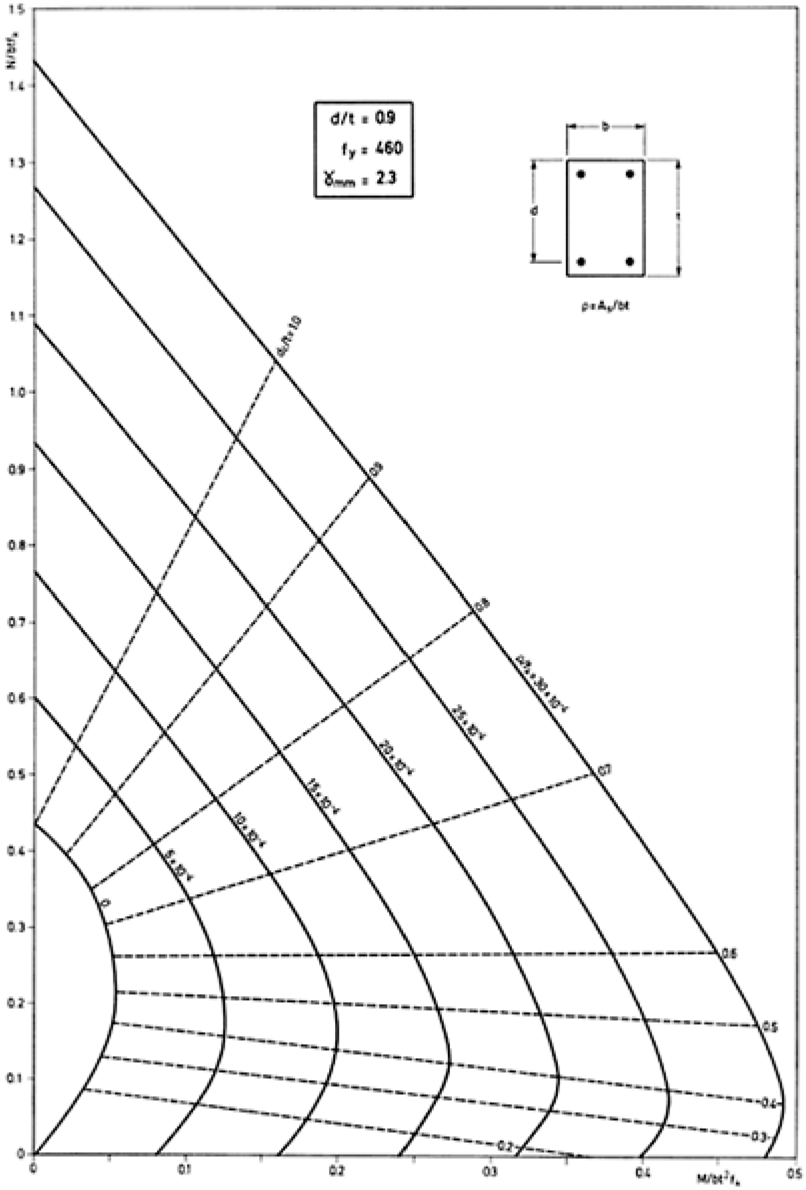
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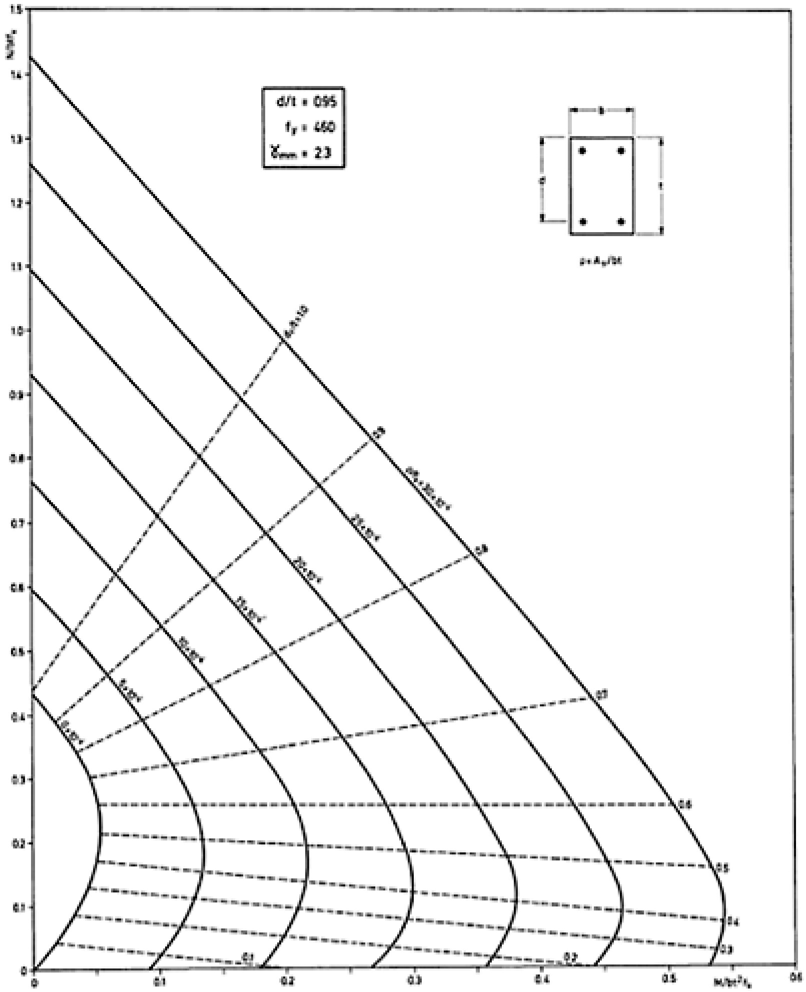
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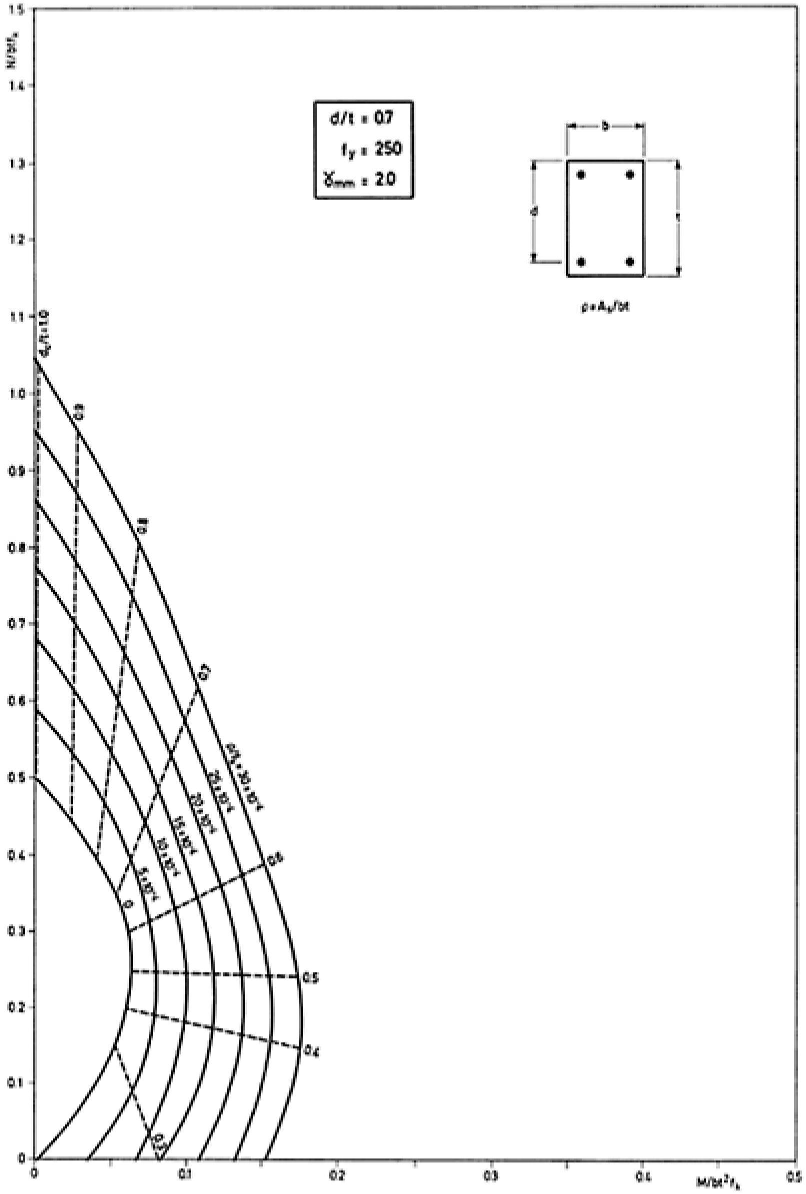
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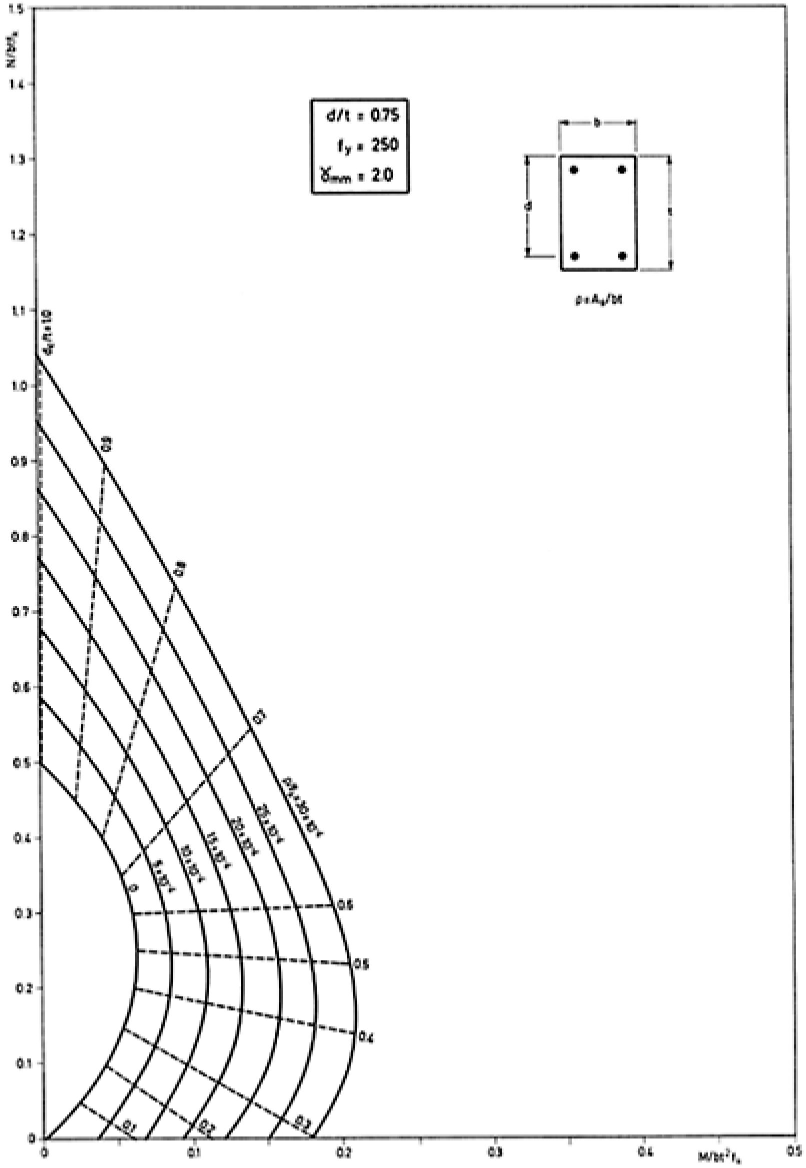
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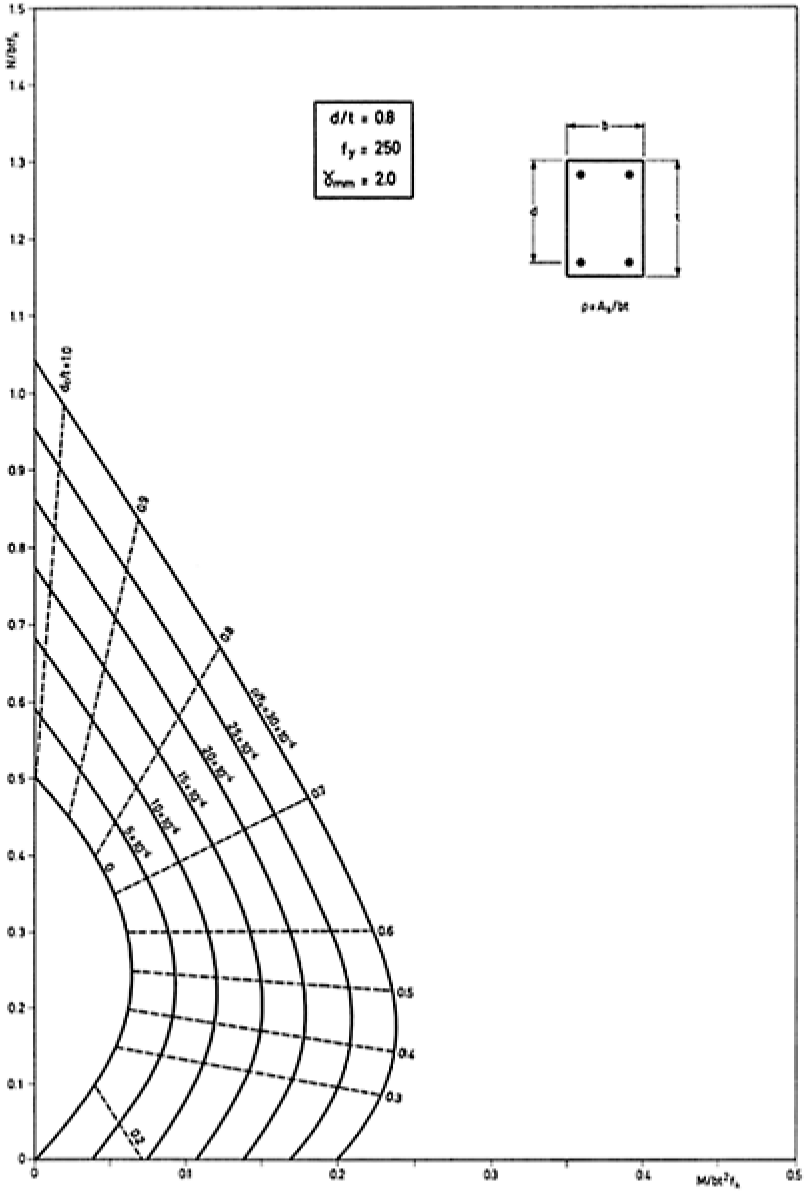
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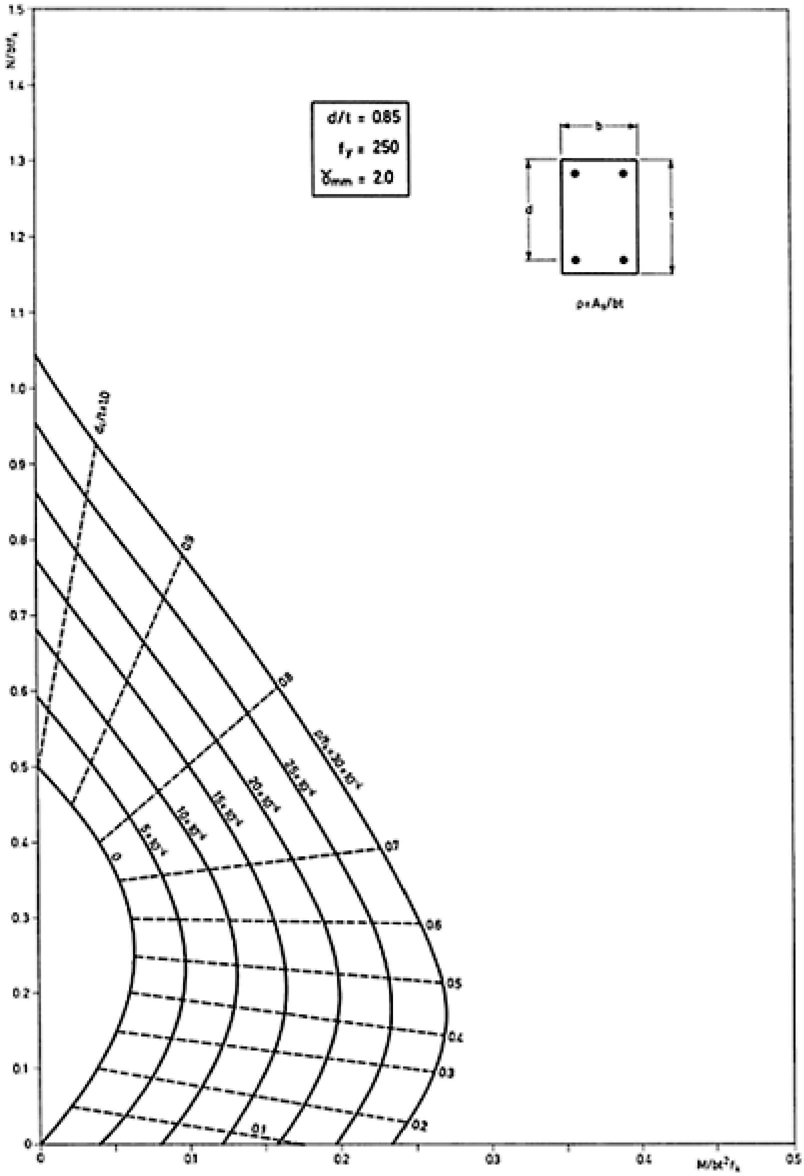
M/bt^2f_k
$p = A_s/bt$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γ_{mm} = 2.0
$\rho = A_s/bt$
$N/bt f_k$
$M/bt^2 f_k$
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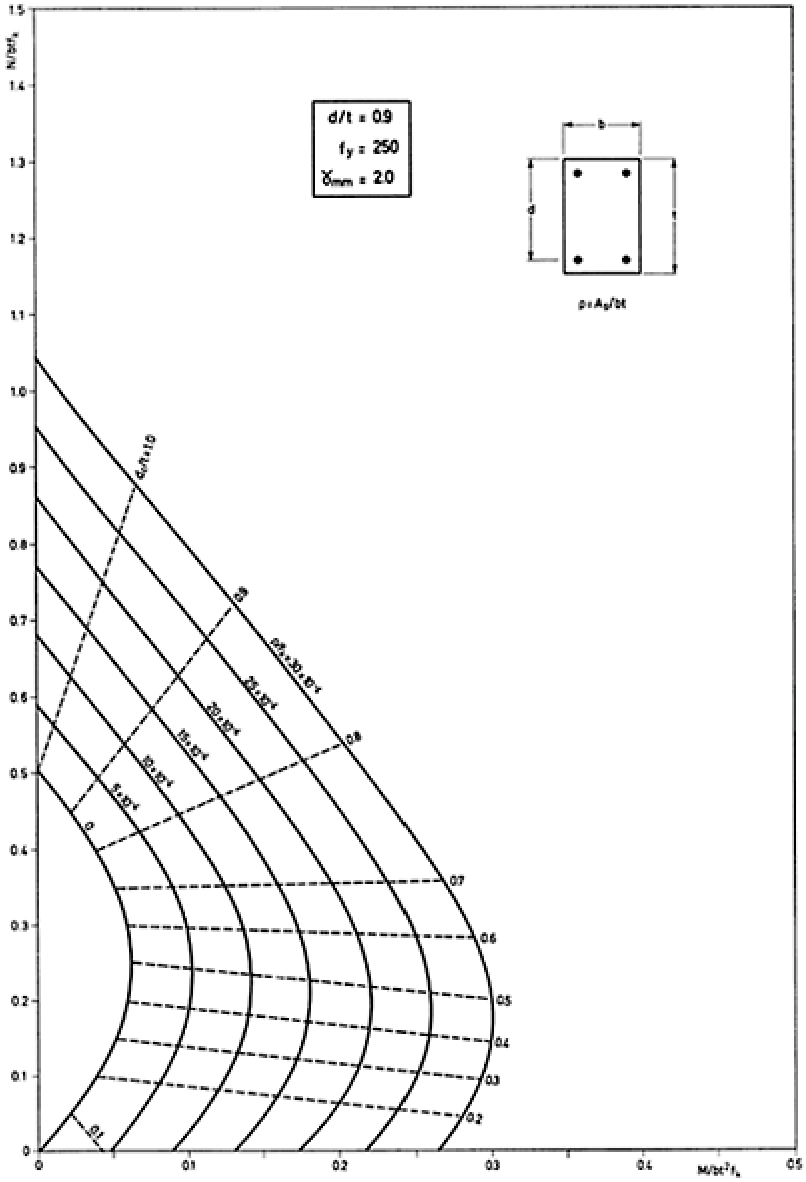
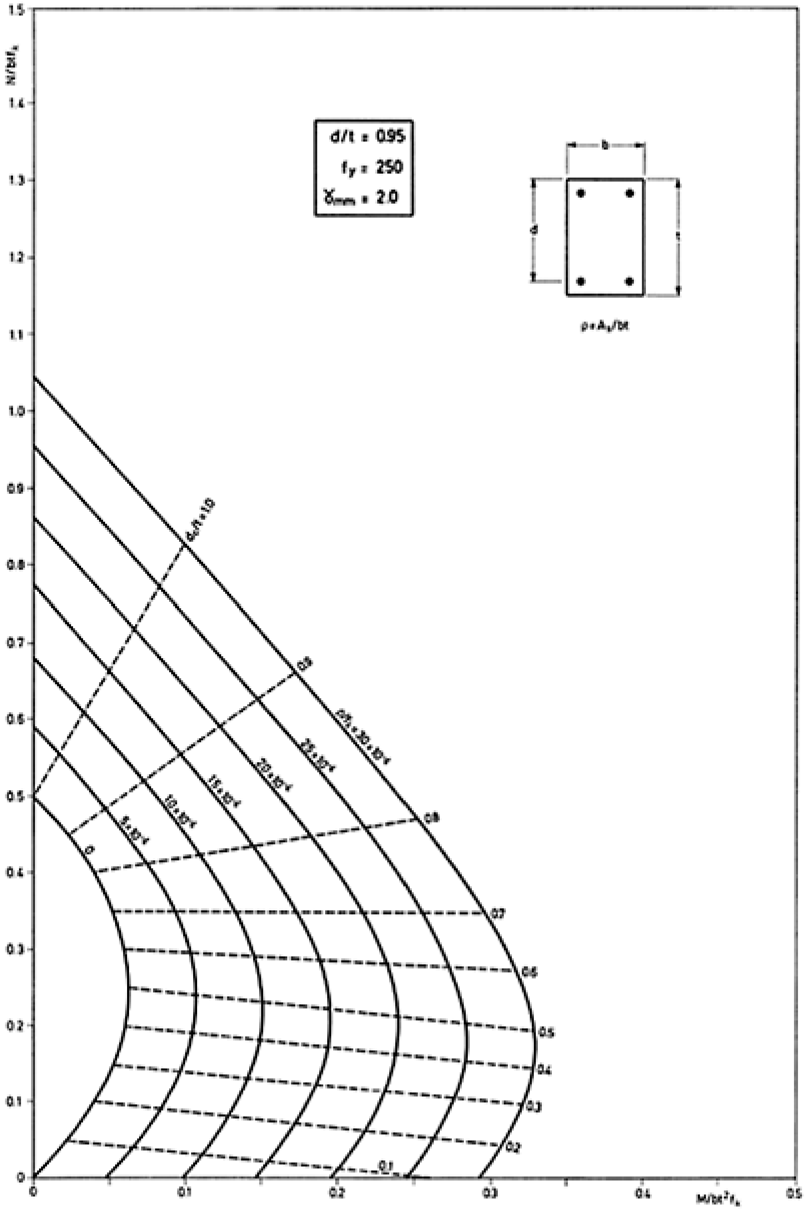
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
1.3
1.4
1.5


d/t = 0.9
f_y = 250
γ_{mm} = 2.0
b
d
t
p = A_s/bt
N/btf_k
M/bt^2f_k
d_c/t = 1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
5×10^{-4}
10×10^{-4}
15×10^{-4}
20×10^{-4}
25×10^{-4}
pf$_y$ = 30×10^{-4}
0
0.1
0.2
0.3
0.4
0.5
1.0
1.1
1.2
1.3
1.4
1.5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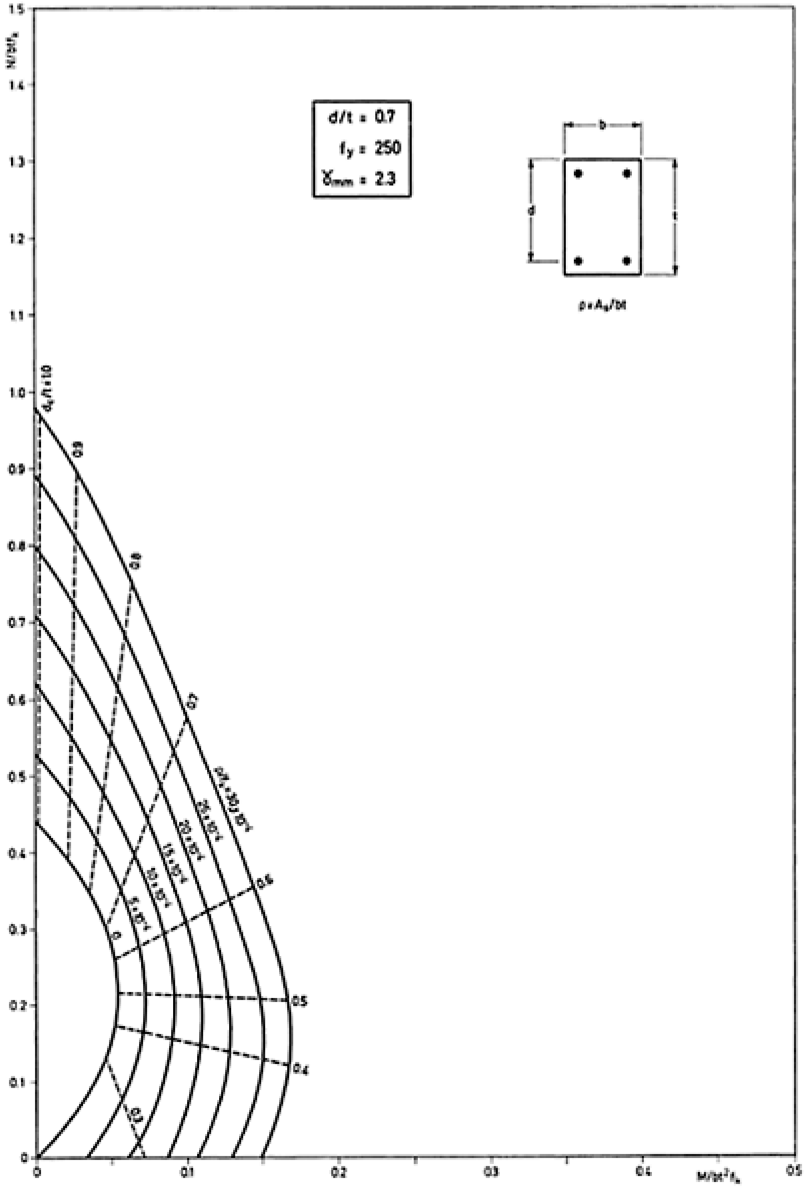
N/btf_k
M/bt^2f_k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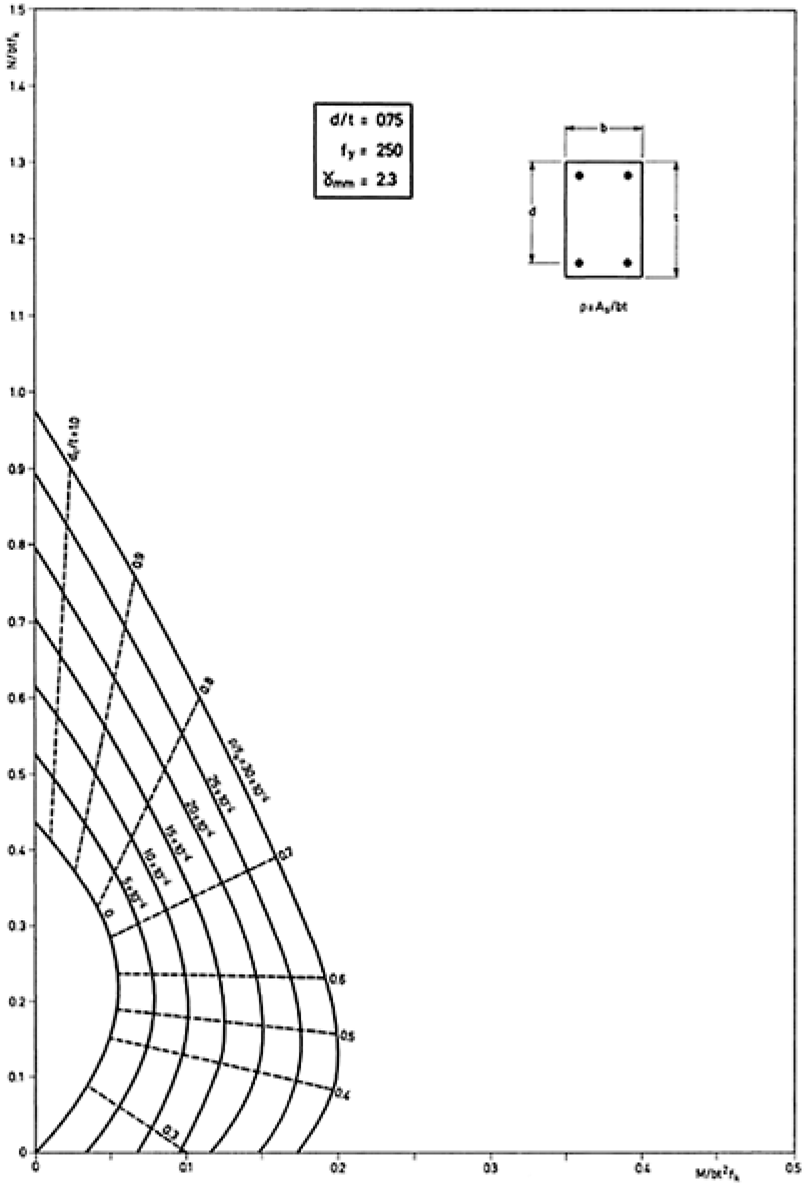
f_y = 250
γ_{mm} = 2.3
b
d
t
$\rho = A_s/bt$
$N/bt f_k$
$M/bt^2 f_k$
$d_n/t = 1.0$
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0
1.5
1.4
1.3
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0.1
0.2
0.3
0.4
0.5


d/t = 0.75
f_y = 250
γ_{mm} = 2.3
b
d
t
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0.7
0.6
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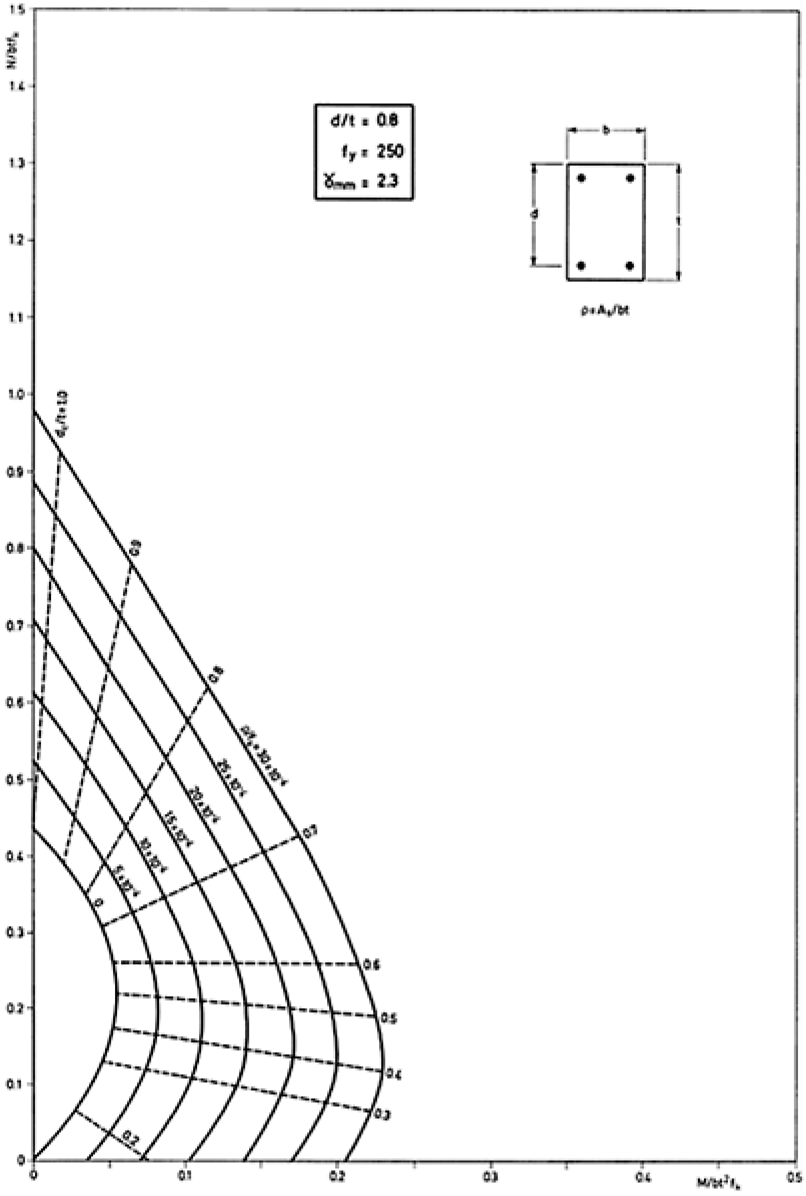
0.4
0.3
0
0.1
0.2
0.3
0.4
0.5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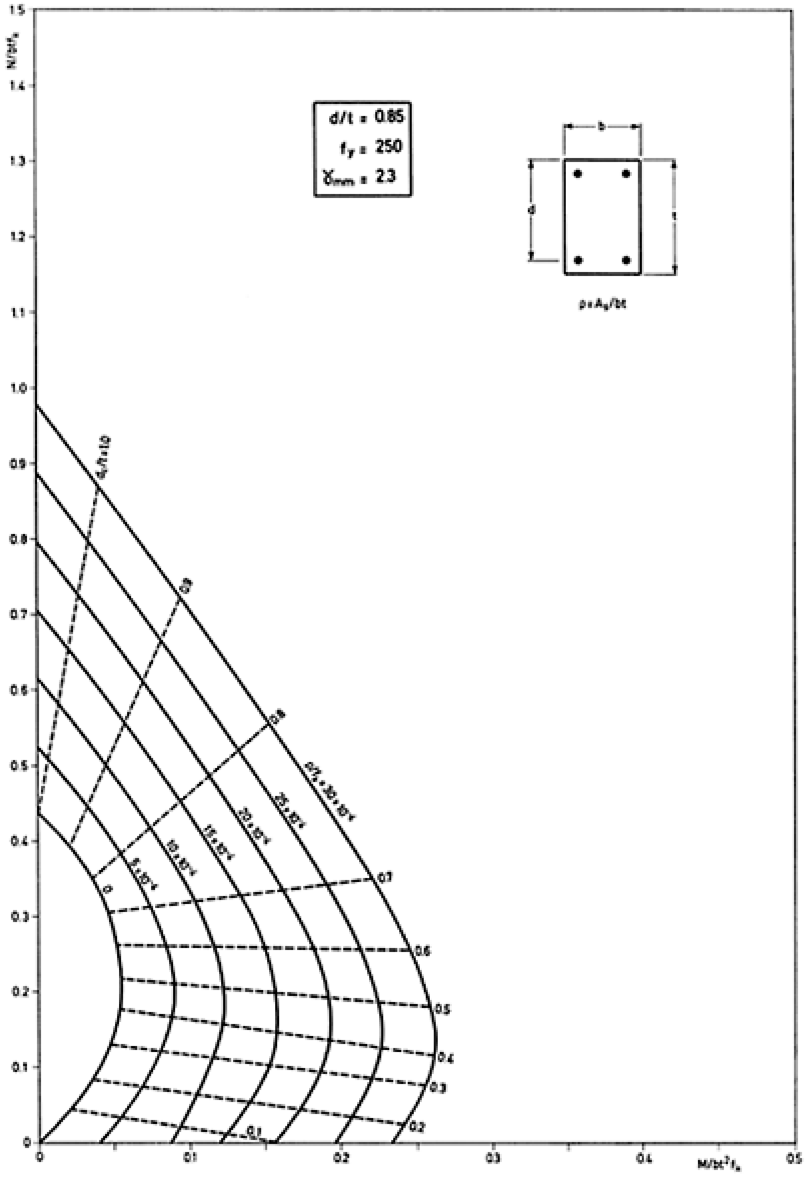
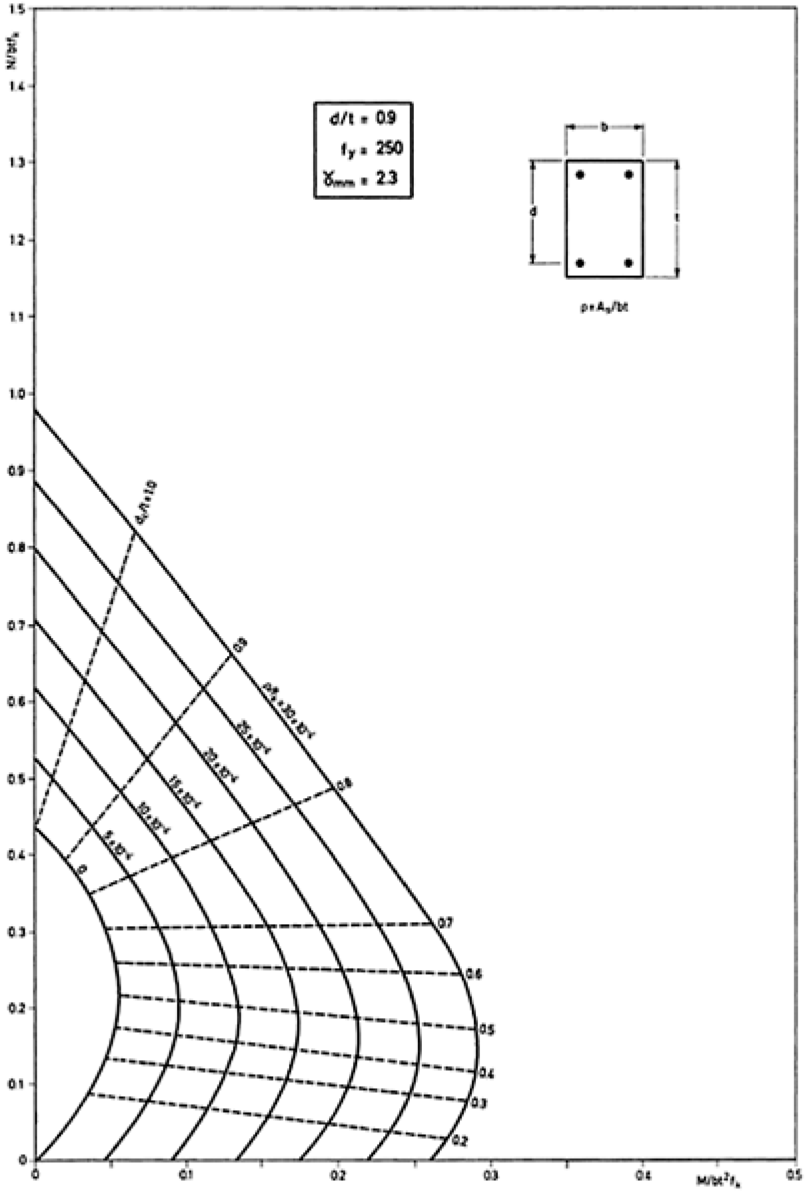
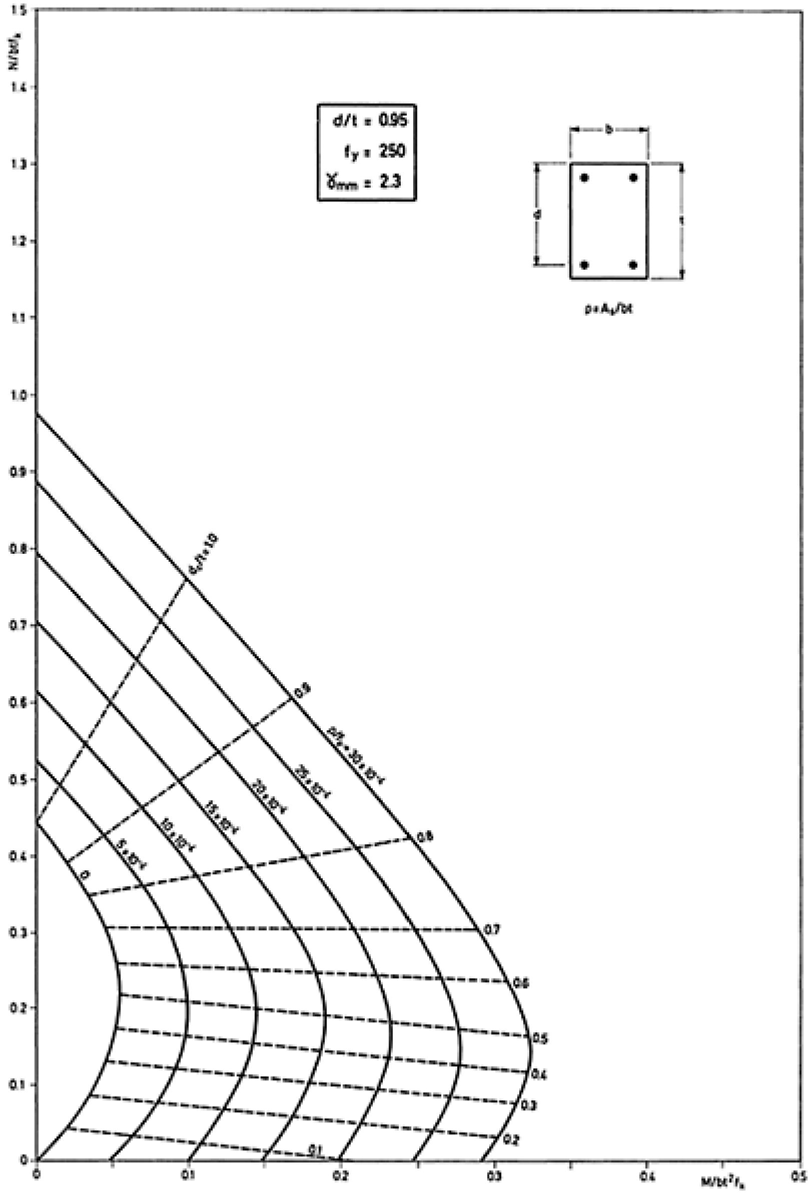
f_y = 250
γ_{mm} = 2.3
b
d
t
ρ = A_s/bt
N/btf_k
M/bt^2f_k
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
1.3
1.4
1.5
d_c/t = 1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
ρf_y = 30 × 10^{-4}
25 × 10^{-4}
20 × 10^{-4}
15 × 10^{-4}
10 × 10^{-4}
5 × 10^{-4}
0


d/t = 0.85
f_y = 250
γ_{mm} = 2.3
p = A_s/bt
$N/bt f_k$
$M/bt^2 f_k$


d/t = 0.9
f_y = 250
γ_{mm} = 2.3
b
d
t
$p = A_s/bt$
N/btf_k
M/bt^2f_k
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
1.3
1.4
1.5
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9


d/t = 0.95
f_y = 250
γ_{mm} = 2.3
$p = A_s/bt$
N/btf_k
M/bt^2f_k

Section Five: Design of prestressed masonry

27. General

As is the case with reinforced concrete, masonry may be either pre- or post-tensioned. Pre-tensioned masonry is probably best suited to the prefabrication of elements in a factory, as in the case of masonry flooring units. The process involves prestressing tendons being tensioned and anchored to independent points either end of the masonry element. In some cases the anchorages will be at either end of a long building and a number of masonry elements will be constructed in a line about the tendons. When the masonry has reached sufficient strength, the tendons are released from the anchorages and the force in them is consequently applied to the masonry through the bond between the tendon and the surrounding concrete or mortar.

Post-tensioning of masonry is better suited to on site construction and involves the force in the tendons being generated simultaneously with the compressive force in the masonry. The masonry is used as the reactor for the device used to stress the tendon. The simplest way to apply the prestressing force is to tighten a nut on the end of a prestressing rod against a plate bearing on the masonry. This method is simple and a calibrated torque spanner may be used to tighten the nut. It is essential to minimise the effects of friction at the anchorage as this could lead to inconsistent results using the spanner and an alternative which has been suggested is to use a specified number of turns of the nut, i.e., apply a fixed displacement. There is a very wide scope for the use of this sort of low technology prestressing. However, the prestressing force which can be applied is limited by the ability of the operator to turn the nut. In the case of the George Armitage Office Block, shown in Figure 5.1,

Figure 5.1 Head office block of George Armitage & Sons plc

20 mm bars were tensioned to a force of 100 kN and this was considered to be the practical maximum.

From the designer's point of view one of the main differences between pre- and post-tensioning is that in the former the elastic compression of the masonry at the moment of transferring the force from the independent anchorages to the masonry causes a reduction in the stress in the tendon which must be considered. In post-tensioned masonry the elastic compression occurs as the tendon is loaded and the force specified can be developed without considering the effects of the strain in the masonry on the force in the tendon.

It is necessary for the designer to consider both the ultimate and serviceability limit states, as for the latter there are recommendations for the maximum compressive stress in the masonry both at transfer and in service, i.e., after any losses have occurred, and it is not possible at the outset to identify which limit state will control the final design.

28. Design for the ultimate limit state

28.1 Bending

The resistance moment of sections is determined using essentially the same philosophy as for reinforced sections. The assumptions made regarding the stresses and strains in the masonry are, as would be expected, similar to those for reinforced sections, which are described in *Section 4*. The exception is that the strain at the outer compression fibre is considered to be at the value regarded as the maximum for reinforced sections, i.e., 0.0035. The assumptions made about the steel behaviour differ slightly in that the stress/strain relationship for the steel must be appropriate to its designation and some typical curves are shown in Figure 5.2. An important distinction is made, dependent upon whether the steel is pre- or post-

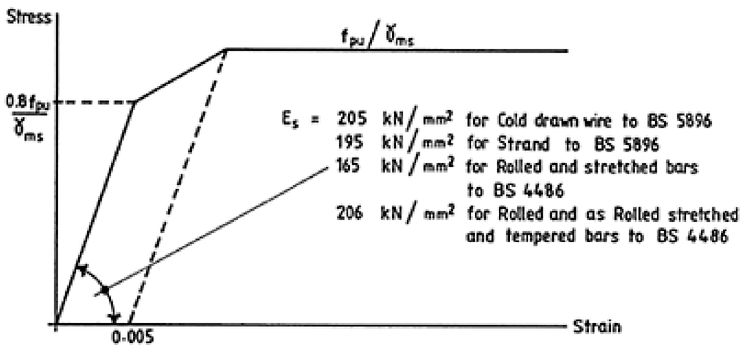


Figure 5.2 Typical short term design stress/strain curves for normal and low relaxation tendons

tensioned. In the latter case the linear deformation of the masonry at the level of the tendons is the same as that of the steel over the whole length of the tendon. However, at any particular section, including that of particular interest where the maximum design moment occurs, the strain in the masonry and steel will differ. The procedure which is adopted in the case where the tendons are bonded is essentially one of trial and error. An assumption is made about the depth of the compression block, d_c, at the ultimate limit state. This depth defines the force in the compression block. Figure 5.3 shows the situation in a rectangular member. From this figure:

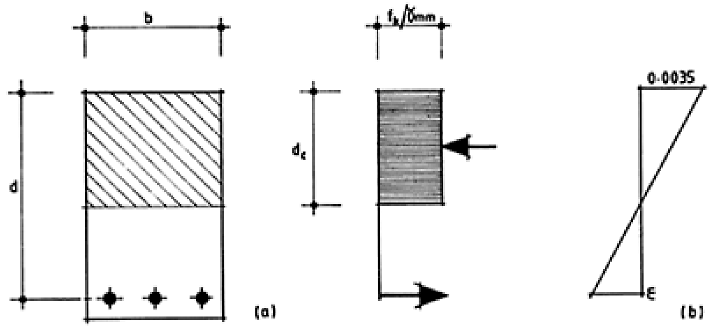


Figure 5.3 Stress and strain distribution in prestressed masonry

due to applied loads (a) stress distribution (b) strain distribution

$$\text{Force in the compression block} = \frac{f_k}{\gamma_{mm}} b d_c$$

This force must be balanced by the force in the tendon, which is defined by the strain due to the prestressing force after losses and the strain, ε, due to the applied loads.

From the assumption that in flexure plane sections remain plane:

$$\varepsilon = \frac{(d - d_c)}{d_c}(0.0035)$$

The total strain in the tendon at failure, ε_{pb}, is given by:

$$\varepsilon_{pb} = \varepsilon + \varepsilon_{pe}$$

where ε_{pe} is the strain corresponding to the effective prestressing force, that is after all losses. The stress, f_{pb}, corresponding to the tendon strain, ε_{pb}, is taken from the stress/strain curve for the tendon.

For internal force equilibrium it should be true that:

$$A_s f_{pb} = \frac{f_k}{\gamma_{mm}} b d_c$$

It may be necessary to make a second estimate of d_c and go through the calculation procedure once again or more until equilibrium of forces is established. The design moment of resistance of the section is then given by:

$$M_d = \frac{f_k}{\gamma_{mm}} b d_c \left(d - \frac{d_c}{2}\right) = A_s f_{pb} \left(d - \frac{d_c}{2}\right)$$

When unbonded tendons are used, the strain in the tendon at the section where the bending moment is a maximum is likely to be less than the strain in the masonry at that level. If it is assumed that the strain in the masonry at the level of the tendon varies along the length of an element in the same way as the bending moment, it is possible to determine the relative strain in masonry and tendon. In the case of a uniformly distributed load on a simply supported beam, the average strain in the masonry is $\frac{2}{3}$ of the maximum value and hence at mid span it is reasonable to take the strain, due to flexure, in the tendon as $\frac{2}{3}$ of the strain in the masonry, derived from the assumption that plane sections remain plane. Whatever the assumption made about the tendon strain, it is recommended that the predicted strain in the tendon at failure is limited in relation to the effective prestress and the assumed depth of compression block. The limiting stress ratios are indicated in Figure 5.4. The values given in this figure do not differ greatly to the empirically determined values in CP 110[1] for a span/depth ratio of 10, but have been chosen simply as what

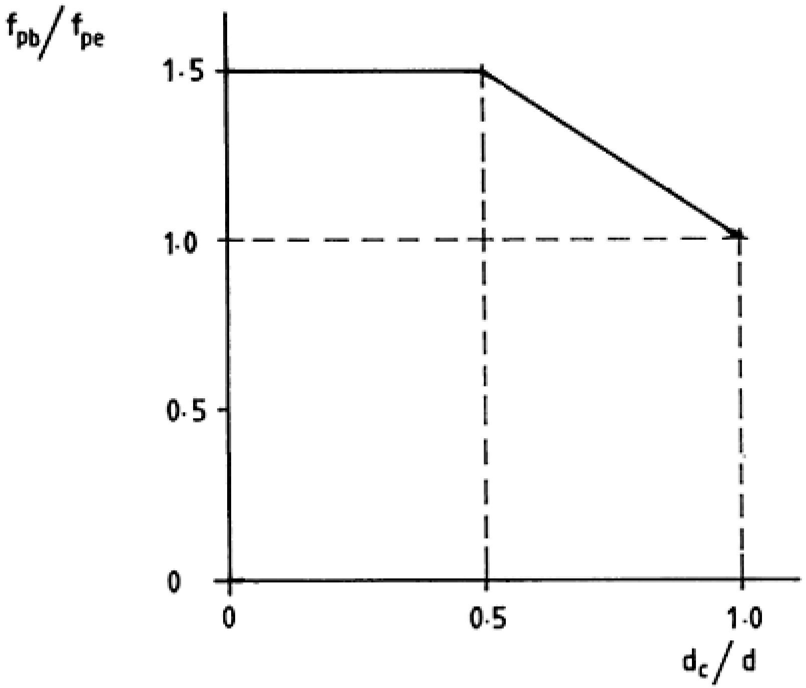


Figure 5.4 Limiting values for stresses in tendons

might be considered reasonable limits with some limited experimental support[2]. The enhancement $\frac{f_{pb}}{f_{pe}}$ assumes that the effective prestress after all losses $\ngtr 0.58\, f_{pu}$ since if this were not so the yield strength, $\frac{f_{pu}}{\gamma_{ms}}$, would be exceeded (1.5×0.58 =0.87) at the ultimate limit state.

Where unbonded tendons are used, the reduction in effective depth of a member due to deflection of the masonry relative to the tendon should be considered or the tendons should be restrained. In the case of diaphragm walls such restraint is relatively straightforward as prestressing bars may run in pvc tubes which are grouted into cores in the web rather than running free in the voids.

28.2 Axial loading

Prestressed masonry elements which are subjected to vertical loads that are either axial or have very small eccentricities (less than 0.05×thickness) are considered as unreinforced masonry and the design is in accordance with Clause 24. In such cases the prestressing force is considered to act as part of the vertical load.

As a general rule it is unlikely that prestressed masonry elements will be used in direct tension, however, where this occurs the design axial load resistance is considered to be that of the tendons. In this case, the prestress in the tendons at any particular stage should be subtracted from the tensile strength to determine the tensile force due to applied loads which can be resisted. One useful application[3] is where post-tensioned columns may be used to support a roof structure which may be subjected to high upward wind forces, for example, where the structure has no walls, as in a market, pedestrian precinct or bandstand.

28.3 Shear

Shear in prestressed masonry is dealt with in a similar way to shear in unreinforced masonry in bending, with the exception that the area considered to be effective in resisting the shear force due to design loads, V, is that in compression alone. Hence, with the usual notation:

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

if

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

then shear reinforcement is required and is designed and detailed as for reinforced masonry.

29. Design for the serviceability limit state

The design for prestressed masonry at the serviceability limit state is based on the assumption that at the stress levels considered the materials behave as if linearly elastic. Loads are generally taken as being the characteristic values (see *20.3.1*) and when calculating deflections or stresses, γ_{mm}, is taken as 1.5 and γ_{ms} as 1.0.

It is necessary to consider the situation at transfer of prestressing force to the masonry and in-service when all losses have occurred. However, the designer should consider whether, for example, in the case of a pre-tensioned element, stresses induced during handling may lead to a more critical situation than either of the usual ones.

A limit is placed on the maximum compressive stress which is dependent on whether the compressive stress is distributed approximately uniformly or triangularly. The values are given in Table 5.1.

Consider the stresses induced in a cellular wall section due to a tendon located at a fixed position in the rib (Figure 5.5):

Table 5.1: Stress limitations

Situation	Uniform distribution	Triangular distribution
At transfer	$0.4\ f_k$	$0.5\ f_k$
After all losses	$0.33\ f_k$	$0.4\ f_k$

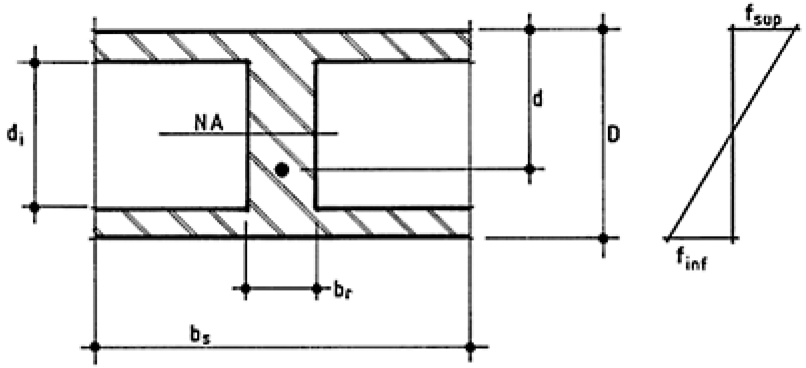


Figure 5.5 Design for serviceability: cellular wall

Prestressing force $=P$ (at time considered)

Area of section $=b_sD-(b_s-b_r)d_i=A$

Moment of inertia $=I$

$$= \frac{b_s D^3}{12} - \frac{(b_s - b_r)\, d_i^3}{12}$$

f_{sup}
$$\frac{P}{A} - P\cdot\left(d - \frac{D}{2}\right)\cdot\frac{D}{2}\frac{1}{I}$$

f_{inf}
$$\frac{P}{A} + P\cdot\left(d - \frac{D}{2}\right)\cdot\frac{D}{2}\frac{1}{I}$$

In this case f_{inf} is compressive and should be checked against the stress taken from Table 5.1 (0.5 f_k or 0.4 f_k respectively). In a real assessment it would be necessary to include load effects, for example, self-weight of wall, which tend to increase f_{inf}.

Where the concrete infill, which is likely to be stiffer than the masonry, occupies more than 10% of the section, the assessment against the serviceability requirements should be made using the transformed section. In addition, where two different masonry types are used in the same wall, particularly when their values of f_k and therefore E_m differ significantly, the transformed section should be used.

It is recommended that the designer should check the deflection of the member at the serviceability limit state. A limit of $\frac{\text{span}}{300}$ has been set for any upward camber before the application of finishes.

30. Design criteria for prestressing tendons

30.1 Maximum initial prestress

As there is much less experience in the use of prestressed masonry than prestressed concrete, many of the requirements have been taken, sometimes directly, from CP 110. This does not mean that they are necessarily less relevant to masonry. For example, the maximum initial prestress is limited by the jacking force being not more than 70% of the characteristic breaking load of the tendon which is the normal limit in prestressed concrete design. In the case of masonry, no reference has been made to the possibility of using a jacking force in excess of this amount as there is in CP 110, and this is consistent with the more limited experience.

30.2 Loss of prestress

30.2.1 General

Generally speaking, it is necessary only to calculate the loss of prestress at the initial condition, that is at transfer, and after all losses have occurred. It is probably fair to say that if losses in prestress are inaccurately assessed the effect will not significantly alter the performance of the element in relation to the ultimate limit state requirements, any errors are likely to be in relation to the serviceability requirements. Although it is fairly straightforward to identify the various reasons for losses in prestress in masonry, no great precision is possible in their evaluation. Consequently the designer must accept that estimations of losses are only approximate and are likely to be conservative. The causes of loss in prestress referred to in the Code are:

1. relaxation of tendons
2. elastic deformation of masonry
3. moisture movement of masonry
4. creep of masonry
5. “draw-in” of the tendons during anchoring
6. friction
7. thermal effects

A warning is also given that in some cases the accumulated losses may significantly reduce the effects of the prestress.

30.2.2 Relaxation of tendons

When steel tendons are subjected to constant high strain, the stress in the steel gradually decreases and the loss of prestress in the masonry due to this must be taken into account. In the case of wire or strand, stress relieving under conditions of plastic deformation may be used to improve the relaxation characteristics and under BS 5896[4] the manufacturer is required to carry out relaxation tests to establish the required design data. In a prestressed masonry structure the strain in the tendon will vary due to time-dependent and load-dependent effects and conse-quently the choice of a figure for the stress relaxation in the tendon is arbitrary. The value chosen in this respect is for a 1000 hour test and data should be available from the manufacturer's test certificate. Alternatively, values are available from the results of tests at 20°C in the relevant British Standard, and are given in Table 5.2.

For initial loads less than 60% of the breaking load, the values are assumed to decrease linearly from the value at 60% to zero at 30% of the breaking load.

Table 5.2:1000 hour relaxation figures (%)

Material	Cold drawn wire or strand to BS 5896[4]		Cold drawn wire in mill coil to BS 5896[4]	Bar to BS 4486[5]
Initial load (% of breaking load)	Relaxation class 1	Relaxation class 2		
60	4.5	1.0	8.0	1.5
70	8.0	2.5	10.0	3.5

30.2.3 Elastic deformation of masonry

The loss of prestressing force in the tendon due to elastic deformation of the masonry depends on whether the tendon is pre-tensioned or post-tensioned. In the case of a pre-tensioned tendon, the loss of stress in the tendon is considered to be equal to the modular ratio times the stress in the concrete infill immediately adjacent to the tendon. It should be realised that the elastic strain developed in this concrete will be dependent on the area and configuration of masonry in the section, although because of the approximate nature of the calculation it will often be adequate to assume a linear stress distribution where the moduli of the materials do not differ significantly. Modular ratios for various combinations of tendon type and 28-day concrete strength are given in Tables 5.3 and 5.4.

In the case of post-tensioned masonry, the elastic strain in the masonry occurs as the tendon is stressed and there is consequently no loss of prestress. If more than one tendon is stressed, then the elastic strain caused by tensioning the steel will cause losses of prestress in previously anchored tendons. If there is no retensioning to offset these losses, they may be taken as half the product of the modular ratio and the stress in the adjacent masonry or concrete. Values for the modular ratio for various tendon types and masonry strengths are given in Table 5.4. Note that the value chosen for f_k must be that appropriate to the age considered.

Generally speaking, if there is a group of tendons it is necessary only to use the masonry or concrete stress adjacent to the centroid of tendons.

Table 5.3: Modular ratios: Concrete infill

28 day cube strength (N/mm^2)		20	25	30	40	50	60
E_m (kN/mm^2)		24	25	26	28	30	32
Steel type	E_s (kN/mm^2)						
Cold drawn wire to BS 5896	205	8.5	8.2	7.9	7.3	6.8	6.4
Strand to BS 5896	195	8.1	7.8	7.5	7.0	6.5	6.1
Rolled and stretched bars to BS 4486	165	6.9	6.6	6.3	5.9	5.5	5.2
Rolled and as rolled and stretched and tempered bars to BS 4486	206	8.6	8.2	7.9	7.4	6.9	6.4

Table 5.4: Modular ratios: Masonry

f_k (N/mm^2)		2	5	10	15	20	25	30	35	38.4
E_m (kN/mm^2)		1.8	4.5	9	13.5	18	22.5	27	31.5	34.6
Steel type	E_s (kN/mm^2)									
Cold drawn wire to BS 5896	205	113	45.5	22.7	15.2	11.4	9.1	7.6	6.5	5.9
Strand to BS 5896	195	108	43.5	21.6	14.4	10.8	8.7	7.2	6.2	5.6
Rolled and stretched bars to BS 4486	165	91.6	36.6	18.3	12.2	9.2	7.3	6.1	5.2	4.8
Rolled and as rolled and stretched and tempered bars to BS 4486	206	114	45.8	22.9	15.2	11.4	9.2	7.6	6.5	6.0

30.2.4 Moisture movement of masonry

In concrete and calcium silicate masonry, the effect of continual migration of moisture into and out of the capillaries of the material leads to an overall shrinkage and consequently a reduction in the prestressing force in the tendons. The reduction of the stress in the tendons is taken as the product of the shrinkage strain and the modulus of elasticity for the tendon which may be taken from Tables 5.3 and 5.4. The value to be taken as the shrinkage strain for both concrete and calcium silicate masonry is 500×10^{-6}. In the case of clay masonry, it is to be expected that the long term moisture expansion would cause an increase in the stress in the tendon. However, it is recommended that this should not be considered in design.

30.2.5 Creep of masonry

Creep of the masonry leads to a reduction in the stress in the tendons and is dealt with in design in the same way as elastic strain. The loss is considered to be a multiple of the elastic strain. The value to be used is given in Table 5.5.

Research results indicate that in the case of brick masonry the creep in single leaf walls and in piers or columns is different. It has been suggested[6] that for clay brick masonry

$$\frac{\text{creep strain}}{\text{elastic strain}} = 4.46 - 0.33\sqrt{\chi}$$

when the length/thickness ratio of the element is greater than 9,

and

$$\frac{\text{creep strain}}{\text{elastic strain}} = 1.73 - 0.14\sqrt{\chi}$$

where the length/thickness ratio is 1. In these formulae χ is a coefficient which may be taken as equal to the brick strength in N/mm^2. Values derived from the formulae tend to be greater than 1.5 for wall sections and less than 1.5 for piers or columns. Bearing in mind the approximate nature of the calculation, the selection of the single value 1.5 is not unreasonable. A value of 2.5 for the strain ratio has been reported in a limited experiment[7] on calcium silicate masonry.

Table 5.5: Factor by which loss is multiplied to give creep loss

Masonry type	Multiplier
Clay or calcium silicate masonry	1.5
Dense aggregate concrete block masonry*	3.0

*No information is currently available on lightweight aggregate block masonry

30.2.6 Anchorage draw-in

The loss of tension in the tendons due to “set” in the grips of the anchorage system used in post-tensioning is more important in short members. The allowance which should be made for any particular system can generally be obtained from the manufacturer.

30.2.7 *Friction*

Where tendons are placed in ducts such that they may be in contact with the duct or any spacers, it will be necessary to allow for any reduction in the force in the tendon due to friction. Little information is available which is related strictly to masonry and it is recommended that the information in CP 110* be used. This is simply that (a) friction in the jack and anchorage will be dependent on the jack pressure and needs to be established for any particular system, and (b) friction due to unintentional variation of the duct from the specified profile is considered to vary exponentially with the distance from the jack. The formula to be used in the

*Section 4.9 of BS 8110: Part 1:1985.

latter case is:

$$P_\chi=P_0e^{-K\chi}$$

where:

P_χ=the force in the tendon at distance from the jack

P_0=the force in the tendon at the jack

e=the base of Napierian logarithms (2.178)

K=a coefficient dependent on, among other things, the type of duct

The value chosen for K in concrete practice should normally exceed 33×10^{-4}. In many cases in masonry construction where the tendons are vertical it may be possible to justify the use of a lower value.

30.2.8 Thermal effects

It is necessary to allow for any thermal movement of the masonry relative to the tendon. The change in stress in the tendon will be the relevant elastic modulus taken from Tables 5.3 and 5.4 and the strain due to thermal expansion or contraction. The change in strain in the masonry is the product of the coefficient of linear thermal expansion and the mean change in temperature. BS 5628: Part 3[8] gives data on the coefficients of linear thermal expansion for masonry made from various units. These values are given in Table 5.6.

The values given are for masonry units and the coefficient of linear thermal expansion for the masonry parallel to the bed joints is taken to be the same. The Code gives a value of $11-13\times10^{-6}$ for the coefficient of linear thermal expansion of mortar and this is used when assessing the thermal expansion perpendicular to the bed joints. In this case the movement is considered to be the sum of the expansions of the mortar and the units calculated using the relevant coefficient.

It is important to realise that the change in temperature is not necessarily the change in air temperature. For example, the surface temperature of a South facing brickwork wall could be very high, say 40°C. A diaphragm wall which was post-tensioned at this temperature could contract by an amount equivalent to a change in tendon stress of 300 microstrain when the temperature fell to 0°C. The accompanying loss in prestress could be 8% or 9% of the applied stress. Clearly this is an extreme example, but where low prestressing forces are used, particular care must be taken.

Table 5.6: Coefficients of linear thermal expansion for masonry for various unit types

Material	Coefficient of linear thermal expansion$\times10^{-6}$ per °C
Fired clay bricks and blocks	4–8

Concrete bricks and blocks	7–14
Calcium silicate bricks	11–15

30.3 Transmission length in pre-tensioned members

Relatively little is known about the required length of a member to transmit the force from the tendon to concrete infill in prestressed masonry compared with that for prestressed concrete*. Consequently, although the factors on which the length depends are likely to be similar to those for prestressed concrete (and some further advice may be found in CP 110), the Code recommends that in the absence of experimental data where the initial prestressing force is less than 75% of the characteristic strength of the tendon, the value should be derived from the formula:

$$l_t = \frac{K_t \phi}{\sqrt{f_{ci}}}$$

*In the early days of prestressed concrete there were a number of failures which were overcome by trial and error. This experience has not been fully codified in CP 110 and for pre-tensioned masonry members it is suggested that trials be conducted on prototypes of proposed sections.

where: l_t=the transmission length

ϕ= the nominal diameter of the tendons

f_{ci}=the strength of the concrete or grout at transfer

K_t=the coefficient to allow for the type of tendon

The number of diameters required for various types of tendon and concrete or grout strength at transfer is given in Table 5.7.

Table 5.7: Number of diameters transmission length required

Concrete or grout strength at transfer N/mm^2	Plain or indented wire Crimped wire with small wave height (K_t=600)	Crimped wire with wave height <0.15 ϕ (K_t=400)	7 wire standard or super strand (K_t=240)	7 wire drawn (K_t=360)
25	120	80	48	72
30	100	73	44	66
35	101	68	41	61
40	95	63	38	57

31. Detailing prestressed masonry

31.1 Anchorages and end blocks

The situation beneath an end block in a prestressed member is considered to be similar to that in unreinforced masonry when a concentrated load is applied. Some local overstressing of the masonry is permitted beneath the end block and the designer should ensure that the average stress is less than 1.5 $\frac{f_k}{\gamma_{mm}}$ or 0.65 $\frac{f_k}{\gamma_{mm}}$ depending upon whether the stress is normal or parallel to the bed joints.

In an end block it would be normal to provide reinforcement to carry all of any bursting tensile stress. The force considered is the jacking force or the tendon force at the ultimate limit state, whichever is the more critical. In this design it would be normal to assume that the force is being applied under carefully controlled conditions and so no partial safety factor would be applied to the force in the tendon. Where end blocks or bearing plates are of a different shape to the member it may be necessary to provide reinforcement to resist flexural or shear stresses.

31.2 Tendons

As previously mentioned, where more than one tendon is to be stressed, this may affect the way the designer deals with elastic losses at transfer. It is consequently important for the designer to consider the sequence to be used in loading the tendons. It is also necessary in this context to ensure that the masonry is not overstressed.

Tendons which are surrounded by concrete should not be placed so close to one another that it is difficult to compact any concrete about them. A minimum spacing of the maximum aggregate size plus 5 mm has been recommended.

31.3 Links

Column links, where necessary, should be provided using the same criterion as for reinforced masonry. In this case the prestressing force should be considered as part of the design load, although it should not be subjected to a partial safety factor for loads.

References

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6. LENCZNER, D and FOSTER, D. *Strength and deformation of brickwork prisms in three directions.* Proceedings 5th IBMAC, Paper II-4. p. 49.
7. BOOKS, J J. *Time dependent behaviour of calcium silicate and fletton clay brickwork walls.* Paper presented to 8th International Loadbearing Brickwork Symposium, London, 1983.
8. BRITISH STANDARDS INSTITUTION. BS 5628: Part 3:1985 *Code of Practice for the use of masonry.* Part 3: Materials and components. Design and workmanship. BSI, London, pp. 100.

Section Six:
Other design considerations

32. Durability

32.1 Masonry units and mortars

The Code refers the designer to BS 5628: Part 3[1] for guidance on durability of masonry units and mortar. To assist the designer, however, some general notes are given here, although where any doubt exists or is indicated, then the fuller guidance in Clause 22 of BS 5628: Part 3 should be consulted. The guidance given relates to the masonry itself and not to the durability of reinforcement, which is covered later.

There are a number of factors which will affect the durability of masonry, including

1. the degree of saturation
2. the potential for frost attack
3. the susceptibility to sulphate attack
4. the characteristics of the unit

A major factor influencing the durability of masonry is the degree to which it becomes saturated with water. It may become saturated directly by rainfall, indirectly by water moving upwards from the foundations, or laterally from retained material as in a retaining wall. Particular attention should be paid to the choice of masonry units and mortar in the following, and similar, situations, where the masonry is likely to become, and may remain, saturated for long periods:

in sills, copings and cappings
in parapets, freestanding and retaining walls
below dpc, at or near ground level and in foundations.

The durability of masonry depends upon:

1. Exposure to the weather: The Local Spell Index[2] or the Driving Rain Index[3] is a good indication of the general exposure of the site and, therefore, the amount of rain which is available to saturate the masonry. It must be noted, however, that different parts of the same structure may be subjected to different degrees of exposure.
2. The adequacy of design details and methods taken to prevent the masonry becoming saturated: External masonry is much less likely to become saturated where projecting features have been provided to shed run-off water clear of the walling, for example:
   protection to wall heads by roof overhangs or projecting throated copings
   projecting throated sills
   bell mouths to rendering, tile hanging and so on

On the other hand, where these features are not incorporated, for example, where flush copings are used, increased wetting and potentially longer periods of saturation will occur. External masonry will be maintained in a drier condition by a moderately porous uncracked rendering. On the other hand, dense rendering may lead to entrapment of moisture if imperfections develop or if water is able to get behind the finish via any path. Depending on the masonry substrate, this could lead to frost or sulphate attack.

3. Exposure to aggressive conditions:

   a) Frost attack: if freezing occurs either during construction or shortly after completion of the work, frost may cause damage to mortar and even to the masonry units themselves, depending on their type and whether they become saturated. Damage is caused by the volumetric expansion which occurs on the formation of ice within the saturated masonry in freezing conditions. It is important, therefore, to protect stored masonry units and newly erected masonry adequately, both from saturation and from frost. Frost attack can also occur later in the life of the structure, although the actual process by which this occurs is very complex. Neither strength nor water absorption are reliable guides for the assessment of resistance to freezing of clay bricks and if they are to be used in situations where saturation and subsequent freezing are likely, their frost resistance should be in accordance with the requirements for bricks of Special Quality, as specified in BS 3921[4]. Many clay bricks of Ordinary Quality and most engineering bricks are, in fact, frost resistant. As there is no standard test, the designer should consult the brick manufacturer when using other than Special Quality bricks in situations where cyclic freezing of the saturated brickwork may occur. Evidence as to the potential durability of particular brick types may, of course, be found by examining buildings in the locality or exposure panels which manufacturers often have available for inspection. Bricks of Ordinary Quality may clearly be used in locations where saturation and freezing do not occur at the same time. Precast concrete masonry units possess good frost resistance.

      Additional consideration should be given to the choice of any masonry unit and mortar if the walling is liable to be splashed with de-icing salts or if the structure is to be located in conditions of extreme exposure to weather, for example, on the coastline.

   b) Sulphate attack: sulphate attack on set mortars is generally due to the expansive reaction of tricalcium aluminate in cement with calcium, sodium or potassium sulphates to form Ettringite.

      Sulphate may derive from certain clay bricks if they have become sufficiently wet to allow any sulphates to be dissolved and for these subsequently to migrate to the mortar joints. Sulphate attack for this reason is not likely in internal work or external work which is not subjected to prolonged saturation and in these situations Ordinary Quality bricks may be used. If, however, the brickwork is likely to remain saturated for lengthy periods, the soluble salt content of the bricks used could be limited by selecting bricks to the Special Quality classification in BS 3921. Alternatively in such conditions, bricks of

Ordinary Quality may be used provided that the mortar is made from sulphate-resisting cement.
Sulphates may also be derived from the ground, ground-water, hardcore or fill. The amount of sulphates should be limited and guidance on the limits for the use of concrete in such situations is given in CP 110[5] and BRE Digest 250[6]. The designer should also consider whether advice is required from the manufacturer of the masonry units or from other authoritative bodies, for example, BRE, C & CA, BCRA, BDA, AACPA, ACBA.

4. The characteristic of the masonry units and mortars: Tables 6.1 and 6.2 give general guidance only and do not necessarily give the minimum requirement (see BS 5628: Part 3 for more detailed information). For example, units other than those indicated may be used, provided that the manufacturer is able to produce authoritative evidence that they are suitable for the intended purpose. It is often desirable to consult the manufacturer in any event. The Tables do not apply where sulphates are present in the ground or groundwater in significant quantities.

Table 6.1: Recommended qualities of precast concrete masonry units to ensure durability

Location	Strength of unit (N/mm^2)		Mortar designation
	Blocks	Bricks	
All internal; external above dpc	any	15	(iii)
External below dpc; freestanding walls; parapets	3.5 dense† 7.0 lightweight	20	(ii)
Earth retaining walls; sills and copings*	7.0 dense†	30	(ii)

*Where the retaining face is waterproofed and an adequate coping provided, the quality may be that as for freestanding walls. Indeed, most masonry is not waterproof in this situation and the provision of proper waterproofing is always desirable.
†Units other than those indicated may be used provided that the manufacturer is able to produce authoritative evidence that they are suitable for the intended purpose.

Table 6.2: Recommended qualities of fired-clay and calcium silicate units to ensure durability

Location	Quality or class of unit		Mortar designation	
	Fired-clay	Calcium silicate	Fired-clay	Calcium silicate
All internal	any	2 to 7	any	(iii)
External—above dpc	Ordinary*	2 to 7	(i) or (ii)	(iii)
—below dpc	Special†	3 to 7	(i) or (ii)	(ii)
Freestanding walls, parapets	Special*	3 to 7	(i) or (ii)	(iii)

Earth-retaining walls, sills and copings	Special‡	4 to 7	(i)	(ii)

*Special quality fired-clay units may be necessary, particularly where the masonry is not adequately protected from saturation.
†Exceptionally ordinary quality bricks and grade (iii) mortar may be used where there is a low risk of saturation.
‡Where the retaining face is waterproofed and an adequate coping provided, the quality may be that as for freestanding walls. Indeed, most masonry is not waterproof in this situation and the provision of proper waterproofing is always desirable.

32.2 Resistance to corrosion of metal components

32.2.1 General

As the UK experience of durability of reinforced masonry in terms of resistance to corrosion is limited and the results of the various field and experimental studies are quite variable, the approach which has been adopted in the Code is cautious. Much notice has been taken of the recent experiences with reinforced concrete construction in the UK.

The process by which steel placed in mortar or concrete corrodes is well understood. Steel is thermodynamically unstable and can corrode in both acid and neutral environments when oxygen and moisture are present. The moisture in the pores of the mortar or concrete act as an electrolyte and consequently metal ions are released by steel placed in contact with it and the steel takes up a negative potential relative to the electrolyte. In concrete the electrolyte varies from place to place, as do the properties of the steel, and so it is possible for the steel to adopt different potential differences with respect to the electrolyte along its length. Consequently in the right conditions an electrical current flows and corrosion cells are set up, leading to metal ions being continually released at the anodes and these then combine with hydroxyl ions to form rust.

However, in the alkaline conditions produced during the hydration of the cement in mortar or concrete, the steel is passivated. An oxide film forms on the surface and even in the presence of moisture and oxygen, will not corrode. Problems arise when the passive oxide layer is disrupted and corrosion may then take place. Disruption of the oxide layer occurs due to two main reasons, namely carbonation and chlorides.

Carbonation is the process by which the acid gases in the environment, in particular carbon and sulphur dioxides, neutralise the hydroxides which provide the alkalinity of the concrete or mortar. The rate at which this occurs depends on a number of factors, one of which is the gas permeability of the material. The subsequent loss of alkalinity depends on the initial cement content of the mix. Naturally, factors such as temperature and relative humidity also have an effect. In reinforced concrete it is relatively simple to say that if strong, high cement content concrete with low permeability and hence low water/cement ratio is used, problems are not likely to occur provided enough cover is allowed. It is less simple to do this in reinforced masonry as, for example, in grouted cavity or hollow block construction, the initial water/cement ratio should be high to ensure that the mix is workable enough to fill all the voids adequately. The units do, of course, draw water from the mix, depending upon their own porosity, and the effective water/ cement ratio from the point of view of defining the permeability will be lower than that in the original mix. Once carbonation has taken place, corrosion does not necessarily commence unless both moisture and oxygen are available. Also, if the masonry was saturated its permeability to oxygen should be low and thus corrosion would be limited. Similarly, if the masonry was very dry, the resistance of the concrete or mortar forming the electrolytic cell would be high and the corrosion current restricted.

The other major cause of disruption of the passive oxide coating to steel is the presence of chlorides. It is probably true to say that a major cause of corrosion in reinforced concrete structures has been the excessive use of calcium chloride as an accelerator in the mix, but their use has been excluded in this Code. Chlorides are also available from the environment, such as near the coast or in boundary walls likely to be in contact with de-icing salts.

Although there is some understanding of the mechanisms causing corrosion, there remain a number of situations where there is a fear that unprotected low carbon steel might corrode, even if a high standard of workmanship has been maintained. These situations arise where reinforcement is placed in bed joints or in special units, such as pistol bricks and in grouted cavity or Quetta bond construction. In these situations, the Code recommends the minimum level of protection for reinforcement. The recommendation is linked to the severity of exposure and this is defined in terms of location of the site in relation to the Local Spell index as defined in DD 93[2] or the Driving Rain index[3]. An additional exposure situation is also defined which is based on the availability of certain chemicals. Certain other situations, such as reinforced hollow blockwork, can be more easily treated by providing an adequate thickness of concrete cover. In any situation, the designer may choose to do this in any case, for example a grouted cavity wall could be dimensioned such that the cavity would allow the appropriate cover to the steel to be maintained.

32.2.2 Classification of exposure situations

Three definitions of site exposure condition (E1, E2, E3) have been defined which relate to wind driven rain, viz.

E1 very sheltered or sheltered

E2 sheltered/moderate or moderate/severe

E3 severe or very severe

The definitions, such as *severe,* are based on the Local Spell index as described in DD 93. Essentially the basis is the amount of wind driven rain falling on a vertical surface during the worst likely spell of bad weather in a three year period. CP 121: Part 1:1973[7] included three definitions of site exposure based on the Driving Rain Index, and the relationship between these and the definitions based on Local Spell index are shown in Table 6.3.

Where exposure conditions overlap, i.e., where the Local Spell index is between 68 and 85 and 29 and 37, the designer should use his own judgement based on any local knowledge to determine which exposure condition to consider. It may be that such a decision is simplified by the following sections.

There are, in addition, certain local conditions to which masonry may be exposed which can be classified in a similar way to site exposure but are not dependent upon it. Consequently, the locations where such conditions exist may be defined in an increasing order of severity as:

E1 reinforcement in the inner skin of ungrouted external cavity walls and behind surfaces protected by an impervious coating which can readily be inspected

E2 reinforcement in buried masonry and masonry continually submerged in fresh water

E3 reinforcement in masonry exposed to freezing while wet or subjected to heavy condensation.

A further set of conditions are so severe that whatever the site classification, the only suitable reinforcement is that which is solid or coated with at least 1 mm of austenitic stainless steel. These conditions are where the masonry is exposed to salt or moorland water, corrosive fumes, abrasion or de-icing salts. This exposure situation is defined as E4.

Table 6.3: Definitions of site exposure

Exposure condition	Definition of site exposure	Local Spell Index L/m^2/spell	Exposure category CP 121
E1	Very severe Severe	98 and over 68 to 123	Severe (DRI>7 m^2/s)
E2	Moderate/severe Sheltered/moderate	46 to 85 29 to 58	Moderate (7 m^2/s>DRI> 3 m^2/s)
E3	Sheltered Very sheltered	19 to 37 24 or less	Sheltered (3 m^2/s>DRI)

32.2.3 Situations of exposure requiring special attention

Within the broad classification based on Local Spell index, there are buildings with certain features and also certain positions of buildings, which need special consideration. If in any building there are features which are more severely exposed than the general building, for example sills, where run-off is a factor, parapets which are exposed on both faces above the roof line and such like, they should be considered as in site exposure condition E3.

32.2.4 Effect of different masonry units

Carbonation and electrolytic corrosion are dependent on the migration of oxygen and moisture through the masonry. Consequently there is a tendency for the protection to the reinforcement offered by the masonry to be greater when low porosity, low permeability materials are used. In this respect the Code is based on an interpolation of a limited amount of evidence, and (if reinforcement is being provided in accordance with Table 13 of the Code) recommends that where bricks of any material have a water absorption of greater than 10% or concrete blocks having a nett density of less than 1500 kg/m^3 are used, the type of reinforcement should be that recommended for the next more severe condition of site exposure.

Exposure condition chart:

The chart, as illustrated in Figure 6.1, is an aid to establishing the condition, E1, E2, E3 or E4 from which the type of reinforcement should be selected. This chart accounts for the initial definition of site exposure, modifications due to special localised conditions, as well as the modifications based on materials selection. The chart may be used by starting either at the top or bottom of the left hand column of boxes and moving from box to box dependent on the applicability of the classification within any particular box.

Selection of type of reinforcement

Once the correct exposure condition is established, the type of reinforcement should be selected from Table 13 of the Code (reproduced here as Table 6.4), which is intended to give the minimum acceptable degree of protection for each classification. Alternatively concrete cover may be provided in accordance with Table 14 of the Code when low carbon steel is used.

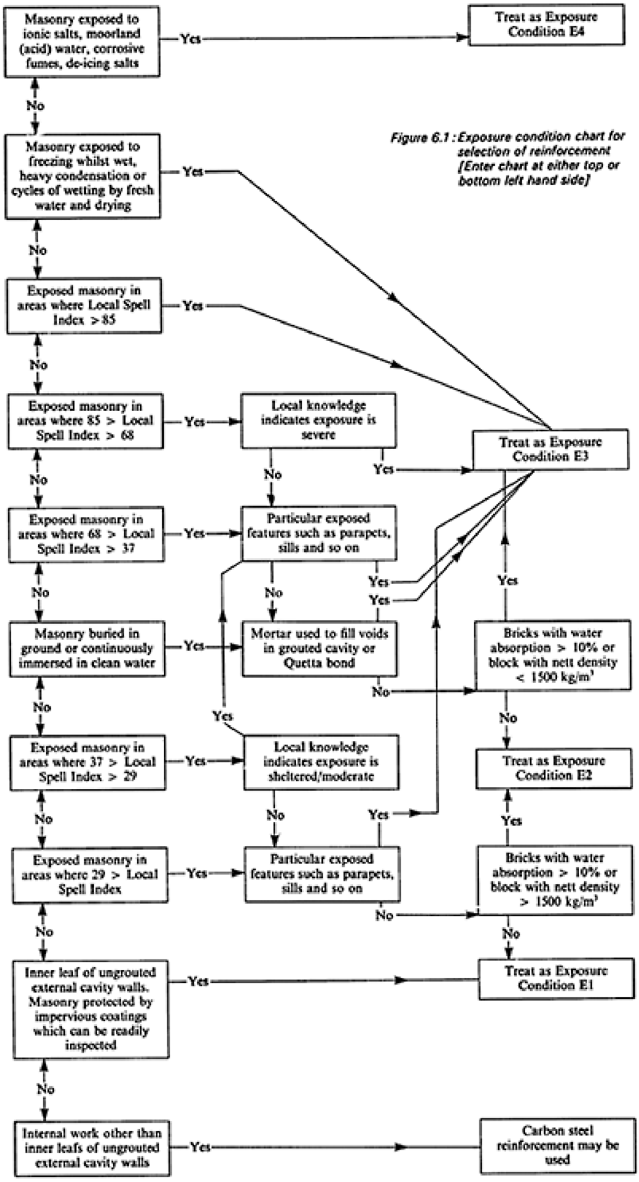

Figure 6.1: Exposure condition chart for selection of reinforcement [Enter chart at either top or bottom left hand side]

Table 6.4: Selection of reinforcement for durability

Exposure condition (see 32.2.2)	Minimum level of protection to reinforcement to be located in:	
	Bed joint or special clay units	Grouted cavity or Quetta bond construction
E1	Carbon steel galvanised following the procedure given in BS 729[8]. Minimum mass of zinc coating 940 g/m^2.	Carbon steel
E2	Carbon steel galvanised following the procedure given in BS 729. Minimum mass of zinc coating 940 g/m^2.	Carbon steel. Where mortar is used to fill the voids the steel should be galvanised following the procedure given in BS 729 to give a minimum mass of zinc coating of 940 g/m^2.
E3	Austenitic stainless steel or carbon steel coated with at least 1 mm of stainless steel.	Carbon steel galvanised following the procedure given in BS 729. Minimum mass of zinc coating 940 g/m^2.
E4	Austenitic stainless steel or carbon steel coated with at least 1 mm of stainless steel.	Austenitic stainless steel or carbon steel coated with at least 1 mm of stainless steel.

Note: In internal masonry other than the inner leaves of external cavity walls, carbon steel reinforcement may be used.

32.2.5 Cover

The use of protected steels in certain circumstances and the allowance which is made for the degree of protection given to reinforcement by the thickness of masonry about any cover in concrete or mortar, allows a lower thickness of cover to be specified than in normal concrete structures. In grouted cavity or Quetta bond construction the minimum cover to the steel should be 20 mm, this applies whether the cover is concrete or mortar as the amount of zinc required is varied in the two situations. When using bed joint reinforcement it is recommended that there be at least 15 mm cover to the bar from the face of the masonry.

The types of austenitic stainless steels or stainless coated steels which have been recommended do not require any cover to ensure their durability. However, if they are required to transmit force into masonry through their bond with mortar or concrete then there should normally be a cover of at least one bar diameter.

There are certain circumstances which lend themselves readily to a specification for corrosion protection based on concrete cover to low carbon steel reinforcement. For example, in reinforced hollow blockwork construction where the shell is relatively thin, in pocket type retaining walls, possibly at the bottom of grouted cavity beams. In these cases the required thickness of concrete cover may be varied with the concrete grade as shown in Table 6.5. The designer may, of course, elect to design for durability by providing this cover in any situation to carbon steel. However, it is considered that in exposure condition E2 where concrete cover is being provided in grouted cavity or Quetta bond, 20 mm is adequate. A number of examples where Table 6.5 may be used are illustrated in Figure 6.2.

Table 6.5: Minimum concrete cover for carbon steel reinforcement

Exposure condition	Thickness of concrete cover (mm)			
E1	20	20	20	20
E2		30	30	25
E3		40	35	30
E4				60
Concrete grade in BS 5328[9]	25	30	35	40
Minimum cement content (kg/m^3)	250	300	350	350

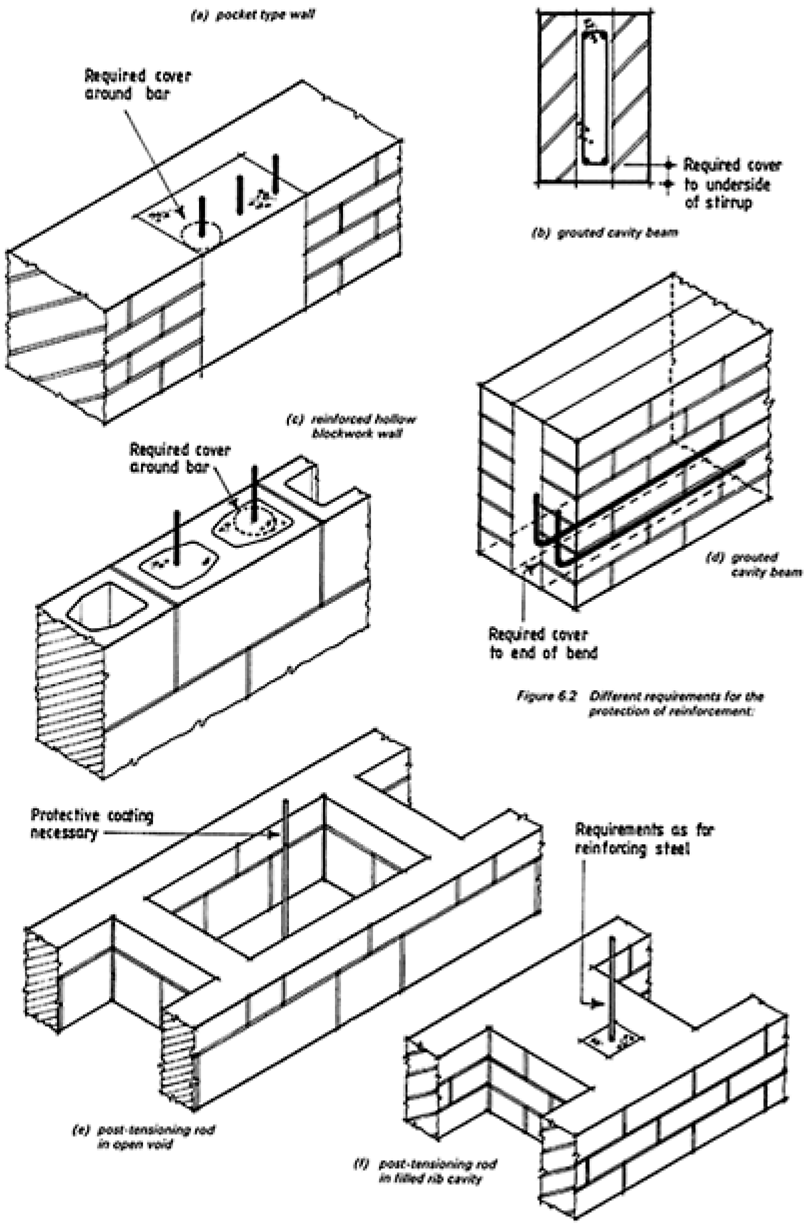

Figure 6.2 Different requirements for the protection of reinforcement:

32.2.6 Prestressing tendons

Prestressing tendons which are surrounded by concrete or mortar should be treated in the same way as reinforcing steel and all the considerations so far described should be made. In some cases, for example, in diaphragm walls or post-tensioned cavity walls, low carbon steel tendons will be placed in open cavities. In these cases it is recommended that they be protected, for example by galvanising with a minimum zinc coating of 940 g/m^2. It is also essential that there is some means of draining open cavities.

32.2.7 Wall ties

Wall ties should be considered in the same way as reinforcement and protected in the same way as would be necessary for steel in the same location. This can lead to a different type of steel being required for the wall ties to that for the main steel. For example, wall ties in a grouted cavity wall are located in the bed joints and in exposure condition E1 would need to be galvanised with 940 g/m^2 of zinc, although the main reinforcement in the cavity need be carbon steel only. In this situation it is essential that dissimilar metals are not allowed to come into contact. In the example quoted the sacrificial coating of zinc protects the wall ties by being more active electrochemically than the carbon steel. If the wall ties touched the main reinforcement the zinc would act as a sacrificial electrode for the carbon steel of both the wall tie and the main bars and consequently would react more quickly and the protection would not last as long as anticipated. In other situations where dissimilar metals are in contact, bimetallic corrosion cells may be set up, leading to unexpected corrosion.

33. Fire resistance

This section requires no further detailed comment.

34. Accommodation of movement

There is little information on the accommodation and control of movement in reinforced masonry. It is likely that the reinforcement will help in resisting the forces set up by movement restraint, although it is unlikely that this contribution will be significant or reliable because of the relatively small amount and possibly poor position of the reinforcement within the reinforced masonry (see BS 5337 for water-retaining structures[10]). It is for these reasons that the Code suggests adopting the recommendations in Clause 20 of BS 5628: Part 3 accepting that in many cases these will be conservative.

All structures are subject to small dimensional changes after construction which may be caused by one or more of the following:

1. change in temperature
2. change in moisture content
3. adsorption* of water vapour
4. chemical action, carbonation

5. deflection and deformation under loads
6. ground movement and differential settlement

In general, because restraints are present, masonry is not completely free to expand or contract and compressive or tensile stresses will develop which may cause bowing or cracking. The provision for movement must be considered at the design stage, bearing in mind, of course, the overall stability of the structure. The effects of movement may be reduced by:

**adsorption* is the term used to describe the bonding of water molecules to the molecules of the masonry material. It should not be confused with *absorption* which refers to the entry of water molecules into the pores of the masonry. Adsorption is the physiochemical process by which fired-clay masonry expands.

1. using the correct mortar: for certain masonry a weaker mortar is preferable to control cracking but for durability reasons, designations (i) and (ii) are suggested for reinforced masonry—the latter being acceptable because the presence of the reinforcement will help to redistribute some of the restraint stresses
2. keeping the units and the wall protected during construction: this is particularly important for calcium silicate and concrete masonry in terms of reducing overall potential movement but is important for all masonry to reduce the risk of other problems, such as efflorescence and frost attack
3. providing reinforcement: the reinforcement provided in the masonry (particularly that placed horizontally) will help reduce overstressing, for example at openings (Figure 6.3)
4. providing movement joints: the spacing of movement joints in unrestrained, unreinforced fired-clay masonry walls, such as parapets, may be calculated based on the ultimate expansion of 1 mm/m run of wall. For walls which are restrained, reinforced or are constructed of fired-clay units of low expansion, this value may be reduced considerably; advice should be sought from the manufacturer. In general expansion joints should be placed at intervals not exceeding 15 m and the width of joint should be approximately 30% more than required by straight calculation since most fillers are not infinitely compressible. For example, if by calculation 12 mm of movement requires accommodating, the joint should be 16 mm wide. Present evidence suggests that vertical expansion will be smaller in magnitude to that horizontally. For calcium silicate masonry, joints to accommodate shrinkage movement should be placed at between 7.5 and 9 m centres. The ratio of the length to height of a panel separated by joints should generally be less than 2:1. As a general rule, for concrete masonry, vertical joints to accommodate horizontal contraction should be provided at intervals of 6 m, although this spacing can be varied slightly to suit the layout of the structure. For certain types of masonry, for certain situations and for reinforced masonry wider spacing or larger length/height ratios may be justified, although advice should be sought from the manufacturer.

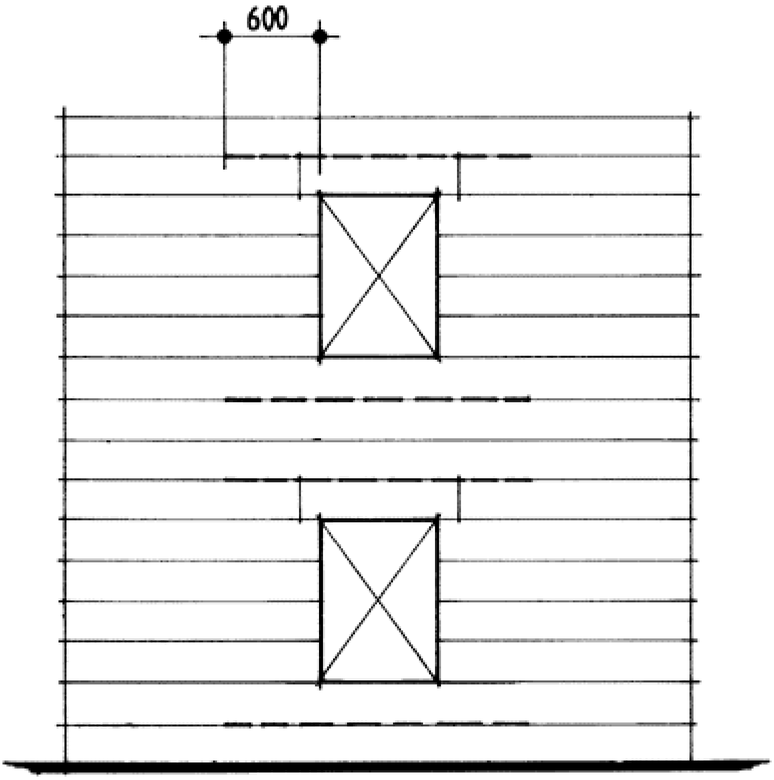

Figure 6.3 Reinforcement at openings

For concrete and calcium silicate masonry in which the contraction joints are not designed to act as expansion joints, separate expansion joints should be provided in freestanding or retaining walls at intervals of 30 m. These expansion joints may be omitted, however, if fibre board is used in all joints. It must be noted at this point that fibre board is not sufficiently compressible to be used in a full expansion joint.

For all masonry the suggested spacings are between free movement joints. The spacing of the first movement joint from an external or internal angle, particularly where the return is stiff, should be approximately half the general spacing due to the effect of end restraint. Consideration should be given to tieing across the joint with debonded dowels or strips which are capable of carrying the lateral shear at that point. This is particularly important in retaining walls. Further, in earth retaining walls where the temperature and moisture content of the masonry remains sensibly constant, joint spacings of up to 20 m may be justified.

Features of the structure which should be considered when determining joint positioning are as follows:

1. intersecting walls, piers, floors, and so on
2. window and door openings

3. changes in height or thickness of the wall
4. deep chases in the wall
5. movement joints in the rest of the structure, for example, floor slabs
6. the juxtaposition of flexible elements of the structure to the more “brittle” masonry elements, for example:

Where masonry is built over a floor slab the deflection of the slab can create tensile stresses within the masonry. For reinforced masonry with a reasonable amount of horizontal steel this should not present a problem.

Where walls are non-loadbearing vertically a gap should be left between the bottom of the floor slab above and the top of the wall.

This gap may need to be filled with compressible, non-combustible material to provide fire separation and various details exist for providing lateral restraint to the wall at this level if required. In concrete framed structures, consideration should be given to differential movement of the frame and masonry particularly for fired-clay units where the overall expansion of the masonry is opposed to the overall shrinkage of the frame.

Providing eccentric loads and short returns are avoided, panel walls of fired-clay masonry in steel framed structures not subject to sway, can usually be built into, and rigidly tied to, the frame. Concrete and calcium silicate masonry should not be tied rigidly to the frame, but it is essential to provide adequate lateral restraint. Where sway can occur, particularly in single-storey framed structures, more complex details will be required to provide adequate lateral restraint, accommodate the movement of the masonry and accommodate the sway movement of the structure. Where masonry walls are provided to resist sway they should be designed to accommodate the stresses induced by the required full restraint as well as those produced by the imposed loads.

35. Spacing of wall ties

In ungrouted cavity walls and low-lift grouted cavity walls where bursting forces due to placement of concrete infill are low, the spacing of wall ties should follow the recommendations in BS 5628: Part 1[11], as follows:

> The two leaves of a cavity wall should be tied together securely by metal ties. Where the width of the cavity is more than 75 mm or the ties are for low lift grouted cavity work, only twist type wall ties complying with BS 1243[12] should be used. Ties should preferably be embedded simultaneously in both leaves with a slight fall to the outer leaf. They should be placed in the mortar joints as the units are laid—not pushed in after the unit is bedded. They should be embedded at least 50 mm which, for unfilled hollow blockwork, means they must coincide with the web of the block. Cellular blocks are usually laid with their closed end uppermost and so a 50 mm embeddment is achievable anywhere along their length. Wall ties of both butterfly and double-triangle type should be laid drip down.

Spacing should be in accordance with Table 6.6, but because of the variety of dimensions of various units, the spacing may be adjusted slightly so as to align the course heights. The total number of wall ties per m^2 must not be less than the value given in Table 6.6. In addition to their normal spacing, ties should also be provided within 225 mm either side of an opening, movement joint or external corner, at vertical centres not exceeding 300 mm.

Table 6.6: Spacing of wall ties in ungrouted and low lift grouted cavity walls

Leaf thickness mm	Cavity width mm	Spacing of ties		Number of ties per m^2
		Horizontally mm	Vertically mm	
Less than 90	50–75	450	450	4.9
90 or more	50–150	900	450	2.5

In high lift grouted cavity walls, the wall ties (see Clause 2.9) 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, and so on, at not greater than 300 mm centres vertically. Guidance on the durability of ties is given in *Section 32.2.7.*

36. Drainage and waterproofing

Reasonable guidance is given in the Code on this subject and further information can be obtained from CP 2, the Code of Practice for earth-retaining structures[13]. It must be remembered, however, that reinforced masonry is not generally as impervious as good quality, well compacted, concrete and that its imperviousness will depend on the form of construction. For example, grouted cavity walls have a continuous “membrane” of high quality infill concrete, reinforced hollow blockwork on the other hand has many “bridges” at mortar joints and the webs of the blocks.

Guidance is given on the necessity for and positioning of weepholes, but it must be borne in mind that some staining will inevitably occur below weepholes. Where this is unacceptable aesthetically the reinforced masonry wall can be faced with a veneer of masonry which is non-loadbearing and which is separated by a cavity. The cavity can then be detailed to transfer any water to one or both ends of the wall. This facing should be tied back to the main wall using light ties such as butterfly or double-triangle ties to minimise any load shedding into the facing.

37. Damp-proof courses and copings

The Code refers the designer to Clause 21 of BS 5628: Part 3 for information on damp-proof courses and copings.

Damp-proof courses

A warning is given in the Code regarding the possible effects the choice of material will have on the bending and shear strength of the member. BS 5628: Part 3 gives no guidance on the latter and indeed little guidance exists: therein lies the first difficulty in using dpc's in reinforced masonry. The second difficulty is that for horizontal dpc's to function correctly they should form a complete break in the structure. In certain instances, however, this is contrary to what is being achieved structurally, for example, in reinforced hollow blockwork cantilever retaining walls where the reinforcement and its surrounding infill must be continuous down to foundation level.

A range of damp-proof course materials is available and these vary in thickness from as little as 0.5 mm up to 2 mm and more. Bitumen polymer dpc's[14] are designed to be used in any position where a flexible dpc is required and are particularly suitable for heavy load situations. Dpc's of this type also tend to retain flexibility at low temperatures. Standard dpc's complying with BS 743[15] are for general purpose use with moderate loadings, but may need to be handled with care in wintry conditions. Polythene dpc's are not suitable for use in freestanding or other lightly load situations because the strength of the bond with mortar is low. One type of dpc available which will give very good adhesion to mortar is an asbestos based dampcourse surfaced with a coarse sand finish which gives good resistance to slip along the length of the wall and to tensile stress across the thickness of the wall.

Damp-proof courses can be put into three main groups:

Flexible	Polyethylene
	Bitumen and pitch polymers
	Bitumen—abrasion fibre or asbestos based
	Lead
Semi-rigid	Mastic asphalt
Rigid	Slate
	Dpc brick
	Epoxy resin/sand

This list is in approximate order of adhesion strength and, therefore, shear strength[16], but several may be subject to extrusion under vertical load, namely mastic asphalt and bitumen. It is generally accepted that the former may extrude under pressures above 65 kN/m^2. Methods of testing dpc's in flexure and in shear are given in DD 86[17]. As stated earlier, little quantitative information exists and it is, therefore, always wise to consult the manufacturer or specialist bodies.

The installation of dpc's should comply with CP 102[18] and BS 743. A dpc is usually placed to extend through the full thickness of the wall. In cold weather bitumen dpc's should be warmed. Bitumen polymer dpc's should be joined with 100 mm overlap. Most BS 743 types of dpc require an 100 mm overlap sealed with a proprietary cement, hot bitumen, or by the careful application of a blow-lamp. Polythene dpc's should be welted or welded.

Flexible dpc's should be sandwiched between two layers of mortar, the dpc being laid on the first bed whilst still wet and the second preferably being applied immediately. It should extend the full thickness of the masonry and preferably project from it. Flush dpc's may be acceptable in some circumstances if accurate positioning can be relied upon. They should never be recessed behind the face of the mortar. Where a fired-clay dpc brick is used in conjunction with concrete or calcium silicate masonry, the possibility of differential movement should be considered, although in most cases this will not be a problem. BS 5628: Part 3 gives full guidance on positioning of dpc's but is not summarised here since much of it is inapplicable to the type of structure likely to be built in reinforced masonry.

Copings (and cappings)

Freestanding walls, retaining walls, and so on, exposed to the weather should preferably be provided with a coping. The coping may be a preformed unit or it may be built up using creasing tiles. In either case, the drip edge(s) should be positioned a minimum of 40 mm away from the face(s) of the wall. Where, for aesthetic reasons, a capping is used, special care is needed in the choice of materials for capping and for the walling beneath *(Section 32.1)*.

A continuous dpc should be used in conjunction with copings or cappings and should be bedded in a designation (i) mortar in the case of fired-clay units, or designation (ii) mortar in the case of concrete and calcium silicate units. In cappings, the dpc may be positioned 150–200 mm down rather than immediately below the capping course to obtain greater weight on the dpc. Dpc's for both cappings and copings should preferably extend 12–15 mm beyond the face(s) of the wall to throw water clear of the wall. Alternatively, a suitable flashing may be used.

Copings may be displaced by lateral loads, vandalism, and so on, and consideration should be given in this aspect. L-shaped and clip-over copings may be more satisfactory in these situations, but where necessary any coping should be dowelled or joggle-jointed together and/or suitably fixed down. Provision for movement should be provided in long coping runs; more frequent movement joints may be required owing to increased solar absorption. Any movement joints detailed in the masonry below must be continued through the coping or capping.

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Section Seven: Work on site

38. Materials

The specification of materials to be used in reinforced and prestressed masonry is provided in *Section Two* and a model specification is included as *Section Ten*.

On site the storage and handling of masonry units and associated materials should follow the recommendations contained in BS 5628: Part 3[1]. The equivalent requirements for reinforcement and prestressing tendons and concrete are contained in CP 110: Part 1:1972[2].

Particular attention should be paid to covering masonry units on site, and during construction. Failure to protect many facing units from excessive moisture may well lead to subsequent efflorescence problems and ideally masonry should be covered as building progresses.

39. Construction

39.1 General

The general requirements for the execution of reinforced and prestressed masonry are similar to unreinforced masonry and are described in detail in BS 5628: Part 3. The following additional requirements should be considered.

The workability of infill concrete should be very high when filling vertical cores or narrow cavities in masonry walls. It is essential that such mixes should be largely self-compacting, although small mechanical vibrators, compacting rods and so on, should also be used to ensure the complete filling of all sections. There are some reinforced masonry elements, such as shallow lintels or beams, in which it is comparatively easy to determine the efficiency of the filling by inspection. Walls filled in fairly low lifts are also reasonably easy to inspect as described below.

The reinforcement should be free from deleterious material as described in the Code. Care should be taken with the fixing and location of reinforcing steel to ensure that the correct cover is maintained and that the steel cannot be displaced during the filling process. This can usually be achieved, in a wall for example, by locating main vertical reinforcement by means of the horizontal distribution steel. Conventional plastic type bar spacers may be used quite readily in beams and other “open” elements, but should not be allowed to obstruct the core, for example, of hollow blockwork.

A set of typical construction detail drawings is provided at the end of this section as Figures 7.1 to 7.14 inclusive.

39.2 Grouted cavity construction

39.2.1 General

During the construction of cavity walls, care needs to be taken to keep the cavity clean. For narrow cavities this may be achieved by the use of a timber lathe which may be placed in the cavity and "drawn up" with the mortar droppings. For wider cavities it will usually be simpler to remove mortar droppings through "clean out" holes left at the bottom of the wall. All mortar extrusions which infringe into the cavity space should be removed before filling.

39.2.2 Low lift

In this method of construction the infill concrete is placed as construction proceeds, usually in lifts of 450 mm, i.e., two courses of blockwork or six courses of brickwork. The "construction joint" in the core should be at mid-unit height rather than corresponding with the top of the unit. To maintain the appearance of facing masonry, care should be exercised in filling the cores and in preventing grout loss detracting from the appearance. The concrete should be compacted as each layer is placed. It may be necessary to limit the rate of construction and filling to avoid disruption of the masonry due to the pressure exerted by the fresh concrete infill. Any disruption due to the placing process will result in the necessity to rebuild the wall.

39.2.3 High lift

The clean out holes at the base of the wall should be at least 150 mm×200 mm and spaced at intervals of 500 mm. They are used to remove all mortar and other debris prior to placing the concrete. Before the wall is filled, the brickwork must either by replaced in the clean out holes or temporary shuttering fixed to prevent the loss of infill concrete. The latter technique provides a means of checking efficient filling at the base of the wall.

The infilling concrete should not be placed until after three days have elapsed since the brickwork was constructed—longer in adverse weather conditions. The maximum height to be filled by this technique in one pour is 3 m, usually in two lifts. The concrete in each lift should be recompacted after initial settlement due to water absorption by the masonry.

There are examples in the USA where extremely high pours (up to 10 m) have been carried out in a single lift, the mix containing a lot of cement and a great deal of water. However, this is not usual and the practice recommended above is similar to many American recommendations.

39.3 Reinforced hollow blockwork

39.3.1 General

There are essentially two techniques for filling the cores of hollow concrete blocks, low lift and high lift grouting. In the low lift technique the cores are filled as the work proceeds so that not more than a few courses of blockwork are built up before filling. In the high lift technique the cores are filled in lifts of up to 3 m, care being taken to ensure that the cores are fully filled and that the pressure exerted by the infilling concrete does not disrupt the wall.

39.3.2 Low lift

The reinforcing steel within the cores may be located by tieing the main steel to the distribution steel. If necessary the face shell of appropriate blocks may be removed to facilitate the tying of vertical steel for laps and so on. The use of plastic spacers which might tend to block up the cores should be avoided. The general aspects applying to low lift grouted cavity construction apply to this technique except that the maximum vertical interval at which concrete is placed may be 900 mm.

39.3.3 High lift

In the high lift technique it is particularly important to ensure that all mortar extrusions are removed from the core of the blocks.

This is commonly achieved by leaving clean out holes at the base of the wall as shown in Figure 7.2. Excess mortar is knocked off the side of the cores and is removed through the holes in the base of the wall. Before filling with concrete these holes need to be securely blocked to prevent the loss of the infilling concrete.

The concrete itself may be placed by hand, skip or pump. Whichever method is used, particular care should be taken with facing work to prevent grout running down the face of the wall. The mixes specified in the Code are such that they are intended to have a high level of workability, and should be virtually self-compacting when a tamping rod is employed. With some mixes it may be necessary to use a small poker vibrator to compact the concrete properly. A 25 mm diameter poker vibrator may be used in most situations.

Once a wall has been filled using the high lift grouting technique it will be noticed that after a period of some 15 minutes (depending on the mix, absorption of the masonry and weather conditions), the concrete in each core has slumped. At this stage further concrete should be added and some limited recompaction carried out. An alternative approach is to use a proprietary additive in the mix to prevent this slump taking place.

When infilling concrete is placed by a grout pump, the rate of placing should not exceed 0.2 m^2 per minute.

39.3.4 Bond beam construction

When using a bond beam within an otherwise unreinforced section of walling, it will be necessary to seal the openings in the bottom of the blocks using an appropriate material. In the USA these are known as “grout stop” materials. Typical materials used are expanded metal lathe, thick mesh screen and asphalt saturated felt.

Horizontal reinforcing steel will need to be supported to give the appropriate cover by either plastic saddle supports, reinforcing steel or prefabricated brackets. Where it is necessary to splice bars, this should be done vertically (i.e., one bar above and one bar below), rather than side by side, to provide less restriction to the flow of the infilling concrete.

39.4 Quetta and similar bond walls

In this method of construction the reinforcement is usually placed progressively, in advance of the masonry. The cavities are filled with mortar or concrete as the work proceeds. In some circumstances, where large voids are produced (as in Figure 7.8), either low or high lift techniques may be used.

39.5 Pocket type walls[3]

Pocket type walls are usually built to their full height, the starter bars only projecting from the base into the pocket space. The main steel is then fixed and may be held in position using wires fixed into bed joints. Shuttering may be propped against the rear face of the wall, although it has in the past, been successfully fixed to the wall with masonry nails. The concrete is normally placed in lifts with a maximum height of about 1.5 m; this may be vibrated by poker vibrator or compacted using a rod.

39.6 Prestressing operations

This section requires no further detailed comment.

39.7 Forming chases and holes and provision of fixings

See BS 5628: Part 3:1985, Clause 19.

39.8 Jointing and pointing

The Code recommends that joints should only be raked out with the approval of the designer. Deeply raked joints are often considered to provide an attractive finish, but since the mortar is not as well compacted as when finished with a steel, in exposed situations their use could lead to problems of durability. In addition in external work raked joints expose the bed faces of the units which may lead to excessive water being absorbed. The above considerations are equally relevant to unreinforced masonry and reference should be made to BS 5628: Part 3. In sections which are critical in terms of the structural design, it may be necessry for the designer to consider the section as being reduced in size by the dimensional extent of any recess in the mortar joint profile.

40. Quality control

There are two requirements in the Code for the quality control of the workmanship in reinforced and prestressed masonry construction. The first requirement relates to site supervision and requires “either frequent visits to the site by the designer or the presence of his permanent representative on site, to ensure that the work is built in accordance with the requirements of this Code and any such specification as he may prescribe”. The second requirement is for “preliminary and site testing and sampling”. It should be noted that these requirements, although representing good practice, do not necessarily always have to apply to masonry incorporating bed joint reinforcement to enhance lateral load resistance when more conservative partial safety factors may be more appropriate in accordance with BS 5628: Part 1[6].

Tolerances which may be reasonably specified for structural masonry are indicated in Table 7.1.

Table 7.1: Acceptable tolerances for masonry

Length	
Up to and including 5 m	±10 mm
Over 5 m, up to and including 10 m	±15 mm
Over 10 m	±20 mm
Level of bed joint	
At 3 m high	+15 mm
[Errors of level not to be cumulative over the height of a building]	
Height	
Up to 3 m	±35 mm
Straightness	
In any 5 m [not cumulative]	+7 mm
Verticality	
Up to 3 m	±10 mm
[No masonry shall deviate more than 30 mm from the vertical in its full height]	
The above figures are based upon those in SP 56[4] and BS 5606[5].	

Site supervision is particularly important at the various “critical” stages of construction, particularly the location of reinforcing steel and the filling of cores or cavities. The usual recommendations relating to construction practice can apply, particularly that the units are laid on a full bed of mortar with properly filled perpend joints. The element must be built accurately with due regard for the control of alignment and plumb. Excess mortar must be carefully removed from voids and cavities which are to be subsequently filled. In the case of high lift grouting, clean out holes and other precautions to ensure clear hollows or cavities are usually necessary, as described in Clause 39. The infilling concrete must be prepared to achieve the correct slump and placed with the aid of tamping rods or mechanical vibration. During the placing of the concrete the supervisor should ensure that the reinforcing steel is not displaced—the location and fixing of the steel should be carefully checked before filling.

The “preliminary site testing and sampling” applies both to the mortar and infilling concrete. The necessary procedures for mortar are given in Appendix A of BS 5628: Part

1[6]. This allows the use of one of three specimen types: 75 mm cubes, 100 mm cubes or 100 mm×25 mm×25 mm prisms. The average compressive strengths required for the various mortar designations are shown in Table 1 (page 79) of BS 5628: Part 1, both for preliminary tests and site tests. Site specimens comprise of six prisms or four cubes for every 150 m^2 of masonry built with any one mortar designation. The storage and testing of the specimens should be carried out in accordance with BS 4551[7].

The specimens both for preliminary and site testing of the infill concrete should be 100 or 150 mm cubes made and tested in accordance with BS 1881[8] if it is proposed to employ a designed mix. A prescribed mix should normally be assessed on the basis of the specified mix proportions and required workability.

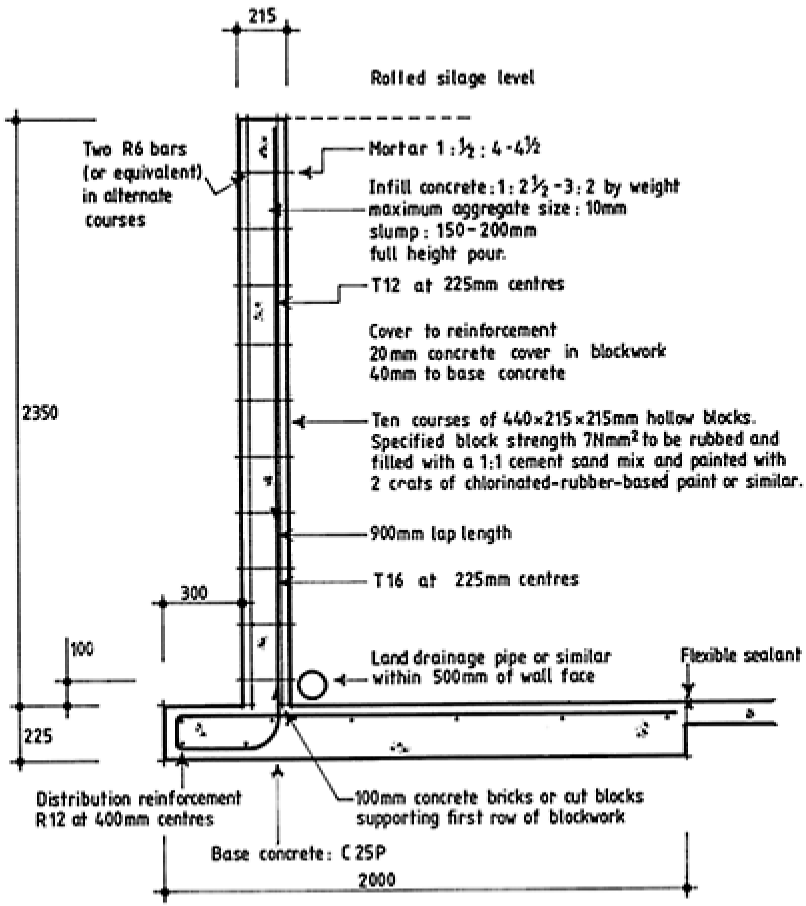


Figure 7.1 Typical requirements for 2.35 m high concrete blockwork retaining wall

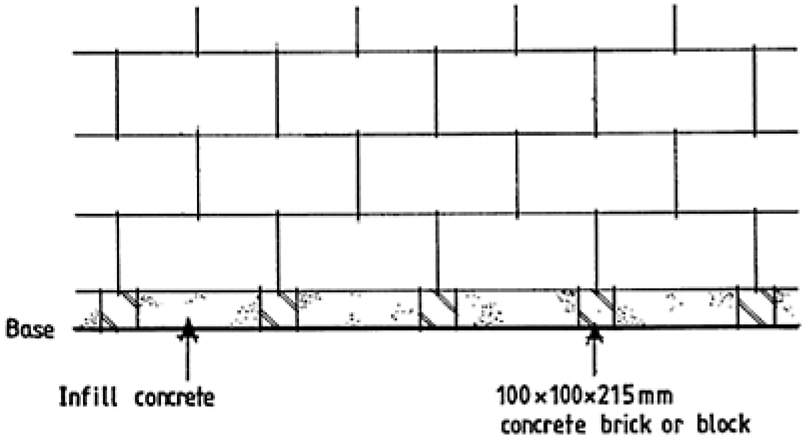


Figure 7.2 Alternative to cutting "face shells" to provide clean-out holes

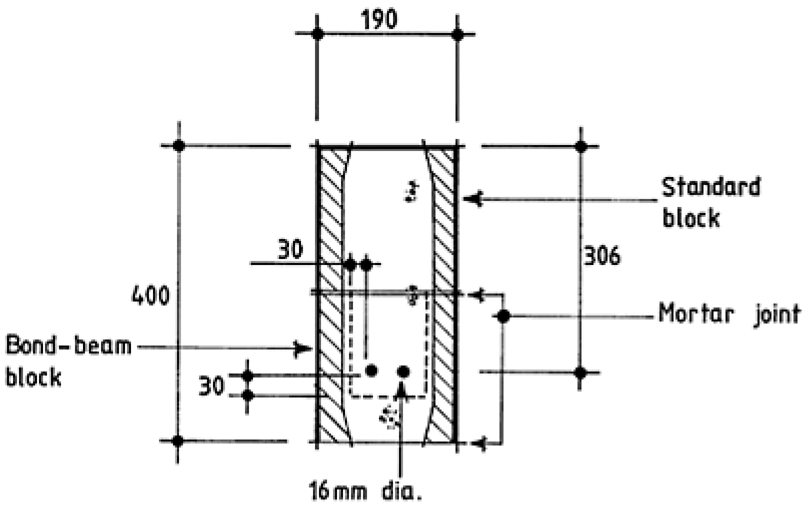


Figure 7.3 Typical cross-section of bond beam

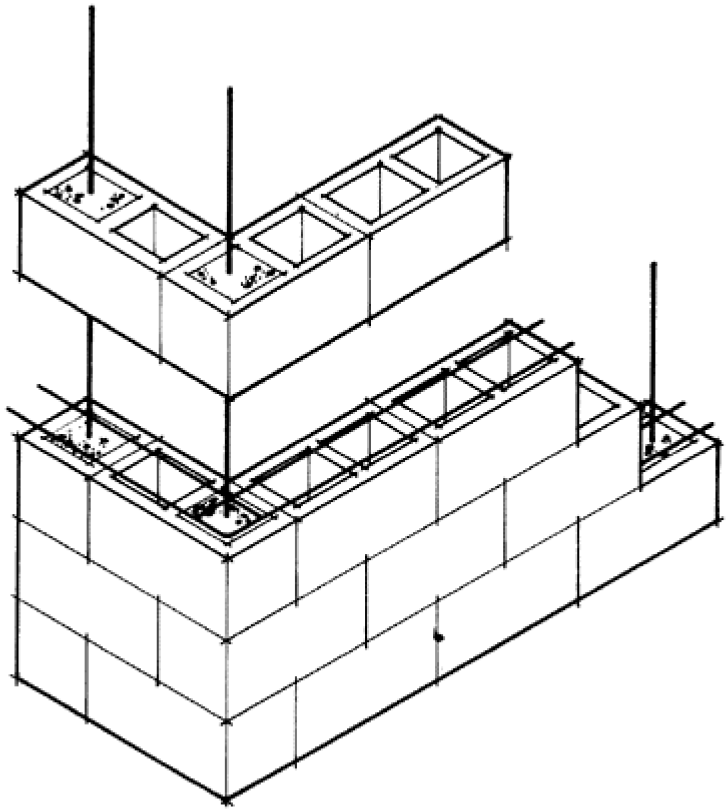


Figure 7.4 Reinforced hollow blockwork corner detail

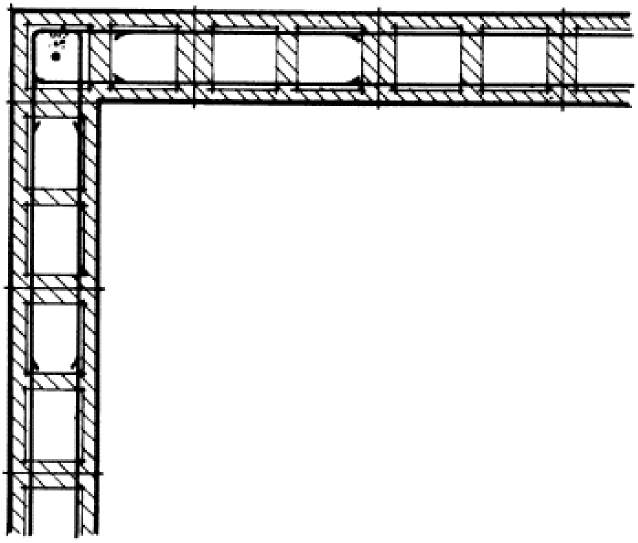


Figure 7.5 Wall corner detail

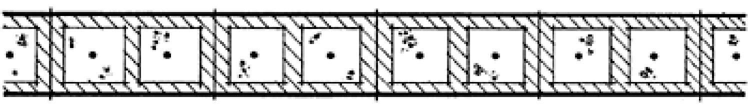


Figure 7.6 Typical plan of a reinforced hollow blockwork wall

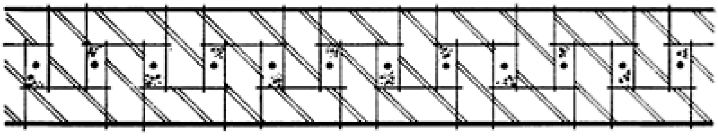


Figure 7.7 Plan of a Quetta Bond wall

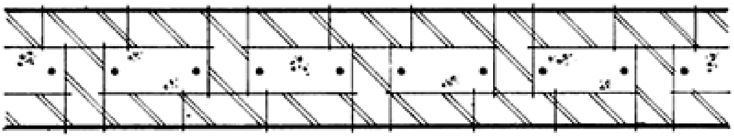


Figure 7.8 Quetta Bond: modified arrangement

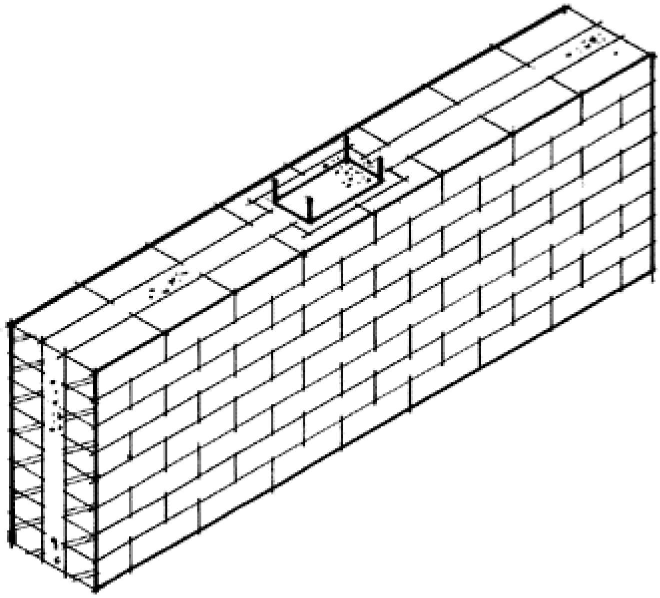


Figure 7.9 Use of pistol bricks to form a column in a grouted cavity wall

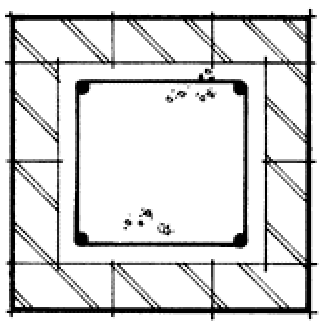


Figure 7.10 Brick column using conventional reinforcement cage

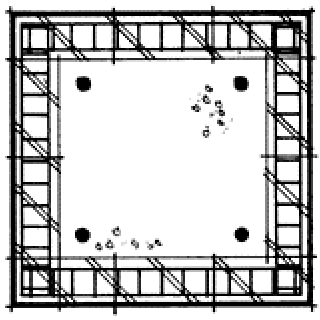
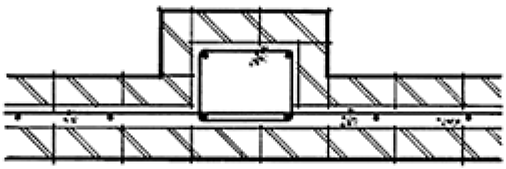


Figure 7.11 Brick column formed using bed joint reinforcement



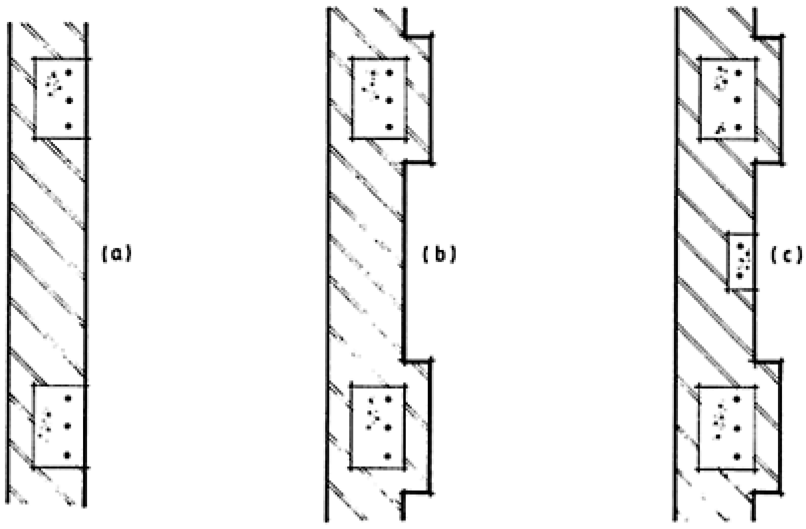


Figure 7.12 Column formed in a cavity wall of 300 mm bricks

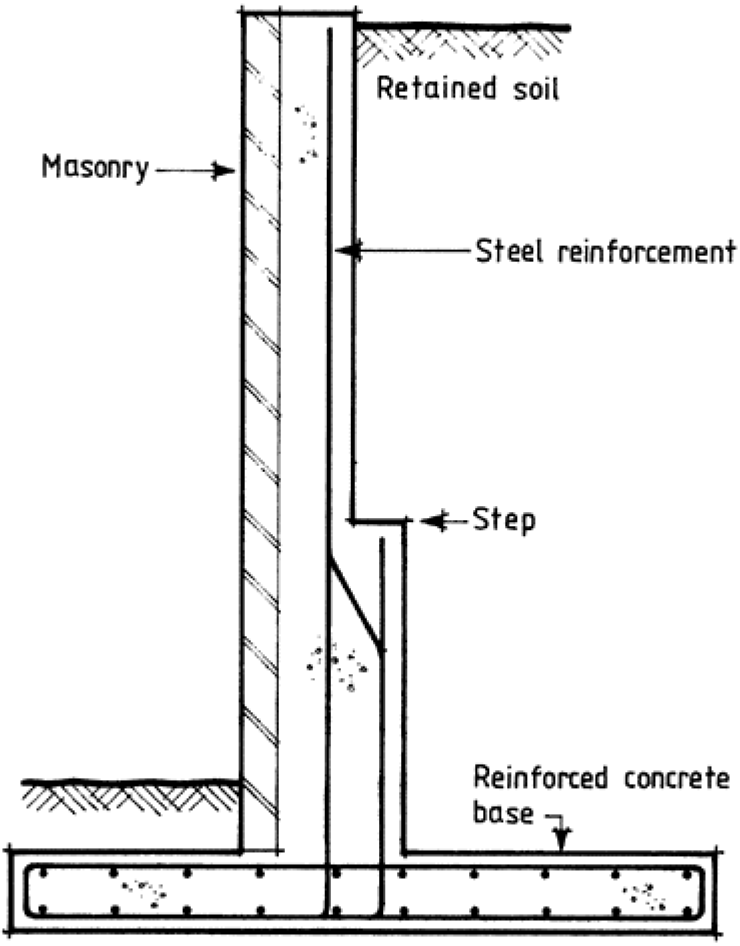


Figure 7.13 Pocket type retaining wall

(a) simple type

(b) projecting pockets

(c) composite type

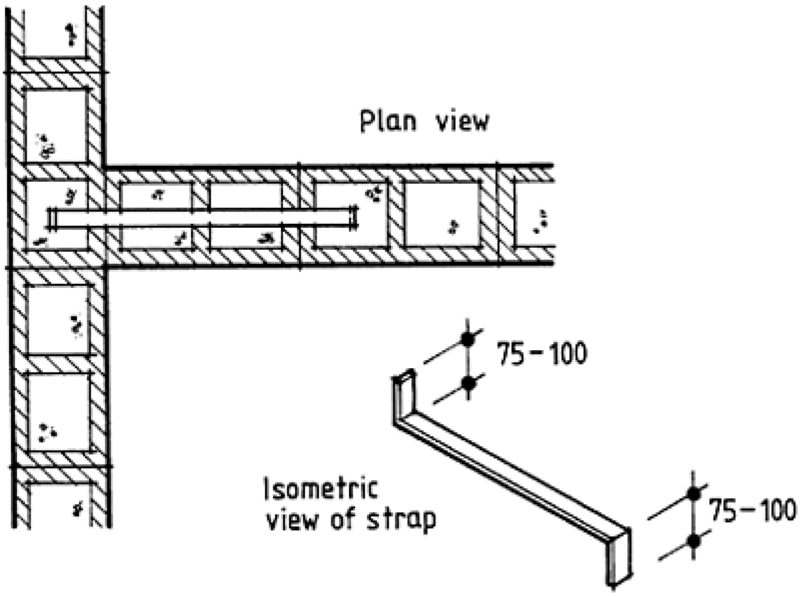


Figure 7.14 Intersecting partially reinforced walls tied together—tie detail

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Section Eight: Appendices

A. Design methods for walls incorporating bed joint reinforcement to enhance lateral load resistance

A.1 General

The use of bed joint reinforcement over openings in brickwork walls or in buildings where settlement is likely has been established practice for many years. The design in these situations has traditionally been based on elastic theory, although now this clearly can, and is, being changed to limit state design. Although the use of bed joint reinforcement to resist lateral loading is also common, there is no single accepted design method. Rules of thumb do exist and, since the development of a method for the design of unreinforced masonry walls subjected to lateral load was published in BS 5628: Part 1[1] in 1978, designers have often used bed joint reinforcement in walls which could not be justified as unreinforced but where the deficit in load carrying capacity was not too great.

As a result, the Code adopts a pragmatic approach and in Appendix A gives four alternative methods of design. A limited amount of research data was available and this was considered in formulating the four methods. The use of bed joint reinforcement is considered strictly as a way of enhancing the capacity of unreinforced walls and consequently, the partial safety factor for the compressive strength of masonry is chosen from those applicable to unreinforced masonry which are given in Table A. 1. It is also considered that the design methods may be used in walls containing grade (iii) mortar.

Table A.1: Partial safety factors for material strength, γ_m, in bed joint reinforced walls

		Category of construction control	
		Special	Normal
Category of manufacturing	Special	2.5	3.1
Control of structural units	Normal	2.8	3.5

Bed joint reinforcement, unless in internal walls, is considered to be subjected to a high degree of exposure and the Code advises that only galvanised or stainless steels be used. Although there is no recommended overall minimum percentage of reinforcement such that bed joint reinforcement may be considered to be effective, it is recommended that a steel cross sectional area of at least 14 mm^2 should be provided at vertical intervals not exceeding 450 mm.

A.2 Design recommendations

The dimensions of panels which may be designed using Appendix A to the Code are limited in the same way as they are for unreinforced panels. That is, they are dependent on the number of supported sides and whether the support is simple or continuous. The sizes are enhanced by about 10% from those for unreinforced walls and this is to allow increases in the moment of resistance of about 20% to be fully utilised. The limits are given in Table A.2.

Table A.2: Limiting dimensions—height×length

Number of sides supported	Types of support	
3	Two or more sides continuous 1800 t_{ef}^2	All other cases 1600 t_{ef}^2
4	Two or more sides continuous 2700 t_{ef}^2	All other cases 2400 t_{ef}^2

NOTE: No dimension to exceed 60 t_{ef}.

The degree of restraint which any particular kind of support is considered to provide is the same as for unreinforced masonry. In the case of vertical supports where the wall is supported from behind, by walls, piers or columns to which it is bonded or fully tied, then there is considered to be direct force restraint. Where such a pier or column is at the end of the wall, the support is considered to be simple. Supporting piers or walls which are intermediate in the sense that they divide up the horizontal span, can provide moment restraint if they are bonded in. However, this is limited by the tensile strength of the masonry. Where ties in the plane of the wall connect it to an intersecting wall, the supporting force is limited by the shear strength of the connection. Some degree of moment restraint may be possible.

Although in the case of cavity construction it is not essential that both leaves be directly connected to the vertical supporting elements, the wall ties must be adequate to ensure composite action and support must be given to the stronger leaf.

Floors at the base of the wall may provide moment restraint. However, this will be limited by the shear and flexural capacity of any damp-proof course material[2,3]. Where floors span onto or parallel to a wall at its top, simple support will generally be assumed where vertical anchors are used or where significant rotation needs to occur before resistance against further rotation is met. Examples of the degree of restraint assumed in various practical situations are illustrated in Figures A1 and A2.

The four methods covered by the Code are:

1. design as horizontally spanning wall
2. design with reinforced section carrying extra load only
3. design using modified orthogonal ratio
4. design based on cracking load

Of these four methods, the first is probably the most conservative and hence the Code permits a greater increase in load carrying capacity above that for an unreinforced wall than for the other methods. The second method is one which has been used in the past by designers and as it has no real theoretical justification it will probably only survive until the other methods are further developed. Method 3. is an extension of the yield-line approach as used in Part 1. Method 4. was developed by G.D.Johnson[4] and is based on the fact that reinforced panels crack at similar loads to those for unreinforced panels. Consequently, the ultimate load of unreinforced panels is considered to be the serviceability load for reinforced panels.

A.3 Method one: Design as horizontally spanning wall

This method is probably the most conservative and treats the wall as a horizontally spanning beam. In practice, except for walls that are tall in relation to their length, there will be some element of two-way spanning and this will enhance the lateral load carrying capacity. As this is likely to be the most conservative of the four methods, the maximum permitted enhancement in lateral load resistance above that for an unreinforced wall is greater at 50% than for the other methods. In cavity

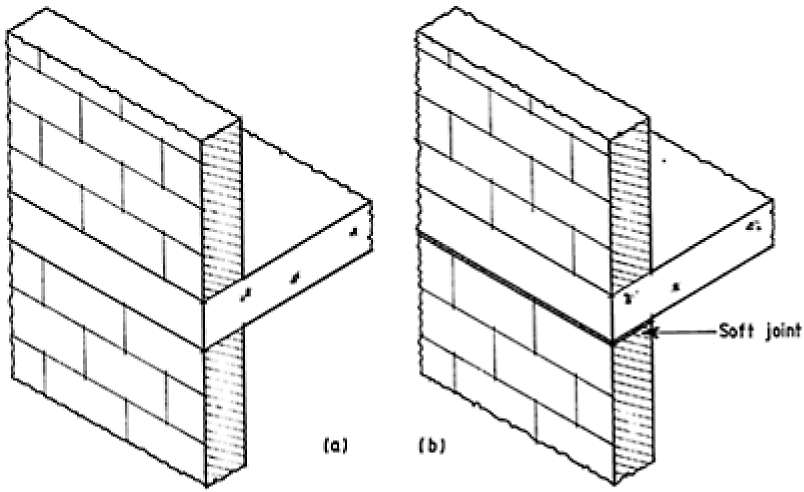


Figure A1 Horizontal support for panels:

(a) cast in situ floor slab—generally continuous support

(b) vertical anchors with adequate shear strength give simple support, else considered free

walls, where both leaves are reinforced, the design strength is considered to be the sum of the design strengths of the separate leaves.

A.4 Method two: Design with reinforced section carrying extra load only

This approach is one which, although relatively easy to use and may lead to economical solutions in some situations, cannot be justified theoretically. The resistance to lateral load of the similar unreinforced panel is calculated according to Clause 36 of Part 1 of BS 5628. A maximum enhancement in lateral load resistance of 30% of this amount is permitted. The frequency of reinforcement is determined by assuming that the load enhancement only is resisted by the reinforced section, the design is then as for any other rectangular reinforced section as in *Section 4*. Although bed joint reinforcement is considered as a means of enhancing the lateral load carrying capacity of walls and hence there is a logical basis to the method, it does attempt to combine the resistance of the uncracked unreinforced section with the capacity of the reinforced section which can only be fully mobilised when the section is cracked. Consequently, it is unlikely that this method will survive when greater experience in using the recommendations in the Code is gained.

A.5 Method three: Design using modified orthogonal ratio

This method is essentially an extension of the method in Part 1 of BS 5628 for unreinforced walls and can be used both for single leaf or cavity walls. For unreinforced walls the bending moment in the wall for given support conditions when it is subjected to uniform lateral load is calculated using the recommendations in Part 1 using the height/length ratio of the wall and the orthogonal ratio of the masonry. The orthogonal ratio, μ, is defined as the ratio between the characteristic strength for failure parallel to the bed joint to that when failure is perpendicular to the bed joint. The bending moments acting in a wall subjected to a characteristic wind load, W_k, are shown in Figure A3. The ratio of the bending moments acting

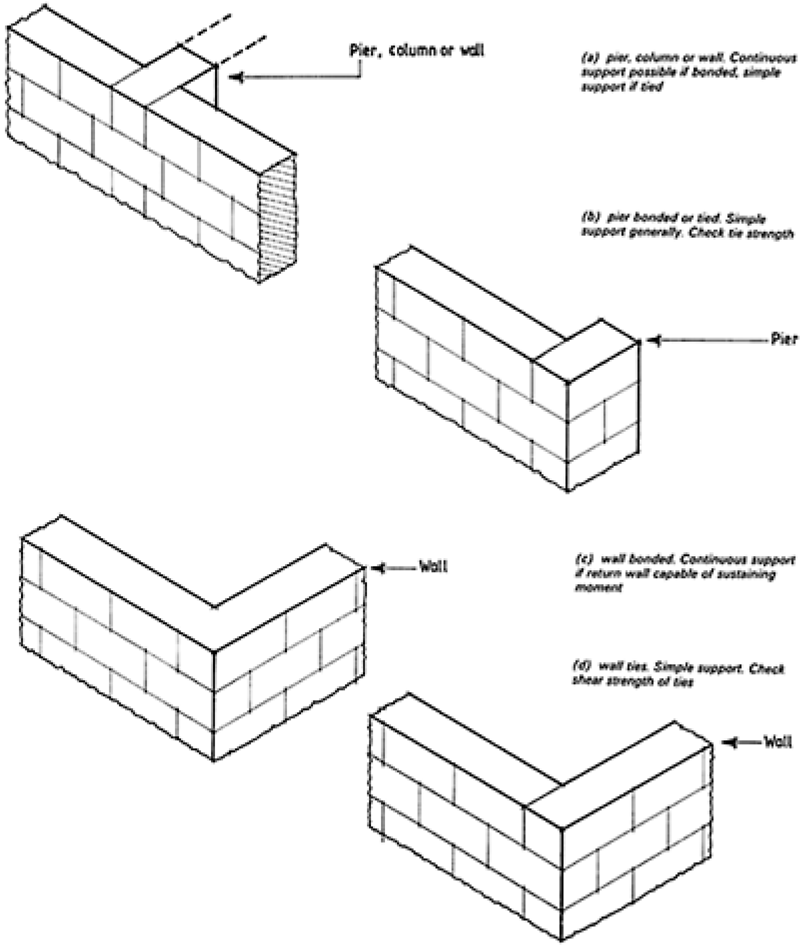


Figure A2 Vertical supports to panels:

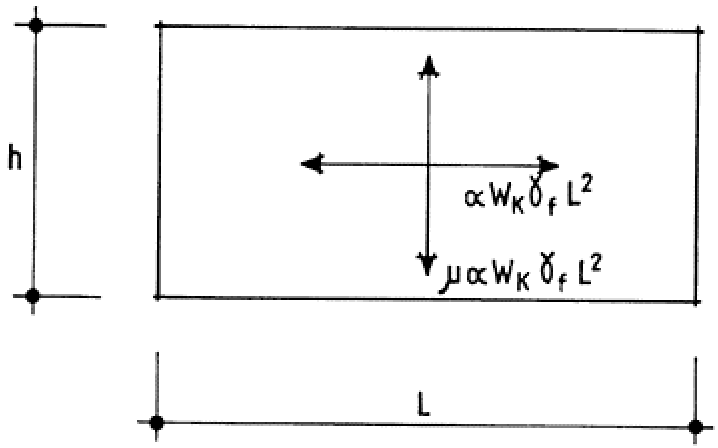


Figure A3 Design bending moments in orthogonal directions:

Characteristic wind load W_k Partial safety factor γ_f Orthogonal ratio μ Bending moment coefficient α

in the two orthogonal directions is the same as that of the flexural strengths in those directions. Consequently the design moment and design strength need only be compared in one direction. When bed joint reinforcement is used, the effect is to increase the moment of resistance about a vertical axis and this is considered as equivalent structurally to enhancing the flexural strength in that direction. In design, the simplest way to allow for this is to use the ratio of the moments of resistance in the orthogonal directions as the orthogonal ratio. The moment of resistance about the horizontal axis is given by:

$$\frac{f_{kx}}{\gamma_m} \cdot z$$

where f_{kx} is the characteristic flexural strength about a horizontal axis and z is the section modulus. The moment of resistance about the vertical axis is as used in Method 1. The design moment in the panel and the treatment of cavity walls are as for unreinforced walls in BS 5628: Part 1.

The addition of bed joint reinforcement to a wall enhances the moment of resistance in one direction only. Consequently the range of modular ratios will generally be outside that given in Part 1 of the Code. Table A.3 gives values of the bending moment coefficient for modular ratios below 0.3, these values have been derived using the same basic equations as those in Table 9 of Part 1 and are relevant to this method of design. The Table should be used with caution as, although based on the same theory as that which has been tested experimentally for unreinforced walls, the values have not yet been verified by testing.

Table A.3: Bending moment coefficients in laterally loaded wall panels

Note 1:

Linear interpolation of μ and $\frac{h}{L}$ is permitted

Note 2:

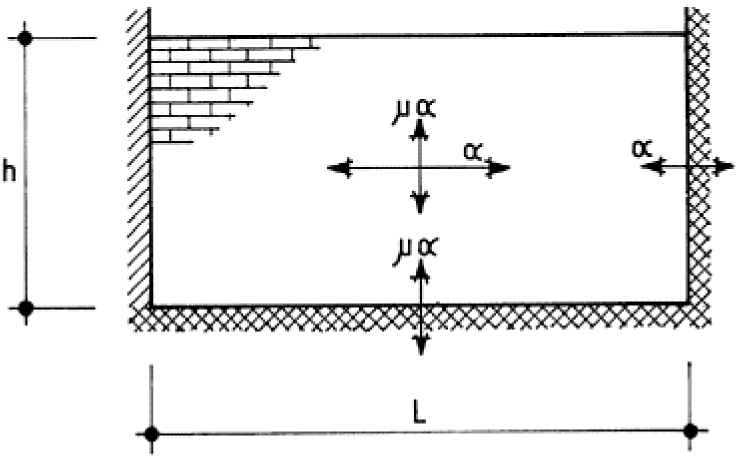
When the dimensions of a wall are outside the range of $\frac{h}{L}$ given in this Table, it will usually be sufficient to calculate the moments on the basis of a simple span. For example, a panel of Type “A” having $\frac{h}{L}$ less than 0.3 will tend to act as a freestanding wall, whilst the same panel having $\frac{h}{L}$ greater than 1.75 will tend to span horizontally.

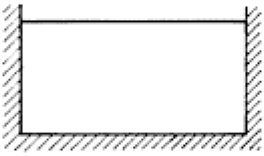
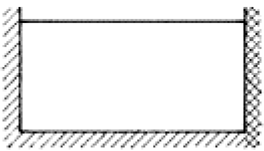
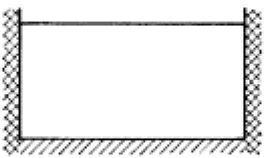
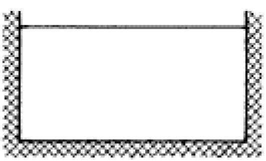
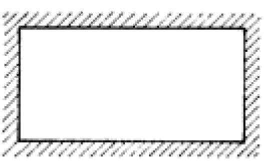
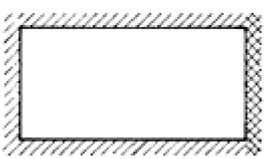
Key to support conditions:

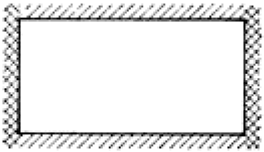
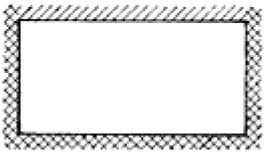
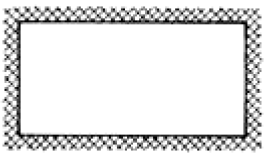
denotes simply supported edge

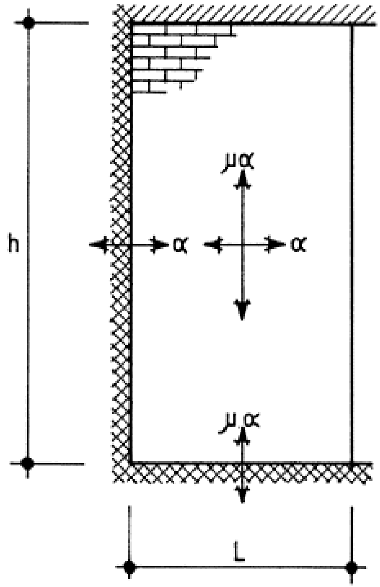
denotes free edge

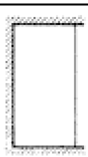
denotes an edge over which full continuity exists

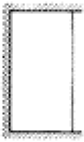
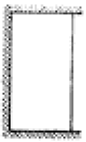


		$\mu/\frac{h}{L}$	Values of α						
			0.30	0.50	0.75	1.00	1.25	1.50	1.75
		0.25	0.050	0.071	0.085	0.094	0.099	0.103	0.106
		0.20	0.054	0.075	0.089	0.097	0.102	0.105	0.108
	A	0.15	0.060	0.080	0.093	0.100	0.104	0.108	0.110
		0.10	0.069	0.087	0.098	0.104	0.108	0.111	0.113
		0.05	0.082	0.097	0.105	0.110	0.113	0.115	0.116
		0.25	0.039	0.053	0.062	0.068	0.071	0.073	0.075
		0.20	0.043	0.056	0.065	0.069	0.072	0.074	0.076
	B	0.15	0.047	0.059	0.067	0.071	0.074	0.076	0.077
		0.10	0.052	0.063	0.070	0.074	0.076	0.078	0.079
		0.05	0.060	0.069	0.074	0.077	0.079	0.080	0.081
		0.25	0.032	0.042	0.048	0.051	0.053	0.054	0.056
		0.20	0.034	0.043	0.049	0.052	0.054	0.055	0.056
	C	0.15	0.037	0.046	0.051	0.053	0.055	0.056	0.057
		0.10	0.041	0.048	0.053	0.055	0.056	0.057	0.058
		0.05	0.046	0.052	0.055	0.057	0.058	0.059	0.059
		0.25	0.025	0.035	0.043	0.047	0.050	0.052	0.053
		0.20	0.027	0.038	0.044	0.048	0.051	0.053	0.054
	D	0.15	0.030	0.040	0.046	0.050	0.052	0.054	0.055
		0.10	0.034	0.043	0.049	0.052	0.054	0.055	0.056
		0.05	0.041	0.048	0.053	0.055	0.056	0.057	0.058
		0.25	0.023	0.042	0.059	0.071	0.080	0.087	0.091
		0.20	0.026	0.046	0.064	0.076	0.084	0.090	0.095
	E	0.15	0.032	0.053	0.070	0.081	0.089	0.094	0.098
		0.10	0.039	0.062	0.078	0.088	0.095	0.100	0.103
		0.05	0.054	0.076	0.090	0.098	0.103	0.107	0.109
		0.25	0.020	0.034	0.046	0.054	0.060	0.063	0.066
		0.20	0.023	0.037	0.049	0.057	0.062	0.066	0.068
	F	0.15	0.027	0.042	0.053	0.060	0.065	0.068	0.070
		0.10	0.032	0.048	0.058	0.064	0.068	0.071	0.073
		0.05	0.043	0.057	0.066	0.070	0.073	0.075	0.077

	G	0.25	0.018	0.028	0.037	0.042	0.046	0.048	0.050
		0.20	0.020	0.031	0.039	0.044	0.047	0.050	0.052
		0.15	0.023	0.034	0.042	0.046	0.049	0.051	0.053
		0.10	0.027	0.038	0.045	0.049	0.052	0.053	0.055
		0.05	0.035	0.044	0.050	0.053	0.055	0.056	0.057
	H	0.25	0.014	0.024	0.033	0.039	0.043	0.046	0.048
		0.20	0.016	0.027	0.035	0.041	0.045	0.047	0.049
		0.15	0.019	0.030	0.038	0.043	0.047	0.049	0.051
		0.10	0.023	0.034	0.042	0.047	0.050	0.052	0.053
		0.05	0.031	0.041	0.047	0.051	0.053	0.055	0.056
	I	0.25	0.011	0.021	0.030	0.036	0.040	0.043	0.046
		0.20	0.013	0.023	0.032	0.038	0.042	0.045	0.047
		0.15	0.016	0.026	0.035	0.041	0.044	0.047	0.049
		0.10	0.020	0.031	0.039	0.044	0.047	0.050	0.052
		0.05	0.027	0.038	0.045	0.049	0.052	0.053	0.055



	J	0.25	0.032	0.071	0.122	0.180	0.240	0.300	0.362
		0.20	0.038	0.083	0.142	0.208	0.276	0.344	0.413
		0.15	0.048	0.100	0.173	0.250	0.329	0.408	0.488
		0.10	0.065	0.131	0.224	0.321	0.418	0.515	0.613

		0.05	0.106	0.208	0.344	0.482	0.620	0.759	0.898
	K	0.25	0.028	0.056	0.091	0.123	0.150	0.174	0.196
		0.20	0.033	0.064	0.103	0.136	0.165	0.190	0.211
		0.15	0.040	0.077	0.119	0.155	0.184	0.210	0.231
		0.10	0.053	0.096	0.144	0.182	0.213	0.238	0.260
		0.05	0.080	0.136	0.190	0.230	0.260	0.286	0.306
	L	0.25	0.021	0.044	0.073	0.101	0.127	0.150	0.170
		0.20	0.025	0.052	0.084	0.114	0.141	0.165	0.185
		0.15	0.031	0.061	0.098	0.131	0.159	0.184	0.205
		0.10	0.041	0.078	0.121	0.156	0.186	0.212	0.233
		0.05	0.064	0.114	0.164	0.204	0.235	0.260	0.281

A.6 Method four: Design based on cracking load

The basis of this method is the experimental evidence that a wall containing bed joint reinforcement first cracks at a similar load to the ultimate loads of a similar unreinforced wall when tested in the laboratory. The ultimate load of the panel is calculated ignoring the reinforcement using the method in BS 5628: Part 1 but setting the partial safety factor, γ_m, to unity. The characteristic wind load for the reinforced panel is then found by dividing this load by the partial safety factor for material strength appropriate to the serviceability limit state. A check is then made that the wall is satisfactory at the ultimate limit state by using one of the other methods but not restricting the enhancement in strength due to the reinforcement. As this method is based on serviceability conditions and has the potential for the maximum load enhancement of all the methods, the designer should check that deflections will not be excessive. For this purpose, the wall may be considered to act as an elastic plate.

B. Wall ties for high-lift grouted cavity walls

Self-explanatory.

C. Estimation of deflection

The Appendix gives some guidance on the approach one should take to *estimate* the deflection of a member. The word *estimate* is emphasised here since accurate calculation of deflection for reinforced masonry is not possible with the present state of knowledge. Further guidance on the approach, or approaches, which can be taken is given in the following clauses, although the designer can, of course, consider other alternatives since the behaviour of reinforced masonry is analogous to reinforced concrete. It must be emphasised, however, that deflection checks will not usually be required.

The Code points out that a number of factors may be difficult to allow for in assessing deflection. The designer should consider their likely effect and, if necessary, adjust the estimated deflection accordingly. The following notes give some guidance:

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

For the purposes of design, supports are usually considered as simple or fixed; this is not often the case. For example, the amount of masonry above and to the side of a lintel can influence the rotation which can occur at bearings. Simple supports will usually be assumed for design purposes, whereas the surrounding masonry will inhibit free rotation. Where the height of the masonry above is greater than or equal to half the clear span and the supporting wall continues $\frac{l}{5}$ beyond the supports, the mid span deflection will be significantly reduced. This is due to the proximity to the support of the thrust from arching action of the masonry above and to the "immovable" restraint offered by the masonry. Any restraint which does occur will also induce bending moments at the supports.

Guidance on the assessment of load on lintels is available in BS 5977: Part 1:1981[5]. This states that experience has shown that it is safe to design a lintel to carry less than the sum of the applied loads and the weight of the masonry immediately above the lintel, the remainder being dispersed through the masonry on either side of the lintel, provided that:

1. all the weight of the masonry within a 45° load triangle is carried as a load on the lintel
2. any point or distributed loads applied to the masonry within the load triangle are dispersed at 45° and carried by the lintel
3. any point or distributed loads applied to the masonry within a 60° triangular interaction zone are reduced by 50%, dispersed at 45° and carried by the lintel
4. the weight of the masonry in the interaction zone is not carried by the lintel

It is assumed also that the following limiting conditions are satisfied:

1. the masonry is constructed following the recommendations of BS 5628: Part 3[6]
2. the height of masonry above the lintel at mid span is not less than 0.6 times the clear span of the lintel
3. the height of the masonry above the supports is not less than 600 mm
4. the masonry is continuous within the area defined by the conditions given in 1. and 2. above
5. where there is a single opening spanned by the lintel, the width of masonry on either side of the opening is not less than 600 mm or 0.2 times the clear span of the lintel, whichever is the greater
6. where there are a series of openings at the level of the opening spanned by the lintel, the length of the masonry between the external corner of the wall and the side of the adjacent opening is not less than 600 mm or 0.2 times the longest clear span, whichever is the greater.

In cases where conditions 2., 3. and 4. are not satisfied (for example, where the lintel directly supports a roof or a point load) the load on the lintel is required to be taken as the full value of the imposed load, plus the self-weight of the lintel.

Deflection of retaining walls, on the other hand, may be increased by $h\theta$ where h is the height of the wall and θ any rotation about the base. This rotation may be caused by long term settlement of the base (overturning) or by the fact that the cantilever is not, in practice, rigidly fixed at the base. Constructing the wall to a slight batter can compensate for this additional deflection and, in this case, rotation can often be ignored.

(b) *Precise loading or that part of it which is of long duration is unknown*

The dead load is the major factor determining the long term deflection of a member, and because the dead load is often known to within quite close limits, lack of knowledge of the precise long term imposed load is not likely to be a major cause of error in estimations of deflection. As a guide, all the dead load should be considered as permanent and for normal domestic, office, hospital, etc., occupancy, 25% of the imposed load should be considered as permanent. Structures such as warehouses and libraries with a significant amount of storage should have at least 75% considered as permanent.

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

In most members, some parts are likely to be cracked and others not. Analysis of such a member would required a knowledge of the critical cracking moment and the use of two or more values for stiffness. This approach is unnecessarily sophisticated for normal design purposes. The alternatives usually adopted are based on: (i) the uncracked section, (ii) the cracked section, ignoring the tensile strength of the masonry, or (iii) the partially cracked section in which the masonry is "given" some tensile strength. Method (iii) is that currently adopted in CP 110[7] but with the present state of knowledge is too sophisticated to apply to reinforced masonry. Method (i) is currently adopted in the Code, but in assessing the section modulus the contribution of the reinforcement is ignored. This simplifies matters with some loss in accuracy but nevertheless has been shown to correlate reasonably well with experimental results[8]. Method (ii) has been used with some success in predicting deflections in reinforced blockwork[9,10,11].

Further guidance on method (i) is given below:

An elastic analysis should be used to estimate deflections with the design loads being appropriate to those at the serviceability limit state. The following assumptions may be made for method (i) as given in the Code:

1. the section to be used for the calculation of stiffness is the gross section of the masonry, no allowance being made for the reinforcement, i.e., for rectangular sections, $I=\frac{bt^3}{12}$ where t is the depth of the masonry and b is the breadth of the section
2. plane sections remain plane
3. the masonry in compression is elastic, under short term loading the moduli of elasticity may be taken as the appropriate values given in *Section 19.1.7*. The long term elastic modulus, E_m, allowing for creep and shrinkage where appropriate, may be taken as $E_m=450\,f_k$ for clay masonry and $E_m=300\,f_k$ for calcium silicate and concrete masonry, where f_k is the characteristic compressive strength of masonry obtained from *Section 19.1*.

The deflected shape of a member is related to the curvature by the equation:

$$\frac{1}{r_x} = \frac{d^2y}{d_x^2}$$

where *y* is the deflection and x is the distance along the member from the origin, and $\frac{1}{r_x}$ is the curvature at point x.

The Code allows deflections to be calculated directly from this equation by calculating the curvatures of successive sections along the member and using numerical integration techniques to obtain the deflection at a point. Alternatively, since it is usually maximum or mid span deflection which is required, the following simplified approach may be used:

The deflection, a_d, is calculated from the equation:

$$a_d = k\, l^2 \frac{1}{r}$$

where: *l*=the effective span of the member

$\frac{1}{r}$ =the curvature at mid span or, for cantilevers, at the support section

k=a constant which depends on the shape of the bending moment diagram, some examples of which are given in Figures C1 to C3.

The curvature of $\frac{1}{r}$ may be taken simply as:

$$\frac{M}{E_m I}$$

where: *M*=the design bending moment (at service loads)

E_m=the elastic modulus of the masonry

I=the moment of inertia of the section, $\frac{bt^3}{12}$

For short term deflection:

$$\frac{1}{r_{it}} = \frac{M_t}{E_{mi} I}$$

where: $\frac{1}{r_{it}}$ =instantaneous curvature due to total load

M_t=total design bending moment

E_{mi}=initial or short term modulus, 900 f_k

For long term deflections:

(i) calculate the instantaneous curvatures due to the total load (as above) and that due to the permanent load, viz:

$$\frac{1}{r_{ip}} = \frac{M_p}{E_{mi} I}$$

where: $\frac{1}{r_{ip}}$=instantaneous curvature due to permanent load

M_p=permanent load

(ii) calculate the long term curvature due to the permanent loads:

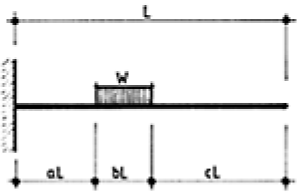
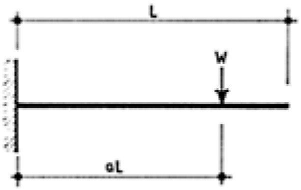
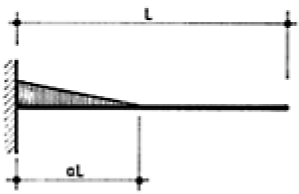
$$\frac{1}{r_{lp}} = \frac{M_p}{E_{ml} I}$$

where: $\frac{1}{r_{lp}}$=long term curvature due to the permanent loads

E_{ml}=long term modulus taking account of creep and shrinkage where appropriate, 450 f_k or 300 f_k

(iii) to obtain overall long term curvature, $\frac{1}{r_{O/A}}$, add to (ii) the difference between

Figure C1 Cantilevers

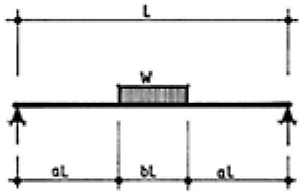
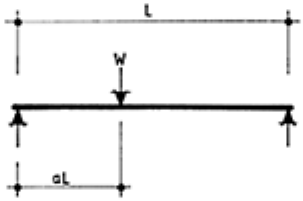
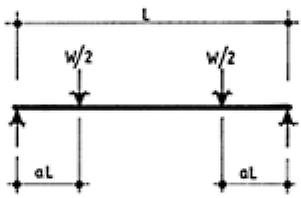
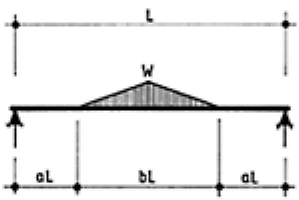
	$M_{max} = WL\left(a + \frac{b}{2}\right)$	$k_{max} = \frac{1}{24\left(a + \frac{b}{2}\right)}[8a^3 + 18a^2b + 12ab^2 + 3b^3 + 12a^2c + 12abc + 4b^2c]$
	When $a = 0$ $M_{max} = \frac{WbL}{2}$	$k_{max} = \frac{(4 - b)b}{12}$
	When $b = 1$ $M_{max} = \frac{WL}{2}$	$k_{max} = 0.25$
	$M_{max} = WLa$	$k_{max} = \frac{a(3 - a)}{6}$
	when $a = 1$ $M_{max} = WL$	$k_{max} = 0.333$
	$M_{max} = \frac{WaL}{3}$	$k_{max} = \frac{a^2}{20}(5a - 1)$
	When $a = 1$ $M_{max} = \frac{WL}{3}$	$k_{max} = 0.20$

curvatures obtained in (i), i.e.:

$$\frac{1}{r_{O/A}} = \frac{1}{r_{lp}} - \frac{1}{r_{ip}} + \frac{1}{r_{it}}$$

$$= \frac{1}{I}\left(\frac{M_p}{E_{ml}} - \frac{M_p}{E_{mi}} + \frac{M_t}{E_{mi}}\right)$$

Figure C2 Simply-supported members

	$M_{max} = \frac{WL}{8b}(1-4a^2)$ When $b = 1$ $M_{max} = \frac{WL}{8}$	$k_{max} = \frac{b}{48}\frac{(8-4b^2+b^3)}{(1-4a^2)}$ $k_{max} = 0.104$
	$M_{max} = WLa(1-a)$ When $a = \frac{1}{2}$ $M_{max} = \frac{WL}{4}$	$k_{max} = \frac{(4a^2-8a+1)}{48a}$ $k_{max} = 0.083$
	$M_{max} = \frac{WaL}{2}$	$k_{max} = 0.125 - \frac{a^2}{6}$
	$M_{max} = \frac{WL}{6}(1-a)$ When $a = 0$ $M_{max} = \frac{WL}{6}$	$k_{max} = \frac{1}{80}\frac{(8-7a-4a^2-4a^3)}{(1-a)}$ $k_{max} = 0.10$

Substituting the values for the short and long term moduli, we get: For concrete and calcium silicate masonry:

$$\frac{1}{r_{O/A}} = \frac{M_t}{900 f_k I}(1 + 2\alpha)$$

and for clay masonry:

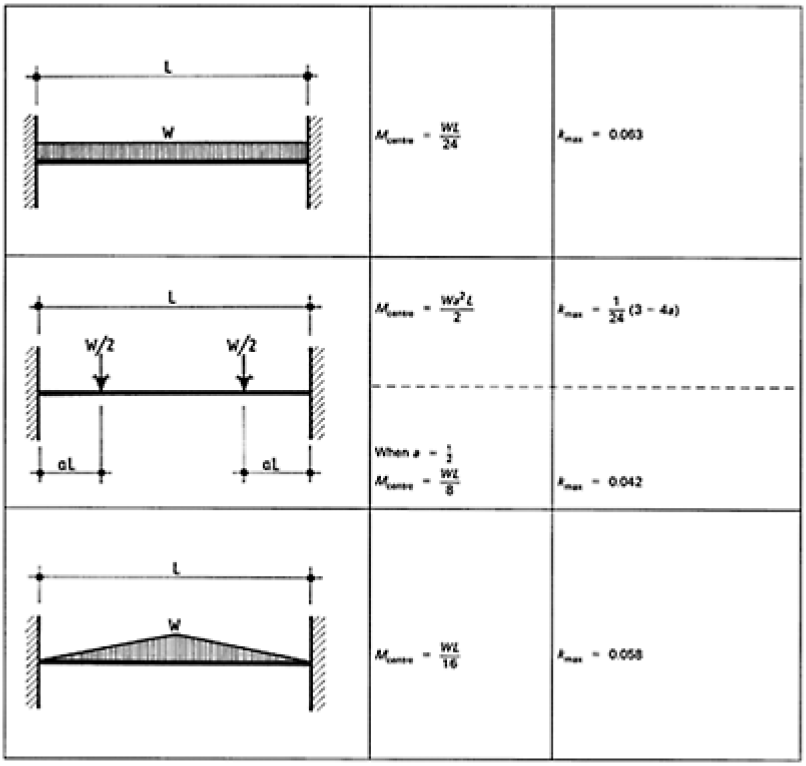
$$\frac{1}{r_{O/A}} = \frac{M_t}{900 f_k I}(1 + \alpha)$$

where α=the proportion of *total* load considered permanent.

Sample calculation for both long and short term deflection are given in Examples 14, 15 and 16.

It must be noted that, in general, the formulae tend to overestimate the deflection and in practice deflections are likely to be less. In those sections containing a high proportion of concrete infill, the elastic modulus of the infill will be much higher than that assumed for the masonry and this will cause estimations of deflection to be conservative.

Figure C3 Built-in members



D. Method for determination of characteristic strength of brick masonry, f_k

Appendix A2 of BS 5628: Part 1 describes an experimental method of determining the characteristic compressive strength of masonry. This involves testing two panels from 1.2 to 1.8 m in length and from 2.4 to 2.7 m high with a minimum cross sectional area of 0.125 m^2. Such a test specimen is clearly very expensive and is rarely used for design purposes. Furthermore, the correction factors applied to establish the characteristic compressive strength of the masonry from the test results were determined for squat specimens such as bricks and are disadvantageous when applied to units with a high aspect ratio.

Like Part 1[1], it is anticipated that most designers will use the Tables or Curves to determine the characteristic strength to be used for design purposes, and indeed the research which has been carried out to verify the design procedures contained in Part 2 has been related to the characteristic strength determined in this way. Reinforced and prestressed masonry will, however, often be used to produce elements in which the units are loaded in directions other than would occur in a wall. It is quite possible that some units (such as perforated or hollow units) are significantly weaker in directions not normally subjected to load. This Appendix has, therefore, been included to enable the direct determination of the characteristic compressive strength of brick masonry when loaded in any particular direction.

The assumption has been made that the various prism specimens may be taken to represent a large enough element of masonry to accurately determine f_k. Within the procedure specified, however, the size variations and test parameters could give rise to significant differences in results between laboratories. The lack of experience in the use of these procedures in the UK has resulted in the Code committee incorporating the qualification that the characteristic strength determined by test should not exceed the value obtained for the unit from the corresponding table for the normal direction of loading.

References

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Section Nine: Examples

EXAMPLE 8

EXAMPLE 1

PROBLEM

Design a 1.8 m high cantilever retaining wall, 215 mm thick, to resist a moment at the base of 9.4 kN m/m run with a shear force of 16.2 kN/m run.

Blocks		440×215×215, hollow, of unit strength 7 N/mm^2 with 55% solid
Mortar		Designation (ii)
Reinforcement		f_y=250 N/mm^2
Notes:	1.	γ_{mm}=2.3
	2.	exposure condition E3
	3.	place the steel in centre so that moment may be resisted equally from either side
	4.	use a concrete grade 30 to BS 5328
	5.	web thickness of block=40 mm

SOLUTION

Cover required (Table 14) =40 mm

Cover provided =107−40−10=57 mm $\therefore$ OK

The ratio of span to effective depth of this wall should be checked.

Effective span =length to face of support+ $\frac{1}{2}$ effective depth

$$= 1.8 + \frac{0.107}{2} = 1.85\,\mathrm{m}$$

$$\frac{\text{span}}{\text{effective depth}} = \frac{1.85}{0.107} = 17.3 < 18 \therefore$$

OK, and it is not necessary to check deflection and cracking by calculation.

For 7 N/mm^2 block with 55% solid, the net strength=12.7 N/mm^2, thus, f_k=6.2 N/mm^2 (Table 3(b))

The maximum design moment, M_d, should not exceed that of the balanced section. Hence:

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

$$= \left(\frac{0.4 \times 6.2 \times 1000 \times 107^2}{2.3}\right) \times 10^{-6}\ \text{kN m}$$

=12.34 kN m>9.4 $\therefore$ OK

Consider now the required area of steel:

$$M_d = \frac{A_s f_y z}{\gamma_{ms}} \text{ i.e., } z = \frac{M_d \gamma_{ms}}{A_s f_y}$$

and, z

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

Therefore

$$\frac{9.4 \times 10^6 \times 1.15}{A_s \times 250} = 107\left(1 - \frac{0.5 A_s \times 250 \times 2.3}{1000 \times 107 \times 6.2 \times 1.15}\right)$$

Hence, A_s =497 mm^2

Therefore, use R12 every core (225 mm)=502 mm^2/m run

Shear

Design shear force,

V =16.2 kN/m run

Therefore, v

$$= \frac{V}{bd} = \frac{16.2 \times 10^3}{1000 \times 107} = 0.15\ \text{N/mm}^2$$

*

$$\varrho = \frac{A_s}{bd} = \frac{502}{1000 \times 107} = 0.0047$$

Therefore, characteristic shear strength (Clause 19.1.3),

$$f_v = (0.35 + 17.5\varrho)\left(2.5 - 0.25\frac{a}{d}\right)$$

$$a = \frac{M}{V} = \frac{9.4}{16.2} = 0.58$$

$$\therefore f_v = (0.35 + 17.5 \times 0.0047)\left(2.5 - 0.25 \times \frac{0.58}{0.107}\right)$$

$=0.49$ N/mm^2

Thus, $$\frac{f_v}{\gamma_{mv}} = \frac{0.49}{2}$$

$=0.245$ N/mm^2

Therefore the wall has adequate resistance to shear.

Horizontal steel

The minimum horizontal steel required
$=0.0005\times107\times1000$
$=54$ mm^2/m run

Therefore, use one 6 mm diameter bar in alternate joints (63 mm^2/m) or use proprietary joint reinforcement.

For durability (exposure E3) these must be austenitic stainless steel or carbon steel coated with at least 1 mm of stainless steel.

Detailing

Its is possible to calculate the change point for providing, say, 12 mm starter bars to lap with 10 mm bars which run for the full height of the wall, but this may not be economical if the lap length is long. The required anchorage length should also be calculated. The horizontal steel should not touch the vertical steel if they are of dissimilar materials.

* The minimum value of $f_v = 0.35 \therefore \frac{f_v}{\gamma_{mv}} = \frac{0.35}{3} > v = 0.15\,\text{N/mm}^2$, which is adequate. The full calculation is shown for information.

EXAMPLE 2

PROBLEM

Axial load capacity of 2.8 m high wall

Blocks		390×190×190, hollow, of unit strength 21 N/mm^2 with 60% solid
Mortar		designation (i)
Reinforcement		one T12 each core, f_y=460 N/mm^2
Notes:	1.	simple lateral support provided top and bottom
	2.	γ_{mm}=2.3
	3.	exposure condition E2

SOLUTION

Cover required	=30 mm
Cover provided	=85−6−web thickness (35 mm)
	=44 mm $\therefore$ adequate

Simple lateral support provided, therefore $h_{ef}=h$

For single leaf wall $t_{ef}=t$

Therefore, slenderness ratio $= \dfrac{2.8}{0.19} = 14.7$

From Table 7 of Part 1, β=0.87

For 21 N/mm^2 block with 60% solid, the net strength=35 N/mm^2,

thus, f_k =14.7 N/mm^2 (Table 3(b))

$$N_d = \frac{\beta b t f_k}{\gamma_{mm}}$$

$$= \frac{0.87 \times 1000 \times 190 \times 14.7}{2.3 \times 10^3}$$

=1056 kN/m run

Note: This approach makes no allowance for the contribution of the reinforcement. It is possible to use the approach provided for columns in Part 2, but this is unlikely to give a more favourable result.

EXAMPLE 3

PROBLEM

Design a 4.0 m high column, 390×390, with axial load of 500 kN and moment of 60 kN m

Blocks		390×190×190, hollow, of unit strength 14 N/mm^2 with 55% solid
Mortar		designation (i)
Reinforcement		f_y=460 N/mm^2
Notes:	1.	lateral restraint in both directions top and bottom
	2.	γ_{mm}=2.3
	3.	exposure condition E2
	4.	web thickness of block=40 mm

SOLUTION (A)

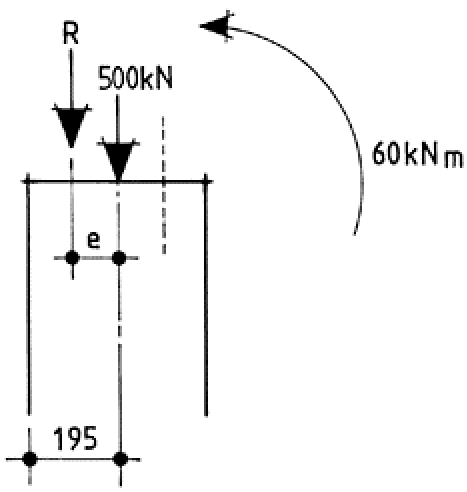
Lateral support is provided, therefore $h_{ef}=h$

$$\therefore \text{slenderness ratio} = \frac{4.0}{0.39} = 10.3 < 12$$

It is therefore a short column

For a 14 N/mm^2 block with 55% solid, the net strength=25.5 N/mm^2, thus, f_k=11.4 N/mm^2 (Table 3(b))

Assume T20 steel which, for exposure condition E2, requires 30 mm cover with a grade 30 concrete to BS 5328.



Resultant eccentricity, e $= \frac{60}{500} = 0.12\,\text{m} = 120\,\text{mm}$

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

$$= \frac{11.4}{2.3} \times 390 \times (390 - 240) \times 10^{-3}$$

$=290$ kN

Design axial load exceeds this, therefore, need to carry out a full analysis.

$$N_d = \frac{f_k}{\gamma_{mm}} b d_c + \frac{f_{s_1} A_{s_1}}{\gamma_{ms}} - \frac{f_{s_2} A_{s_2}}{\gamma_{ms}}$$

It is now necessary to choose a value of d_c, which should not be chosen as less than $2d_1$, where d_1 is the depth from the surface to the reinforcement in the more highly compressed face. Assume T20 steel, therefore, with exposure condition E2 (cover =30 mm) and block web thickness of 40 mm, gives

d_1 $=40+30+10=d_2$

Thus, $2d_1$ $=160$ mm

Choose d_c=250 mm. This value is between $(t-d_2)$=390−80=310, and $\frac{t}{2} = 195$ (where d_2 is the depth to the reinforcement from the least compressed face). In this range, f_{s2} is varied linearly between 0 and f_y, i.e., f_y when d_c=195 and 0 when d_c=310.

$$\therefore f_{s2} = \frac{60}{115} \times f_y = 0.52 f_y$$

$$\therefore N_d = \frac{11.4 \times 390 \times 250}{2.3 \times 10^3} + \left(\frac{0.83 \times 460}{1.15} - \frac{0.52 \times 460}{1.15}\right)\frac{628}{10^3}$$

$$= \frac{11.4 \times 390 \times 250}{2.3 \times 10^3} + \frac{0.31 \times 460}{1.15} \times \frac{628}{10^3}$$

$=483+78$

$=561$ kN

$N_d>500$ $\therefore$ this is adequate

$$M_d = \frac{0.5 f_k}{\gamma_{mm}} b d_c (t - d_c) + \frac{0.83 f_y}{\gamma_{ms}} A_{s_1} (0.5t - d_1) + \frac{f_{s_2}}{\gamma_{ms}} A_{s_2} (0.5t - d_2)$$

$$= \frac{0.5 \times 11.4}{2.3 \times 10^6} \times 390 \times 250 (390 - 250) + \frac{0.83 \times 460 \times 628}{1.15 \times 10^6} (195 - 80) + \frac{0.52 \times 460 \times 628}{1.15 \times 10^6} (195 - 80)$$

$=33.83+23.98+15.02$

$=72.8$ kN m

∴ this is adequate. Thus need 4 No T20 bars, one in each core.

SOLUTION (B): ALTERNATIVE SOLUTION USING INTERACTION CURVES

Lateral support is provided, therefore $h_{ef}=h$

∴ slenderness ratio $= \frac{40}{0.39} = 10.3 < 12$

It is therefore a short column.

For a 14 N/mm² block with 55% solid, the net strength=25.5 N/mm²,

thus, f_k $=11.4$ N/mm²

$$\frac{N}{bt f_k} = \frac{500 \times 10^3}{390 \times 390 \times 11.4}$$

$=0.29$

$$\frac{M}{bt^2 f_k} \quad \frac{60 \times 10^6}{390 \times 390^2 \times 11.4}$$

$=0.09$

d $=390-40-30-10$

=310 mm

Therefore, $\dfrac{d}{t} = \dfrac{310}{390}$

=0.8

From interaction diagram for f_y=460 N/mm^2

$\dfrac{\varrho}{f_k}$ $=6\times10^{-4}$, where $\varrho = \dfrac{A_s}{bt}$

Therefore, A_s $=6\times10^{-4}\times11.4\times390\times390$

$=1040$ mm^2

Therefore, use 4 No T20 (=1260 mm^2), one each core.

EXAMPLE 4

PROBLEM

As *Example 3,* but design 6.0 m high column.

SOLUTION (A)

Lateral support is provided, therefore $h_{ef}=h$

$\therefore$ slenderness ratio= $= \dfrac{6.0}{0.39} = 15.4 < 27$

The slenderness ratio is greater than 12 and it must, therefore, be designed as a slender column with account taken of the additional moment induced by vertical load due to lateral deflection. This may be taken as:

$$\frac{N\,(h_{ef})^2}{2000\,t} = \frac{500 \times 6.0^2}{2000 \times 0.39}$$

$$= 23.1 \text{ kN m}$$

Assume Y25 steel which, for exposure condition E2, requires 30 mm cover with a concrete grade 30 to BS 5328.

$\therefore d_2=d_1$=83 mm

As before, assume d_c=250 mm. By consideration of previous example, N_d is adequate.

$$M_d = \frac{0.5 \times 11.4}{2.3 \times 10^6} \times 390 \times 250\,(390 - 250) + \frac{0.83 \times 460 \times 982}{1.15 \times 10^6}(195 - 83) + \frac{0.52 \times 460 \times 982}{1.15 \times 10^6}(195 - 83)$$

=33.83+36.51+22.88

=93.2 kN m

This is greater than 60+23.1 kN m, therefore adequate.

SOLUTION (B): ALTERNATIVE SOLUTION USING INTERACTION CURVES

Lateral support is provided, therefore $h_{ef}=h$

$$\therefore \text{slenderness ratio} = \frac{6.0}{0.39} = 15.4 < 27$$

The slenderness ratio is greater than 12 and it must, therefore, be designed as a slender column with due account taken of the additional moment induced by the vertical load due to lateral deflection. This may be taken as:

$$\frac{N\,(h_{ef})^2}{2000\,t} = \frac{500 \times 6.0^2}{2000 \times 0.39} = 23.1 \text{ kN m}$$

$$\frac{N}{bt\,f_k} = \frac{500 \times 10^3}{390 \times 390 \times 11.4}$$

=0.29

$$\frac{M}{bt^2 f_k} = \frac{(60 + 23.1) \times 10^6}{390 \times 390^2 \times 11.4}$$

=0.12

As with *Example 3, d/t,* f_y=460 N/mm^2

Therefore, from interaction diagram for f_y=460

$\frac{\varrho}{f_k}$ $=8\times10^{-4}$, where $\varrho = \frac{A_s}{bt}$

Therefore, A_s $=8\times10^{-4}\times11.4\times390\times390$

$=1388\ \text{mm}^2$

Therefore, use 4 No T25 ($=1960\ \text{mm}^2$) one each core.

EXAMPLE 5

PROBLEM

Design a beam to span a 3.8 m opening in a blockwork wall. The beam is subjected to a moment of 20 kN m and a shear force of 18 kN

Blocks		390×190×190, hollow, of unit strength 7 N/mm^2 with 55% solid
Mortar		designation (i)
Reinforcment		f_y=460 N/mm^2
Notes:	1.	γ_{mm}=2.3
	2.	exposure condition E1
	3.	use bond beam blocks with 50 mm of web left intact

SOLUTION

Initial attempt: single course beam

For 7 N/mm^2 block with 55% solid, the net strength=12.7 N/mm^2,

thus, f_k $=6.8\ \text{N/mm}^2$

Web is 50 mm thick, cover is 20 mm, say 20 mm dia. bar. Therefore, effective depth,

d=190–50–20–10

=110 mm

Using charts:

$$\frac{M}{f_k\,bd^2} = \frac{20\times10^6}{6.8\times190\times110^2}$$

=1.28>0.174

Try again, two course beam

Effective depth, d $=390-50-20-10$

$=310$ mm

Check lateral stability first, $3.8\text{ m} \not> 60\,b_c$ or $\dfrac{250\,b_c^2}{d}$ whichever is the lesser:

$60\,b_c$ $=60\times0.19$

$=11.4$ m

$$\frac{250\,b_c^2}{d} = \frac{250\times190^2}{310}$$

$=29.1$ m

both >3.8 m $\therefore$ OK

Using charts:

$$\frac{M}{f_k\,bd^2} = \frac{20\times10^6}{6.8\times190\times310^2}$$

$=0.16<0.174 \therefore$ OK

This gives $\dfrac{\varrho}{f_k}$ $=5.3\times10^{-4}$, where $\varrho = \dfrac{A_s}{bd}$

Therefore, A_s $=5.3\times10^{-4}\times6.8\times190\times310$

$=212\text{ mm}^2$

Provide 2 No T12 ($=226\text{ mm}^2$)

Shear

Design shear force,

V $=18$ kN

$$v = \frac{V}{bd} = \frac{18\times10^3}{190\times310} = 0.31\text{ N/mm}^2$$

$$\frac{A_s}{bd} = \frac{226}{190\times310} = 0.0038$$

Therefore, characteristic shear strength (Clause 19.1.3.1.2),

$$a = \frac{M}{V} = \frac{20}{18} = 1.11$$

$$\therefore f_v = (0.35 + 17.5 \times 0.0038)\left(2.5 - 0.25\,\frac{1.11}{0.31}\right)$$

$=0.67\ \text{N/mm}^2$

Thus, $$\frac{f_v}{\gamma_{mv}} = \frac{0.67}{2.0} = 0.34 > 0.31$$

Consider providing nominal shear reinforcement for 1 m in from each end:

$$\frac{A_{sv}}{s_v} = 0.002\,b_t$$

A_{sv} $=200\times0.002\times190$

$=76\ \text{mm}^2$ (two legs)

Therefore, provide nominal throughout, R8 @ 200 (=100.6 mm^2 two legs). Detail as for reinforced concrete to CP 110.

EXAMPLE 6

PROBLEM

Design the stem of a freestanding reinforced concrete masonry perimeter wall to be built from 440×215×215 mm, two core hollow blocks with a net strength of 10 N/mm^2, made to normal category of manufacturing control. Mortar designation (i) will be employed. The wind load is as follows:

Basic wind speed, v	=46 m/s
Wall height	=2.65 m
Topography factor, s_1	=1.0
Roughness, size and height factor, s_2	=0.74
Statistical factor, s_3	=1.0
s_4	=1.0

SOLUTION

Span $$= \text{height} + \frac{d}{2}$$

Check span/depth ratio $= \frac{2.65 + 0.05}{0.107} = 25$

Table 8 permits 18+30% for wind load only=23.4 ∴need to check deflection and cracking:

Design wind speed,

V_s =46×1.0×1.0×0.74×1.0

=3.40 m/s

Dynamic wind pressure, q $= \frac{0.613 \times 34^2}{10^3}$

=0.71 kN/m^2

For worst possible case, C_f=2

∴total load on wall,

F =2×0.71×2.7

=3.83 kN/m run

∴moment to beresisted by wall $= 3.83 \times \frac{2.7}{2} \times \gamma_f$

Choose γ_f=1.2, not 1.4, because the wall does not affect the stability of a structure.

∴moment =6.2 kN m

The wind is incidental from either direction ∴place steel in the centre of the core

d =107 mm

Consider 1 No 12 mm ϕbar in each alternate core (area of each bar=113 mm^2) Steel is at 450 centres=2.2 bars/metre

A_s =248 mm^2/m

Now, z $= d\left(1 - \frac{0.5 A_s f_y \gamma_{mm}}{bd\, f_k \gamma_{ms}}\right) < 0.95\, d$

∴z $= 107\left(1 - \frac{0.5 \times 248 \times 460 \times 2.3}{1 \times 0.107 \times 5.7 \times 1.15 \times 10^6}\right)$

=107 (1−0.19)

87 mm<0.95 d

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

$$= \frac{248 \times 460 \times 87}{1.15 \times 10^6}$$

$= 8.6$ kN m

which is greater than the moment produced by the wind load.

Check that the masonry strength is adequate:

$$\frac{0.4 f_k bd^2}{\gamma_{mm}} = \frac{0.4 \times 5.7 \times 1 \times 107^2}{2.3 \times 10^3}$$

$= 11.3$ kN m $\therefore$ OK

Note: secondary reinforcement of 0.05% *bd* is required.

Shear

$$v = \frac{V}{bd} = \frac{3.83 \times 10^3}{10^3 \times 107}$$

$= 0.036$ N/mm^2

$f_v = 0.35 + 17.5\varrho$ $\therefore$ adequate

Deflection

Now deflection needs to be checked:

deflection, $a = kl^2 \dfrac{M}{EI}$

k may be determined from Table…to this handbook, or derived using the moment/area theorem—"the deflection of a point on a member, measured from the tangent at another point on the member, is equal to the moment of the $\dfrac{M}{EI}$ diagram between the two points about the point whose deflection is sought."

$$a = \left(\frac{lM}{EI}\frac{l}{2}\right) - \left(\left[\frac{2}{3} l \frac{M}{EI}\right]\left[\frac{3}{8} l\right]\right)$$

$$= \frac{1}{2} l^2 \frac{M}{EI} - \frac{1}{4} \frac{l^2 M}{EI} = \frac{1}{4} l^2 \frac{M}{EI}$$

∴k =0.25

The moment applied to the wall $= \frac{Wl}{2}$

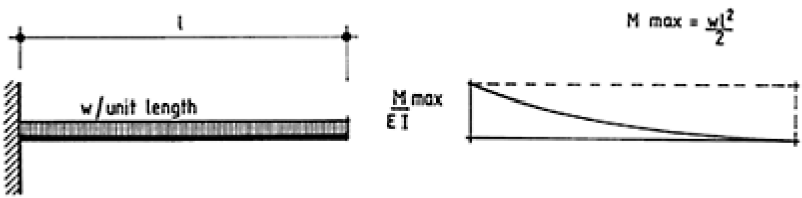
∴deflection, *a* $= \frac{Wl^3}{8EI}$

I is based on the gross cross section ignoring the steel.

I $= \frac{bt^3}{12}$

$$= \frac{1000 \times 215^3}{12} \text{ mm}^4$$

$=828\times10^6$ mm^4



For short term loading, which is applicable to this example since wind loads are neither continuous nor from one direction only:

E $=900 f_k$ N/mm^2

$=900\times10^{-3}\times5.7$ kN/mm^2

∴*a*
$$= \frac{3.83 \times 2700^3}{8 \times 900 \times 10^{-3} \times 5.7 \times 828 \times 10^6}$$

=2.2 mm

Note that Clause 16.2.2.1(a) is not applicable when short term loading is considered, neither does part (b) of this clause apply if no applied finishes are to be placed on the wall. Assume blockwork is to be rendered (i.e., (b) applies) and limiting deflection is therefore the lesser of $\frac{\text{span}}{500} = 5.2\,\text{mm}$ or 20 mm. Thus the deflection in this example is acceptable at $\frac{\text{span}}{1080}$. Unacceptable cracking is not likely at this deflection.

EXAMPLE 7

PROBLEM

Design a two course bond beam to support a uniformly distributed load of 9.75 kN/m run over a span of 2.8 m. The blocks to be used have a block strength of 5.5 N/mm^2 and are 55% solid. They are to be laid with a mortar of designation (ii). The exposure category is E2. The blocks to be used are of size 440×215×215.

SOLUTION (A)

Loads

Imposed load	=9.75 kN/m run
Self weight	$=0.215\times0.44\times2300\times10^{-3}\times9.81$
	=2.2 kN m/run
Design load	$=1.6\times9.75+1.4\times2.2$
	=18.68 kN/m run
Effective span	=the lesser of 2.80+0.215=3.01 or 2.81+0.354=3.16

Therefore take effective span as 3.0 m

$$\frac{\text{span}}{\text{effective depth}} = \frac{3.0}{0.354} = 8.47$$

This is less than the value of 20 in Table 9 and no detailed calculation of deflection is required.

The lateral stability requirement is that the clear distance between lateral restraints does not exceed 60 b_c or 250 $\frac{b_c^2}{d}$ whichever is the lesser:

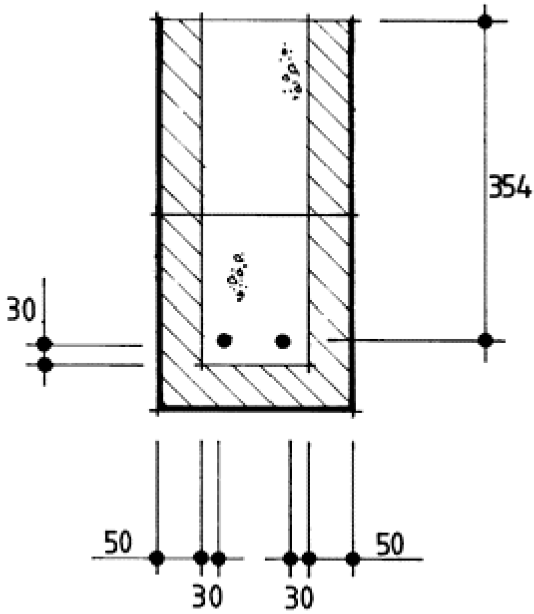
60 b_c $=60\times0.215=12.9$ m

$$250\,\frac{b_c^2}{d} = \frac{250 \times 0.215^2}{0.354} = 32.6 \text{ m}$$

both of which are greater than the span (3 m).

Design bending moment $= \dfrac{18.68 \times 3^2}{8} = 21.02$ kN m

For 5.5 N/mm^2 block with 55% solid, the net strength=10 N/mm^2. For mortar designation (ii), f_k=5.4 N/mm^2



Assume 2 No 12 mm ϕ bars OK.

A_s =226 mm^2

z
$$= d\left(1 - \frac{0.5 A_s f_y \gamma_{mm}}{bd\, f_k \gamma_{ms}}\right) < 0.95$$

$\therefore z$
$$= 354\left(1 - \frac{0.5 \times 226 \times 460 \times 2.3}{215 \times 354 \times 5.4 \times 1.15}\right)$$

=264 mm<0.95d

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

$$= \frac{226 \times 460 \times 264 \times 10^{-6}}{1.15}$$

=23.9 kN m

This is adequate to resist design bending moment but it is necessary to check the capacity of the masonry is not exceeded:

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

$$= \frac{0.4 \times 5.4 \times 215 \times 354^2}{2.3 \times 10^6}$$

$= 25.3$ kN m $\therefore$ OK

Shear

Design shear stress, v

$$= \frac{18.68 \times \frac{3}{2} \times 10^3}{215 \times 354}$$

$= 0.37$ N/mm^2

Shear span

$$= \frac{21.02}{28}$$

$= 0.75$ m

$\therefore f_v$

$$= (0.35 + 17.5\varrho)\left(2.5 - 0.25\frac{a}{d}\right)$$

$$= \left(0.35 + \frac{17.5 \times 226}{215 \times 354}\right)\left(2.5 - \frac{0.25 \times 750}{354}\right)$$

$= 0.79$ N/m^2

$\therefore \frac{f_v}{\gamma_{mm}}$

$$= \frac{0.79}{2}$$

$= 0.39$ N/m^2 which is greater than the design shear stress

Although shear reinforcement is not necessary to satisfy the calculations, it is suggested that nominal shear reinforcement be provided in a beam of this size. Therefore, provide beam links as indicated in Clause 26.5.2, such that

$$\frac{A_{sv}}{s_v} = 0.0012\, b_t$$

for high yield steel, noting that the spacing of the reinforcement, s_v should not exceed 0.75 *d*.

SOLUTION (B): ALTERNATIVE SOLUTION USING DESIGN CHARTS

B1. Using design chart provided in Section 4

Calculate $\frac{M}{bd^2 f_k}$ assuming 12 mm ϕ bars as before for the purpose of assessing d

$$= \frac{21.02 \times 10^6}{215 \times 354^2 \times 5.4}$$

$=0.144$

From chart, $\frac{\varrho}{f_k}$ $= 4.4 \times 10^{-4}$ where $\varrho = \frac{A_s}{bd}$

$\therefore \frac{A_s}{bd}$ $=4.4 f_k \times 10^{-4}$

A_s $=215 \times 354 \times 4.4 \times 5.4 \times 10^{-4}$

$=180.8\ \text{mm}^2$

Therefore, use 2 No 12 mm bars $A_s=226\ \text{mm}^2$. Otherwise complete example as before.

B2. Using design chart provided in Part 2

M_d $Q\,bd^2$

and Q $= 2c(1-c)\frac{f_k}{\gamma_{mm}}$

where $c = \frac{z}{d}$

$\frac{f_k}{\gamma_{mm}}$ $= \frac{5.4}{2.3}$

$=2.35$

Q $= \frac{M_d}{bd^2}$

$$= \frac{21.02 \times 10^6}{215 \times 354^2}$$

$=0.780$

again assuming d for 12 mm diameter bars.

$\therefore$ from chart, $\dfrac{z}{d}$ $=0.75$

A_s $= M_d \dfrac{\gamma_{ms}}{f_y} \dfrac{1}{z}$

$$= \frac{21.02 \times 1.15 \times 10^6}{460 \times 0.75 \times 354}$$

$=198 \text{ mm}^2$

Therefore, use 2 No 12 mm bars $A_s=226 \text{ mm}^2$. Otherwise complete example as before.

PROBLEM

Consider a cavity wall consisting of an external leaf of brickwork and an internal leaf of 150 mm thick solid concrete blockwork. The cavity is 50 mm wide. The wall is 8 m long and 4 m high and may be considered as simply supported along the vertical sides and at the base. The wall is in a relatively sheltered position in London and the design wind load calculated in accordance with CP 3: Chapter V: Part 2 and applying a partial safety factor of 1.2 is 0.45 kN/m^2.

SOLUTION (A): WITHOUT REINFORCEMENT

A1. Brickwork leaf

Bricks	Compressive strength 20 N/mm^2
Water absorption	15%
Mortar	designation (ii)
γ_{mm}	$=2.5$
Hence:	characteristic compressive strength=6.4 N/mm^2 characteristic flexural strengths (a) parallel to bed joints=0.3 N/mm^2 (b) perpendicular to bed joints=0.9 N/mm^2 orthogonal ratio, $\mu=0.33$
Wall panel	$\dfrac{h}{L} = 0.5$

Hence, from Table 9, BS 5628: Part 1:

Moment coefficient, α $=0.065$

$\alpha\,(W_k\gamma_f)\,L^2$ $= \dfrac{f_{kx}}{\gamma_m} \cdot Z$

Thus:

$$W_k\gamma_f = \frac{0.9 \times 10^3 \times 102.5^2}{2.5 \times 6 \times 10^6 \times 64 \times 0.065}$$

$=0.15\ kN/m^2$

A2. Blockwork leaf

Blocks 3.5 N/mm^2

Mortar designation (iii)

γ_{mm} =2.5

Hence: characteristic compressive strength=2.7 N/mm^2 characteristic flexural strengths
(a) parallel to bed joints=0.22 N/mm^2
(b) perpendicular to bed joints=0.38 N/mm^2 othogonal ratio, μ=0.58

From Table 9, BS 5628: Part 1:

Moment coefficient, =0.054

$$\alpha(W_k\gamma_f)\ L^2 = \frac{f_{kx}}{\gamma_m} \cdot z$$

Thus:

$$W_k\gamma_f = \frac{0.38 \times 10^3 \times 150^2}{2.5 \times 6 \times 10^6 \times 64 \times 0.054}$$

$=0.16\ kN/m^2$

Design strength of cavity wall

$=0.16+0.15\ kN/m^2$

$\therefore$ design strength $=0.31\ kN/m^2 < 0.45$ $\therefore$ not adequate.

SOLUTION (B): WITH REINFORCEMENT

Consider now the strength when bed joint reinforcement is placed at a spacing of 225 mm vertically in both leaves. The design being carried out according to Method 1 (Appendix A), i.e., horizontally spanning.

B1. Brickwork leaf

Bed Joint reinforcement 60 mm spacing to two 10 mm^2 parallel wires

Characteristic strength $=485\ N/mm^2$

Partial safety factor =1.15

effective depth, d

$$= \frac{102.5}{2} + \frac{60}{2}$$

=81 mm

Design of under-reinforced section, ignoring compression in reinforcement

lever arm, z

$$= d\left(1 - \frac{0.5A_s f_y \gamma_{mm}}{bd f_k \gamma_{ms}}\right) < 0.95d$$

$$= d\left(1 - \frac{1}{2}\frac{10}{225 \times 81}\frac{485}{6.4}\frac{2.5}{1.15}\right)$$

=0.95d (upper limit=formula gives 0.96d)

M_d

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

$$= \frac{485}{1.15} \times \frac{10 \times 81 \times 0.95}{10^6}$$

=0.32 kN m per 225 mm height

$\therefore M_d$ =1.42 kN m per m height

B2. Blockwork leaf

Bed joint reinforcement 100 mm spacing to two 10 mm^2 wires

effective depth, d

$$= \frac{150}{2} + \frac{100}{2}$$

=125 mm

Design of under-reinforced section, ignoring compression in reinforcement

lever arm, z

$$= d\left(1 - \frac{10 \times 485 \times 2.5}{2 \times 225 \times 125 \times 3.5 \times 1.15}\right)$$

=0.93d

$\therefore M_d$

$$= \frac{485}{1.15} \times \frac{10 \times 125 \times 0.93}{10^6}$$

=0.49 kN m per 225 m height

$\therefore M_d$ =2.18 kN m per m height

Design moment resistance of cavity wall

$=1.42$ kN m$+2.18$ kN m

$\therefore$ design moment $=3.60$ kN m/m height

For a horizontally spanning simply supported wall panel:

$$(W_k \gamma_f)\frac{L^2}{8} = 3.60$$

$$\therefore W_k\gamma_f = \frac{3.60 \times 8}{8^2}$$

$=0.45$ kN/m^2

Hence design strength of reinforced wall$=0.45$ kN/m^2. Strength enhancement above that for unreinforced wall

$$= \frac{0.45 - 0.31}{0.31} \times 100\%$$

$$= 45\%$$

Hence strength enhancement<50% (upper limit)

Design strength (0.45 kN/m^2)=Design load (0.45 kN/m^2)

WALL TIES

Consider the load carried by the wall ties:

Load carried by inner leaf $=2.18$ kN/m^2

Tie spacing $=900$ m horizontally
450 mm vertically

i.e., 2.5 ties/m^2

Design force per tie $= \dfrac{2.18}{2.5} = 0.9\,\text{kN}$

Partial safety factor for strength of wall ties=3

Hence required characteristic strength of wall tie=2.7 kN. From Table 8 of BS 5628: Part 1, vertical twist ties, whose characteristic load is 4.0 kN, are required.

CONCLUSION

Consequently, the design consists of bed joint reinforcement of two wires 10 mm^2 at 60 mm and 100 mm width in the brickwork and blockwork respectively. The vertical spacing is 225 mm in both leaves. The wall ties are vertical twist type at 900 mm and 450 mm spacing. Care is necessary to ensure that the reinforcement and wall ties have adequate protection against corrosion and dissimilar metals should not make contact.

EXAMPLE 9

PROBLEM

Consider the assessment of the panel in *Example 8* using Method 4, i.e., based on cracking load. The design strength of the unreinforced cavity wall was found to be 0.31 kN/m^2, taking γ_m=2.5. Hence, with γ_m=1, $W_k\gamma_f$=0.78 kN/m^2.

SOLUTION

The partial safety factor for masonry strength for the serviceability limit state is 1.5. Therefore, characteristic wind load, W_k=0.52 kN/m^2. Hence to check ultimate limit state design load=0.52×1.2. Design load=0.62 kN/m^2.

In the previous Example the design strength of the reinforced wall was 0.45 kN/m^2 which represented a strength enhancement of less than 50%. Consequently, although Method 4 will allow a greater characteristic working load, in this case 0.52 kN/m^2 as opposed to the 0.46 kN/m^2 in the previous Example,. more reinforcement is required to do this.

In this particular case, assuming that the blockwork courses are 225 mm in height, the only possibility, apart from using heavier guage reinforcement, is to place the reinforcement at a closer vertical spacing in the outer leaf. For this Example the necessary spacing is 75 mm and the design strength is then 0.94 kN/m^2. Deflection calculations for this type of wall are not straightforward, however, if a simple crude check is made in the first instance on each leaf considered as acting separately and spanning horizontally, using 450 f_k (ignoring the reinforcement) the deflections are clearly low at the serviceability load.

EXAMPLE 10

PROBLEM

Consider the same panel as in the previous examples, in this case using the method based on the modified orthogonal ratio (Method 3).

SOLUTION

(i) Brickwork leaf

Moment of resistance about a horizontal axis

$$= \frac{0.3}{2.5} \times \frac{10^3 \times 102.5^2}{6} \frac{1}{10^6}$$

	=0.21 kN m/m run
Moment of resistance about a vertical axis	=1.42 kN m/m run
Modified orthogonal ratio, μ	=0.15

(ii) Blockwork leaf

Moment of resistance about a horizontal axis	$= \frac{0.22}{2.5} \times \frac{150^2}{6} \times \frac{10^3}{10^6}$
	=0.33 kN m/m run
Moment of resistance about a vertical axis	=2.18 kN m/m run
Modified orthogonal ratio, μ	=0.15

From the table, $\frac{h}{L} = 0.5$ for the brickwork leaf the moment coefficient is α=0.080 (by interpolation). Similarly, for the blockwork leaf α=0.080.

Thus, for the brickwork leaf:

$$W_k\gamma_f = \frac{1.42}{64 \times 0.080} = 0.28 \text{ kN/m}^2$$

and for the blockwork leaf:

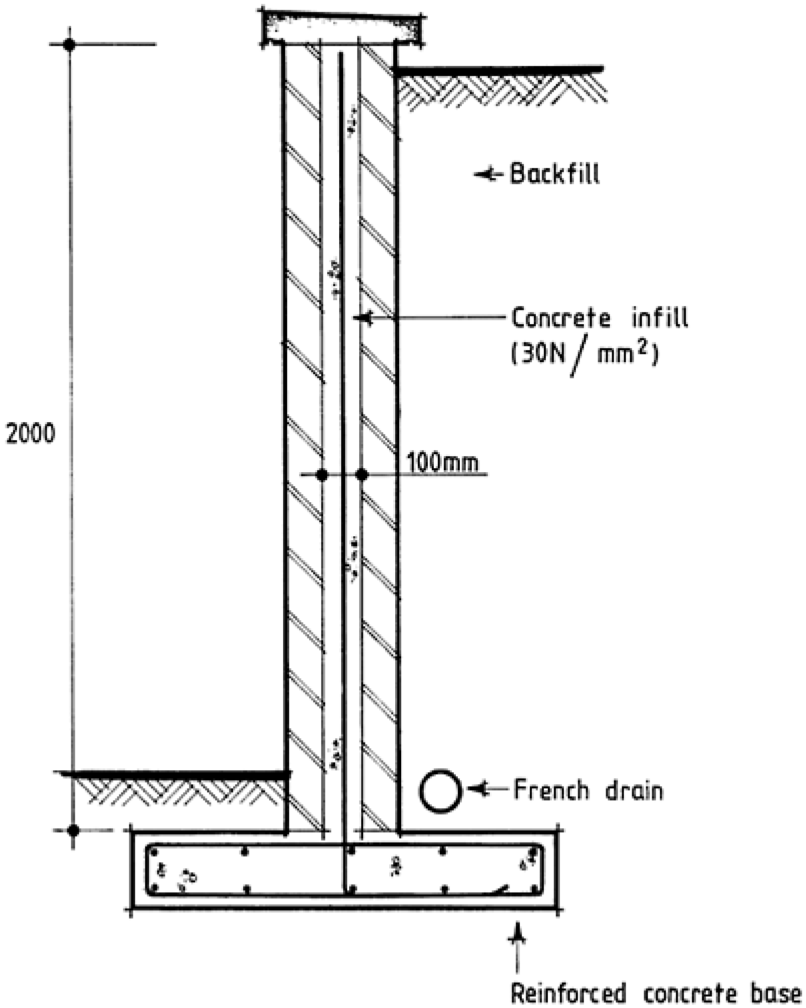
$$W_k\gamma_f = \frac{2.18}{64 \times 0.080} = 0.43 \text{ kN/m}^2$$

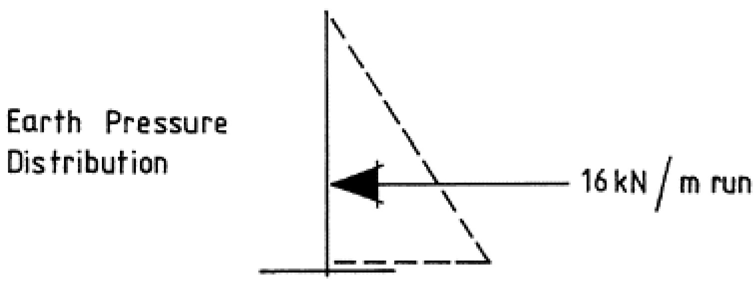
and the design strength of the cavity wall is 0.28+0.43 kN/m^2; so design strength of the cavity wall is 0.71 kN/m^2. In this particular instance the calculated design strength of the wall is greater than that from Method 1 and also at 0.71 kN/m^2, greater than the limiting value for Method 4 (0.62 kN/m^2).

EXAMPLE 11

PROBLEM

Consider the design of a reinforced brickwork retaining wall. The wall, illustrated below, is to be built in grouted cavity construction and is to be 2 m high.





Let the service load due to earth pressure be a triangular pressure distribution, the overall force being 16 kN/m acting at a point one third the height of the wall from the base.

Hence, as γ_f =1.4

Design bending moment

$$= 1.4 \times 16 \times \frac{2}{3}$$

=15 kN m/m run

Design shear force =1.4×16

=22 kN/m run

Effective depth =152.5 mm

Effective span =2.0 m+76.25 mm

=2076 mm

$$\frac{\text{span}}{\text{effective depth}} = 13.6$$

As this is less than 18, assuming for the moment that the steel percentage will be less than 0.5%, then there is no further need to check against deflection and cracking.

SOLUTION

Ultimate limit state

Resistance moment of section

$$= 0.4 f_k \frac{bd^2}{\gamma_{mm}}$$

Take γ_{mm} =2.0

Hence, if $0.4 f_v \frac{bd^2}{\gamma_{mm}} \geqslant 15$ kN m/m

$$0.4 f_k\ 1000 \frac{152.5^2}{2} \geqslant 15\times10^6$$

$f_k \geqslant 3.2$ N/mm^2

Assuming that for architectural reasons a brick of strength 35 N/mm^2 has been chosen and for reasons of durability a grade (i) mortar is to be used, then f_k=11.4 N/mm^2, which is clearly adequate.

Now, M_d

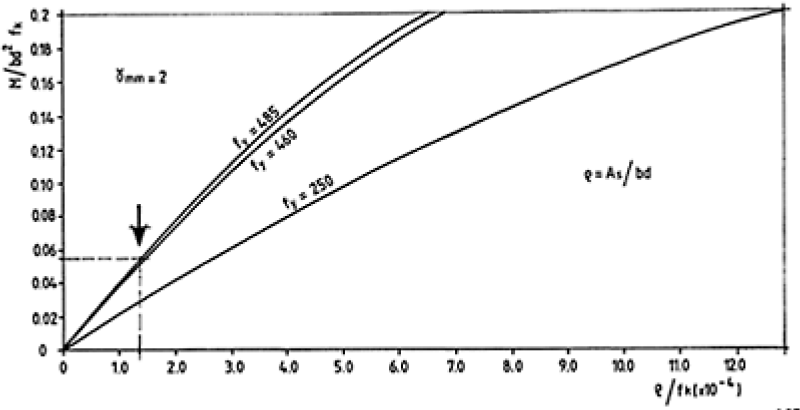
$$= \frac{A_s f_y z}{\gamma_{mm}}$$

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

where, γ_{ms} =1.15

Assume that fabric reinforcement is to be used, hence f_y =485 N/mm^2.

Now assume z =0.8d



hence, 15×10^6 $$= A_s\,\frac{485}{1.15} \times 0.8 \times 152.5$$

A_s =292 mm^2

and

z $$= d\left(1 - 0.5\,\frac{292 \times 485 \times 2}{10^3 \times 152.5 \times 11.4 \times 1.15}\right)$$

=0.93d

Using this as a second estimate

A_s $$= 292 \times \frac{0.8}{0.93}$$

=251 mm^2

and z =0.94d

Clearly, A_s=251 mm^2 is close to the exact solution. The reinforcement chosen can, therefore, be B283 structural mesh which provides 283 mm^2/m, and this satisfies the flexural requirement (a check shows that the section is under-reinforced).

As an alternative to the iterative method, the design could choose to obtain the required steel area by solving the quadratic equation:

$$\frac{A_s}{bd} = \frac{f_k \gamma_{ms}}{f_y \gamma_{mm}}\left(1 - \sqrt{1 - 2\frac{M}{bd^2}\frac{\gamma_{mm}}{f_k}}\right)$$

which gives:

$$\frac{A_s}{bd} = \frac{11.4}{485}\frac{1.15}{2}\left(1 - \sqrt{1 - 2\frac{15 \times 10^6 \times 2}{10^3 \times (152.5)^2 \times 11.4}}\right)$$

$=1.63\times10^{-3}$

$=248\ \text{mm}^2$

Alternatively, the design chart may be used:

Now, $\frac{M}{bd^2 f_k}$ $=0.0565$

hence, from the chart for f_y=485 and γ_{mm}=2, read (see Figure)

$\frac{Q}{f_k}$ $=1.40\times10^{-4}$

A_s $=1.40\times10^{-4}\times11.4\times10^3\times152.5$

$=243\ \text{mm}^2$

The remaining alternative is to use the moment coefficient *(Q)* method:

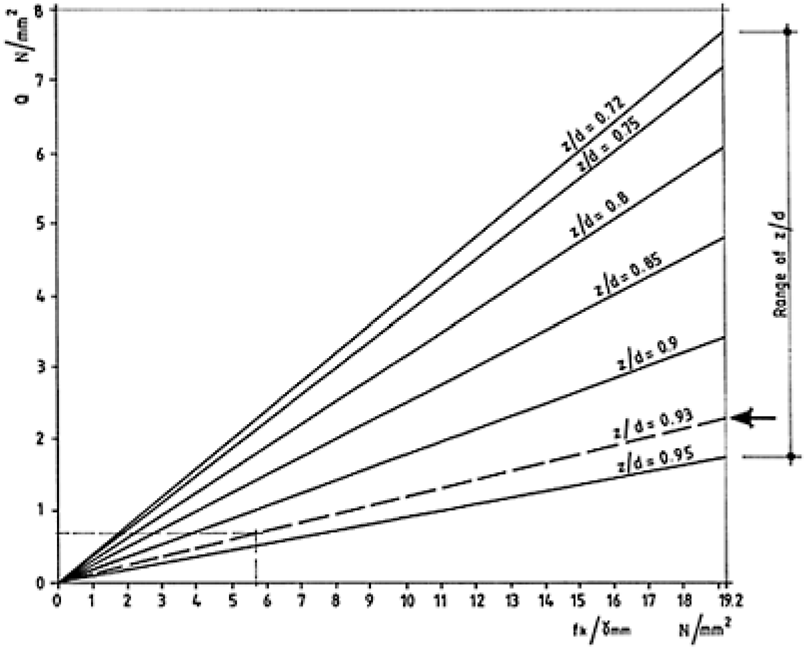
The required Q $= \frac{M}{bd^2}$

$$= \frac{15 \times 10^6}{10^3 \times (152.5)^2}$$

$=0.645$

Now, $\frac{f_k}{\gamma_{mm}}$ $=5.7$

So, interpolating from the graph fixes $\frac{z}{d}$ as 0.93.



$$A_s = M\frac{\gamma_{ms}}{f_y}\frac{1}{z}$$

$$= 15 \times 10^6 \times \frac{1.15}{485} \times \frac{1}{0.93 \times 152.5}$$

$= 251 \text{ mm}^2$

Clearly the results obtained by the various methods are similar, slight differences being due to interpolation from the graphs and rounding errors.

Shear

Consider now the shear resistance of the section. The basic shear resistance is given by:

$$f_v = 0.35 + 17.5\varrho \text{ N/mm}^2$$

where $\varrho = \dfrac{A_s}{bd}$

hence, in the case

$$f_v = 0.35 + 17.5\frac{283}{1000 \times 152.5}$$

$=0.38\ \text{N/mm}^2$

As this is a cantilever retaining wall, this value may be enhanced by the factor $2.5-0.25\frac{a}{d}$, where a is the shear span and *d* is the effective depth.

The shear span $= \dfrac{\text{maximum moment}}{\text{maximum shear}} = \dfrac{1}{3}\text{ height} = 666\ \text{mm}$

hence, $\frac{a}{d}$ $=4.4$

so, f_v $=0.38\times1.4$

$=0.53\ \text{N/mm}^2$

The shear stress due to design loads $= \dfrac{22\times10^3}{10^3\times152.5}$

$=0.14\ \text{N/mm}^2$

Now we require $\frac{f_v}{\gamma_{mm}}$ >shear stress due to design loads, i.e.,

$\dfrac{0.53}{2} > 0.14$

consequently, shear reinforcement is not required.

The use of mesh ensures that there is some secondary reinforcement and this exceeds the minimum 0.05% recommended. The main reinforcement represents 0.18% of breadth×effective depth and, as this is less than 0.5%, the serviceability requirements will be met.

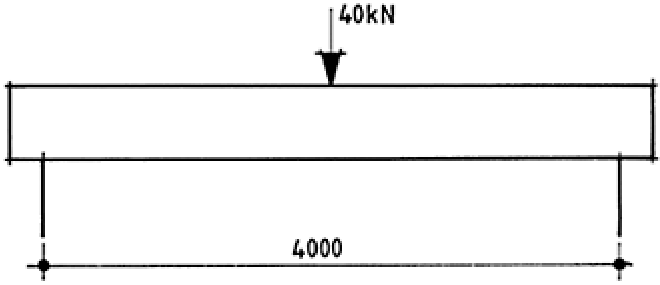
In the central 100 mm cavity, the cover will be 40 mm to the mesh and hence, assuming that the exposure condition may be considered as E2 and a grade 30 concrete is specified, there is no need for further protection to the steel.

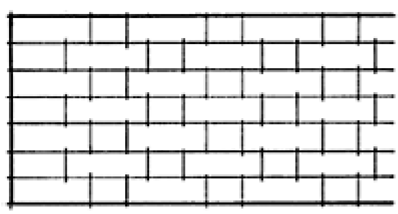
There are, of course, a number of other considerations such as lap of starter bars, design of base, movement joints, weatherproofing of the head of the wall, draining the soil adjacent to the rear face, detailing of wall ties and specification of grouting procedure, and although these are all important they will not be discussed here.

EXAMPLE 12

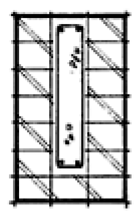
PROBLEM

Consider the design of a reinforced brickwork beam. The beam is simply supported and carries a point load of 40 kN, the clear span is 4 m. The external appearance of the beam is to be of Flemish bond throughout. It's depth, which is seven brick courses, is to be achieved using Quetta bond for the bottom course and the top five courses. The second course is to be of two outer wythes of alternate full and cut bricks to maintain the appearance and, at the same time, provide a central space for tensile reinforcement. The vertical cores enable stirrups to be used for shear reinforcement and the voids will be filled with plasticised mortar. Small wooden blocks are to be used in the voids in the bottom course so that there will be a small recess in the beam soffit at the position of the cores. The beam is shown in the following figure:





Bonding pattern



Section

Loads

Imposed load =40 kN

Self weight

$$= 0.515 \times 0.328 \times 2153 \times \frac{9.8}{10^3} \text{ kN/m run}$$

=3.6 kN/m run

Design load =1.6×40 kN=64 kN centrally

=1.4×3.6 kN=5.0 kN/m u.d.l.

Effective span =the lesser of 4+0.225=4.225 m

or 4+0.42=4.42 m

$\therefore$ =4.2 m

$$\frac{\text{span}}{\text{effective depth}} = \frac{4.2}{0.42}$$

=10

Consequently, the deflection should be acceptable and no detailed calculations are required. Lateral restraint to the beam is available at the supports and consequently the limits $60b_c$ (19.6 m) and $250\frac{b_c^2}{d}$ (63.8 m) are not exceeded.

Design bending moment, M_d

$$= \frac{64 \times 4.2}{4} + \frac{5.0 \times 4.2^2}{8}$$

=78.2 kN m

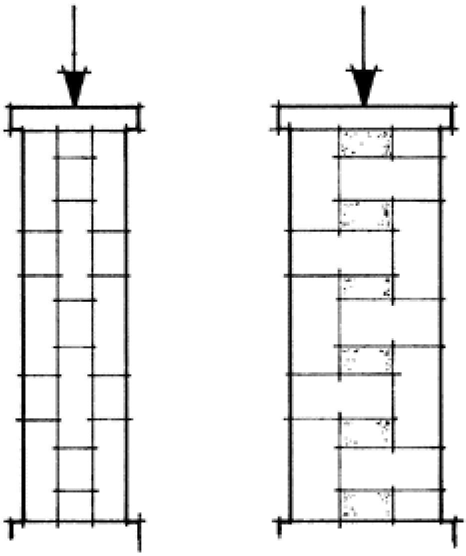
Brick strength =45 N/mm^2

Mortar designation (i)

SOLUTION

In this situation some of the bricks in the compression block are loaded through the stretcher face and some through the header face. Although the bricks are solid and, for want of better information, the value of f_k to be used would be taken from the Tables as 13.8 N/mm^2, as the bonding is unusual it would be advisable to do a direct determination of the characteristic strength. Consequently, the value of f_k will be taken from the results of tests on five specimens of the type shown in the following figure. The test results are as follows:

Specimen number	Test strength (N/mm^2)
1	16.2
2	14.0
3	13.1
4	15.9
5	16.3



Test specimen format

Hence, the average strength is 15.1 N/mm^2. The logarithm of each value is required (y_n) and the characteristic strength is given by:

antilog (y_c)

where y_c $=y_{mean}-k\cdot s$

where s

$$=\sqrt{\frac{(y_1^2+y_2^2\ldots\ldots y_5^2)-\left(\frac{y_1+y_2\ldots\ldots y_5)}{5}\right)^2}{5-1}}$$

In this case, for five specimens, k=2.335 and the characteristic strength calculated must be multiplied by 0.95 to allow for the fact that the height/thickness ratio of the specimens tested is less than 5. The calculation is tabulated as follows:

Computation of characteristic strength

Specimen number	Strength	log (strength) y_n	y_n^2
1	16.2	1.210	1.464
2	14.0	1.146	1.313
3	13.1	1.117	1.248
4	15.9	1.201	1.442

5	16.3	1.212	1.469
		$\Sigma y_n = 5.886$	$\Sigma y_n^2 = 6.938$

s	$= \sqrt{\dfrac{6.938 - \dfrac{5.886^2}{5}}{4}}$
	$=0.047$
y_c	$=1.177-0.047\times2.335$
	$=1.067$
antilog (y_c)	$=11.68$
Characteristic strength	$=0.95\times11.68$
	$=11.1$ N/mm^2
To check that M_d	$\not> 0.4 \dfrac{f_k}{\gamma_{mm}} bd^2$
$0.4 \dfrac{f_k}{\gamma_{mm}} bd^2$	$= 0.4 \times \dfrac{11.1}{2} \times \dfrac{328 \times 420^2}{10^6}$
	$=128.2$ kN m

Hence the upper limit on the moment of resistance is not exceeded.

Determination of steel area

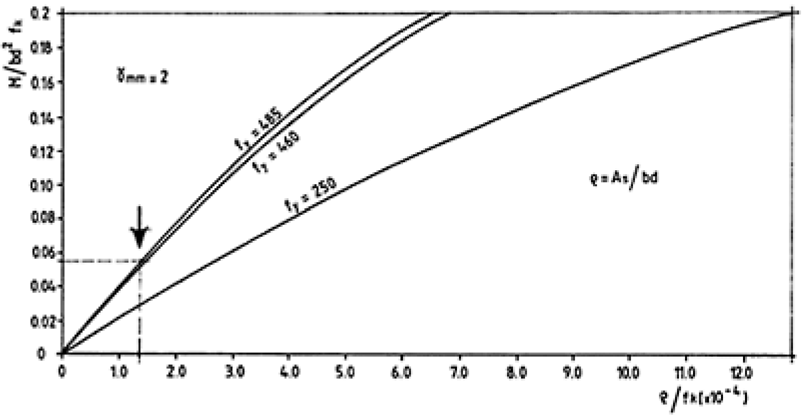
78.2×10^6	$= A_s \dfrac{f_y}{\gamma_{ms}} z$
Assume, z	$=0.9\,d$
f_y	$=460$
γ_{ms}	$=1.15$
Hence, A_s	$= \dfrac{78.2 \times 10^6 \times 1.15}{460 \times 0.9 \times 420}$
	$=517$ mm^2
substitute, z	$= d\left(1 - 0.5 \dfrac{A_s}{bd} \dfrac{f_y}{f_k} \dfrac{\gamma_{mm}}{\gamma_{ms}}\right)$
	$=0.86\,d$

Hence, the original estimate was quite close, a second iteration gives A_s=541 mm^2 and z=0.86d. The alternatives to the iterative method above will now be considered.

A_s may be found by solution of the quadratic equation:

$$\frac{A_s}{bd} = \frac{f_k}{f_y}\frac{\gamma_{ms}}{\gamma_{mm}}\left(1 - \sqrt{1 - \frac{2M}{bd^2}\frac{\gamma_{mm}}{f_k}}\right)$$

$$= \frac{11.1}{460}\frac{1.15}{2}\left(1 - \sqrt{1 - \frac{2 \times 78.2 \times 10^6 \times 2}{328 \times 420^2 \times 11.1}}\right)$$



	$=3.94\times10^{-3}$
A_s	$=3.94\times10^{-3}\times328\times420$
	$=543$ mm^2

From the design chart:

for f_y	=460
γ_{mm}	=2
$\dfrac{M}{bd^2 f_k}$	$= \dfrac{78.2 \times 10^6}{328 \times 420^2 \times 11.1}$
	=0.122
and so $\dfrac{\varrho}{f_k}$	$=3.5\times10^{-4}$
ϱ	$=11.1\times3.5\times10^{-4}$
	$=3.9\times10^{-3}$
A_s	$=537$ mm^2

The moment coefficient (Q) method is as follows:

$$Q = \frac{M}{bd^2}$$

$$= \frac{78.2 \times 10^6}{328 \times 420^2}$$

$$=1.35$$

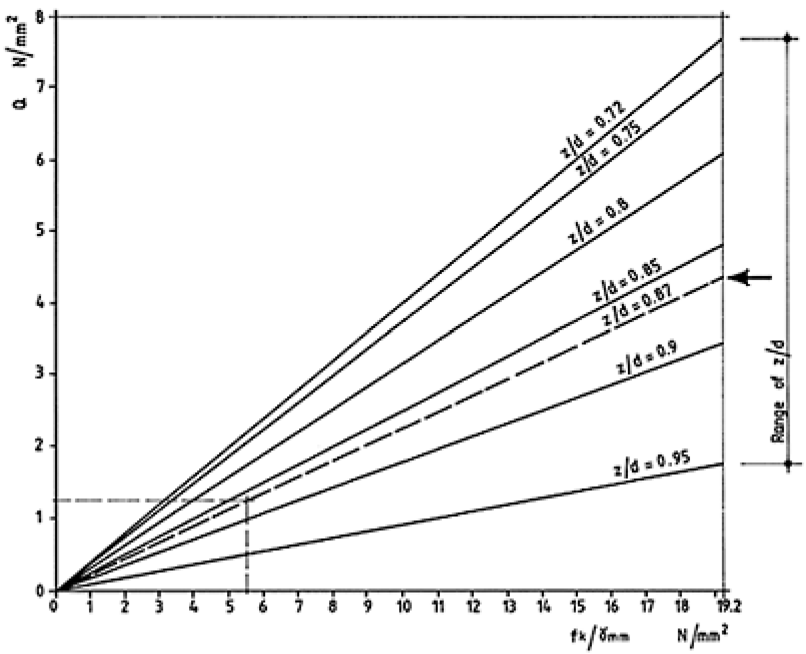
$$\frac{f_k}{\gamma_{mm}} =5.55$$

Thus, from the chart:

$$A_s = M \frac{\gamma_{ms}}{f_y} \frac{1}{z}$$

$$= 78.2 \times 10^6 \times \frac{1.15}{460} \left(\frac{1}{0.86 \times 420} \right)$$

$$=541 \text{ mm}^2$$



Clearly the results using the available methods are comparable. The area of steel supplied will be 2 No. 20 mm bars (628 mm^2)

Now, z $$= d\left(1 - 0.5\,\frac{628}{328 \times 420}\,\frac{460}{11.1} \times \frac{2}{1.15}\right)$$

$=0.92\,d$

Shear

Consider the design for shear:

Maximum shear force $=32+5\times 2.1$

$=42.5$ kN

Design shear stress $$= \frac{42.5 \times 10^3}{328 \times 420}$$

$=0.31$ N/mm^2

Now, f_v $= 0.35 + 17.5\varrho$

where ϱ $$= \frac{A_s}{bd}$$

Hence, f_v $$= 0.35 + 17.5 \times \frac{628}{420 \times 328}$$

$=0.43$ N/mm^2

The shear span $$= \frac{78.2 \times 10^6}{42.5 \times 10^3}$$

$=1840$ mm

and the shear span/effective depth ratio is 4.4

The enhancement factor

$2.5 - 0.25\,\dfrac{a}{d}$ $=1.41$

f_v $=0.60$ N/mm^2

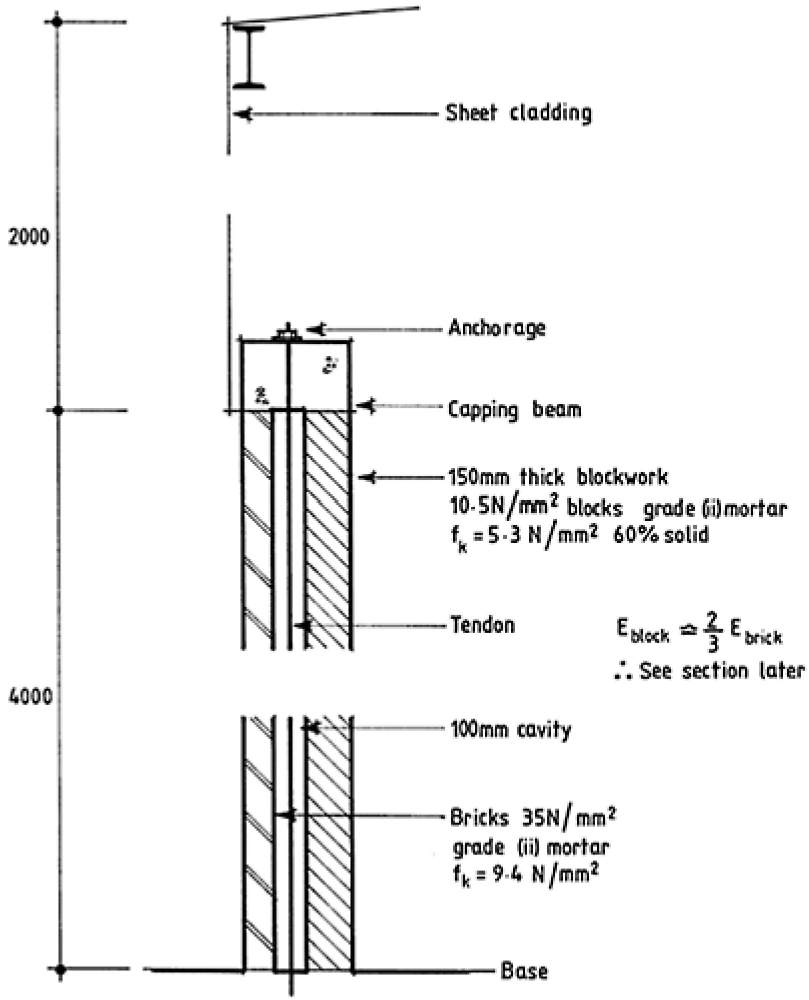
and $\dfrac{f_v}{\gamma_{mv}}$ $=0.30$ N/mm^2

This is nominally the same as the design shear stress. Nominal stirrups will be adequate.

EXAMPLE 13

PROBLEM

Consider a cavity wall subjected to wind load only. The wall is required to cantilever from the base and to carry the wind load from a profiled sheet cladding above. It is considered that there should not no tensile stress in the outer leaf when the wall is subjected to service loads. The wall is illustrated below:



SOLUTION

Serviceability limit state

Loads:

Characteristic wind load, W_k $=1.0\ \text{kN/m}^2$

Design wind load $=1.0\ \text{kN/m}^2$ (serviceability limit state with $\gamma_f=1.0$)

Maximum bending moment at base due to uniform wind load on wall and point load from $\frac{1}{2}$ uniform wind load on sheeting acting at the top of the wall

$$M = \frac{1.0 \times 4^2}{2} + 1.0 \times 1.0 \times 4$$

$=8.0+4.0$

$=12$ kN m/m run

Section properties

The leaves are of dissimilar materials and, therefore, in this case their values of elastic modulus differ significantly (by the ratio of their respective f_k values [5.3 and 9.4] since these are multiplied by a constant of 0.9 to give moduli in kN/mm^2). It is necessary, therefore, to consider a transformed section. The equivalent brickwork area will be used in each case although the same resultant stresses would be calculated if the equivalent blockwork area was considered. The short term elastic moduli have been used in determining the transformed section; this is reasonable for both transfer and service conditions. Hence, taking moments about the inside face of the inner leaf:

$$\Sigma Ay = \bar{y}\sum A$$

$$150 \times 10^3 \times 75 \times \frac{5.3}{9.4} + 102.5 \times 10^3 \times 301.25 = [150 \times 10^3 \times \frac{5.3}{9.4} + 102.5 \times 10^3]\bar{y}$$

$$37.22\times10^6 = 187.1 \times 10^3\bar{y}$$

$\therefore\ \bar{y}$ $=199$ mm

and the transformed area $=187.1\times10^3\ \text{mm}^2$

The second moment of area of the transformed section is given by:

$$I = \frac{150^3}{12} \times 10^3 \times \frac{5.3}{9.4} + 150\,(199 - 75)^2 \times 10^3 \times \frac{5.3}{9.4}$$

$$+ \frac{102.5^3}{12} \times 10^3 + 102.5\,(352.5 - 199 - 51.25)^2 \times 10^3$$

$=[158.6+1300.4+89.7+1071.7]\times 10^6$

$=2620\times 10^6$ mm^4

The stress at any point within a material, x

$$= \frac{M_y E_x}{EI}$$

where, for this example, E in the denominator has previously been chosen as that for brickwork.

Calculation of the stresses in this way implies that the two leaves can behave as a composite section and provision must be made to transfer shear between the two leaves, in the case of the cavity wall referred to earlier and shown in the figure, this was achieved by the use of special steel spacer plates.

The maximum stresses induced by the wind load are given by:

at the outer face:

stress

$$= \frac{12.0 \times 10^6 \times 153.5}{2620 \times 10^6}$$

$=0.70$ N/mm^2 (tension)

at the inner face:

stress

$$= \frac{12.0 \times 10^6 \times 199 \times 5.3}{2620 \times 10^6 \times 9.4}$$

$= 0.51$ N/mm^2 (compression)

The stresses due to the weight of the masonry are given by:

outer leaf:	0.05 N/mm^2
inner leaf:	0.03 N/mm^2

Assuming that no vertical load from the cladding above the masonry need be considered and similarly that there is no wind uplift transferred to the wall, the maximum stresses are given by:

outer leaf:	0.65 N/mm^2 (tension)
inner leaf:	0.54 N/mm^2 (compression)

It is necessary to apply sufficient prestressing force to ensure that after all losses have occurred the tensile stress at the outer face is zero. Assume that in the first instance the loss in prestress amounts to 30% of the initial prestress. The eccentricity of the prestressing tendons, which is at the centre of the cavity, is 1 mm. Hence, if required prestressing force is *P:*

$$\frac{P}{187.1\times10^3}+\frac{P\times1.0\times153.5}{2620\times10^6}=0.65$$

$$P(5.34+0.06)\times10^{-6}=0.65$$

Therefore, P=120 kN.

Hence, initial prestressing force, $=\frac{120}{0.7}=171\text{ kN}$ $\therefore$ at transfer:

stress at outer leaf

$$=\frac{171\times10^3}{187.1\times10^3}+\frac{171\times10^3\times1.0\times153.5}{2620\times10^6}+0.05$$

$=0.91+0.01+0.05$

$=0.97$ N/mm^2

$<0.4\,f_k$ for rectangular dist. (=3.76 N/mm^2)

stress at inner leaf

$$=\frac{171\times10^3\times5.3}{187.1\times10^3\times9.4}-\frac{171\times10^3\times1.0}{2620\times10^6}\times\frac{5.3}{9.4}\times199+0.03$$

$=0.52-0.01+0.03$

$=0.54$ N/mm^2

$<0.4\,f_k$ (=2.12 N/mm^2)

After losses have occurred, these values will be:

at outer leaf $=0.7\times0.92+0.05$

$=0.69$ N/mm^2

$<0.3\,f_k$ for rectangular distribution (=2.82)

at inner leaf $=0.7\times0.51+0.03$

$=0.39$ N/mm^2

$<0.3\,f_k$ for rectangular distribution (=1.59)

With service loads applied, check the maximum compressive stress at the inner leaf:

$0.51+0.39=0.9$ N/mm^2

$<0.4\,f_k$ for rectangular distribution (=2.12)

Consider now the losses:

(i) Relaxation

If the bar is initially stressed to 70% of breaking load, the loss due to relaxation is 3.5%.

(ii) Elastic strain

It may be assumed that the bars are stressed progressively such that the loss due to elastic strain is half the product of the modular ratio and the stress in the masonry section adjacent to the centroid of the tendons (If the tendons were restressed or were stressed simultaneously no losses due to elastic deformation would occur).

Normally a linear strain distribution would be assumed across the section and the strain at the tendon position calculated. In this case, however, the eccentricity of the tendon is very small and, therefore, constant strain across each material may be assumed and the strain on the tendon is simply the meanof these two values since it is placed centrally in the cavity.

strain in outer leaf $= \dfrac{0.92}{9.4 \times 900} = 109 \times 10^{-6}$

Strain in inner leaf $= \dfrac{0.51}{5.3 \times 900} = 107 \times 10^{-6}$

$\therefore$ strain in bar $= 0.5\left(\dfrac{109 + 107}{2}\right) \times 10^{-6}$

$=54\times10^{-6}$

Loss of bar stress $=206\times54\times10^{-6}$

$=11.1$ N/mm^2

(iii) Moisture movement

Assuming the shrinkage of the concrete masonry is 500×10^{-6} and ignoring the moisture expansion of the clay brickwork, the effect at the tendon position is a loss of stress equivalent to a strain of 250×10^{-6}.

Hence, loss of prestress due to shrinkage $=206\times250\times10^{-6}$

$=51.5$ N/mm^2

(iv) Creep

For the brickwork leaf, creep strain=1.5×elastic strain. The elastic strain in the brickwork at transfer may be taken as 109×10^{-6}, usually a linear strain distribution is assumed but in this case the elastic strain does not vary greatly across the section. Hence, for the brickwork leaf, creep strain

$=164\times10^{-6}$

For the blockwork leaf, creep strain=3×elastic strain, Hence

creep strain $=321\times10^{-6}$

Mean creep strain $=242\times10^{-6}$

Loss of bar stress $=50\ \text{N/mm}^2$

Clearly the creep strain could have been estimated in other ways, for example by considering the elastic strain due to a reduced level of prestress, as occurs in practice due to losses. However, the values used as creep coefficients are estimates only and are applied without discrimination between types of brickwork and blockwork and as a result the calculated loss of stress is approximate, consequently the finer points which might be considered are not really relevant.

(v) Anchorage draw-in

With a positive connection such as a threaded nut there is no anchorage draw-in.

(vi) Friction

In this design there is no loss due to friction.

(vii) Thermal effects

Assuming the change in temperature to be considered is 30°C for the outer leaf and 20°C for the inner leaf, taking the coefficient of linear thermal expansion as 10×10^{-6} for both leaves, the thermal strain in the bar is given by:

strain $=25\times10\times10^{-6}$

and the corresponding bar stress is 51.5 N/mm^2

Therefore, losses due to elastic strain, moisture movement, creep and thermal effects are:

$(11.1+51.5+50+51.5)=164.1\ \text{N/mm}^2$

Now, if the tensile strength of the tendon is 1030 N/mm^2 and it is tensioned to 70% of its ultimate value, this loss of stress is equivalent to:

$$\frac{164.1\times100}{1030\times0.7}=22.8\%$$

Consequently, adding the relaxation loss (3.5%), the total is 26.3%. This is close to the assumed value of 30% and is, therefore, acceptable. Some tension may occur under service loads but due to the approximate nature of the calculation of losses and the very short duration of wind loads, this can be disregarded.

The required initial prestressing force is 171 kN. Hence, the required tendon area

$$=\frac{171\times10^3}{0.7\times1030}$$

$$=237\ \text{mm}^2$$

20 mm ϕ bars at 1.2 m centres provide 262 mm^2 of tendon. These points are worthy of note:

1. the area provided is greater than the area required and thus if stressed fully to 70% of its breaking load, higher stresses in the masonry will result with less chance of tension occurring
2. by providing an excess area of tendons the force in each bar could be reduced from 70% of breaking load but this would effectively increase losses and a balance would need to be achieved
3. the spacing of tendons could be changed (greater spacing) to suit a construction module

None of the above would, in general, require recalculation of stresses and losses since, in this case "inspection" would be sufficient.

Ultimate limit state

With losses at 26.3%, the strain in the tendon due to prestress:

$$= \frac{0.737 \times 1030 \times 0.7}{206 \times 10^3}$$

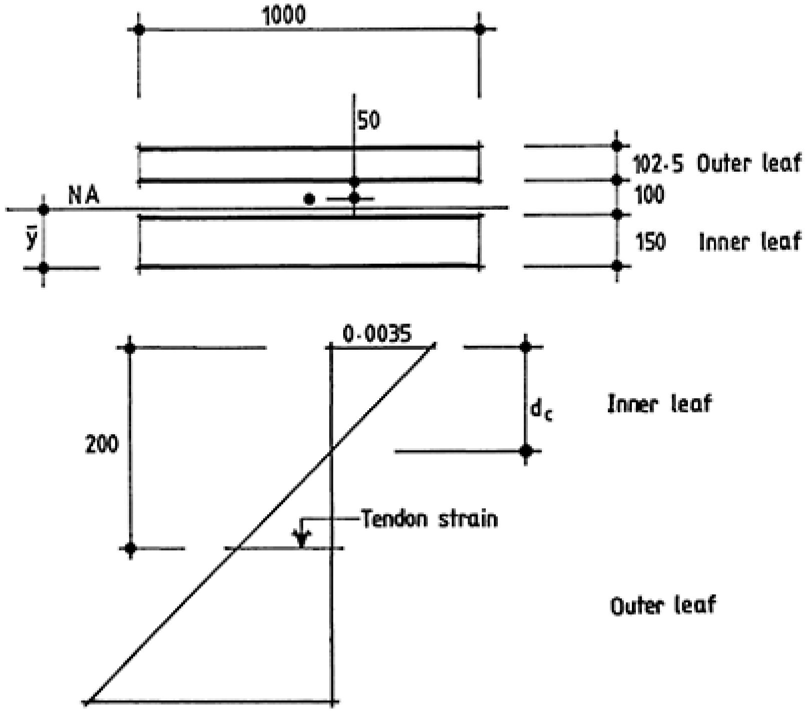
$$= 0.0026$$

Assume the depth of compression block (d_c)=60 mm. The strain distribution based on a maximum masonry strain of 0.0035 is as shown:

Maximum strain in tendon due to bending, ε_s $= \frac{(200 - 60)}{60} \times 0.0035$

$=0.0082$

As the tendon is unbonded the strain in the bar will be the average strain in the masonry over its height. Assuming that the strain takes the same distribution as the bending moment diagram, the strain in the tendon due to flexure will be between $\frac{1}{2}$ and $\frac{2}{3}$ of that calculated above ($\frac{1}{2}$ for the moment due to the point load and $\frac{2}{3}$ for the moment due to the u.d.l.). Take $\frac{1}{2}$ as conservative.



Total tendon strain $= \dfrac{0.0082}{2} + 0.0026$

$=0.0067$

From the idealised stress/strain curve for tendons this is equivalent to a stress

f_{pb} $=815$ N/mm^2

Steel force, $A_s f_{pb}$ $=815\times314$

$=256$ kN

Masonry force,

$$\frac{f_k}{\gamma_{mm}} b d_c = \frac{5.3}{2} \times 1200 \times 60 \times 10^{-3}$$

$=190.8$ kN

Clearly 60 mm is too small. Try 80 mm:

$$\varepsilon_s = \frac{(200-80)}{80} \times 0.0035$$

=0.0053

Total tendon strain $= \dfrac{0.0053}{2} + 0.0026$

=0.0052

from the stress/strain curve, f_{pb} =769 N/mm^2

$\therefore$ steel force, $A_s f_{pb}$ =769×314

=242 kN

and masonry force,

$$\frac{f_k}{\gamma_{mm}} bd_c = \frac{5.3}{2} \times 1200 \times 80 \times 10^{-3}$$

=254 kN

This is obviously a close estimate and the resistance moment will be:

$A_s f_{pb} \times 1_a$ $= 242 \times \dfrac{160}{10^3}$ kN per 1.2 m length

$$= 242 \times \frac{160}{10^3} \frac{1}{1.2} \text{ kN m/m}$$

=32.3 kN m/m run

The design ultimate moment is γ_f times the serviceability moment. Taking γ_f as 1.4:

design moment =16.8 kN m/m run

<32.3 $\therefore$ OK

Check maximum stress in tendon is not exceeded (Figure 6):

$$\frac{d_c}{d} = \frac{80}{200}$$

=0.4

$$\frac{f_{pb}}{f_{pc}} = \frac{769}{0.737 \times 1030 \times 0.7}$$

=1.45<1.5 $\therefore$ OK

Tendon will not be overstressed.

Shear

Design shear force, V =1.4 [1×4+1×1] kN

=7 kN

$\therefore$ design shear stress, v

$$= \frac{7 \times 10^3}{10^3 \times 80}$$

=0.09 N/mm^2

For prestressed sections

f_v $=0.35+0.6\, g_B$

where g_B is the design load due to loads acting normal to bed joints. Clearly with $\gamma_{mv} = 2, \frac{f_v}{\gamma_{mm}} > v$ and the wall is safe.

Obviously there are a number of other factors to be included in the complete design, such as movement joints, capping beam, spreader plates, and so on. In particular, as mentioned earlier, there is the problem of tieing the two leaves together. The ties must be capable of transmitting vertical shear from one leaf to another. As also mentioned previously, in the factory at Darlington specially designed connectors were used for this purpose.

EXAMPLE 14

This example demonstrates a deflection calculation for a 4.5 m cantilever wall inside a factory unit being used as a fire separation partition. The wall may be subject to a wind load inside the building of 0.5 kN/m^2.

Design parameters:

γ_f=1.0

Blocks 440×215×215 hollow, 55% solid, of unit strength 7 N/mm^2

Mortar Designation (ii)

Table 3(b), compressive strength of unit

$$= \frac{7.0}{0.55}$$

=12.7 N/mm^2

f_k =6.2 N/mm^2

Wind load is intermittent, therefore, check for short term deflection only, i.e.,

E_m $=900\ f_k$

a $$= kl^2 \frac{1}{r_{it}}$$

$$\frac{1}{r_{it}} = \frac{M_t}{E_{mi} I}$$

M_t $$= \frac{wL^2}{2}$$

$$= \frac{0.5 \times 4.5^2}{2}$$

50.6 kN m

E_{mi} $=900\times6.2$

$=5580\ \text{N/mm}^2$

I $$= \frac{bt^3}{12}$$

$$= \frac{1000 \times 215^3}{12}$$

$=828\times10^6$

Figure C1, k $=0.25$

$\therefore a$ $$= \frac{0.25 \times 4.5^2 \times 10^6 \times 5.06 \times 10^6}{5580 \times 828 \times 10^6}$$

$=154$ mm

Allowable deflection $$= \frac{\text{length}}{125}$$

$$= 36\,\text{mm} > 5.58 \therefore \text{OK}$$

Note: span/effective depth ratio for steel at centre

$$= \frac{4500}{107}$$

$=42>18$, Table 8.

EXAMPLE 15

This example demonstrates a deflection calculated for a single course beam spanning an internal opening (span=3.355 m) supporting hollow blockwork only.

Blocks 440×215×215 hollow, 55% solid, of unit strength 7 N/mm^2

Mortar Designation (ii)

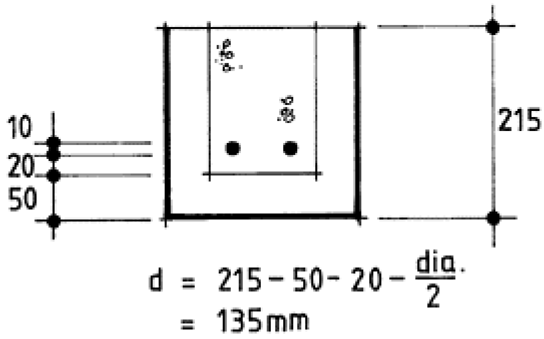


Table 3(b), as in the previous example, compressive strength of unit,

f_k =6.2 N/mm^2

All load is permanent.

$$a = kl^2 \frac{1}{r_{lt}}$$

$$\frac{1}{r_{lt}} = \frac{M_t}{E_{ml} I}$$

For 45° triangle above opening, the total load on the beams, *W*, is given by:

$$W = \frac{3.355}{2} \times \frac{3.355}{2} \times \frac{0.215 \times 2000 \times 9.81 \times 0.55^*}{1000}$$

(*only 55% solid)

=6.53 kN

Figure C2, *M*

$$= \frac{WL}{6}$$

$$= \frac{6.53 \times 3.355}{6}$$

=3.65 kN m

E_{ml} $=300\, f_k$ for concrete masonry

$=300\times6.2$

$=1860$ N/mm^2

I $$= \frac{bt^3}{12}$$

$$= \frac{215 \times 215^3}{12}$$

$=178\times10^6$

Figure C2, k $=0.10$

$\therefore a$ $$= \frac{0.1 \times 3.355^2 \times 10^6 \times 3.65 \times 10^6}{1860 \times 178 \times 10^6}$$

$=12.4$ mm

Allowable deflection $$= \frac{\text{span}}{250}$$

$=13.42$ $\therefore$ OK

Note: span/effective depth ratio

$$= \frac{3355}{135}$$

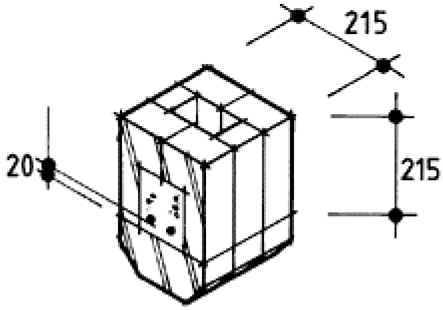
$=24.9>20$, Table 9.

EXAMPLE 16

This example illustrates a deflection calculation for a single course "pistol" brick beam spanning an internal opening (span=2.775 m) supporting a floor slab over. Assume brick strength is 50 N/mm^2 (/J.=15.0), moment=16.6 kN m due to design service loads, ratio of dead to live load=2:1.

Overall long term curvature $$= \frac{1}{(r_0/A)}$$

$$= \frac{1}{r_{lp}} + \frac{1}{r_{it}} - \frac{1}{r_{ip}}$$



for clay masonry.

$$\frac{1}{(r_0/A)} = \frac{M_t}{900 f_k I}(1 + \alpha)$$

where α is a proportion of *total* load which is permanent. Assume all dead+25% of live load is permanent (dead:live=2:1), therefore, 75% of *total* load is permanent.

$$I = \frac{215 \times 215^3}{12}$$

$=178\times10^6$ mm^4

$$\frac{1}{(r_0/A)} = \frac{16.6 \times 10^6}{900 \times 15 \times 178 \times 10^6}(1 + 0.75)$$

$=12.09\times10^{-6}$

for u.d.l. over simple suupports, k =0.104 (Figure C2)

$\therefore a$

$$= kl^2 \frac{1}{(r_0/A)}$$

$=0.104\times2775^2\times12.09\times10^{-6}$

=9.68 mm

Allowable deflection

$$= \frac{\text{span}}{250}$$

$$= \frac{2775}{250}$$

$= 11.1>9.68 \therefore$ OK

Section Ten: Model specification for reinforced and prestressed masonry

1. MATERIALS AND PROPERTIES

1.1 MASONRY UNITS

1.1.1 Clay bricks

Clay bricks shall comply with BS 3921.

1.1.1.1 Facing bricks

Facing bricks shall comply with the sample panels located at…. The (common, facing, loadbearing, engineering) bricks for…(location) shall be…(description or manufacturer's designation) obtained from (manufacturer) or other equal approved, and be…(perforated, frogged, and so on) having a minimum strength of…N/mm^2.

It is advisable that where appearance is paramount, sample panels of not less than 1 m^2 should be constructed by the contractor. The requirements for such panels should be included in the Specification.

1.1.1.2 Loadbearing bricks

Loadbearing bricks shall be classified in accordance with Clause 10: Table 6 of BS 3921:1974. The…(common, facing, loadbearing, engineering) bricks for …(location) shall be…(description or manufacturer's designation) obtained from (manufacturer) or other equal approved, and be… (perforated, frogged, and so on) having a minimum strength of…N/mm^2.

1.1.1.3 Engineering bricks

Engineering bricks shall be of Class A or B in accordance with Clause 10 of BS 3921:1974. The…(common, facing, loadbearing, engineering) bricks for …(location) shall be…(description or manufacturer's designation) obtained from…(manufacturer) or other equal approved, and be… (perforated, frogged, and so on) having a minimum strength of…N/mm^2.

The purpose of including reference to location is simply to indicate that it is of benefit to make it quite clear where each type of unit is to be used within the structure. It would not strictly form part of the Specification clause. Some of the information requested may be superflous, for example, the manufacturer's designation may automatically cover whether or not the brick is perforated. Although strength may be associated with the type of brick, it has to be specified separately for the following reasons:

1. the structural design will be based on a particular strength
2. certain minima are recommended for reinforced and prestressed masonry (see *Section 2.6*) except where used with bed joint reinforcement to enhance lateral load resistance
3. certain minima are required for durability reasons (see *Section 6.32.1*)

1.1.2 Special requirements for clay bricks

1.1.2.1 Initial rate of suction

★Before orders for clay bricks are placed, the contractor shall satisfy the engineer either that the initial rate of suction of the brick, when determined according to the method set out in *BCRA Special Publication 56*, does not exceed 1.5 kg/m^2/min, or that he is able to adjust it so as not to exceed this value.

★No special requirements.

For any given clay brick and mortar there will be a specific initial rate of suction at the time of laying which leads to the development of optimum bond. However, the bond and therefore flexural resistance is not likely to differ greatly from the optimum if at the time of laying it is not greater than 1.5 kg/m^2/min. The designer should consider invoking this clause therefore, where reliance is being placed on the flexural tensile strength of the masonry to resist primary stresses, for example, in a post-tensioned member where some tension may be designed to occur at the extreme fibres of the section for long periods.

1.1.2.2 Frost resistance

★The frost resistance of clay bricks shall be in accordance with the requirements for bricks of special quality in Clause 12.5 of BS 3921:1974.

★No special requirements

If clay brickwork in use is likely to become saturated and then be subjected to freezing conditions, the bricks will need to satisfy one or other of the requirements for frost resistance for bricks of special quality as set out in Clause 12.5 of BS 3921:1974. The final choice of brick and mortar will depend also on whether the construction is likely to be frozen during or soon after construction. Full guidance on the choice of brick and mortar is given in BS 5628: Part 3 and these requirements are summarised in *Section 6* herein.

1.1.2.3 Soluble salt content

The soluble salt content of clay bricks shall be in accordance with that for bricks of special quality as defined in Clause 12.3 of BS 3921:1974.

Where clay brickwork is being used for internal work, or for external work not liable to become and remain saturated, bricks of ordinary quality may be used, since the conditions for sulphate attack of the mortar are not present, i.e., there is no medium (water) into which to take the salts contained in the bricks into solution. Where the above conditions do not prevail then either (a) bricks of special quality shall be specified, or (b) bricks of ordinary quality may be used with mortars containing sulphate-resisting cement (providing the mortar is not weaker than designation (ii), $1:\frac{1}{2}:4\frac{1}{2}$ or equivalent).

1.1.3 *Concrete blocks*

All blocks shall comply with BS 6073: Part 1 and any additional requirements. The …(concrete blocks, facing blocks) for (location) shall be…(description or manufacturer's designation) obtained from…(manufacturer) or other equal approved, and be…(solid, hollow, cellular) having a minimum compressive strength of…N/mm^2 in the following sizes….

The same general notes apply here as in *1.1.1* where appropriate, but in addition several other points are worthy of note. A standardised method of specifying precast concrete masonry units (including concrete bricks, as discussed later) is given in BS 6073: Part 2, which indicates certain specific or additional requirements to the basic requirements in BS 6073: Part 1. This is particularly the case for reinforced and prestressed masonry regarding strength—the minimum requirement in BS 6073: Part 1 is 2.8 N/mm^2, which will, in most cases, need to be exceeded.

1.1.4 Concrete bricks

All concrete bricks shall comply with BS 6073: Part 1 and any additional requirements as specified.

The general clauses and commentary for concrete bricks are liable to be similar to those for concrete blocks, although they may not contain as much data.

1.1.5 Calcium silicate bricks

Calcium silicate bricks shall comply with BS 187.

Similar general notes apply here except where calcium silicate brickwork is likely to be saturated and then subjected to freezing conditions, in which case they should be Class 3 or stronger as defined in Clause 6: Table 2 of BS 187: Part 2.

1.1.6 "Special" masonry units

Special…(clay bricks, concrete blocks and so on) shall be in accordance with …(type, description contained in…, or to the drawings) and be within the following tolerances…

The inclusion of tolerances in this clause is to bring the readers attention to the fact that for concrete blocks, BS 6073: Part 1 basically applies to normal rectangular units. It will not often be necessary to refer to tolerances since the manufacturer will adhere to the Standard as far as is practicable.

1.1.7 Stone masonry

Stone masonry shall be in accordance with BS 5390.

1.2 MORTARS

1.2.1 Materials for mortars

1.2.1.1 Cement

The cement for mortar shall be to BS…

The following cements are permitted:

Ordinary Portland cement	BS 12
Portland blast-furnace cement	BS 146
Sulphate-resisting cement	BS 4027

High alumina cement and masonry cement are not permitted for use in reinforced and prestressed masonry.

1.2.1.2 Lime

The lime for mortar shall be non-hydraulic (calcium) limes or semi-hydraulic (calcium) limes or magnesium limes conforming to the requirements of BS 890.

1.2.1.3 Sand

The sand for mortar shall be to BS 1200. The chloride ion content by mass of dry sand shall not exceed 0.15%.

Other sands, apart from sands to BS 1200, can produce acceptable mortars but this needs to be checked with local experience. With certain sands, whether to BS 1200 or not, it will not always be possible to achieve the compressive strengths given in Table 1 of BS 5628: Part 2. Where any doubt exists trial mixes should be requested and, unless testing is required for all mortar in accordance with Appendix A1 of BS 5628: Part 1, a separate clause should be included setting out precisely the trials and testing required.

1.2.1.4 Water

The water shall be from normal mains supply or approval should be obtained before its use.

Where the quality of the supply is doubtful the water should be tested in accordance with BS 3148.

1.2.1.5 Admixtures

(a) calcium chloride shall not be permitted
(b) colouring agents shall comply with the requirements of BS 1014 and shall not exceed 10% by mass of cement in the mortar. Carbon black shall be limited to 3% by mass of the cement.

The tigher limit on carbon black is included because at higher proportions the strength of the mortar can be markedly reduced.

(c) other admixtures including mortar plasticisers may be used subject to approval in writing.

Before approving the use of an admixture the designer should check the appropriate manufacturer's recommendations and then ask the contractor to submit evidence of satisfactory performance of the admixture when used correctly in the appropriate situation, and details of the arrangements for its use. The designer should also confirm in the written approval the agreed procedure for use. Mortar plasticisers should comply with BS 4887.

1.2.1.6 Total chloride content

The total chloride content of mortar expressed as % of chloride ion by mass of cement shall not exceed 0.2% for mortar made with cement complying with BS 4027, or 0.4% for mortar containing embedded metal and made with cement complying with BS 12 or BS 146.

1.2.2 Preparation of mortars

1.2.2.1 Recommended mortar

Mortar for…(location) shall be…(mix proportions) by…(mass/ volume).

★Marine aggregates shall not be used in concrete infill for prestressed masonry and shall have a mean compressive strength of…N/mm^2.

The specified mix should take into account the requirements of the structural design and durability *(Section 6).* Where testing is to be specified the expected mean compressive strength of the mortar at 28 days is given in Table 6 of the Code. Volume proportions will be the norm, but increased accuracy will result when batching is by mass. Sand proportions should be specified as a single value depending upon the grading of the sand; uniformly coarse or uniformly fine sands should, in general, be gauged at the lower proportion, for example, at 1 Where a range of sand proportions is given, then the grading conditions should also be specified. If the source of sand is not known the lower proportion will err on the side side.

1.2.2.2 Equivalent mortar mixes

Alternative mortar mixes may be used subject to written approval.

1.2.2.3 Ready-mixed mortars

Ready-mixed lime: sand for mortar shall comply with the requirements of BS 4721. The appropriate addition of cement shall be gauged on site. Wet ready-mixed retarded cement: lime: sand mortar may be used subject to written approval.

1.2.2.4 Batching of mortars

Materials should be accurately measured by…(weigh batching, gauge boxes).

1.2.2.5 Mixing of mortars

The mortar shall be mixed by machine and be used before initial set takes place (usually within two hours). Mortar, except coloured mortars, may be retempered within this time. When using coloured mortars, mixing should be such that the colouring agent is evenly distributed throughout the mix.

1.3 CONCRETE INFILL

1.3.1 Materials for concrete infill

1.3.1.1 Cement

The cement for concrete infill shall be to BS….

The same cements are permitted and prohibited as for mortars

1.3.1.2 Lime

The lime used for concrete infill shall be non-hydraulic (calcium) or semi-hydraulic (calcium) or magnesium limes conforming to the requirements of BS 890.

1.3.1.3 Fine aggregate

Fine aggregate shall comply with BS 882 and shall have a chloride content by mass of dry fine aggregate not exceeding 0.15%.

1.3.1.4 Coarse aggregate

★Coarse aggregate shall comply with BS 882 and be of nominal size…mm. Aggregates shall be of the non-shrinkable type and have a chloride ion content by mass of dry coarse aggregate not exceeding 0.2%. The maximum shell content shall not exceed 15% by mass and aggregates containing hollow shells or shells of unsuitable shape in quantities which may adversely affect the quality or durability of the concrete shall not be used.

★Marine aggregates shall not be used in concrete infill for prestressed masonry or in concrete made with cement complying to BS 4027.

The nominal size of the coarse aggregate should be chosen in relation to the size of void to be filled. Guidance is given in *Section 2*.

1.3.1.5 Water

The water shall be from normal mains supplies or approval should be obtained before its use.

Where the quality of water supply is doubtful, the water should be tested in accordance with BS 3148.

1.3.1.6 Admixtures

Admixtures may be used subject to written approval. Calcium chloride shall not be permitted.

The same notes apply here as in *Section 1.2.1.5* and furthermore combinations of admixtures should be compatible. The written permission should state the relevant British Standard where appropriate with a rider that no admixture shall have a chloride ion content exceeding 2% by mass of the admixture or 0.03% by mass of the cement.

1.3.1.7 Total chloride content

The total chloride content of concrete, expressed as % of chloride ion by weight of cement, shall not exceed:

(a) 0.1% for prestressed, heat cured concrete containing embedded metal

(b) 0.2% for concrete made with cement complying with BS 4027

(c) 0.4% for concrete containing embedded metal and made with cement complying with BS 12 or BS 146.

1.3.2 Mixes

1.3.2.1 Recommended mix

★The concrete infill shall be…(prescribed, designed) mix to BS 5328 of strength grade…(25, 30, etc.) with a slump of…mm and with a minimum cement content of…kg/m^3.

★The concrete infill shall be 1:0—$\frac{1}{4}$:3:2 of cement: lime: sand: 10 mm maximum size aggregate (proportioned by volume of materials) and shall have a slump of…mm.

The minimum grade of concrete is 25 for reinforced masonry and 40 for prestressed masonry in which the infill acts structurally. The alternative specified mix of 1:0—$\frac{1}{4}$ 3:2 is approximately equivalent to a grade 25 prescribed mix. Slump should be specified between 75–175 mm for unplasticised concrete as appropriate to the size and configuration of the space to be filled. Where the designer intends plasticisers or superplasticisers to be used, the target slump before the addition of admixture should be indicated.

1.3.2.2 Alternative mix

Alternative concrete mixes may be used subject to written approval.

1.4 STEEL

1.4.1 Reinforcing steel

(a) Reinforcement for…(location) shall be…(type, size) complying with BS….
(b) Bed joint reinforcement for…(location) shall be…(type, size, manufacturer's designation) complying with BS….

For type indicate type of steel (i.e., high-yield, mild). For size indicate length and diameter (reference to drawings/schedules is often required). In the case of proprietary bed joint reinforcement, it may be necessary to indicate width, manufacturer's reference number, manufacturer, and so on. A list of the relevant British Standards for reinforcing steel is given in Clause 7.1 of the Code and the steel appropriate to the type or grade of reinforcement assumed in the design for strength and durability *(Section 6)* reasons should be chosen.

1.4.2 Additional protective coatings

★The reinforcement for…(location) shall be galvanised after manufacture in accordance with the requirements of BS 729 with a coating…(mass, thickness) of not less than…(g/m^2, μm).

★The reinforcement for…(location) shall be clad with a layer of austenitic stainless steel of nominal thickness not less than 1 mm.

1.4.3 Prestressing steel

The prestressing steel for (location) shall comply with BS

The relevant British Standards are isted in Clause 7.2 of the Code.

1.5 WALL TIES

Wall ties for low lift grouted cavity construction shall be the vertical twist type to BS 1243. Wall ties for high lift grouted cavity construction shall be of the type described in BS 5628: Part 2: Appendix B, or other similar approved.

★The ties shall be galvanised with a minimum mass of zinc coating not less than 940 g/m^2.

★The ties shall be austenitic stainless steel.

Alternative wall ties to those described in Appendix B may be approved providing they are assessed as adequate to resist the bursting tensile forces which occur during filling and compacting operations. The resistance to corrosion of the wall ties should be at least equal to that specified for the reinforcement used in the same position. Protection systems other than those indicated here may be available.

1.6 DAMP-PROOF COURSES

Damp-proof courses shall be…(description) complying with BS 743.

Other materials not covered by BS 743 may need to be considered due to special problems associated with the incorporation of dpcs in reinforced and prestressed masonry (see *Section 6*).

2. WORK ON SITE

2.1 HANDLING AND STORAGE OF MATERIALS

2.1.1 Masonry units

Masonry units shall be carefully unloaded to minimise damage, placed on the site in different stacks according to strength and type and marked accordingly. The stacks shall be on prepared level areas to avoid ground contamination and shall be protected from rain or snow.

2.1.2 Cement

Cement shall be stored off the ground in a dry structure so as to permit inspection and used in the sequence of delivery. Separate storage, clearly marked, shall be provided for different cements. Cement which has been adversely affected by dampness shall not be used.

2.1.3 Lime

Lime shall be stored in the same way as cement.

2.1.4 Sand and aggregate

Sand and aggregate shall be stored separately according to type, on hard paved areas where they will not become contaminated.

2.1.5 Lime: sand mixture

Lime: sand mixture shall be stored separately according to type, on hard paved areas where it will not become contaminated and it shall be protected from drying out.

2.1.6 Metals

All metal components shall be stored on site in a safe and workmanlike manner. Wall ties and post-tensioning steel shall be stored to prevent metal from becoming rusty or contaminated. Reinforcement and pre-tensioning steel shall be free from loose rust, scale, dirt, paint, oil and grease, or other harmful materials immediately prior to fixing.

2.1.7 Damp-proof courses

★Rolls of dpc materials shall be stored so as to avoid damage and distortion.

This clause may not be necessary where, for example, clay bricks are to be used as a damp-proof course.

2.2 ACCURACY OF CONSTRUCTION

2.2.1 General

The methods of controlling accuracy and setting out shall be in accordance with BS 5606 and BS 5628:Part 3. Care shall be taken to ensure the accuracy and alignment of pockets, cores and cavities containing reinforcement.

Tighter tolerances on construction than those in BS 5606 may be specified providing it is felt they are practicable and achievable without excessive costs.

2.2.2 Dimensions

All masonry shall be set out and built to the respective dimensions, thickness and height indicated.

2.2.3 Uniformity

All work shall be plumbed and levelled as work proceeds.

2.3 LAYING OF MASONRY UNITS

2.3.1 General

The laying of masonry units shall generally be in accordance with BS 5628: Part 3 or BS 5390 as appropriate. The maximum height of masonry built in any one day shall be 1.5 m with no one portion of any section of masonry more than 1.2 m above the level of the adjacent masonry. All masonry units shall be laid and adjusted to final position while the mortar is still plastic.

2.3.2 Bricks

Clay bricks having an initial rate of suction greater than 1.5 $kg/m^3/min$ shall be:

★wetted so as not to exceed this figure at the time of laying

★laid in a mortar containing a water retentive admixture to the approval of the designer

★laid as received except in very warm dry summer conditions when they shall be wetted prior to laying.

Concrete and calcium silicate bricks shall not be wetted. All bricks shall be laid on a full bed of mortar and all vertical joints shall be solidly filled with mortar. Joints shall be nominally 10 mm thick and for…(fair faced work, standard work) shall be…(concave, tooled, struck flush, etc.). Single frogged bricks shall be laid frog up and double frogged bricks with the deeper frog uppermost. All frogs shall be filled with mortar at the time of laying. The coursing and bond of brickwork shall be as shown on the drawings.

★The specially shaped bricks provided shall be used where indicated to accommodate reinforcement and maintain a true and regular bond. Cutting of bricks shall be kept to a minimum.

Where no special requirement for initial rate of suction has been specified in *1.1.2.1*, then the third option here must be chosen; otherwise one of the first two will be appropriate. Care should be taken in detailing reinforced and prestressed elements when the cores or cavities for reinforcement or prestressing steel are to be formed by special shaped bricks or special bond patterns.

2.3.3 Concrete blocks

Blocks shall be laid on a full bed of mortar and all vertical joints shall be solidly filled. Joints shall be nominally 10 mm thick and for…(fair faced, standard) work, shall be…(concave, tooled, struck flush, etc.). Where cores in blockwork are to contain reinforcement they are to be kept clean and clear of any mortar. Clean-out holes should be provided at the bottom of the wall to permit the removal of any droppings. The coursing and bond of reinforced blockwork shall be as shown on the drawings. Cutting of blocks shall be kept to a minimum.

2.4 CONCRETE INFILL

The mixing of infill concrete shall be in accordance with CP 110: Clause 6.7.4.

2.5 WALL TIES, REINFORCEMENT AND PRESTRESSING STEEL

2.5.1 General

Contact between components of dissimilar materials or metal coatings shall be avoided.

This clause is required to avoid bi-metallic corrosion.

2.5.2 Wall ties

Wall ties shall be embedded at least 50 mm into the mortar joint and spaced at intervals of…mm vertically and…mm horizontally. The spacing may be varied provided that the number of ties per unit area is maintained. Additional ties at vertical centres not exceeding…mm shall be provided within 225 mm of any discontinuity of the wall, for example, an opening.

Guidance on the required spacing of ties is given in *Section 6.*.

2.5.3 Reinforcement

2.5.3.1 Bed joint reinforcement

Bed joint reinforcement shall have a side cover of not less than mm, be continuous except where indicated, and be located as shown on the drawings. The method of placing bed joint reinforcement shall be such to ensure each bar is completely surrounded by mortar.

2.5.3.2 Main and secondary reinforcement

The main and secondary reinforcement shall be of the size and number shown on the drawings. All cutting and bending shall be to the dimensions and tolerances specified in BS 4466. Reinforcement shall be tightly bound together with 1.5 to 2.0 mm annealed tieing wire and shall be firmly fixed and maintained in the correct position during concrete infilling. Concrete cover shall be maintained by suitable spacers.

Bar bending schedules will be supplied by the designer and the contractor shall check these against the drawings and be responsible for their accuracy. Minimum concrete cover to reinforcement shall be as indicated on the drawings.

2.5.4 Prestressing steel

The positioning, tensioning and protection of prestressing tendons shall be carried out in accordance with CP 110: Part 1 and any additional requirements indicated on the drawings.

The procedure for tensioning and locking off the force in the steel should be discussed with the contractor and the whole prestressing operation itself may require separate specification.

2.6 INFILLING OF VOIDS

2.6.1 General

Any mortar droppings or scrapings entering cavities or voids to be filled with concrete shall be kept to a minimum and any that does so enter shall be removed at the end of each day through openings left for this purpose at the base of the cavity or void. Where the size of void permits, compaction of the concrete shall be by poker vibrator, otherwise at the discretion of the designer, compaction shall be by hand using suitable tamping rods.

This is a particularly important aspect since if mortar is allowed to accumulate in cores and cavities, the bond and effective compaction of the concrete may be impaired. Similarly if mortar accumulates at the base of the wall the resistance to bending and shear forces at this often critical point may be drastically reduced.

2.6.2 Grouted cavity construction

2.6.2.1 Low lift construction

In low lift construction the concrete infill or mortar shall be placed as part of the process of laying the units at maximum vertical intervals of 450 mm. Each layer of concrete or mortar shall be placed to within 50 mm of the level of the top of the last course laid and shall be placed using receptacles with guards to avoid splashing and staining of face work. The concrete infill shall be compacted immediately after pouring. Care shall be taken to avoid dismption resulting from raising the walls too rapidly. Any wall disrupted in this way shall be taken down and rebuilt.

2.6.2.2. High lift construction

In high lift construction the walls shall be built to a maximum height of 3.0 m. Clean out holes of minimum size 150×200 mm spaced at approximately 500 mm centres shall be provided at the bottom of the wall. After cleaning of the cavity these holes shall be blocked off and the concrete infill placed not sooner than three days after building. The infill concrete shall be placed and compacted in… (one, two, etc.) lifts. Where initial settlement of the infill concrete occurs due to water absorption by the masonry, recompaction of the concrete before initial set occurs will be permitted.

The number of lifts required will be dependent upon the overall height of the wall; with walls up to 3 m high, two lifts will generally be sufficient.

2.6.3 Reinforced hollow blockwork

2.6.3.1 Low lift construction

In low lift reinforced hollow blockwork construction the concrete infill shall be placed as part of the process of laying the blocks, at maximum vertical intervals of 900 mm. Each layer of infill concrete shall be placed to within 50 mm of the level of the top of the last course laid, and shall be placed using receptacles with spouts to avoid splashing and staining of face work. The concrete infill shall be compacted immediately after pouring.

2.6.3.2 High lift construction

In high lift reinforced hollow blockwork construction the walls shall be built to a maximum height of 3.0 m. Clean out holes of a minimum size 100×100 mm shall be provided at every core to be filled.

The alternative to cutting clean out holes in the blocks is to sit the ends of the first row of blocks on concrete bricks.

2.6.4 Quetta and similar bond walls

Main reinforcement shall be fixed sufficiently in advance of the masonry construction so that other work can proceed without hinderance.

★The cavities formed around the reinforcement by the bonding pattern shall be filled with mortar or concrete infill as the work proceeds.

★The cavities formed around the reinforcement shall be filled by the low lift technique as described in 2.6.2.1.

★The cavities formed around the reinforcement shall be filled by the high lift technique as described in 2.6.2.2.

The low and high lift techniques should only be specified when the voids are sufficiently large.

2.6.5 Pocket type walls

In pocket type wall construction, the main reinforcement shall be fixed in advance of wall construction. Care shall 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 concrete and to avoid grout loss.

Main reinforcement may only need to be fixed in advance of wall construction where bed joint reinforcement is to be incorporated.

2.7 PROTECTION

2.7.1 Stability

Precautions shall be taken to ensure the stability of walls during construction, during concrete infilling operations and during backfilling.

Props shall not be removed from beams until…days after concrete infilling except by express permission of the designer.

Guidance on the length of time work requires propping is given in Table 52: CP 110. The time indicated above will, in general, be conservative. Therefore, if approached by the contractor to remove props early the designer should consider this in relation to the in service stresses required, the grade of concrete and an assessment of the rate of gain of strength.

2.7.2 Weather

All newly constructed masonry shall be protected to prevent rain falling on its top surface or water being channelled into it until the work has its finally intended protection. Fair faced work shall be protected against striking or other damage resulting from construction operations.

The last sentence is particularly relevant to filling cores and cavities with concrete.

The contractor shall be responsible for avoiding the harmful effects of frost to materials stockpiles and to the masonry, during and immediately after construction. Frozen materials shall not be used.

This part of the clause of frost attack is non-specific. If the designer wishes to be more precise then the following notes should be useful.

The general precaution during cold weather is to either stop laying or concreting when the air temperature is close to freezing (say 3° or less) or ensure a minimum temperature of 4° in the work when laid and thereafter prevent the mortar or concrete from freezing until they have gained sufficient strength. Problems can still exist even when initial air temperatures are above 3° if overnight temperatures are expected to be lower or the units are very cold. Further guidance is given in BS 5628: Part 3.

2.8 MOVEMENT JOINTS

Movement joints shall be formed where specified in accordance with the details given. Care shall be taken to ensure that the joints are free from debris. On no account shall any expansion joint be pointed with mortar.

Further guidance is given in *Section 6*.

2.9 DAMP-PROOFING

The provision and detailing of damp-proof courses, membranes and copings shall be in accordance with the details shown. All damp-proof courses shall be laid with an even bed of mortar both sides and, whilst exposed, shall be protected from damage.

Reinforced or prestressed masonry retaining walls shall be drained by weepholes of not less than 75 mm diameter, at not more than 2 m centres and approximately 300 mm above the lower finishes, ground level, or as indicated on the drawings.

The inclusion of weepholes may not always be necessary.

2.10 FORMING CHASES AND HOLES AND PROVISION OF FIXINGS

Chasing of completed walls, the formation of holes or the inclusion of fixings, shall only be carried out with the approval of and to the requirements of the designer.

3. QUALITY CONTROL

3.1 WORKMANSHIP

Quality control of the workmanship in reinforced and prestressed masonry construction shall be maintained by:

(a) either frequent visits to site by the designer/engineer or the presence of his permanent representative on site, to ensure that the work is built in accordance with the requirements of the Specification
(b) preliminary and site testing and sampling

These requirements may be waived at the discretion of the designer where bed joint reinforcement is included in the masonry for the enhancement of lateral load resistance only.

3.2 MATERIALS

3.2.1 General

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

3.2.2 Masonry units

Compressive strength tests shall be carried out by an approved authority in accordance with BS 3921 for clay bricks, BS 187 for calcium silicate bricks, or BS 6073: Part 1 for concrete blocks and concrete bricks.

The requirements for special category manufacturing control are given in Clause 20.2.2 of BS 5628: Part 2.

3.2.3 Mortar

Trial mixes and site control of mortar shall be carried out following the procedures given in Appendix A of BS 5628: Part 1.

3.2.4 Infill concrete

Fresh and hardened infill concrete shall be sampled and tested in accordance with the requirements of BS 1881. A prescribed mix shall, unless otherwise notified by the designer in writing, not be sampled and tested for strength. A designed mix shall be tested for compressive strength of the hardened concrete. The contractor shall bear all costs in connection with the supply and testing of the test cubes required by the preliminary and works tests procedures. The contractor shall also allow for the making and testing of the number of cubes required by the designer.

The compliance of prescribed mixes is based on specified mix proportions and required workability.

3.2.5 Grout in prestressed members

The quantity of grout used shall be checked to ensure that the prestressing ducts are completely filled.

3.2.6 Reinforcement

When directed by the designer, the contractor shall provide samples taken from steel on site and have them tested for compliance with the appropriate British Standards at an accredited testing laboratory.

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