DEVELOPMENT OF DESIGN GUIDANCE FOR REINFORCED MASONRY IN THE U.K.

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ABSTRACT
At the First Canadian Masonry Conference in Calgary in 1976 I presented two research papers one of which dealt with the behaviour of vertically reinforced blockwork [1&2]. Since that time a Code of Practice for reinforced masonry, BS 5628 Part 2, was introduced in the UK and subsequently updated and developed [3]. In 2010 the BS 5628 standards were withdrawn in favour of BS EN 1996-1-1(Eurocode 6) [4]. This paper traces the development of reinforced masonry in the UK from some of the earliest test sources to the current recommendations contained in Eurocode 6 and the associated UK National Annex [5].

KEYWORDS: reinforced, masonry, Eurocode, design, code

INTRODUCTION
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 [6]. Some guidance on reinforced masonry was provided in CP 111 [7] but it was not until the introduction of BS5628: Part 2 in 1985 [3] that detailed design guidance became available in the UK. Subsequently BS5628 was amended and new editions published in 1995, 2000 and 2005. This paper highlights some of the principal differences between BS 5628 and Eurocode 6.

BS 5628 PART 2
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 first draft of the standard was produced by BCRL[1] and the C&CA[2] under contract.

The document gave 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, was guidance on design methods for walls containing bed joint reinforcement to enhance their resistance to lateral load.

Since this Code was essentially structural in content, attention was drawn to the need to satisfy

[1] British Ceramic Research Limited
[2] The Cement and Concrete Association

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 assumed 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 [8] which dealt with unreinforced masonry, only recognized the special category of construction control. The definition of masonry in the standard permitted units to either be laid in situ or as prefabricated panels. In both cases the units had to be bonded and solidly put together with concrete and/or mortar so as to act compositely.

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 defined the four types of construction most likely to be employed, but the many other possibilities are equally valid. The types defined were:
(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 as shown in Figure 1. 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.

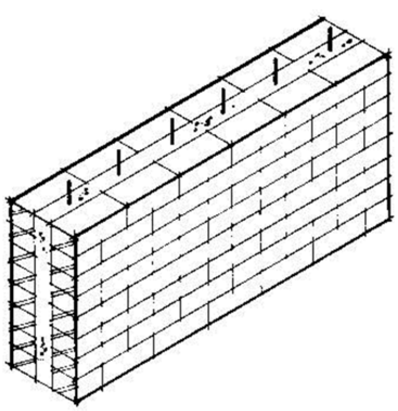

Figure 1: Grouted cavity masonry.

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.

POCKET TYPE MASONRY

This type of construction is so named because the main reinforcement is concentrated in vertical pockets formed in the masonry as shown in Figure 2. 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.

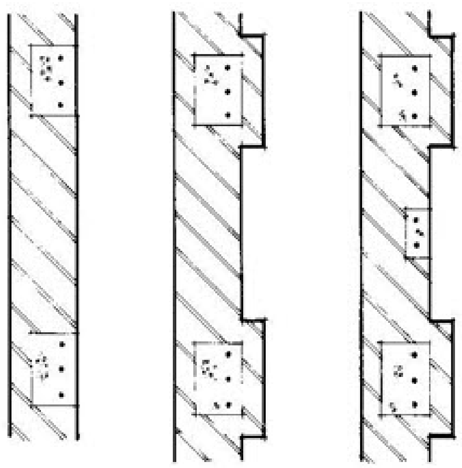

Figure 2: Plan showing different arrangements of Pocket Type masonry walls

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 fixed 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 Pakistan following earthquake damage. 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. This is shown in Figure 3.

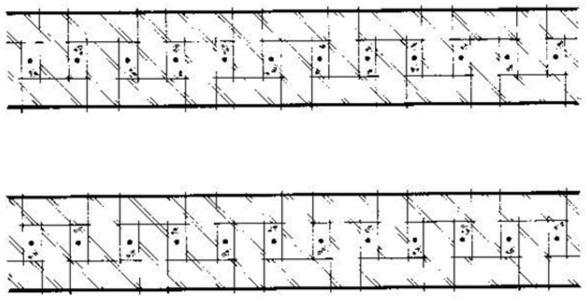

Figure 3: Quetta bond

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. 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.

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.

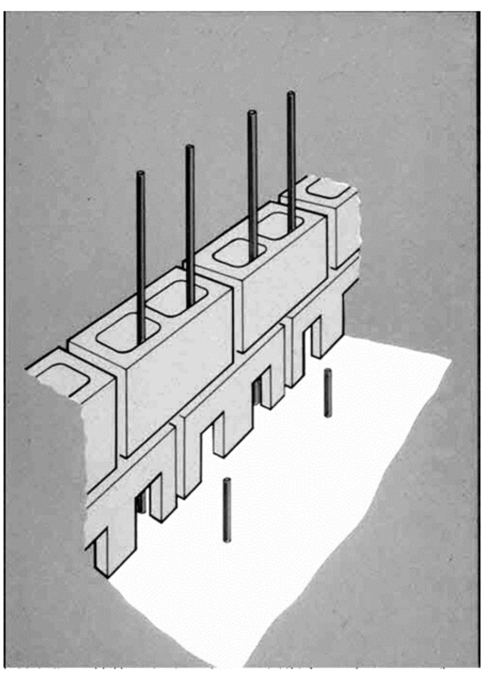

Figure 4: Reinforced hollow blockwork

In addition to the standard two core hollow blocks, specials such as lintel and bond beam blocks are available. 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.

REINFORCED MASONRY SUBJECTED TO BENDING

The design of elements subjected only to bending 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. 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 both BS 5628 and Eurocode 6 to the design of members subjected to bending has been developed from the simplified approach developed for concrete.

The designer may calculate deflections 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. Both codes adopt this approach.

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 idealized 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 5.

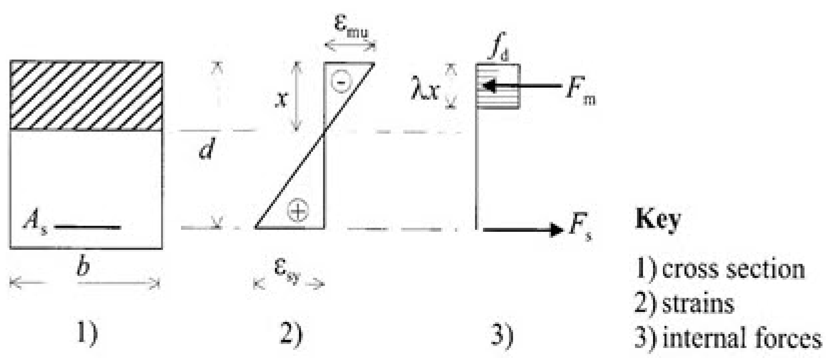

Figure 5: Stress and strain distribution in Eurocode 6 and BS 5628.

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 factor for the compressive strength of masonry. This partial 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 for all types of masonry material 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 both codes.

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

In the case of 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) they may be assumed 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 in BS 5628 is given in Equation 1:

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

this must not exceed the limit set in Equation 2:

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

z is determined using equation 3:

$$z = d(1 - 0.5\frac{A_s}{bd}\frac{f_y}{f_k}\frac{\gamma_{mm}}{\gamma_{ms}}) \tag{3}$$

Where:
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 factor for strength of masonry
γ_{ms} = partial factor for strength of steel

In Eurocodes these equations are expressed as design values and thus equations 1 and 3 are presented in the form of equations 4 and 5.

$$M_{Rd} = A_s f_{yd} z \tag{4}$$

$$z = d(1 - 0.5\frac{A_s}{bd}\frac{f_{yd}}{f_d}) \leq 0.95d \tag{5}$$

A difference in Eurocode 6 is that additional limits are placed on the capacity of the masonry depending upon the type of masonry unit employed as shown in equations 6 and 7:

$$M_{Rd} \text{ limited to} \leq 0.4 f_d bd^2 \tag{6}$$

for Group 1[3] masonry units – this is the same as BS 5628 Part 2.

M_{Rd} limited to $\leq 0.3\, f_d bd^2$ (7)

for Group 2 masonry units and Group 1 lightweight aggregate masonry units– this is additional to BS 5628 Part 2.

Unlike BS 5628 no convenient interaction formulae of graphs are provided in Eurocode 6.

FLANGED MEMBERS.

There are a number of situations where reinforced masonry elements may be considered to act as flanged members and both codes include recommendations for the more usual cases. 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:

- for pocket-type walls, the width of the pocket or rib plus 12 x the thickness of the flange
- the spacing of the pocket or ribs
- 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 in Equation 8:

$$M_d = \frac{f_k}{\gamma_{mm}} bt_f (d - 0.5t_f) \qquad (8)$$

The limit shown in Equation 9 applies:

M_{Rd} limited to $\leq f_d b_{ef} t_f (d\text{-}0.5t_f)$ (9)

[3] A wide range of masonry units is available across Europe and for structural design purposes these are classed into four groups as defined in Table 3.1 of Eurocode 6. The groups reflect aspects such as the proportions of voids and hollows, the direction in which they run and the thicknesses of webs and shells. Solid units would fall into Group 1 and most two core hollow units into Group 2. When hollow units are completely filled with concrete Eurocode 6 treats them as Group 1 units for design purposes.

which is also effectively the same as BS 5628 Part 2. Limits for effective width of the T section are the same as BS 5628 Part 2. There are additional limits for the effective width of an L section– this is additional to BS 5628 Part 2.

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

Both codes include clauses dealing 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. 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 pre-compression.

The shear stress at any cross section, v, is calculated in BS 5628 from Equation 10:

$$v = \frac{V}{bd} \tag{10}$$

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

Equation 10 treats the shear stress as if it were uniform across the whole cross section as far as the tensile reinforcement. This is not strictly true and many researchers have found that the shear resistance is made up of a number of component forces. The same approach is, however, adopted in Eurocode 6.

The shear resistance of the section includes contributions from the un-cracked 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.

It is known 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 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. In BS 5628 an enhancement to the characteristic shear strength of masonry, f_v, 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 as shown in Equation 11.

$$f_v = 0.35 + 17.5\rho \qquad (11)$$

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 Equations 12 and 13:

$$f_v = [0.35 + 17.5\rho]\left[2.5 - 0.25\frac{a}{d}\right] \qquad (12)$$

$$\rho = \frac{A_s}{bd} \qquad (13)$$

Where $\frac{a}{d} =$ 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 use of Equation 11 is not included in the body of Eurocode 6 but the approach is covered in Informative Annex J which has been adopted for use in the UK.

WHAT IS OMITTED FROM EUROCODE 6?

Eurocode 6 does not provide the following:

- Design guidance for slender walls and columns
- Design guidance for biaxial bending
- Only limited lateral load enhancement options

There is, however, guidance on the design of deep beams whereas there is no specific guidance in BS 5628 for deep beams

For enhancing the flexural strength of panels Eurocode 6 permits two of the four methods recognized in BS 5629 Part 2, namely:

- Design as horizontally spanning wall
- Design using modified orthogonal ratio

The other two methods have been retained for use in the UK by incorporating them in PD 6697 [9].

- Design with reinforced section carrying extra load only
- Design based on cracking load

Eurocode 6 gives the principle for the design of prestressed masonry but no application rules to assist the designer. In the UK those parts of BS 5628 which are not covered by Eurocode 6, and do not conflict with Eurocode 6, have been retained in a BSI publication PD 6697.

CONCLUDING REMARKS

Reinforced masonry has not been widely used in many parts of Europe and the requirements in Eurocode 6 have been heavily influenced by BS 5628 Part 2. There has been a dearth of recent research into the behaviour of reinforced masonry in the UK, primarily because of the difficulty in attracting research funding for what is seen as established technology, and this has had an adverse effect on introducing new material into the codes. Enhanced regulatory requirements for the avoidance of accidental damage have reduced the use of unreinforced masonry structures over 3 storeys high in the UK whilst in Europe in general seismic requirements have been enhanced. Both these factors could eventually lead to renewed research interest and updated application rules.

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