REINFORCED MASONRY

The Development of Reinforced 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. 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 ½ 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. This was used until 1897 and was eventually demolished in 1932[4].

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 the San Merino Pavilion for the 1900 Paris Exhibition, the Church of St Jean de Montmarre and a fashionable house in the Avenue Rapp, Paris. Figure 2 illustrates 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 x 73 mm deep x 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 3, 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.

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 4 shows a beam being subjected to a "live" load. At Quetta reinforced brickwork was built in a special bond (Quetta bond) 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.

More recent developments include the widespread use of reinforced hollow block masonry, particularly in seismic areas. Other typical applications for vertical reinforced masonry include increasing the resistance of walls to wind loading.

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. A great deal of attention has been given to the possibility of producing pre-stressed brickwork[9,10,11] and bonding arrangements have been devised which permit the introduction of both prestressing tendons and shear reinforcement. 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 pre-stressing in building construction is the vertical post-tensionsing of walls to resist lateral loading from either wind, stored material or retained earth[13,14,15].

Post-tensioned diaphragm walls were also used by W G Curtin and Partners[16] for the 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. Vertically prestressed walls which act compositely with connected floors have been laboratory tested' and also used in the George Armitage office block to build storey height box section cantilevers[19]. 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[(21,22)].

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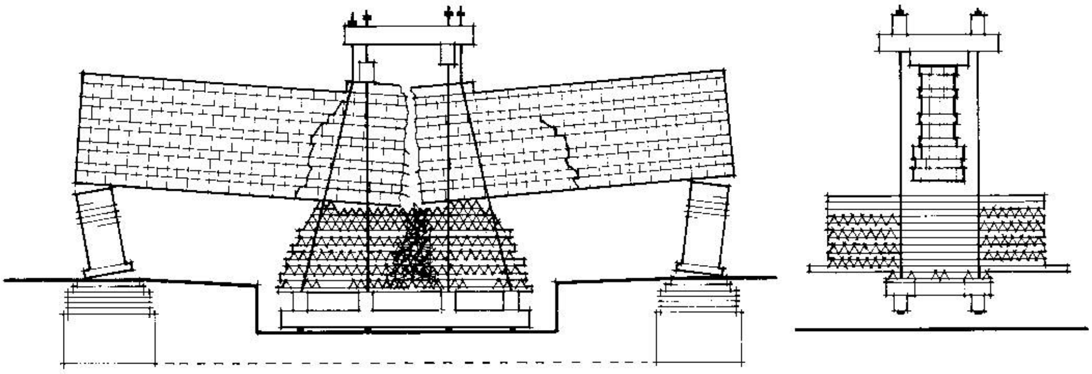


Figure 1 Nine Elms beam test, 1838

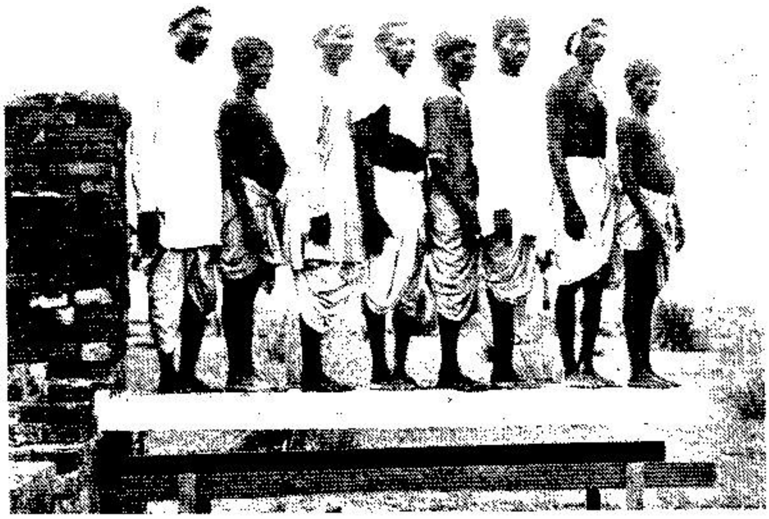


Figure 2: An early load test.

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

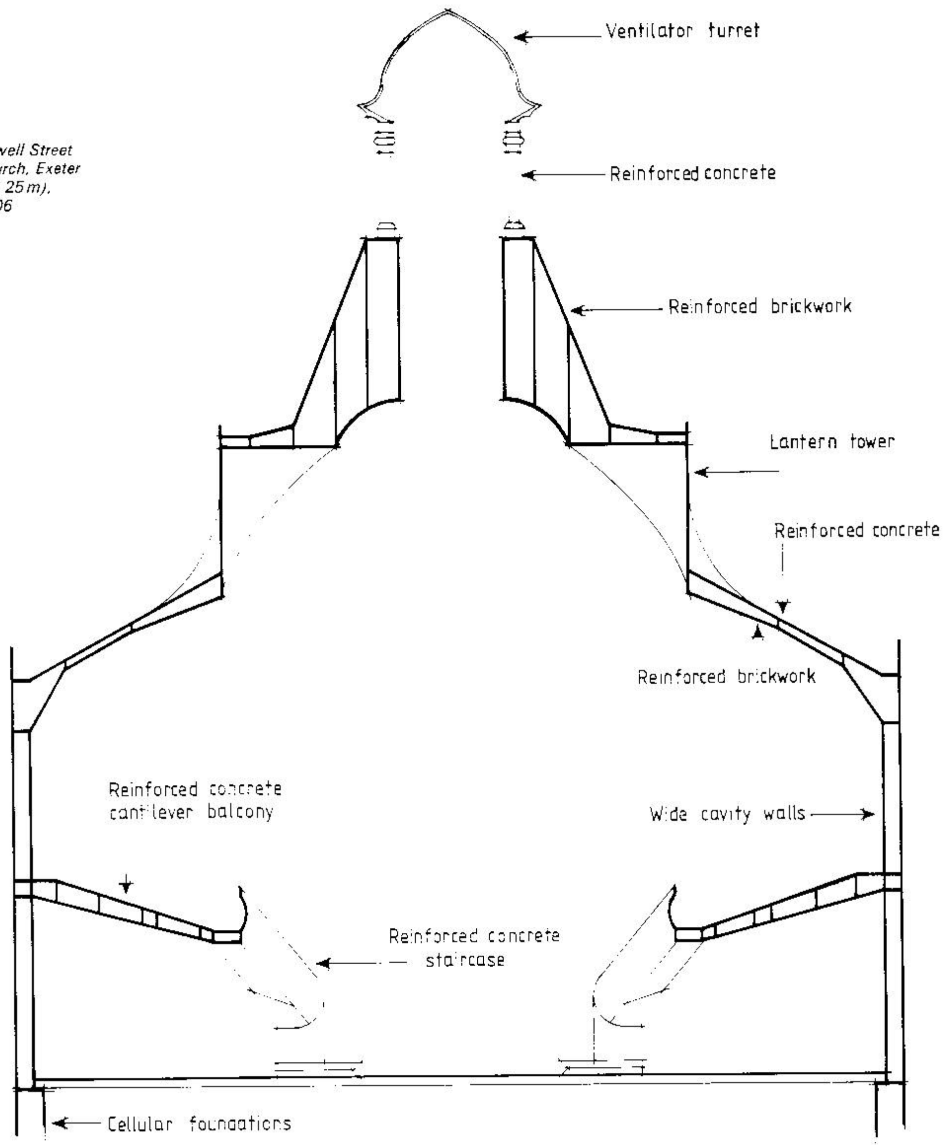
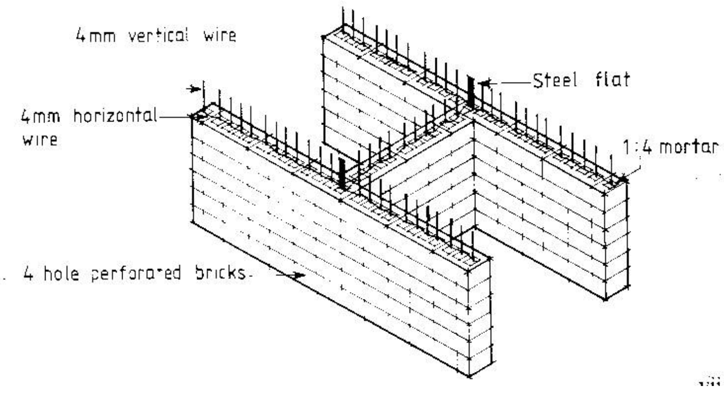


Figure 4 Detail of wall and cross rib of Sidwell Street Methodist Church



Design to BS 5628 Part 2

Preparation of the first design guidance for reinforced masonry (more specifically reinforced brickwork) commenced in 1937 but was not issued until 1943 in the form of a British Standard BS 1146. Some guidance on reinforced masonry was provided in CP 111[2] but it was not until the introduction of BS5628: Part2 in 1985 that detailed design guidance became available in the UK. Subsequently BS5628 was amended and new edition published in 1995 and the current version in 2005.

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.

Definitions

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.

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.

Earlier guidance on reinforced brickwork did not include the concrete or mortar in the cavity as contributing to the compressive strength 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. 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.

Pocket type masonry

This type of construction is so named because the main reinforcement is concentrated in vertical pockets formed in the masonry. 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.

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.

Quetta bond

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[8]. The section produced by this bond is at least one and a half units thick 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. In thicker walls the steel may be placed nearer to the faces to resist lateral loading more efficiently.

Reinforced Hollow Blockwork

In this form of construction the cores of hollow blocks are reinforced with steel and filled with in situ concrete. The work size of the most common blocks is 440 x 215 x 215 mm, although 390 x 190 x 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.

Materials and components

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. 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. 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 to check compliance with any minimum strength requirement.

Wall ties

When the low lift grouting technique is employed in conjunction with cavity construction, the vertical twist type of tie 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.

Concrete infill and grout

The minimum grade of concrete infill which may be employed in reinforced masonry is a Grade 30. As an alternative to the Grade 30 mix, a mix of the following proportions by volume of the dry materials may be used or grouted cavity and quetta bond reinforced masonry construction:

1: 0 – $^{1}/_{4}$:3: 2 cement lime : sand:l0 mm maximum size aggregate

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 the Code. In some situations a concrete of a Grade better than 30, up to a Grade 50, may be required.

Chlorides

Limits are placed on both the percentage of chloride ion present in sands and in concrete and mortar mixes. 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.

Basis of design

Limit state design

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

1.final deflection not to exceed length/125 for cantilevers or span/250for all other elements

2.limiting deflection 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 span/300 if finishes are to be applied, unless uniformity of camber between adjacent units can be achieved.

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.

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. 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).

Shear

For simply supported beams or cantilevers an enhancement factor of 2 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. The maximum factor of 2 implies a cut off in the shear strength at a ratio $a_v/d=1.0$.

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**.**

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:

$$f_v = 0.35 + 17.5\rho$$

where $\rho = A_s/bd$

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

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

$$[\ 2.5 - 0.25\ (a/d)\ ]$$

Here the shear span is defined as the ratio of the maximum design bending moment to the maximum design shear force, i.e., M/V. An upper limit of 1.75N/mm^2 is applied, i.e., a maximum enhancement of 2.5 when a/d = 0; the enhancement factor equals 1.0 when a/d = 6. Much below *ad*=2, the masonry would act as a corbel not a beam, above 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.

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 is due to an increased "friction effect" preventing sliding.

Characteristic anchorage bond strength, f_b

Reinforcement exhibits better bond strength in concrete than in mortar and this is reflected in the values given in the code. 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. 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.

Elastic moduli

For all types of reinforced masonry the short term elastic modulus, *Em,* may be taken as $0.9f_k$ 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. 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.

Partial safety factors

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.

Ultimate Limit State

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.

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.

Serviceability limit state

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.

Moments and forces in continuous members

In continuous members and their supports it is necessary to consider the effects of
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 ($1.4\ G_k + 1.6\ Q_k$) and minimum dead load ($0.9\ G_k$)
2. all spans loaded with maximum combination of dead and imposed load.

Design of reinforced masonry

Reinforced masonry subjected to bending

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 watts 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. 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 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.

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

Limiting dimensions

Attention is drawn to the fact that the limiting ratios given in the Code should not be used when more stringent limitations on deflection and/or cracking are required.

Walls subjected to lateral loading

Limiting values of the ratio span to effective depth for walls subjected to lateral loads are given in 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.

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 used 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 the Code are fairly conservative.

For simply supported or continuous beams the distance should not exceed the lesser of 60 b_c and 250 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 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.

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 under-reinforced. 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.

Analysis of sections

The mean stress at failure of the masonry in compression is assumed to be f_k/γ_{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.

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. 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. 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. 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.

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
f_k = characteristic compressive strength of masonry
z = lever arm, which should not exceed *0.95*
γ_{mm} = partial safety factor for strength of masonry
γ_{ms} = partial safety factor for strength of steel

Design formulae for walls with the reinforcement concentrated locally

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 x 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. 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. 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.

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 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 x the thickness of the block.

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, $e_x = \frac{M}{N}$ 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.

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)
V = the shear force due to design loads

This equation treats the shear stress as if it were uniformly whole cross section as far as the tensile reinforcement.

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.

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 recognized 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.

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:

$$f_v = 0.35 + 17.5\rho$$

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:

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

where $\rho = \frac{A_s}{bd}$

$\frac{a}{d} =$ 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 $\frac{M}{V}$

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 be enhanced in relation to any precompression which exists.

<u>Provision of shear reinforcement</u>

If $\frac{f_v}{\gamma_{mv}}$ is ≥ v then for many structures (for example, retaining walls) shear reinforcement is not generally needed. For beams in which $v \langle \frac{1}{2}\frac{f_v}{\gamma_{mv}}$, 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.

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, $\frac{f_v}{\gamma_{mv}} \langle v$, and it is not possible to adjust the section as previously described, shear reinforcement should be provided according to the requirement:

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

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 ≤ *0.75 d*
v= shear stress due to design loads $\leq \frac{2.0}{\gamma_{mv}}$ N/mm^2

This formula has been developed from the truss analogy and has been shown experimentally 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 (when large shear forces are being supported it is possible that the diagonal compressive force could cause failure). Experimental observations of cracking indicated that the inclined compression struts can be taken at 45^0 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*. 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. 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:

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

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

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. An eccentricity of 0.05 times the depth of the section in the plane of bending is a common reference point.

Slenderness ratios of walls and columns

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.

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 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.

Short columns

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 analyzed using strain compatibility to determine the design moment of resistance and the design axial load capacity, and the second is to use the design formulae given.

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.

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. 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 I, making no allowance for the reinforcement. Recourse to Part I is also recommended for the design of walls subjected to concentrated loads, the implication being that the provision of special reinforcement is impractical.

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.

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.

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 x the breadth of the section. There are no minimum recommendations in the Code, although many of the early drafts included the following limitation:

$A_s \geq 0.002bd$ for mild steel
$A_s \geq 0.015bd$ 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. 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

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.

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 Particular attention should be paid to the durability requirements of a section especially with respect to steel embedded in mortar.

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. 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 x 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.

Classification of exposure situations

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

El very sheltered or sheltered
E2 sheltered/moderate or moderate/severe
E3 severe or very severe

There are, in addition, certain local conditions to which the masonry may be exposed which can be classified in a similar way to site exposure but are not dependent upon it. Examples are:

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 moor land water, corrosive fumes, abrasion or dc-icing salts. This exposure situation is defined as E4.

Construction

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.

Grouted cavity construction

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.

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.

High lift

The clean out holes at the base of the wall should be at least 150 mm x 200mm 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 l0m) 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.

Reinforced hollow blockwork

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.

Low lift

The reinforcing steel within the cores may be located by tying 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.

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. 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 readily compacted when a 25 mm diameter poker vibrator is used.

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.

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.

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, either low or high lift techniques may be used.

Pocket type walls

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.