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Elements of Loadbearing Brickwork

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Preface

EVIDENCE of the vitality of brick masonry and of recent developments in research and in the application of this form of construction are to be found in the *Proceedings* of the International Conferences devoted to the subject which are now being convened at regular intervals. This renewed interest in masonry construction, especially for residential construction, follows on the application of engineering principles, backed by scientific testing of materials, to the structural design of masonry elements. This has resulted in economical buildings, which retain the advantages of brickwork in terms of appearance, durability, sound insulation and fire resistance.

The teaching of brickwork construction in universities and colleges, however, has not kept pace with these important developments, partly because of the lack of suitable textbooks. Dr. Lenczner's book is, therefore, to be welcomed as it will provide a useful introductory review of the subject suitable for students of structural engineering, building and architecture. It will also be found useful by those in practice who are in need of a concise review of brickwork construction. I recommend the book to this readership and hope that it will enjoy the success it deserves.

A. W. HENDRY

Author's Note

BRICKWORK is one of the oldest building materials, but it is only in the last few decades that scientific principles have been successfully applied to it, leading to a new concept of brickwork design and construction. My purpose in writing this book was to keep abreast of these developments. My approach was to depart from the traditional treatment of the subject which, in the past, tended to be oriented mainly towards the craftsman, and to present it as a legitimate field of study for students reading for Civil Engineering, Building and Architecture. To this end I have tried to concentrate mainly on the basic properties of brickwork and the application of analytical thought to the subject. My philosophy throughout has been to place the emphasis on principles rather than detail.

In gathering information for the book I have occasionally borrowed from other sources and in some cases made direct or near direct quotations. My reason for doing this was to ensure the authenticity of the statements and to avoid the risk of misinterpretation. I wish to express my thanks and appreciation to those people whose work I have quoted. A list of references at the end of each chapter gives the names of the authors and details of their publications.

During the writing of the book I received help and encouragement from many friends and colleagues. I wish to thank in particular Mr. K. Thomas and Mr. D. Foster for their help and advice. Finally, I wish to extend my sincerest thanks to Professor A. W. Hendry for his patient and thorough reading of the manuscript and for his many useful comments which will undoubtedly add to the value of the book.

D. LENCZNER

Glossary

bed-joint—the horizontal joint in a wall.

cavity wall—a wall constructed of two separate panels with a cavity between and tied together with metal ties or stays.

chase, chasing—a channel or groove formed or cut in the material.

efflorescence—a chalk-like appearance on a wall, due to the crystallization of the alkaline salts contained in the bricks and mortar.

frog—the indentation in the bedding surface of a brick, to reduce the weight and provide a key for the mortar.

leaf—each continuous, vertical section of a wall, one masonry unit in thickness.

perpend joint—the vertical joint in brickwork.

racking strength—the strength of a member subjected to an in-plane horizontal force.

spall, spalling—breaking away of fragments from the surface of wall.

Specifications and Codes of Practice

British Standard 3921: 1965, *Specifications for Bricks and Blocks of Fired Brickearth, Clay or Shale.*

British Standard 4551: 1970, *Methods or Testing Mortars.*

Model Specification for Loadbearing Clay Brickwork, Special Publication No. 56, The British Ceramic Research Association, 1967.

Code of Practice CP 111: 1970, Part 2, *Structural Recommendations for Loadbearing Walls.*

Code of Practice CP 3: 1970, Chapter V, *Wind Loads.*

Code of Practice CP 114: 1969, Part 2, *The Structural Use of Reinforced Concrete in Buildings.*

Code of Practice CP 115: 1969, Part 2, *The Structural Use of Prestressed Concrete in Buildings.*

Building Regulations (5th Amendment) 1970. Ministry of Housing and Local Government.

CHAPTER 1

Manufacture and Properties of Bricks

1.1. Nature of Ceramics

Bricks belong to a group of materials which we call ceramics. These are inorganic, nonmetallic materials, usually processed at high temperature. They include a wide range of silicates and metal oxides and combinations of silicates and metal oxides. Ceramics can be grouped into three divisions: heavy clay, refractories and pottery. The fact that they are compounds of oxides means that they are chemically stable. Physically, ceramics are hard, brittle, non-ductile and highly temperature-resistant. In common with many other ceramic materials, bricks are therefore very suitable as building materials.

1.2. Raw Materials

The raw materials for the manufacture of bricks are clay or shale (another type of brick called sand–lime brick is not considered here). Clay as dug out from the ground usually contains the essential minerals silica (SiO_2), alumina (Al_2O_3) and kaolinite ($Al_2O_3.2SiO_2.2H_2O$). Other ingredients often present are: cordierite ($2MgO.2Al_2O_3.5SiO_2$), steatite ($3MgO.4SiO_2.H_2O$), feldspar [$(Na,K)_2O.Al_2O_3.6SiO_2$] and mica ($K_2O.3Al_2O_3.6SiO_2.2H_2O$). The familiar red building-brick clays may contain up to 50% kaolinite with quartz, mica and about 5 to 10% of iron oxide which gives the brick its red colour and has important effects on the firing.

1.3. Processing

The various stages in the processing chain are grinding, mixing and screening to remove small boulders. Following this the clay is passed on to a wet pan, which is a revolving roller mill with one or more grid opening in the base. Water is added to produce the required degree of plasticity in the clay which is now ready for being pressed or extruded.

In the extrusion process the plastic clay is put through a pug mill which contains a screw extruder. The clay is extruded through a die and the product is a firm clay column, whose depth and length is of a size calculated to give the correct width and length in the burnt bricks, after drying and firing contractions have taken place. The clay column is cut by wire into bricks ready for drying and then firing.

In the pressed process, the clay is delivered to a pug which forces it into moulds. From there the rough brick clots are delivered to the brick press which gives the final shape to the brick. At this stage the bricks are again ready for drying and firing.

1.4. Action of Heat

The process of firing is extremely important in determining the properties of the bricks. Considering the case of a single clay consisting solely of the minerals kaolinite, quartz and mica, the first reaction after the loss of mechanically held water at low temperatures is at about 575°C when the quartz (SiO_2), which is chemically unchanged, undergoes a crystallographic change from the α to the β form. The kaolinite ($Al_2O_3.2SiO_2.2H_2O$) breaks down with loss of chemically combined water at about 600°C to yield meta-kaolinite ($2Al_2O_3.4SiO_2$), generating heat in the process. At 1100°C or above, the meta-kaolinite changes into mullite ($3Al_2O_3.2SiO_2$), one of the constituents of the final product, and releases some surplus of free silica which subsequently reacts at 1200°C to form other compounds. At the same time the mica has also broken down with loss of combined water, beginning at about 800°C, to form, at 1100°C, a glass in the system K_2O–SiO_2 and crystals of mullite. Thus the original quartz, kaolinite and mica finish up as three crystalline components—quartz, cristobalite and mullite and a glass bonding them together. In the case of building bricks the firing range lies between 950° and 1220°C.

A considerable amount of fuel is required to generate the necessary reaction. Some types of clays, notably the Bedford and London clays, contain as impurities substantial amounts of organic matter, which at high temperatures undergo spontaneous combustion which saves on the total amount of fuel added, thus making the production of bricks substantially cheaper. The penalty of it is, however, that sizeable pores are left in the brick structure which reduce their strength. The effect of pores on the strength of bricks will be discussed in greater detail in the following section.

1.5. Porosity

During the manufacture of bricks, innumerable fine spaces and passages of irregular shapes and varying size are formed in their interior. We call these spaces pores and they effect almost every important property of the brick. Obviously, a parameter defining the amount of these pores would be extremely useful, as it could then be used as an index for the many properties which are closely related to the amount of pores. We call this parameter porosity which is defined as:

$$\text{porosity} = \frac{\text{volume of pores}}{\text{overall volume of substance}}$$

and is often expressed as a percentage. To obtain porosity, we must measure the volume of the pores and also the overall volume. The first measurement is much more difficult. In the case of bricks, the normal procedure is to weigh the brick dry, then immerse it in water in order to fill all the pore spaces with water, which is subsequently weighed, by deducting the dry weight of the brick from the saturated weight. Knowing the density of water the volume of pores can readily be calculated.

The principle sounds simple, but in practice many difficulties are encountered. The space to be filled is not simply a cavity but consists of a labyrinth of fine passages, most of which already contain air which blocks the passage of water. The air itself is trapped in the narrow channels and cannot easily escape, thus preventing complete saturation of the pores. There are fortunately one or two ways of getting over this difficulty. One is to boil the brick in water. This expands the entrapped air, which comes bubbling out, and also produces steam in the pores which displaces the air from them. Subsequently the brick is allowed to cool under water, when the steam in the pores condenses, and water is drawn in to fill the space occupied by the expelled air. Another method is to place the brick in a vacuum chamber from which air is evacuated. Distilled water is then admitted, which effectively saturates the brick.

Many physical properties, such as compressive strength, modulus of elasticity, thermal conductivity, moisture expansion and frost resistance, are closely related to the porosity of bricks. The relationship between strength and porosity is now fairly well established for simple polycrystalline systems to which bricks can be approximated. Many of the experimental results can be accounted for by an exponential relationship and this is in agreement over a wide range of porosities with the results of a theoretical treatment, which considers a number of spheres in contact progressively coalescing. As the spheres coalesce, the contact area increases and the porosity decreases. Assuming the strength S is proportional to the area of contact, it can be shown that the plot of $\ln S$ against porosity p is well represented by a straight line over a range of porosities from about 25% downwards. This relationship can also be written in an exponential form as:

$$S = S_0 \, e^{-bp} \tag{1.1}$$

where S_0 is the strength at zero porosity and b is a constant.

Similarly there is an almost linear relationship between the Young's or elastic modulus E and porosity p given by:

$$E = E_0(1 - cp) \tag{1.2}$$

where E_0 is the Young's modulus at zero porosity and c is a constant ($c = 2$).

Finally thermal conductivity k varies more or less linearly with porosity according to:

$$k = k_0(1 - p) \tag{1.3}$$

where k_0 is the conductivity at zero porosity.

1.6. Strength of Bricks

Due to their brittleness and relatively high porosity, bricks are generally weak in tension and their compressive strength varies with porosity over a wide range. At the upper bound we have the high-strength engineering bricks, with their compressive strength ranging from 55 to 69 N/mm^2 (8000 to 10,000 lb/in^2) upwards (maximum about 138 N/mm^2 or 20,000 lb/in^2). Medium-strength bricks range from approximately 27 to 48 N/mm^2 (4000 to 7000 lb/in^2) and low-strength bricks range from 14 to 25 N/mm^2 (2000 to 3500 lb/in^2). The variability in strength of bricks for any particular batch is quite considerable. This makes it important to use a statistical method in evaluating their mean strength. A coefficient of variation between 15 and 20% for any particular sample is quite typical. The coefficient of variation is defined as the ratio of standard deviation over the average value and is usually expressed as a percentage:

$$\text{standard deviation } \sigma = \sqrt{\frac{\Sigma(x - \bar{x})^2}{n - 1}}$$

where $\bar{x}$ = arithmetical mean = $\Sigma x/n$,
n = number of values.

Standard method of finding the crushing strength of bricks is laid down in British Standard Specification BS 3921 (1965). This states that from any sample at least 10 bricks should be tested in a compression machine between two 3 mm plywood sheets. The bricks must be immersed in water for at least 24 h prior to testing. Bricks with frogs must have them filled with a cement/sand mortar of suitable strength. The bricks are laid on the bed face in the testing machine and the load is applied at a rate of 14 N/mm^2 (2000 lb/in^2) per minute. When carrying out these tests reference should always be made to the appropriate specification.

1.7. Elastic Properties

Bricks, in common with other ceramic materials, are characteristically brittle and their stress/strain relationship remains linear almost up to the point of fracture. It is an interesting fact that whereas the stress at fracture can vary over a wide range, the strain at fracture always lies around 10^{-3}. Putting it in another way, the ratio of stress at failure to Young's modulus is constant at approximately 10^{-3}. This critical strain is very considerably less than would be expected from theoretical consideration of the behaviour of an ideal crystal structure. The discrepancy, which is of the order of 100, can be explained by the presence of minute cracks and flaws which are inevitably present in every brick. If it were possible to produce a brick without these small cracks, its strength could theoretically increase a hundredfold. In fact, scientists have produced very thin ceramic whiskers, which showed these fantastic strengths in a laboratory, but at the present state of technology it is not yet possible to manufacture full-size bricks free of cracks and it is doubtful if it will ever become economically feasible. Research is progressing, however, which is aimed at increasing the strength of bricks at least tenfold.

The Young's modulus of bricks ranges from 3·5 kN/mm^2 ($0{\cdot}5 \times 10^6$ lb/in^2) for low-strength bricks up to 34 kN/mm^2 (5×10^6 lb/in^2) for high-strength bricks. Although under a static load bricks show little or no plastic deformation prior to failure, yet under small alternating stresses remote from fracture bricks exhibit considerable plastic or irreversible strains inside the specimen. Under alternating stresses bricks also exhibit quite high internal friction, which is manifested by the time lag between the maximum stress and strain. If the stress/strain curve is plotted for successive instances of time during a loading cycle, the result is a closed loop, whose area is a measure of the energy dissipated in overcoming internal friction during that cycle. Figure 1.1 shows a typical loop obtained during a loading cycle.

1.8. Movements in Bricks

Bricks coming out fresh from the kiln undergo measurable dimensional changes. The movements, as they are called, are caused primarily by changes in moisture content in the brick and, to a lesser extent, by temperature expansion or contraction. A brick straight from the kiln is bone dry, and when it comes into contact with moist air it will absorb moisture until a moisture equilibrium is reached. The absorption of moisture is accompanied by volume expansion in the bricks.

A considerable amount of research has been carried out on moisture movements in bricks and it can be summarized as follows:

Bricks undergo reversible expansion or contraction due to wetting

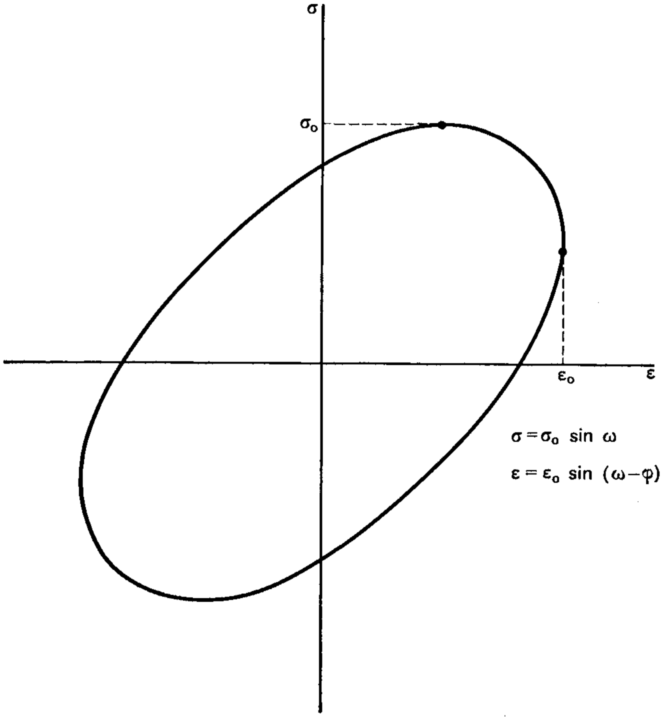


FIG. 1.1. Curve showing lag of strain behind stress.

or drying. Superimposed on this small movement there is a larger, irreversible expansion, taking place over a long period of time. This expansion is greatest during the first day, and subsequently the rate of moisture expansion decreases, reaching a limiting value after approximately 6 months. There is a firing temperature for each clay which gives the maximum moisture expansion. For most of the clays examined, this temperature lies between 900° and 1000°C. The rate of penetration of moisture is slow, but significant, so that the centre of a brick may take 8 days longer to expand than the outside. There is some evidence that redistribution of moisture within the brick may lead to an intermediate contraction, followed by continual expansion. Typical values of linear moisture expansion strains for a 0·229 m (9 in.) long brick after 4 months range from 0·02 to 0·07%.

Dimensional changes in bricks may also be due to thermal expansion or contraction. The expansion varies, in some instances quite considerably, with direction in the brick, depending on direction of pressing or extrusion. The average values of coefficient of thermal expansion for clay bricks lie within the range 3·6 to 5·8 $\times$ 10^{-6} per °C.

1.9. Thermal Conductivity

The thermal conductivity of a material is defined as:

$$k = \frac{q}{A} \cdot \frac{\Delta x}{\Delta T} \tag{1.4}$$

where q is the quantity of heat transferred in unit time through an area A due to a temperature gradient $\Delta T/\Delta x$.

We have already seen in section 1.5 that thermal conductivity of bricks is closely related to its porosity. It is apparent from equation (1.4) that the thermal conductivity is a useful parameter which measures the heat loss through a material by conduction. Generally speaking we wish to keep the value of k as low as possible, and from equation (1.3) we see that this can be achieved by increasing the porosity of the brick. There is a limit, however, to which porosity can be increased, without seriously impairing the strength of the brick. One solution is to use a perforated brick. The loss of area is effectively compensated by increased strength of the solid material due to improved firing in the interior of the brick.

A typical value of thermal conductivity for brick is 0·9 W/m h degC (0·5 Btu/ft h degF). The value increases significantly with moisture content of the brick. The reason for this is that water is a better conductor of heat than air. For effective insulation it is important, therefore, to keep the brick as dry as possible.

1.10. Durability of Bricks

The durability of bricks is a function of their resistance to frost action, chemical attack and moisture penetration. Damage by frost action is caused by expansion of water in the pores as it freezes into ice. Fracture occurs when the expansion of the freezing water cannot be accommodated by the elastic resilience of the brick. Repeated freezing and thawing cycles accelerate the disruptive effect on bricks. Susceptibility of bricks to frost damage increases with their degree of saturation. In fully saturated bricks, the voids are completely filled with water and there is no room for expansion when freezing occurs. In partially saturated bricks, the air in the voids is partially replaced by the expanded ice and frost damage is much less likely.

Results on frost resistance of bricks by direct exposure to outside elements can lead to serious problems in their interpretation. Alternative laboratory tests which have been devised to simulate site conditions include saturation freezing, cyclical freezing and thawing tests.

In the absence of a general consensus of opinion as to what constitutes the most reliable test for frost resistance of bricks, the following empirical rules have been suggested for avoidance of frost damage:

(i) The water absorption, expressed in terms of percentage increase by weight of the dry specimen after 5 h boiling or by vacuum, is not greater than 7 %.

(ii) The saturation coefficient (ratio of absorption after immersion for 24 h to absorption after 5 h boiling) is not greater than 0·6, i.e. if there is 40 % of the pore space difficult to fill with water, so that there remains air space in the voids for the expanding ice.

(iii) The compressive strength of the brick is more than 48 N/mm^2 (7000 lb/in^2).

Among the different agents which cause chemical disintegration, efflorescence is probably the most common. Efflorescence is the visible effect at the surface of a brick of salts which have percolated in solution through the brick, and have precipitated on the surface. The amount seen depends upon the quantity and availability from other sources, including mortar, soluble material and water. The damage it causes depends on the chemical nature of the salts. Thus magnesium sulphate, which crystallizes just behind the face of the brick, causes spalling, while other salts are mainly unsightly.

Most clay bricks contain a small proportion of soluble salts. Well-fired bricks tend to contain a lower amount of soluble salts than under-fired bricks. The presence of soluble salts does not necessarily mean that efflorescence will develop. For efflorescence to occur, salts must be present, water must be available to take them into solution and a dry surface must exist where evaporation can proceed to deposit the crystals.

References

ASTBURY, N. F. (1963) *Introductory Lecture on Ceramics*, Special Publication No. 38, *The Use of Ceramic Products in Buildings*, The British Ceramic Research Association.

BUTTERWORTH, B. (1948) *Bricks and Modern Research*, Crosby Lockwood, London.

BUTTERWORTH, B. (1953) *The Properties of Clay Building Materials*, Symposium held by the British Ceramic Society.

CHAPTER 2

Properties of Mortars

2.1. Development of Brickwork Mortars

The function of mortar is to bed and joint building units, giving them continuity required for stability and exclusion of rainwater. Historically, brickwork mortars have progressed from the sand–lime mortars used early in the century to the present cement–plasticizer–sand or cement–lime–sand mixtures. Sand–lime mortars, which depend on carbon dioxide for carbonation and development of strength, were considered adequate for the massive type of construction then built, using relatively slow-paced construction techniques. The introduction of Portland cement showed that it was compatible with limes, and produced a desirable early setting, which permitted more rapid construction. As the brickwork construction rate increased, the acceptance of Portland cement additions increased, until at the present time sand–lime mortars are seldom used.

For simplicity, the batching of materials to make mortar was done mainly by visual guesswork or, at best, by measurement by volume with the simplest of equipment. In either case, any expression of proportions has been by volume and not by weight. Generally, the proportions by volume of lime or cement to sand has been 1:3 and when both cement and lime were used, the sum of their separate volumes still remained essentially one-third of the volume of sand. The change in properties of the mortar was achieved only by the change in the proportion of cement to lime. A higher proportion of cement would be used where strength was required at an early age or when the ultimate strength had to be fairly high. On the other hand, for mortar which was to be used in situations where freedom from cracking, good bond or resistance to rain penetration were paramount, the proportion of cement to lime was kept low. Thus there has developed a range of mortar mixes which became known as 1:¼:3, 1:1:6, 1:2:9 and 1:3:12, these being the proportions by volume of cement:lime:sand.

In recent times plasticizers other than lime were investigated and ground with Portland cement to improve the economics and to provide a more desirable cement for brickwork construction. Plasticizers such as clay or limestone and air-entraining agents were found to provide mortars with improved workability characteristics without sacrificing setting and hardening properties.

2.2. Properties of Wet Mortar

An ideal mortar is one which possesses the necessary "workability" in its wet state and which on setting attains the strength and good bonding characteristics required for structural stability and durability. The "workability" of a brickwork mortar is difficult to define and even more difficult to measure. The bricklayer's appraisal of workability of wet mortar depends on its ability to be spread easily, its ability to cling to vertical surfaces and its resistance to flow during the placing of bricks or other building units. In laboratory, workability is recognized as a complex rheological property which includes adhesion, cohesion, density, flowability, plasticity and viscosity. Although research continues to measure these individual properties, no single test is available for measuring workability. The bricklayer in performing his task integrates these influences and arrives at a subjective determination of the workability of the mortar.

In the laboratory, the evaluation of mortar properties is made using mortars having the same flow. In the flow test, a truncated cone of mortar is subjected to twenty-five 12·5 mm (½ in.) drops of a standard flow table. The diameter of the disturbed sample is compared with the initial diameter and the ratio of the first over the second is defined as the "flow index".

An important property of wet mortars is their water retention. This property measures the ability of the mortar to retain its mix water when subjected to an absorptive force. Water retention in the field is shown by the mortar's ability to remain workable after contacting an absorptive brick. Mortars possessing low water retentivity lose water rapidly from the mortar bed, making the laying of bricks difficult, whereas mortars with high water retentivity retain the water and make the task of bricklaying much simpler. The water retention of mortars can be increased by adding air-entraining agents and finely ground plasticizers such as limestone, clay or lime.

To measure water retentivity in the laboratory a sample of wet mortar is weighed in a mould. A number of layers of blotting paper are now placed on top of the mortar and are weighed down by a standard weight through a glass plate. The weight of the water retained by the mortar after suction expressed as a percentage of the weight originally present in the mortar is taken as the water retentivity of the mortar.

Another important property of wet mortars is their consistence retentivity. To measure it a ball is allowed to drop through a standard height and its indentation in the mortar is measured. Next the mortar is subjected to a suction by laying blotting paper on its surface as described above. The dropping ball test is then repeated and the indentation measured again. The penetration of the ball after suction expressed as a percentage of the average penetration before suction is the consistence retentivity of the mortar.

2.3. Strength of Mortar

The setting and subsequent gain in the strength of mortar is due to the hydration taking place between the water added to the mix and some of the constituents in the cement. Of these, tricalcium silicate and tricalcium aluminate are the two compounds responsible for the early gain in strength and dicalcium silicate for the subsequent and more gradual strength development.

In testing mortars for strength the most important is the compressive strength, followed by the tensile bond strength. The compressive strength is measured by testing 75 × 75 mm (3 × 3 in.) cubes in a compression machine. The factors which affect the compressive strength of mortars are the cement content of the mix, the water/cement ratio, the proportion of cement to sand and the properties of the sand itself. Generally, a high cement content and a low water/cement ratio yield a mix of high compressive strength. The quality and type of sand have a most pronounced effect on the strength. As a rule, coarse sands tend to yield higher strength than sands with a high proportion of fines. For a 1 : 1 : 6 mix by volume the compressive strength at 28 days could vary from 2·5–7·0 N/mm^2 (400–1000 lb/in^2) for the same workability, according to the type of sand used. One explanation for higher strength, using coarse sand, is that less water is necessary to wet all the surfaces and therefore the required workability can be attained with a relatively low water/cement content, thus leading to a higher strength. Where finer sand is used, the wetted surface is considerably increased, and to maintain the same workability, more water is needed, which leads to a correspondingly lower strength. This explanation, although basically true, is an over-simplification as the distribution of particle sizes in the sand, the particle shape and the impurities, particularly clay content, also play an important part. On the other hand there are practical limitations to the coarseness of sand that may be used, as this can lead to serious loss of cohesion in the mix when in the wet state. Since the strength of mortar increases with time, it is important to specify the age at which a certain strength is attained. The ages which are commonly used are 7, 14 and 28 days.

2.4. Tensile Bond Strength

The tensile bond strength is the adhesive strength developed between the mortar and brick and is influenced by both of them. The mortar must possess the ability to flow and fill the head joints, and wet the surface of the brick to be bonded. The brick must also possess sufficient surface irregularities to provide mechanical bond, and sufficient absorption to draw the wet mortar into these surface irregularities.

Although some tensile bond strength develops immediately after brick and mortar make contact, bond continues to increase as the cement hydrates. Tensile bond is of importance not only from a strength standpoint but also because it helps to resist forces generated by volume and temperature changes.

In the laboratory, tensile strength is measured by determining the tensile force required to separate sandwich brick assemblies. An alternative although less satisfactory method is to place a single leaf brickwork pier on its side and load it as a beam. Using the standard beam bending expression the modulus of rupture can be easily determined.

2.5. Elastic Properties of Mortars

The elastic properties of mortars are important because they effect significantly the elastic properties of brickwork as well as its strength. Generally speaking, in a brickwork assembly, the mortar tends to deform more under an axial load. The vertical compression is accompanied by a lateral expansion which, in unrestrained mortar, tends to be greater than in the brick. When the two are bonded together, they are forced to strain equally, causing the brick to move into a state of tension which, if high enough, could cause tensile cracking and eventually failure.

The stress/strain relationship for mortars is generally a curve showing distinct plastic characteristics. In the absence of direct proportionality between stress and strain it is difficult to give definite values of Young's modulus of mortar and its strength. For a 1:1:6 mortar mix Young's modulus is approximately 2·8 to 4·2 kN/mm² (0·4 to 0·6 $\times$ 10^6 lb/in²) and for a 1:¼:3 mix it is approximately 10·0 to 14·0 kN/mm² (1·5 to 2·0 $\times$ 10^6 lb/in²).

A convenient method of measuring the elastic modulus of mortar is by the electrodynamic test. Here a prism of mortar is clamped at its centre and set into longitudinal oscillations by an excitor placed at one end of the specimen. The frequency of vibration can be varied and the corresponding amplitude of oscillation is read off a meter which is a part of the apparatus. At resonance, the amplitude is at its maximum value and therefore the resonance frequency can be easily established. Using the relationship:

$$E = 4L^2 \rho f^2$$

where E is the dynamic modulus (N/m²),
L is the length of specimen (m),
ρ is the density of material (kg/m³),
f is the frequency at resonance (c/s),

the dynamic modulus can be evaluated.

Great care must be exercised in applying the results of elastic properties of mortar obtained from mortar prisms or cylinders to its behaviour in the brickwork joint. This is due to the overriding importance of the geometry and restraint conditions of the specimen under test. Obviously the elastic behaviour of a thin layer of mortar restrained top and bottom by more rigid bricks bears little relation to that of a corresponding cylinder which is comparatively unrestrained over most of its length.

Changes in dimensions of building materials due to various causes are of importance in relation to the risk of cracking in buildings. In brickwork, the risk of cracking could be brought about either by excessive dimensional changes in the brick or the mortar. However, the latter is considered to be less important partly because mortar is very much the minor constituent in brickwork. In fact some recent studies have shown that dimensional changes of all types in mortars are much reduced below the unrestrained values by small degrees of restraint comparable with the shear and tension restraints existing in brick and blockwork. This would indicate that the component of shrinkage or expansion stresses attributed to the mortar in brickwork can usually be neglected. These findings have not yet been fully accepted and at the moment it is generally recommended that rich mortars should not be used in conjunction with weak bricks in case cracking results.

2.6. Durability of Mortars

The durability of mortars is measured by their ability to endure the exposure conditions to which they are subjected. The mortar may have to withstand damage by frost action, either during construction when it could be particularly vulnerable, or during the service life of the brickwork. Only the weakest mortars such as a 1:3:12 are likely to suffer damage by direct frost action after they have matured. The stronger mortar mixes, i.e. those richer in cement are the more durable against early frost attack. Air-entrainment produces considerable resistance to frost attack so that those mixes which are plasticized with air, either by using a mortar plasticizer or masonry cement, are likely to have adequate durability under normal circumstances. Air-entraining plasticizers can also be used with the weaker mixes to improve their early frost resistance.

Calcium chloride as an admixture for mortar in acceptable proportions serves no useful purpose in protecting it against frost. The amount by which it depresses the freezing point of water in the mortar is negligible. Any effect of accelerating the hardening of the cement at low temperatures is limited by the fact that the temperature of the mortar due to hydration cannot rise appreciably when it is spread out in thin layers in contact with large amounts of other materials at low temperatures. It is recommended, therefore, to stop construction of important work during frost, rather than run the risk that the calcium chloride will be used in proportions grossly in excess of acceptable limits in an attempt to produce a noticeable effect. The presence of excess calcium chloride in mortar may well lead to accelerated deterioration of embedded wall ties or reinforcement and encourage efflorescence. Its use in mortar should be discouraged.

Mortar may also suffer from sulphur compounds in the adjacent units, in the atmosphere or drawn from the ground in solution. Again, the stronger mortars are likely to be more durable, but it may be necessary to use sulphate resisting Portland cement rather than an ordinary cement if deterioration from sulphate attack is envisaged.

Mortar is able to protect metal ties and reinforcement embedded in it because of the alkaline environment it provides. However, the hardening of lime depends on a chemical reaction between the carbon dioxide in the atmosphere and the alkaline material and this carbonation reduces the alkalinity of the mortar. Carbonation progresses inwards from the surface exposed to the atmosphere but its rate of penetration is reduced with increasing impermeability of the mortar. It is important therefore that steel should be protected by fully embedding it in a dense mortar to give adequate cover.

Reference

ISBERNER, A. W. (1967) Properties of Masonry Cement Mortars, *Proceedings of the First International Conferences on Masonry Structural Systems*, University of Texas, Austin.

CHAPTER 3

Properties of Brickwork

3.1. Behaviour of Brickwork in Compression

Brickwork is a composite material with the brick as the building unit and the mortar as the jointing material. It might be thought possible at first glance to deduce the properties of brickwork from the knowledge of its two constituents. However, any such attempt would produce serious discrepancies. An analogy between two elements and their compound may be useful here. As is well known, when two elements combine to form a new compound, the latter acquires its own characteristic properties which may bear little resemblance to either of the constituent elements. To a large extent this may also be said of brickwork, although the metamorphosis is not quite so drastic, and certain derived properties are still noticeable in the final product. For example, a strong brick will usually produce strong brickwork.

Figure 3.1 shows the graph of brick strength against brickwork strength tested at 28 days after casting. The results which came from different sources were obtained by testing 0·229 m (9 in.) brickwork cubes which is one of the methods used for measuring brickwork strength. The graph is in a form of an exponential curve with a limiting brickwork strength of approximately 35 N/mm^2 (5000 lb/in^2). There is very little increase in brickwork strength above a brick strength of 83 N/mm^2 (12,000 lb/in^2).

Figure 3.2 shows a graph of mortar strength and brickwork cube strength both tested at 28 days, with brick strength kept constant. Once again the graph is a curve with the increase in brickwork strength getting progressively smaller as the mortar strength increases. These, and many other tests carried out on brickwork cubes and on full-size brickwork walls, have produced a number of empirical formulae relating brick, mortar and brickwork strength. Most of these indicate that brickwork strength is proportional to the square root of brick strength and the fourth root of mortar strength. Comparison of brickwork wall strength and brickwork cube strength with the same materials shows that a typical value of the ratio wall strength/cube strength is approximately 0·7. This relationship is important when an attempt is made to

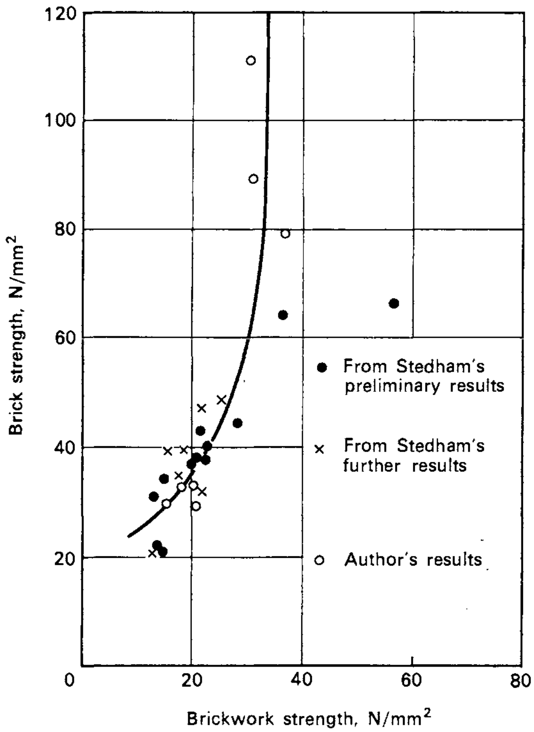


FIG. 3.1. Brick strength against brickwork cube strength.

predict brickwork wall strength from the standard laboratory brickwork cube tests.

There are a number of other useful indices which could be used to predict brickwork strength. One of these is the density of bricks. Figure 3.3 shows a plot of brickwork cube strength against the dry density of bricks. The relationship is almost linear. Another useful index is the dynamic modulus of mortar. In Fig. 3.4 it is seen that there is again a linear relationship between the dynamic modulus of mortar and brickwork cube strength.

Other factors influencing brickwork strength are time of curing, thickness of mortar bed joints, water suction of bricks and workmanship. Tests on brickwork cubes showed that they are less sensitive to

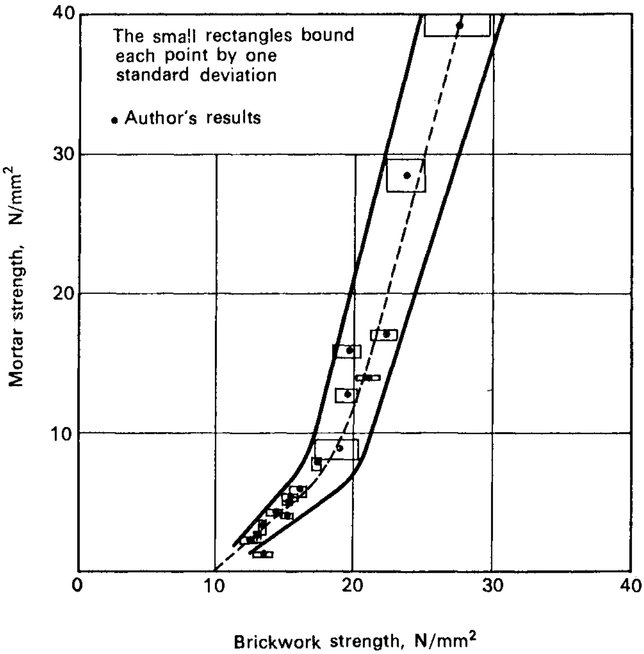


FIG. 3.2. Mortar strength against brickwork cube strength.

curing time than concrete or mortar. This result is not unexpected considering that a high proportion of brickwork are bricks whose strength after firing remains fairly constant. After 7 days brickwork reaches approximately 80% of its ultimate strength and after 14 days, 95%. The standard time for testing brickwork is 28 days when it has almost reached its ultimate strength.

The thickness of mortar bed joints has quite a significant effect on brickwork strength. In one series of tests the strength of brickwork cubes decreased from 23·4 N/mm² (3400 lb/in²) to 15·2 N/mm² (2200 lb/in²) by increasing the thickness of bed joints from 3·17 mm ($\frac{1}{8}$ in.) to 15·9 mm ($\frac{5}{8}$ in.). Current work indicates that substantial increase in brickwork strength can be obtained by using higher bricks in relation to the standard 9·5 mm ($\frac{3}{8}$ in) bed joint.

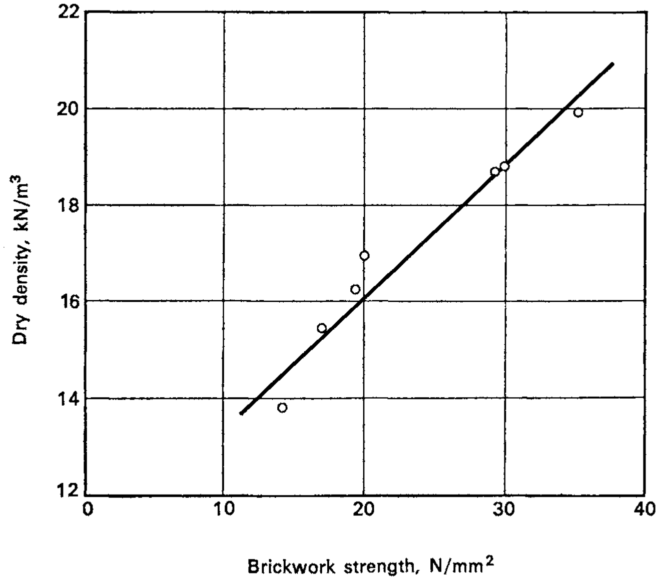


FIG. 3.3. Brickwork cube strength against dry density of bricks.

Excessive water suction in the bricks can lead to considerable reduction in brickwork strength. This is probably due to the bricks absorbing an excessive amount of water from the mortar and thus interfering with complete hydration of the cement. It would appear that an initial rate of absorption not exceeding 20 g/dm²/min should be acceptable. This particular problem is likely to be most critical in laying relatively low strength bricks.

The problem of workmanship and its effect on the strength of brickwork is of paramount importance. With regard to the type of brick used it is well known that there are considerable practical difficulties in ensuring that frogs are completely filled with mortar and tests at the Building Research Station have shown that when single frogged bricks were laid frog down, the resulting brickwork was 20% weaker than

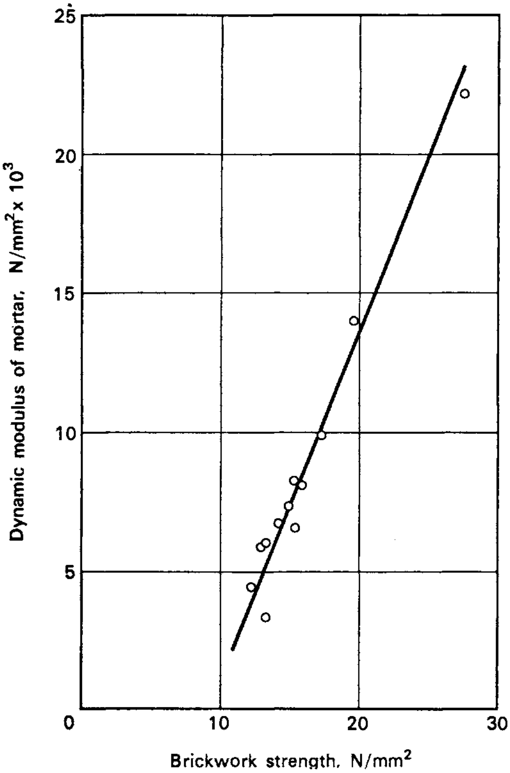


FIG. 3.4. Brickwork cube strength against the dynamic modulus of mortar.

when laid with frogs up. Similarly, recent tests indicate that the strength of two-leaf cavity walls is of the order of 70% of the strength of the two leaves when built separately. This is probably due to the fact that in the cavity wall it is more difficult to ensure the standard of workmanship which is achieved when building a single leaf. Another reason is that the strength of a cavity wall will tend to be limited by the strength of the weaker leaf.

3.2. Mechanism of Failure in Brickwork Under Axial Load

Failure in brickwork under axial compression is normally by vertical splitting due to horizontal tension in the bricks. Figure 3.5 shows a

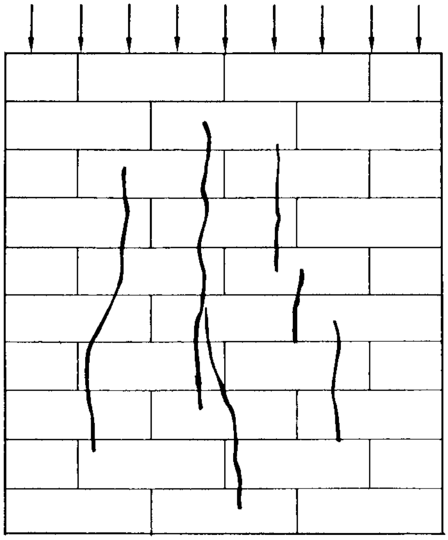


FIG. 3.5. Typical failure pattern in a brickwork wall.

typical failure pattern in a brickwork wall. The reason for this type of failure is due mainly to the widely different strain characteristics of the bricks and mortar joints. Broadly speaking, the mortar is less rigid than the brick and under load its tendency is to spread laterally to a greater extent than the brick. Differential movement is prevented by the bond between the brick and mortar and consequently the mortar is put into a state of biaxial compression and the brick into biaxial tension. Failure in brickwork occurs when the tensile stress in the brick reaches its ultimate tensile strength. It is obvious therefore that the elastic properties of both mortar and bricks influence the ultimate strength of brickwork.

To get a better understanding of the interaction between the brick and mortar, tests were carried out where the bonding material covered a wide range of rigidity, from soft rubber to steel. The lowest strength in brickwork was obtained with the soft rubber jointing and the highest with steel. The differences in strength were quite marked and verified the importance of the elastic properties of the constituents. An interesting fact to emerge from the investigation was that in no case did the strength of brickwork exceed substantially the strength of individual bricks.

3.3. Simple Theory of Failure for Brickwork

In preceding sections we discussed the various factors affecting the strength of brickwork. Amongst these the elastic properties of bricks and mortar were shown to be critical. The following theory put forward by the author is an attempt to relate analytically the compressive strength of brickwork, the elastic properties of bricks and mortar and the ratio of height of brick to thickness of bed joint. The fundamental criterion is that brickwork fails when the tensile strain in the brick reaches its ultimate value. For the sake of simplicity the approach and some of the basic assumptions in the theory have been grossly over-simplified. This is particularly so with the elastic constants which, in brickwork, are difficult to establish and which tend to vary with a number of factors. For this reason the theory should be regarded as qualitative rather than quantitative in its function.

NOTATION

σ_y = applied vertical stress
σ_b = horizontal tensile stress in brick
σ_m = horizontal compressive stress in mortar
σ_0 = unrestrained compressive strength of brick
σ_u = ultimate compressive strength of brickwork
E_b = Young's modulus for bricks
E_m = Young's modulus for mortar
ν_b = Poisson's ratio for bricks
ν_m = Poisson's ratio for mortar

ϵ_{tb} = total lateral strain in brick
ϵ_u = ultimate tensile strain in brick
d = height of bricks
t = thickness of bed joint

THEORY

Consider a brickwork assembly subjected to a vertical compressive stress, σ_y. Next consider a single brick of the assembly bounded top and bottom by mortar joints of thickness t. If there were no bond between the brick and mortar, the free vertical strain in the brick would be σ_y/E_b and in the mortar σ_y/E_m. The corresponding free lateral strains in the brick and mortar would be $(\sigma_y/E_b) . \nu_b$ and $(\sigma_y/E_m) . \nu_m$ where E_b, E_m, ν_b and ν_m are the Young's moduli and Poisson's ratios for brick and mortar respectively. Due to bonding between the brick and mortar, a composite action between the two is set up with both of them straining together as one unit. Since the mortar is generally less rigid than the brick, it would tend to expand laterally more under load than the brick and in composite action it is thrown into a state of biaxial compression σ_m whilst, at the same time, the brick is thrown into biaxial tension σ_b. We have therefore:

Lateral strain in brick due to biaxial tension

$$= \frac{\sigma_b}{E_b} - \nu_b \frac{\sigma_b}{E_b}$$

$$= \frac{\sigma_b}{E_b} (1 - \nu_b). \tag{3.1}$$

Similarly, lateral strain in mortar due to biaxial compression

$$= \frac{\nu_m}{E_m} (1 - \nu_m). \tag{3.2}$$

Figure 3.6 shows diagrammatically the free lateral expansion of brick and mortar due to externally applied stress σ_y and the resultant expansion of the composite. From the geometry we see that:

Lateral compression in mortar + lateral extension of brick = difference in free lateral movements. Since the length dimensions of the brick and mortar joint are equal, we can convert the movements into strains by dividing each term above by the common length and we get:

Lateral compressive strain in mortar + lateral tensile strain in brick = difference in free lateral strains,

or
$$\frac{\sigma_m}{E_m}(1-\nu_m)+\frac{\sigma_b}{E_b}(1-\nu_b)=\sigma_y\left(\frac{\nu_m}{E_m}-\frac{\nu_b}{E_b}\right). \tag{3.3}$$

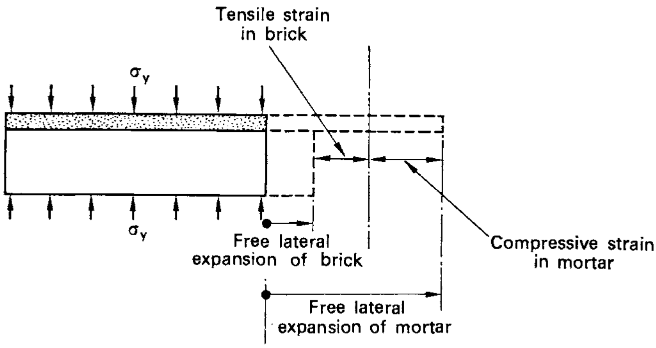


FIG. 3.6. Lateral expansion of brick and mortar under vertical stress σ_y.

Also from horizontal equilibrium, we get:

$$\sigma_b d=\sigma_m t \quad \text{or} \quad \sigma_m=\sigma_b\frac{d}{t}. \tag{3.4}$$

Substituting for σ_m in equation (3.3), we get:

$$\frac{\sigma_b}{E_b}\left[(1-\nu_b)+\frac{E_b}{E_m}\frac{d}{t}(1-\nu_m)\right]=\sigma_y\left(\frac{\nu_m}{E_m}-\frac{\nu_b}{E_b}\right). \tag{3.5}$$

Total lateral strain in brick:

$$\epsilon_{tb}=\frac{\sigma_y\,\nu_b}{E_b}+\frac{\sigma_b}{E_b}(1-\nu_b). \tag{3.6}$$

From equation (3.5):

$$\frac{\sigma_b}{E_b} = \frac{\sigma_y \left(\dfrac{\nu_m}{E_m} - \dfrac{\nu_b}{E_b}\right)}{\left[(1 - \nu_b) + \dfrac{E_b}{E_m}\dfrac{d}{t}(1 - \nu_m)\right]}$$

and therefore:

$$\epsilon_{tb} = \sigma_y \frac{\nu_b}{E_b} + \frac{\sigma_y \left(\dfrac{\nu_m}{E_m} - \dfrac{\nu_b}{E_b}\right)(1 - \nu_b)}{\left[(1 - \nu_b) + \dfrac{E_b}{E_m}\dfrac{d}{t}(1 - \nu_m)\right]} = \epsilon_u \text{ at failure} \tag{3.7}$$

where ϵ_u is the ultimate tensile strain in the brick. If there is no bonding between the brick and mortar, the last term in equation (3.6) disappears and:

$$\epsilon_u = \sigma_0 \cdot \frac{\nu_b}{E_b} \tag{3.8}$$

where σ_0 is the unrestrained compressive strength of the brick, and from equation (3.7), at failure:

$$\sigma_u \cdot \frac{\nu_b}{E_b} + \sigma_u A = \sigma_0 \cdot \frac{\nu_b}{E_b}$$

or

$$\sigma_u = \sigma_0 \left(\frac{1}{1 + A\dfrac{E_b}{\nu_b}}\right) \tag{3.9}$$

where:

$$A = \frac{\left(\dfrac{\nu_m}{E_m} - \dfrac{\nu_b}{E_b}\right)(1 - \nu_b)}{\left[(1 - \nu_b) + \dfrac{E_b}{E_m}\dfrac{d}{t}(1 - \nu_m)\right]}$$

or

$$\sigma_u = \sigma_0 \left(\frac{1}{1 + B}\right) \tag{3.10}$$

where:

$$B = A \cdot \frac{E_b}{\nu_b}.$$

EVALUATION OF CONSTANT B

For known values of E_b, E_m, ν_m, ν_b, d and t constant B can be determined. For most types of brickwork, the ratio d/t can be taken as 7. Also, the value of ν_m for mortar is found to be relatively constant at 0·15 whereas ν_b for bricks is of the same order. Assuming that $\nu_m = \nu_b$ and $d/t = 7$:

$$B = \frac{E_b - E_m}{7E_b + E_m}.$$

The unrestrained compressive strength of bricks σ_0 can be best determined by testing the bricks on their sides with their faces ground and placed between similar bricks with their contact faces also ground.

Equation (3.10) can be used to study the effect of strength of mortar on brickwork strength. For a very weak mortar (say $E_m \to 0$) constant $B \to 1/7 = 0{\cdot}143$ and

$$\sigma_u = \sigma_0 \frac{1}{1{\cdot}143} = 0{\cdot}88\sigma_0,$$

whereas for a strong mortar (say $E_m = E_b$), $B = 0$ and $\sigma_p = \sigma_0$.

Equation (3.10) shows, therefore, that brickwork strength is very approximately equal to the *unrestrained* compressive strength of the brick and that the strength of mortar is relatively unimportant in determining brickwork strength. This is borne out by experimental results.

Note: The unrestrained compressive strength of bricks is very approximately equal to one half of the crushing strength of bricks when tested flat between 3 mm plywood sheets as laid down by B.S. 3921 (1965).

3.4. Strength of Brickwork in Shear

Shear strength of brickwork is of great importance when designing for lateral loads on brickwork walls. When a horizontal load is applied in the plane of a brickwork wall and parallel to the bed joints, failure may occur either by horizontal shear at the brick/mortar interface or by diagonal tension. The resistance of brickwork to horizontal shear increases as the normal load between the brick and mortar increases.

The shear bond is normally independent of mortar strength. Perforated bricks will provide a mechanical shear key with the mortar and hence increase the racking resistance, at least at low precompression. At higher precompression failure may be in diagonal tension where the keying effect may be less important. Experimental evidence is, however, somewhat lacking.

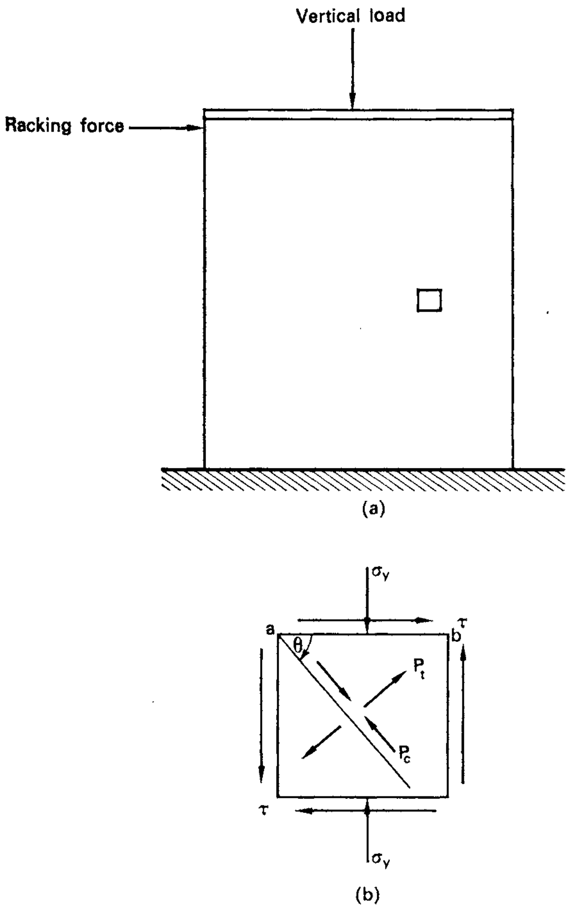


FIG. 3.7. (a) Vertically loaded wall subject to a racking force.
(b) Stresses in element of wall.

Consider a wall loaded vertically and subjected to a racking force *P* as shown in Fig. 3.7a. Due to the external forces an element of wall such as shown in Fig. 3.7a and enlarged in Fig. 3.7b has a vertical stress σ_y and a shear stress τ, acting on it. Under the combined action of σ_y and τ the wall can fail by:

(i) horizontal shear bond failure in the plane of the bed joints,
(ii) diagonal tension failure.

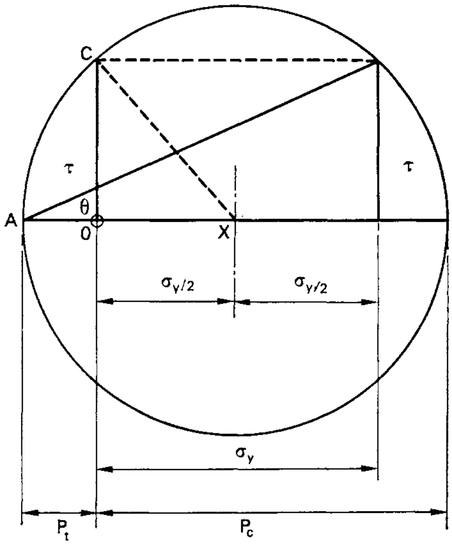


FIG. 3.8. Mohr circle for stresses shown in Fig. 3.7b.

In (i) resistance to shear bond failure is given by:

$$\tau_b = V_{b0} + \mu\sigma_y \tag{3.11}$$

where: τ_b = shear stress at shear bond failure
V_{b0} = shear bond strength at zero precompression due to the adhesive strength of mortar
μ = coefficient of friction between brick and mortar.

In (ii) diagonal tension failure is caused by tensile stresses set up in the element as a result of the combined action of the direct and shear stresses σ_y and τ. The magnitude and direction of the tensile stress can be deduced from a Mohr circle. Figure 3.8 shows a Mohr circle for the stresses acting on the element in Fig. 3.7b. The stresses σ_y and τ produce a major principal stress p_c (compression) and a minor principal stress p_t (tension). Each plane in the element is represented by a point on the Mohr circle. The angle θ in the Mohr circle represents the inclination of the plane of the principal tensile stress p_t (diagonal tension stress) to the plane *ab* on which the stress σ_y is acting. We see from the Mohr circle that:

$$AO = AX - OX$$
$$= CX - OX$$
$$= \sqrt{(OX)^2 + (OC)^2} - OX$$

or

$$p_t = \sqrt{\left(\frac{\sigma_y}{2}\right)^2 + \tau^2} - \frac{\sigma_y}{2}. \qquad (3.12)$$

If τ_t is the shear stress causing diagonal tension failure and σ_t represents the tensile strength of brickwork normal to plane *ab* in Fig. 3.7b then from equation (3.12):

$$\left(\frac{\sigma_y}{2}\right)^2 + \tau_t^{\,2} = \left(\frac{\sigma_y}{2}\right)^2 + \sigma_t\,\sigma_y + \sigma_t^{\,2}$$

and:

$$\tau_t = \sqrt{\sigma_t^{\,2} + \sigma_t\,\sigma_y}. \qquad (3.13)$$

For $\sigma_y = 0$, $\tau_t = \sigma_t$ and $\theta = 45°$. For other values of σ_y angle θ varies and with it the tensile strength of brickwork. The most convenient way of determining σ_t is by a splitting test on a cylindrical disc of brickwork. The disc can be 0·114 m ($4\frac{1}{2}$ in.) thick and 0·381 m (15 in.) in diameter. The orientation of the bed joints can be varied and σ_t plotted against θ. Typical results are shown in Fig. 3.9.

It can be shown that if P is the applied load causing tensile split in the disc then:

$$\sigma_t = \frac{2P}{\pi D t}$$

where: D = diameter of disc,
t = thickness of disc.

Figure 3.10 shows the variation of τ_b and τ_t with σ_y for known values of V_{b0}, σ_t and μ. For $\sigma_y < \sigma_{y1}$ and $V_{b0} < \sigma_t$ the expected failure will be by shear bond and above σ_{y1} by diagonal tension failure. These predictions are to a large extent confirmed by laboratory tests.

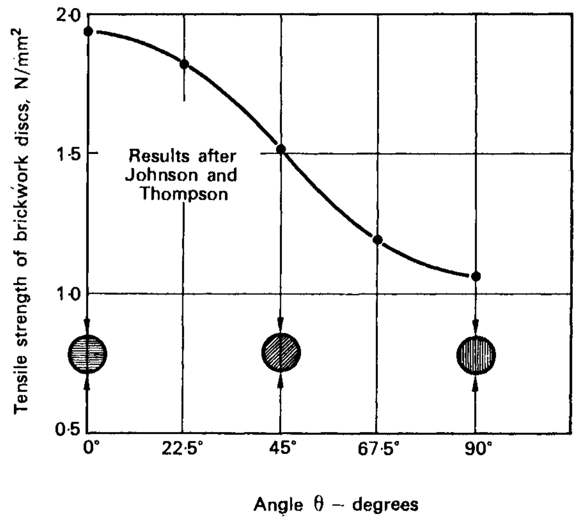


FIG. 3.9. Effect of direction of principal tensile stress bed joint angle on tensile strength of brickwork discs.

3.5. Strength of Brickwork in Tension

Tensile strength of brickwork is governed by the tensile bond at the brick/mortar interface. Among the factors influencing tensile bond strength are absorption or suction rate of the bricks, the initial water content, retentivity of the mortar, type of mortar, type of brick, thickness of mortar joints and workmanship.

The tensile bond strength is markedly reduced by bricks with high suction rate or by weak mortars. The suction rate is easily measured by immersing an oven dried brick in water to a depth of 12·7 mm ($\frac{1}{2}$ in.) and noting the gain in weight due to water absorption over a period of 30 sec.

The tensile strength of brickwork may be increased by adding a suitable plastic additive in powder form to the mortar during mixing, although the cost of it may in some cases make it uneconomical. One-half of a percent of the weight of cement in the mortar is usually found to be quite adequate. In addition to improving the adhesive properties at the brick/mortar interface the plastic additive seals the brick surface and thus reduces its suction. As a rough guide tensile strength of brickwork can be taken as one-tenth of its compressive strength.

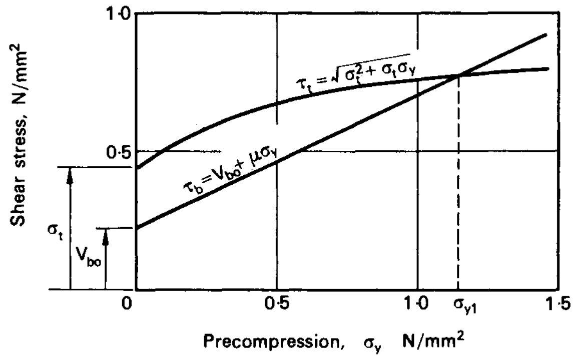


FIG. 3.10. Effect of precompression on shear strength of brickwork.

3.6. Concentrated Loads on Brickwork

A study of the problem of concentrated loads on brickwork has recently been undertaken at the Structural Ceramics Unit, Edinburgh University. Full-scale brickwork piers as well as one-third and one-sixth scale brickwork walls were used in the investigation. An analysis of the vertical and horizontal strain distributions indicated that a concentration of stress exists in the vicinity of concentrated loads, forming bulbs of pressure with peak values immediately below the load point. For centrally loaded piers, the average failure stress was considerably higher than for end-loaded piers. The presence of horizontal reinforcement in the piers seemed to make little difference to the failure stress.

In the piers and model walls it was found that the failure stress increased with decreasing bearing plate length. For bearing plates longer than 115 mm (4½ in.) the size effect became negligible. The failure stress also increased with increasing distance from the end of the wall. Above a certain distance from the end a further increase seemed to make little difference.

Analysis of the test results showed that the lowest load factor, based on the failure stress and the allowable stress as given by the Code of Practice CP 111: 1970 was 2·5. The average values for the full scale and model brickwork fell at approximately 4·0. Failure of the brickwork under a concentrated load may take place by vertical splitting at some distance below the loaded area, by horizontal "tearing" at the surface or by spalling of the brickwork under the load.

3.7. Movements in Brickwork

MOVEMENT DUE TO LOAD

Movement in brickwork is caused by loads, temperature and absorption or loss of moisture. The movement due to load is made up of instantaneous or elastic deformation and delayed movement, also known as creep. The two parameters which together define the elastic properties of a material are the Young's or elastic modulus, E and the Poisson's ratio, ν. The latter is defined as the ratio lateral strain/strain in the direction of the load. The knowledge of elastic properties for brickwork is rather scanty and the problem is complicated by the fact that the stress/strain relationship is not linear and therefore E varies with applied stress. What information is available suggests that for a given brick, the value of E increases with the strength of mortar. In one series of tests using a brick strength of 32·6 N/mm² (4730 lb/in²) the E value for brickwork increased from 1030 N/mm² ($0{\cdot}15 \times 10^6$ lb/in²) using a 1:2:9 mortar mix through 5720 N/mm² ($0{\cdot}83 \times 10^6$ lb/in²) for a 1:1:6 mix, to 15,350 N/mm² ($2{\cdot}23 \times 10^6$ lb/in²) for a 1:¼:3 mix. When the 1:¼:3 mix was used with a much stronger brick (90·2 N/mm² or 13,100 lb/in²) the E value remained almost unchanged. Generally it may be safely assumed that the elastic modulus increases with the strength of brickwork.

Under a sustained load brickwork in common with other visco-elastic materials, is subject to creep. Creep is of importance when assessing the long term movements and in determination of stress distribution in composite materials with different creep properties.

When a load is applied to a brickwork specimen it undergoes instantaneous deformation. This is followed by progressive creep whose rate decreases with time and ceases entirely after a time which varies according to the type of brick and mortar used. In some tests carried out by the author, for a high strength brick (98·5 N/mm^2 or 14,350 lb/in^2) and a 1:1:6 mortar creep ceased after approximately four months and with a 1:$\frac{1}{4}$:3 mortar it ceased after 40 days. Typical results are shown in Fig. 3.11. We see that creep increases at increasing rate with applied stress. The strength of mortar is an important factor in determining the amount of creep in brickwork. For brickwork with the 1:1:6 mortar the maximum creep is 2–3 times as great as for the same brick and a 1:$\frac{1}{4}$:3 mortar. The total strain including creep is 1·2–1·4 of the instantaneous strain for brickwork with the 1:$\frac{1}{4}$:3 mortar and 1·5–1·8 for brickwork with the 1:1:6 mortar.

EFFECTS OF MOISTURE AND TEMPERATURE

The moisture movements in brickwork are caused by the composite effects of moisture movements in bricks and mortar. These movements have already been discussed in some detail in Chapters 1 and 2. The net effects of these movements is to cause moisture expansion in brickwork. The moisture movement in brickwork is highly sensitive to changes in external temperature and relative humidity. Under the fluctuating conditions met in the British climate it is extremely difficult to detect a set trend for these movements except that the expansion rate increases quite noticeably with increase in relative humidity. The movement is of course closely interlinked with thermal expansion or contraction and under normal operating conditions it is virtually impossible to separate the two. The most that can be said is that the overall effect is a gradual expansion of the brickwork and that its magnitude is very significant in the assessment of long term movements, often exceeding the creep strain due to load. Consequently it is quite possible to get an overall expansion under a sustained compressive load with the ultimate total strain being less than the instantaneous compressive strain.

All building materials expand with the rise in temperature and this

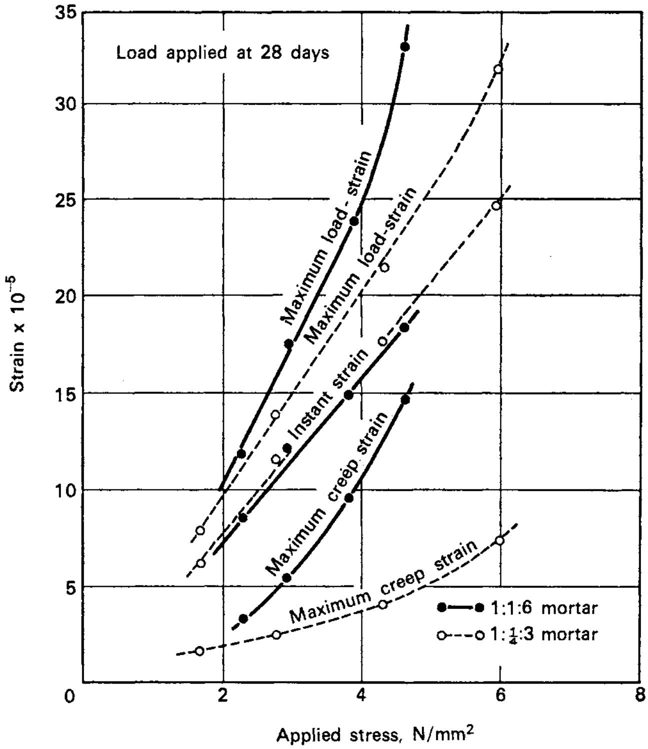


FIG. 3.11. Graphs of maximum load, creep and instantaneous strains against applied stress for brickwork piers with Butterley class 'B' engineering bricks and 1:1:6 and 1:¼:3 mortars.

includes brickwork. For calculation of thermal movements in a brickwork wall the average wall temperature should be used. For solid walls this will be the temperature in the centre of the wall. In cavity walls there may be differential thermal movement between the inner and outer leaves and in such situations provision should be made for maximum thermal movement by considering the average temperature of the outer leaf. Special care should be taken with brickwork walls with a southerly aspect when the surface temperature during summer could easily reach 44°C. Such temperature can give rise to a steep thermal gradient through the wall and in cavity construction differential thermal movement which may cause excessive bending in addition to longitudinal expansion.

The coefficient of thermal expansion for brickwork is $5{\cdot}6 \times 10^{-6}$ per °C in a horizontal direction and vertically may be up to 1·5 times this value. Vertical thermal movements in walls are generally reversible but horizontal movements may only be reversible if the wall does not crack as a result of the expansion or contraction. This depends upon whether the wall is built on a soft damp proof course as the degree of restraint imposed by it appears to be the critical factor.

Because of the various movements which take place in brickwork it is essential to include movement joints when the span of the wall exceeds 15 m (50 ft). They should also be placed at short returns in walls which are susceptible to cracks. The jointing material should be one of high ductility and low stiffness. Suitable materials include premoulded, extruded, closed cell rubber or plastics (polyurethane or polyethylene.)

The sealing of movement joints has always presented a problem. The introduction of polysulphide based sealants has improved their performance considerably. It is impossible to lay down rigid rules for the spacing of movement joints and each building should be considered on its own merits. As a rough guide, for long walls, movement joints should be placed at approximately 12 m (40 ft) centres.

3.8. Durability of Brickwork

The durability of a material can be defined as its ability to function safely and efficiently during its intended working life. In brickwork the durability is impaired more than by any other factor by excessive moisture in its fabric. This can lead in turn to excessive moisture movements, causing cracking, efflorescence, frost damage by spalling, growth of unsightly fungus, lowering of thermal insulation and general discomfort to the occupants of the building. We see therefore that durability of brickwork is closely related to the exclusion of moisture. This can be partly achieved by the proper selection of suitable bricks and mortar but a lot depends also on the workmanship during construction, correct design and attention to detail. Points of special importance are the proper filling of joints with mortar, the correct placing of damp-proof course so as to exclude rising moisture by capillary action in the brickwork and absence of dirt and lumps of mortar in cavity walls which could form "bridges" through which moisture from outside can permeate to the mortar.

So far as design is concerned one should avoid unnecessary projections or breaks in the external facade. The design should ensure the exclusion of water from the tops of walls, particularly parapet walls. Working drawings should clearly indicate constructional details and nothing should be left to the discretion of the site foreman or bricklayer. With proper precautions brickwork, probably more than any other building material, can function satisfactorily with the minimum of maintenance and cost. Its inherent advantages, in particular high resistance to fire and corrosive chemicals as well as its attractive external appearance make it one of the most popular materials used in modern buildings.

References

BRADSHAW, R. E. and THOMAS, K. (1968) Modern Developments in Structural Brickwork, *C.P.T.B. Technical Note*, Vol. 2, No. 3.

HENDRY, A. W. (1967) High Rise Loadbearing Brickwork, *The Architect's Journal*, 1967.

HENDRY, A. W. and SINHA, B. P. (1967) Racking Tests on Storey-height Shear Wall Structures, *Proceedings of the First International Conference on Masonry Structural Systems*, University of Texas, Austin.

HENDRY, A. W., BRADSHAW, R. E. and RUTHERFORD, D. J. (1968) Tests on Cavity Walls and the Effect of Concentrated Roads and Joint Thickness on the Strength of Brickwork, *C.P.T.B. Research Note*, Vol. 1, No. 2.

JOHNSON, F. B. and THOMPSON, J. N. (1967) Development of Diametral Testing, *Proceedings of the First International Conference on Masonry Structural Systems*, University of Texas, Austin.

LENCZNER, D. (1966) Strength and Elastic Properties of the 9 in Brickwork Cube, *Transactions of the British Ceramic Society*, Vol. 65, No. 6.

LENCZNER, D. (1970) Creep in Brickwork, Second International Conference on Brick Masonry, Keele University, Stoke-on-Trent, England.

THOMAS, K. (1966) Movement Joints in Brickwork, *C.P.T.B. Technical Note*, Vol. 1, No. 10.

CHAPTER 4

Design of Brickwork Members

4.1. Historical Background

Until the latter part of the nineteenth century loadbearing walls were commonly used to support the floors in multistorey buildings. This type of construction limited somewhat the style and form of the building, due to the restrictions imposed by the walls, which had to be continuous from the foundations upwards. The situation was aggravated by the fact that in Britain, prior to the issue of the Code of Practice for Loadbearing Walls, multistorey brick design was largely based on rule of thumb, quite often producing structures of enormous proportions which, due to their bulk, were uncompetitive with framed structures in steel and reinforced concrete, for which proper scientific design methods were already available. However, the requirements of residential buildings still favoured the rigid plan, enabling a repetition of the structural supports on each floor for which loadbearing brickwork construction was ideally suited. Coupled with this was the introduction of the British Standard Code of Practice CP 111 (1948) "Structural Recommendations for Loadbearing Walls". The Code was based on the results of tests carried out at the Building Research Station between the years 1926 and 1934. Most of the tests related to piers, which subsequently have been proved to have a lower resistance to failure than walls being more susceptible to local buckling and splitting due to variable workmanship. In the Code rather high load factors were incorporated (load factor is the ratio of collapse load to working load) which were based on the pier tests. In spite of this, brickwork became once again an economically viable and competitive structural material.

Subsequent to the publication of the 1948 Code a considerable amount of testing has been carried out at various research centres and as a result of better knowledge of the behaviour of brickwork, and in particular walls as opposed to piers, a new Code of Practice CP 111 (1964) was published, making calculated brickwork an even more economic material. A metric version of the 1964 Code has recently been issued, called CP 111: Part 2: 1970, and an Amendment Slip added to it to bring some of the information up to date.

Further tests on interconnected walls and floor slabs as well as other aspects of brickwork technology are continuing, with the result that the Code is due for yet another revision, probably in a few years time. It is probable that this document will be in a limit state terminology.

4.2. Materials and Workmanship

The quality of the final structure depends very largely on the type of materials and workmanship used. This is perhaps even more true for brickwork structures where it is generally difficult to apply a high degree of quality control since most, if not all, building processes take place on the site. It is, therefore, essential to take certain safeguards to ensure satisfactory results. These may be summarized as follows:

The water suction of bricks should not exceed 20 $g/dm^2/min$. Where necessary, the suction rate on site should be reduced by wetting the bricks prior to laying. Where perforated bricks are used the perforations should not be filled with mortar, as this reduces the heat insulation of the wall. All joints should be properly filled with mortar. Bricks with frogs should be laid with frogs uppermost on a full bed of mortar. Double frogged bricks should be laid with the deepest frog uppermost.

Cutting of units should be kept to a minimum, where units are cut this should be done squarely and properly. Where possible an abrasive wheel or a saw should be used. Sleeves, chases and holes should, as far as possible, be provided during the erection of the walls. Chasing of completed walls or the formation of holes should only be carried out with the written approval of the designer and then only with a tool designed to cut the units cleanly. No horizontal or diagonal chases should be permitted unless they have been allowed for in the calculations. Designers should pay particular attention to the choice of suitable materials for damp-proof courses. Materials which squeeze out or allow sliding to occur are undesirable, particularly for highly stressed walls. Wall ties should be of galvanized mild steel. Plastic wall ties should not be used.

For the purpose of bricklaying it is recommended to use certain types of mortar in conjunction with the bricks. Generally, it is desirable that the mortar should be substantially weaker than the brick so that any movements in the structure should be accommodated within the mortar joints without cracking the bricks. The use of high-alumina cement in brickwork mortars is not recommended.

4.3. Types and Layout of Building in Loadbearing Brickwork

Loadbearing brickwork construction is most appropriately used for buildings in which the floor area is divided into a relatively large number of rooms of small to medium size in which the floor plan is repeated on each storey throughout the height of the building. The architect and engineer should work closely together in the early stages of design so as to achieve a suitable layout. Stability of the walls is derived primarily from gravity and careful planning of the layout with full utilization of lift shafts and staircases is necessary to provide the required stability in all directions.

The great variety of wall arrangements in a brickwork building makes it rather difficult to define distinct types of structure but a rough classification is as follows:

(a) Cellular wall systems.
(b) Single or double crosswall systems.
(c) Complex systems.

(a) A cellular arrangement is one in which both internal and external walls are loadbearing and these form a cellular pattern in plan. Figure 4.1a shows an example of a cellular structure.

(b) In the crosswall construction shown in Fig. 4.1b partition walls running parallel throughout the length of the building take all the vertical loads and the wind force acting in the plane of the walls. Stability of the building at right angles is normally provided by corridor partition walls and lift shafts. It will be observed that there is a limit to the depth of the building which can be constructed on the crosswall principle if the rooms are to have effective daylighting.

(c) All kinds of combinations between the cellular and crosswall arrangements are possible and these come under the "complex" system (see Fig. 4.1c).

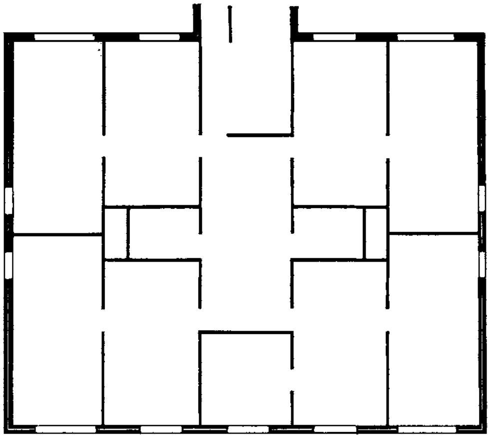


FIG. 4.1a. Example of cellular structure.

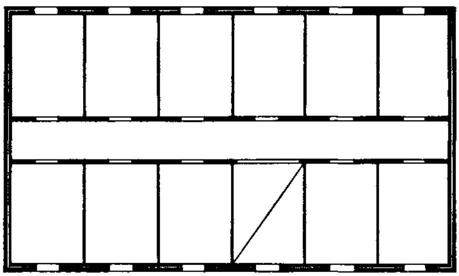


FIG. 4.1b. Example of crosswall construction.

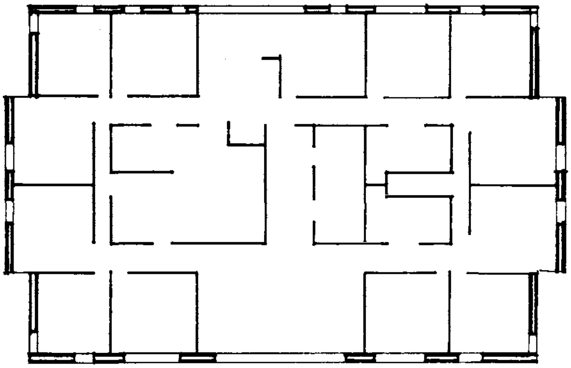


FIG. 4.1c. Example of complex construction.

4.4. Modes of Failure in a Brickwork Building

Figures 4.2a–d show some typical modes of failure in a brickwork building.

Figure 4.2a shows failure due to excessive vertical compression. In Fig. 4.2b failure is caused by excessive compression resulting from the combination of gravity and wind forces. There is also a possibility of tensile failure caused by the wind. In Fig. 4.2c we see the different levels of the building sliding along bed joints at floor levels as a result of wind loadings. This type of failure is most unlikely except perhaps at first floor level. Finally, in Fig. 4.2d we see a shear or diagonal tension failure caused by the combination of vertical and wind loads. Resistance to the last type of failure is dependent on the racking strength of the brickwork walls.

4.5. Loadings on a Building

The loading on a building can be subdivided into three broad categories: dead loads, live loads and wind loads. These are discussed below:

1. DEAD LOADS

The dead load in a building comprises the actual weight of all partitions, floors, roofs and all other permanent fixtures. They are computed from the known densities of materials used and from the assumed

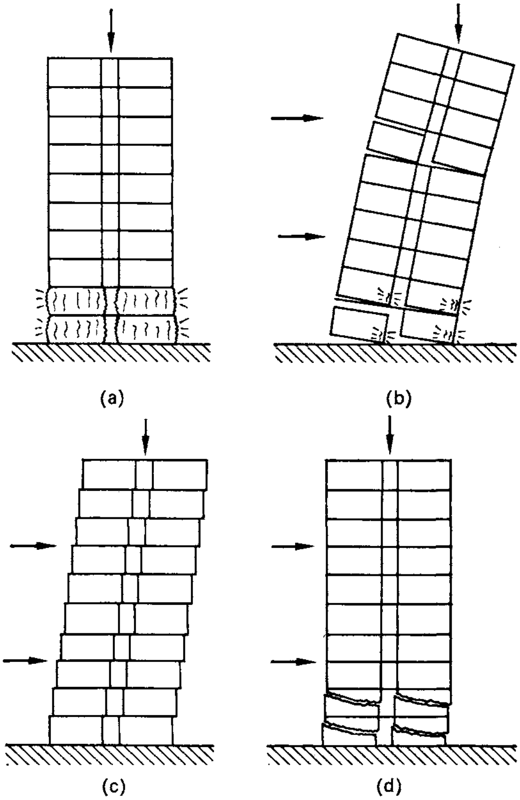


FIG. 4.2. Possible modes of failure of brickwork structures: (a) compression in lower storeys due to dead load, (b) tension developing due to dead load and wind loading, (c) sliding of walls at floor levels due to wind loading, (d) shear or diagonal tension failure due to dead and wind loading.

dimensions of the structural components. They may subsequently need to be modified if the calculated stresses prove to be excessive. In such cases the dead loads will have to be recalculated. It is customary to specify the dead load as a uniformly distributed load in kN/m^2 or lb/ft^2.

2. LIVE LOADS

Live loads are defined as all imposed vertical loads other than dead loads. Details of recommended live loads are given in Code of Practice CP 3: Chapter V: Part 1: 1970, "Loading". According to the intended use of the building, different live loads are recommended and expressed as a uniformly distributed load in kN/m^2 or lb/ft^2. For residential purposes the recommended value is 1·90 kN/m^2 (40 lb/ft^2) of floor area and for office floors 2·40 kN/m^2 (50 lb/ft^2) of floor area. The live loads must be added to the dead loads to get the total vertical loading on the building.

3. WIND LOADS

The wind load on a building depends on the wind speed. Details of calculation of wind pressure are given in CP 3 Chapter V: Part 2: 1970, "Loading" and the following is only a simplified outline of the procedure. Depending on the exposure condition and height of building, a certain design wind speed is selected and is converted to dynamic pressure q using the relationship

$$q = KV_s^2$$

where V_s is the design wind speed,
K is a constant.

The dynamic pressure q is then multiplied by an appropriate pressure coefficient C_p to give the pressure p exerted at any point on the surface of a building. If C_p is negative, this indicates that p is a suction as distinct from a positive pressure.

The resultant wind load F on an element depends on the difference of pressure between opposing forces and is given by

$$F = (C_{pe} - C_{pi})\, qA$$

where C_{pe} and C_{pi} are the pressure coefficient for the external and internal forces and A is the area of the surface.

The stresses due to wind loads must be added algebraically to the values due to vertical loads and their sum must not exceed the permissible values as laid down in CP 111.

4.6. Design of Brickwork Members

The design of loadbearing brickwork members in Britain is carried out in accordance with the Code of Practice CP 111 "Structural Recommendations for Loadbearing Walls". The Code deals with the design of walls, piers and columns with axial and eccentric loadings. The design of laterally loaded panels is not covered by the existing Code although satisfactory methods of design are being developed and will shortly be available for design use. It is hoped that the design of such panels will be covered in the revised Code. What follows is based on the latest edition of the Code at the time of writing.

In the design of walls and columns, CP 111 (1970), Part 2, (metric) makes use of a number of concepts and definitions which are discussed below. For the meaning of some of the terms the reader is referred to the short Glossary on page xi.

1. TYPES OF LOADBEARING MEMBERS

(a) *Solid loadbearing wall.* A wall built of solid or perforated brick designed to carry an imposed load other than its own weight.

(b) *Cavity wall.* Two structural leaves of wall, connected together with metal ties, separated by a continuous cavity.

(c) *Pier.* A thickening integral with a wall, used to provide lateral support or to take concentrated loads.

(d) *Column.* An isolated wall having a length not more than four times its thickness.

2. LATERAL SUPPORT

The restriction at the top of a wall, which prevents horizontal movement. Certain detailed requirements are given in CP 111 (1970), especially in connection with ties to timber floors (Clause 304).

3. SLENDERNESS RATIO

The tendency to buckle under axial or eccentric loading is measured by the slenderness ratio which is defined as:

$$\text{Slenderness ratio (SR)} = \frac{\text{Effective height } (h)}{\text{Effective thickness } (t)}$$

where h and t are defined in items 4 and 5.

For a wall, buckling is only likely to occur in the direction normal to its plane and, therefore, the effective height is determined by the degree of restraint in the direction of its thickness. For a column, buckling is possible in both directions and two values of slenderness ratio must be considered, based on the degree of restraint in each respective direction, and the corresponding effective thickness in the same direction. The larger of the two values of slenderness ratio, indicating a greater tendency to buckle, is used in designing the column.

4. EFFECTIVE HEIGHT

For the purpose of calculating the slenderness ratio, the effective height of a wall or column is obtained from Table 4.1 as follows:

For a wall supporting a floor (or roof) the effective height (h) is equal to three-quarters of the height (H) between the centres of support.

Where the wall has no lateral support at the top $h = 1{\cdot}5H$.

For a wall carrying isolated beams spanning at right angles to it, $h = \frac{3}{4}H$.

For a column, the effective height in the direction of lateral support is given by $h = H$. In the direction without such support, $h = 2H$.

For the illustrations of the above conditions refer to Table 4.1.

5. EFFECTIVE LENGTH

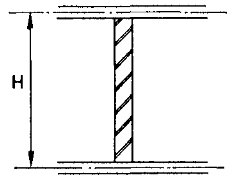
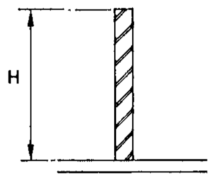
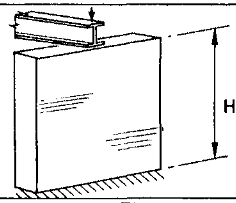
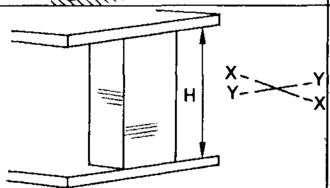
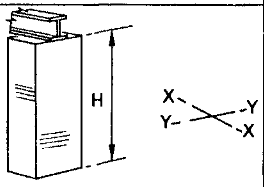
Where a wall is bonded into piers or intersecting or return walls at both ends, the effective length (l) may be taken as the length measured between those piers, or intersecting or return walls. Where the effective length is less than the effective height of the wall it may be used in the calculation of the slenderness ratio of the wall under consideration.

6. EFFECTIVE THICKNESS

For solid walls the effective thickness (t) is equal to the actual thickness. For cavity walls, the effective thickness is taken as two-thirds the sum of the actual thickness of the two leaves.

For walls stiffened by piers at intervals, and provided that the slenderness ratio is based upon the effective height, the effective thickness

TABLE 4.1 EFFECTIVE HEIGHT OF WALLS AND COLUMNS

Wall with concrete floors top and bottom		$h=\frac{3}{4}H$
Wall without lateral support at the top		h=1·5H
Wall carrying isolated beams spanning at right angles		$h=\frac{3}{4}H$
Column with concrete floors top and bottom		For buckling about XX and YY h=H
Column supporting a beam spanning in direction XX		For buckling about XX h=2H For buckling about YY h=H

defined above may be multiplied by the appropriate stiffening coefficient given in Table 4.2.

TABLE 4.2 STIFFENING COEFFICIENT FOR WALLS STIFFENED BY PIERS

Pier spacing (centre to centre) / Pier width	Pier thickness / Effective wall thickness		
	1	2	3
6	1·0	1·4	2·0
10	1·0	1·2	1·4
20	1·0	1·0	1·0

Linear interpolation between the values given in this table is permissible, but not extrapolation outside the limits given.

For a wall stiffened by intersecting walls the appropriate stiffening coefficient may be determined from Table 4.2 on the assumption that the intersecting walls are equivalent to piers of width equal to the thickness of the intersecting wall and of thickness equal to 3 times the thickness of the stiffened wall (see Fig. 4.3).

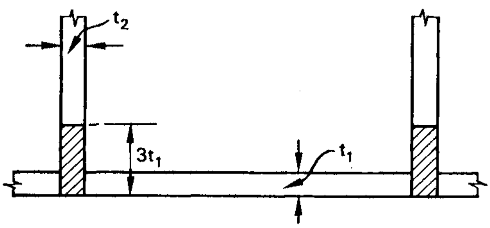


FIG. 4.3. Equivalent piers for wall stiffened by intersecting walls.

7. ECCENTRICITY

This is the distance e from the point of application of a load to the centroidal axis of the member under consideration (see Fig. 4.4). It is normally expressed as a proportion of the thickness of the member.

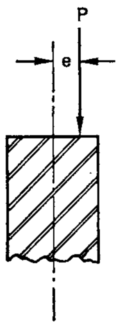


FIG. 4.4. Loadbearing wall with an eccentric load.

8. BASIC STRESS

All compressive stresses in brickwork are related to a basic stress. The value of the basic stress depends on the brick strength and type of mortar and is obtained from Table 4.3.

9. REDUCTION FACTORS

These are factors applied to the basic stress:

(a) Stress reduction factor (F_s). This is a factor applied to the basic stress to allow for the effect of slenderness ratio and eccentricity. Values of F_s are listed in Table 4.4.
(b) Area reduction factor (F_a). Where the cross-sectional area of the wall or column is less than 0·3 m² (375 in²) an additional area reduction factor, F_a, is applied to the basic stress, namely

$$F_a = 0{\cdot}75 + A/1{\cdot}2 \text{ where } A \text{ is the area in m}^2,$$
or
$$F_a = 0{\cdot}75 + A/1850 \text{ where } A \text{ is the area in in}^2.$$

10. PERMISSIBLE STRESSES

(a) *Compressive stresses.* For an axially loaded member the permissible compressive stress should not exceed the product of the appropriate basic stress and the reduction factors F_s and F_a.

Thus, permissible compressive stress = basic stress × F_s or basic stress × F_s × F_a when $A < 0{\cdot}3$ m² (375 in²).

TABLE 4.3 BASIC COMPRESSIVE STRESSES FOR BRICKWORK

Description of mortar	Mix (parts by volume)			Basic stress in N/mm² corresponding to units whose crushing strength (in N/mm²)† is:								
	Cement	Lime	Sand	2·8	7·0	10·5	20·5	27·5	34·5	52·0	69·0	96·5 or greater
Cement	1	0–¼*	3	0·28	0·70	1·05	1·65	2·05	2·50	3·50	4·55	5·85
	1	½	4½	0·28	0·70	0·95	1·45	1·70	2·05	2·80	3·60	4·50
Cement–lime	1	1	6	0·28	0·70	0·95	1·30	1·60	1·85	2·50	3·10	3·80
Cement–lime	1	2	9	0·28	0·55	0·85	1·15	1·45	1·65	2·05	2·50	3·10
Cement–lime	1	3	12	0·21	0·49	0·70	0·95	1·15	1·40	1·70	2·05	2·40

* The inclusion of lime in cement mortars is optional.
† Linear interpolation is permissible for units whose crushing strengths are intermediate between those given in the table.

TABLE 4.4 STRESS REDUCTION FACTORS FOR SLENDERNESS AND ECCENTRICITY

Slenderness ratio	Stress reduction factors*			
	Axially loaded	Eccentricity of vertical loading as a proportion of the thickness of the member		
		1/6	1/4	1/3†
6	1·00	1·00	1·00	1·00
8	0·95	0·93	0·92	0·91
10	0·89	0·85	0·83	0·81
12	0·84	0·78	0·75	0·72
14	0·78	0·70	0·66	0·62
16	0·73	0·63	0·58	0·53
18	0·67	0·55	0·49	0·43
20	0·62	0·48	0·41	0·34
22	0·56	0·40	0·32	0·24
24	0·51	0·33	0·24	—
26	0·45	0·25	—	—
27	0·43	0·22	—	—

* Linear interpolation between values is permitted.

† Where in special cases the eccentricity of loading lies between 1/3 and 1/2 of the thickness of the member, the stress reduction factor whould vary linearly between unity and 0·20 for slenderness ratios of 6 and 20 respectively.

Where there are additional stresses due to eccentricity of loading and/or lateral forces, the permissible compressive stress resulting from the sum of these and those due to axial loading may be increased by 25%.

The Code allows an increase of 50% on the permissible compressive stress for concentrated loads from beam or girder bearings.

(b) *Tensile stresses.* In general, no reliance should be placed on the tensile strength of brickwork in the calculations. For mortar not weaker than a 1:1:6 mix, the permissible tensile stress in bending should not exceed 0·07 N/mm^2 (10 lb/in^2) when the direction of this stress is normal to the bed joints, and 0·14 N/mm^2 (20 lb/in^2) when it acts normal to the perpend joints.

(c) *Shear stresses.* In the case of walls resisting horizontal forces in the plane of the wall, the permissible shear stress should be calculated on the area of the mortar in the horizontal bed joint. For walls built with mortar not weaker than 1 : 1 : 6, the permissible shear stress should range from 0·10 N/mm^2 (15 lb/in^2) when the compressive stress due to dead load at the level under consideration is zero, to an upper limit of 0·50 N/mm^2 (75 lb/in^2) when the compressive stress due to dead load is 2·5 N/mm^2 (300 lb/in^2). Linear interpolation between these values is permitted.

4.7. Design of Members with Axial Loads

The load which an axially loaded wall or column can carry depends basically on five factors. These are:

(i) The thickness of the wall, or cross-sectional area of column.
(ii) The strength of brick.
(iii) The strength of mortar.
(iv) The slenderness ratio.
(v) Lateral support, i.e. stiffening walls or piers.

Factors (i), (ii) and (iii) are usually determined by the designer himself whilst factors (iv) and (v) depend on the configuration of the building, which may have been already determined beforehand. The following examples illustrate the basic principles involved in the design of axially loaded members.

EXAMPLE 4.1

A 0·114 m (4½ in.) wall between concrete floors at 2·60 m (8·5 ft) centres is required to carry a load of 73,000 N/m (5000 lb/ft). Select a suitable brick and mortar for the following conditions:

(a) The wall is unstiffened.
(b) The wall is stiffened by 0·114 m (4½ in.) intersecting walls at 2·74 m (9·0 ft) centres.
(c) The wall is stiffened by 0·114 m (4½ in.) intersecting walls at 1·83 m (6·0 ft) centres.

Solution

(a)

$$H = 2{\cdot}60 \text{ m}$$

$$h = \tfrac{3}{4} \times 2{\cdot}60 = 1{\cdot}95 \text{ m}$$

$$t = 0{\cdot}114 \text{ m}$$

$$\text{SR} = \frac{1{\cdot}95}{0{\cdot}114} = 17.$$

From Table 4.4 the stress reduction factor for axially loaded wall is obtained by interpolation as 0·70.

$$\text{Actual stress in wall} = \frac{73{,}000}{0{\cdot}114 \times 1 \times 10^6}$$

$$= 0{\cdot}64 \text{ N/mm}^2 \text{ (92·5 lb/in}^2\text{)}.$$

The basic stress for which a suitable strength of brick and mortar is selected is obtained by dividing the actual stress in the wall by the appropriate stress reduction factor. In this case

$$\text{Basic stress} = \frac{0{\cdot}64}{0{\cdot}70} = 0{\cdot}915 \text{ N/mm}^2 \text{ (132 lb/in}^2\text{)}.$$

From Table 4.3 the above requirement is met by choosing a 10·5 N/mm^2 (1500 lb/in^2) brick and 1 : 1 : 6 mortar.

(b) The effective length of wall = 2·74–0·114
= 2·626 m (8·6 ft)

which is greater than the effective height and therefore the latter is still used for calculating the slenderness ratio. However, from section 4.6.6 a stiffening coefficient may be applied to obtain the effective thickness.

Width of equivalent stiffening pier = 0·114 m (4½ in.)

$$\frac{\text{Equivalent pier spacing}}{\text{Pier width}} = \frac{2{\cdot}74}{0{\cdot}114} = 24.$$

Therefore from Table 4.2, stiffening coefficient = 1, i.e. the effective thickness is the same as in (a). The basic stress is also the same as in (a).

(c) In this case $l = 1{\cdot}83–0{\cdot}114 = 1{\cdot}716$ m (5·6 ft) which is less than the effective height (1·95 m) and it may therefore be used for calculating the slenderness ratio.

Thus, $$SR = \frac{1\cdot716}{0\cdot114} = 15.$$

From Table 4.4, stress reduction factor = 0·755.

Therefore $$\text{Basic stress} = \frac{0\cdot64}{0\cdot755} = 0\cdot847 \text{ N/mm}^2 \text{ (123 lb/in}^2\text{)}.$$

From Table 4.3 a 10·5 N/mm² (1500 lb/in²) brick with a 1:2:9 mortar would be adequate.

EXAMPLE 4.2

Figure 4.5 shows a brickwork column carrying a steel joist which transmits an axial load of 89,000 N (20,000 lb) to it. The column is 0·572 m by 0·229 m ($22\frac{1}{2}$ in. × 9 in.) in cross-section and is 3·66 m (12·0 ft) high.

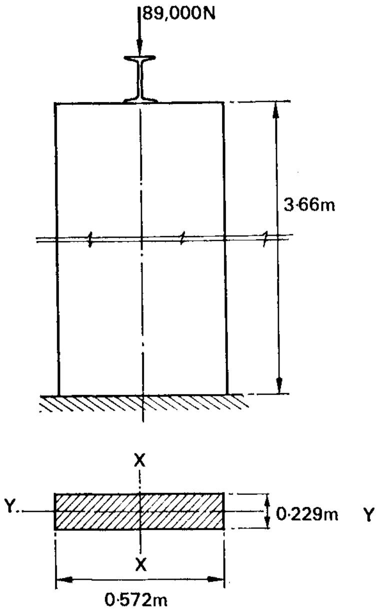


FIG. 4.5. Brickwork pier carrying a steel joist.

Assuming the unit weight of brickwork is 19 kN/m³ (120 lb/ft³) determine the basic stress for the brickwork.

Solution

$$\begin{aligned}\text{Total load on column} &= 89{,}000 + 0{\cdot}572 \times 0{\cdot}229 \times 3{\cdot}66 \times 19{,}000 \\ &= 89{,}000 + 9060 \\ &= 98{,}060 \text{ N } (22{,}025 \text{ lb})\end{aligned}$$

From Fig. 4.5, for buckling about the YY axis, $h = H$ and

$$\text{SR} = \frac{3{\cdot}66}{0{\cdot}229} = 16.$$

For buckling about the XX axis, $h = 2H$ and

$$\text{SR} = \frac{2 \times 3{\cdot}66}{0{\cdot}572} = 12{\cdot}8.$$

Using the highest value of SR, the corresponding stress reduction factor from Table 4·4 is 0·73.

$$\begin{aligned}\text{Cross-sectional area of column} &= 0{\cdot}572 \times 0{\cdot}229 \\ &= 0{\cdot}131 \text{ m}^2 \; (202 \text{ in}^2).\end{aligned}$$

This is less than 0·3 m² (375 in²) and an area reduction factor must be applied.

$$F_a = 0{\cdot}75 + \frac{0{\cdot}131}{1{\cdot}2} = 0{\cdot}86.$$

Allowing a 50% increase on the permissible stress due to the concentrated load,

$$\begin{aligned}\text{Basic stress} &= \frac{89{,}000}{0{\cdot}131 \times 0{\cdot}73 \times 0{\cdot}86 \times 1{\cdot}5 \times 10^6} \\ &+ \frac{9060}{0{\cdot}131 \times 0{\cdot}73 \times 0{\cdot}86 \times 10^6} \\ &= 0{\cdot}722 + 0{\cdot}110 \\ &= 0{\cdot}832 \text{ N/mm}^2 \; (120 \text{ lb/in}^2).\end{aligned}$$

4.8. Design of Members with Eccentric Loads

In a brickwork building the load on a loadbearing wall or column is rarely truly axial. The eccentricity is caused by uneven distribution of load and sagging of the floor slabs. An exact assessment of eccentricity at a wall/slab junction is a complex problem influenced by a number of factors such as the degree of fixity at the junction, the floor loading and the relative stiffness of the floor slabs and the wall. Normally, the eccentricity of wall loading would be most serious in the uppermost storeys, creating tensile stresses in the wall because the direct compressive stresses are low. However, the degree of fixity of the slab/wall junction, and hence the bending moment actually transferred to the wall, is less here than at lower storeys and therefore less likely to be critical.

In buildings having stiff floors and walls stiffened by intersecting walls, it is usual to ignore the eccentricity of floor loading, i.e. assume that the walls are axially loaded.

Where metal hangers are used to support timber joists, the load is normally assumed to act 25 mm (1 in.) from the inner face of the wall. For long spans and for timber and other lightweight floors an eccentricity of 1/6 is often assumed.

In a cavity wall, where both leaves carry an eccentric load, the eccentricity is measured from the centroid of the combined wall and the slenderness ratio is based on the effective thickness of the two leaves. Where only one of the leaves carries an eccentric load, the eccentricity is measured from the centroid of that leaf and the slenderness ratio is calculated from the effective thickness of that leaf. However, the bending moment so produced is assumed to be shared equally by the two leaves.

Once the eccentricity of loading has been determined, the appropriate stress reduction factor may be selected from Table 4.4. The distribution of stress on the cross-section of wall or column subjected to an eccentric load is assumed to be linear and the upper limit of the compressive stress of the resultant trapezoidal or triangular stress block should be used to calculate the required basic stress. As pointed out in section 4.6.10(a), CP 111 allows a 25% increase in permissible stress, provided that such excess is due solely to eccentricity of loading and/or lateral forces.

The principles discussed above are illustrated by the following examples.

EXAMPLE 4.3

A 0·23 m (9 in.) brickwork wall built between concrete slabs measures 2·74 m (9·0 ft) between centres of slabs and carries a load of 365,000 N/m (25,000 lb/ft) at an eccentricity of 0·038 m ($1\frac{1}{2}$ in.). Calculate the stresses in the wall and the corresponding basic stress.

Solution

$$\text{SR} = \frac{\frac{3}{4} \times 2{\cdot}74}{0{\cdot}23} = 9.$$

$$\text{Eccentricity} = \frac{0{\cdot}038}{0{\cdot}23} = 1/6.$$

By interpolation from Table 4.4, stress reduction factor = 0·89

$$\text{Extreme fibre stress in wall} = \frac{P}{A} \pm \frac{Pe}{Z}$$

where P = load, A = cross-sectional area, e = eccentricity of loading and $Z = bt^2/6$ is the section modulus.

For one metre length of wall

$$Z = \frac{1 \times 0{\cdot}23^2}{6} = 0{\cdot}00873 \text{ m}^3$$

$$\text{Cross-sectional area} = 1 \times 0{\cdot}23 = 0{\cdot}23 \text{ m}^2$$

$$\text{Stresses} = \frac{365{,}000}{0{\cdot}23 \times 10^6} \pm \frac{365{,}000 \times 0{\cdot}038}{0{\cdot}00873 \times 10^6}$$

$$= 1{\cdot}595 \pm 1{\cdot}595$$

$$= 3{\cdot}19 \text{ N/mm}^2 \text{ (462 lb/in}^2\text{) compression or nil.}$$

Since CP 111 allows an increase of 25% in stress due to eccentricity (see section 4.6.10).

$$\text{Basic stress} = \frac{3{\cdot}19}{0{\cdot}89 \times 1{\cdot}25} = 2{\cdot}87 \text{ N/mm}^2 \text{ (415 lb/in}^2\text{).}$$

EXAMPLE 4.4

Figure 4.6 shows a 0·282 m (11 in.) cavity wall with the inner leaf carrying a load of 7300 N/m (500 lb/ft) at an eccentricity of 0·025 m (1 in.). Both leaves of the wall are 0·114 m ($4\frac{1}{2}$ in.) thick. Determine the stress distribution in the two leaves of the wall.

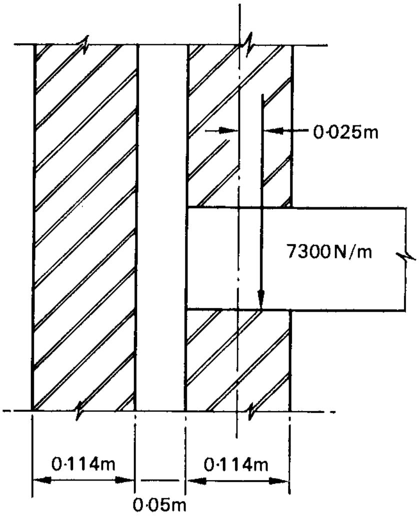


FIG. 4.6. Cavity wall with eccentric loading on the inner leaf.

Solution

Consider one metre length of wall

$$A \text{ (area of each leaf)} = 0{\cdot}114 \times 1 = 0{\cdot}114 \text{ m}^2.$$

$$Z \text{ (each leaf)} = \frac{bt^2}{6} = \frac{1 \times 0{\cdot}114^2}{6} = 0{\cdot}00217 \text{ m}^3.$$

$$\text{Total bending moment} = 7{,}300 \times 0{\cdot}025 = 182{\cdot}5 \text{ Nm}.$$

Bending moment per leaf, $M = 91{\cdot}25$ Nm.

$$\text{Stresses in outer leaf} = \pm \frac{M}{Z} = \pm \frac{91{\cdot}25}{0{\cdot}00217 \times 10^6}$$

$$= \pm 0{\cdot}042 \text{ N/mm}^2 \text{ (6·1 lb/in}^2\text{)}.$$

$$\text{Stresses in inner leaf} = \frac{P}{A} \pm \frac{M}{Z}$$

$$= \frac{7{,}300}{0{\cdot}114 \times 10^6} \pm \frac{91{\cdot}25}{0{\cdot}00217 \times 10^6}$$

$$= 0{\cdot}064 \pm 0{\cdot}042$$

$$= 0{\cdot}106 \text{ N/mm}^2 \text{ (15·4 lb/in}^2\text{) compression}$$

or $0{\cdot}022$ N/mm² (3·2 lb/in²) tension.

4.9. Design of Laterally Loaded Wall Panels

An accurate analysis of laterally loaded brickwork panels is a complex problem and at present only approximate methods are available, based on a limited number of tests, and they are likely to be superseded in the near future as a result of current research work. CP 111 does not give any guidance for the design of such panels except to lay down certain permissible stresses.

There are, broadly speaking, three types of panel support conditions. The first one is met when large windows are used, or where doors separate the panels. In this condition the panel, under horizontal loading, will tend to span vertically between the floors slabs and failure will be usually by tensile bond. For this case the Code allows a tensile stress of up to $0{\cdot}07$ N/mm^2 (10 lb/in^2) but recent panel tests suggest that this may be too conservative and a permissible tensile stress of $0{\cdot}14$ N/mm^2 (20 lb/in^2) is suggested as more realistic. Where the panels are discontinuous, the maximum bending moment may be taken as $WH/10$, where W is the total load on the panel and H is the vertical

TABLE 4.5 MAXIMUM BENDING MOMENTS FOR CONTINUOUS PANELS

$\frac{\text{Span } H}{\text{Width } L}$	1·00	1·25	1·50
Bending moment	$\frac{WH}{18}$	$\frac{WH}{15}$	$\frac{WH}{12}$

span. For a continuous panel the maximum bending moment depends on the panel dimensions, as shown in Table 4.5. If H is the height, and L is the width of the panel then for values of H/L equal to 1·0, 1·25, 1·5 or more, the corresponding maximum bending moments can be taken as $WH/18$, $WH/15$ and $WH/12$ respectively.

The second type of support is obtained when the panels span horizontally between vertical walls or columns. In this case the laterally loaded panel will tend to fail by shear bond and for this condition the Code allows a maximum permissible tensile stress of 0·14 N/mm^2 (20 lb/in^2) but once again recent tests indicate that this stress could also be safely increased to 0·28 N/mm^2 (40 lb/in^2).

The third type of support is obtained when the panels are supported on three or four sides. Under these conditions a panel may fail either by tensile bond at the bed joint, shear bond, or tension in the brick and perpend joint, depending on the panel dimensions. The maximum bending moments will also depend on panel dimensions and conditions of restraint at the sides. At the moment of writing not enough information is available on the safe design of such panels and until such information becomes available, they should be analysed as spanning in the direction of the shorter span.

EXAMPLE 4.5

A brickwork panel 0·229 m (9 in.) thick and 1·83 m (6·0 ft) wide spans vertically and continuously between floor slabs at 2·74 m (9·0 ft) centres. The panel is subjected to a wind loading of 0·77 kN/m^2 (16 lb/ft^2) and its self weight produces a compressive stress of 0·021 N/mm^2 (3 lb/in^2).

Check the maximum tensile and shear stresses in the panel.

Solution

$$\text{Total wind load on panel} = 770 \times 1{\cdot}83 \times 2{\cdot}74$$
$$= 3860 \text{ N (868 lb)}$$

$$\text{The ratio } H/L = \frac{2{\cdot}74}{1{\cdot}83} = 1{\cdot}5.$$

Therefore, from Table 4.5

$$\text{Maximum bending moment} = WH/12$$
$$= \frac{3860 \times 2{\cdot}74}{12}$$
$$= 882 \text{ Nm(7800 lb in).}$$

$$\text{Second moment of area of panel} = \frac{1}{12} \times 1{\cdot}83 \times (0{\cdot}229)^3$$
$$= 0{\cdot}00183 \text{ m}^4 \text{ (4380 in}^4\text{).}$$

$$\text{Maximum bending stresses} = \pm \frac{My}{I} = \pm \frac{882 \times 0{\cdot}1145}{0{\cdot}00183 \times 10^6}$$
$$= \pm 0{\cdot}055 \text{ N/mm}^2 \text{ (8}{\cdot}\text{0 lb/in}^2\text{)}$$

Resultant stresses in panel are 0·034 N/mm^2 (5·0 lb/in^2) tension and 0·076 N/mm^2 (11·0 lb/in^2) compression

$$\text{Maximum shear in panel} = \frac{W}{2} = \frac{3860}{2}$$
$$= 1930 \text{ N (434 lb)}$$

$$\text{and Maximum shear stress} = \frac{1930}{1{\cdot}83 \times 0{\cdot}229} \times 10^6$$
$$= 0{\cdot}0046 \text{ N/mm}^2 \text{ (0}{\cdot}\text{7 lb/in}^2\text{).}$$

References

BRADSHAW, R. E. (1965) The Brick in Slender Crosswall Construction, *C.P.T.B. Technical Note*, Vol. 1, No. 7.

BRADSHAW, R. E. and ENTWISLE, F. D. (1965) Wind Forces on Non-loadbearing Brickwork Panels, *C.P.T.B. Technical Note*, Vol. 1, No. 6.

HASELTINE, B. A. *The Design of Calculated Loadbearing Brickwork*, The Brick Development Association Ltd.

CHAPTER 5

Reinforced Brickwork

5.1. Introduction

One of the limitations of unreinforced brickwork is that it suffers at times from a degree of inflexibility in plan and architectural detail due to its low tensile strength. The use of reinforced brickwork and high-strength mortars removes these restrictions, making the whole concept of design much more flexible. Brickwork reinforcement in the form of steel bars can be used to resist tensile forces, to increase the wall strength, especially under concentrated loads, and where differential settlement is likely to occur. Light reinforcement can also be used to strengthen prefabricated panels which are particularly vulnerable during the handling stage.

Tests on reinforced brickwork have shown that there is very little difference in the structural behaviour between it and reinforced concrete and that the general design principles used for reinforced concrete can be successfully applied to reinforced brickwork. The general principles of design of reinforced brickwork are outlined in Section 5.2.

5.2. Principles of Design

CP 111: 1970 recommends that the design of reinforced brickwork should be based on the same general principles of analysis and elastic design as are adopted for reinforced concrete (see CP 114: 1969, "The structural use of reinforced concrete in buildings"). More particularly it gives a number of guidelines with regard to permissible stresses, strength requirements and modular ratio. Below is a summary of the main points:

1. PERMISSIBLE STRESSES

The maximum stresses in reinforced brickwork should not exceed the following values:

(a) Direct compression. The permissible direct compressive stress should be the same as that for unreinforced brickwork and obtained in the same manner (see Section 4.6 10(a)).
(b) Flexural compression. The permissible flexural compressive stress should be 4/3 of that permitted for direct compression, but it should not exceed 4/3 of the value given for units of 52 N/mm^2 (7500 lb/in^2) strength in the appropriate mortar.
(c) Shear. The permissible shear stress should be as given for unreinforced brickwork (see Section 4.6 10(c)) or 0·21 N/mm^2 (30 lb/in^2), whichever is the greater.
(d) Bond. The permissible bond stress between brickwork and steel should not exceed 0·56 N/mm^2 (80 lb/in^2).
(e) Tensile stress. The tensile stress in steel reinforcement should not exceed 140 N/mm^2 (20,000 lb/in^2) for mild steel.
(f) Eccentric loading. In the case of an eccentrically loaded wall, or column, the maximum permissible compressive stress due to the combined dead and imposed loading should not exceed the values for flexural compression as determined in (b) above, and the maximum permissible stress due to the same loading, calculated as uniformly distributed over the whole area, should not exceed the values for direct compressive stress obtained from (a) above.

2. STRENGTH REQUIREMENTS

In reinforced brickwork, units with strength below 5 N/mm^2 (725 lb/in^2) should not be used, nor mortars weaker than the 1:1:6 mix by volume.

3. MODULAR RATIO

Depending on the compressive strength of the brick, the appropriate modular ratio, i.e. the ratio of elastic modulus of steel to that of brickwork, is listed in Table 5.1.

TABLE 5.1 MODULAR RATIOS FOR REINFORCED BRICKWORK

Crushing strength of unit (N/mm^2)	Maximum modular ratio m (steel to brickwork)
10·5 and under 14·0	33
14·0 and under 20·5	30
20·5 and under 27·5	27
27·5 and under 34·5	24
34·5 and under 41·5	21
41·5 and under 48·5	18
48·5 and under 55·0	15
Over 55·0	12

5.3. Analysis of Reinforced Brickwork

The method of analysis of reinforced brickwork is the same as that for reinforced concrete. Figure 5.1 shows the assumed distribution of strain and stress at any section subjected to a bending moment M. It is assumed that both stress and strain vary linearly with the distance from the neutral axis and that all tensile stresses are taken by the steel reinforcement.

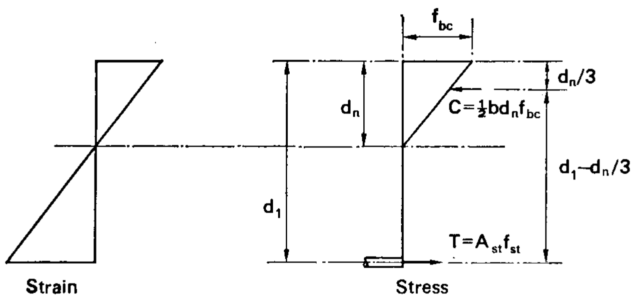


FIG. 5.1. Assumed distribution of stress and strain in reinforced brickwork.

Let A_{st} = area of tensile reinforcement,
b = width of section,
D = overall depth of section,
d_1 = effective depth of section,
d_n = depth of brickwork in compression,
l_a = lever area = $d_1 - d_n/3$,
m = modular ratio = $E_{steel}/E_{brickwork}$,
f_{bc} = extreme fibre stress in compression due to bending,
f_{st} = tensile stress in steel,
p_{bc} = permissible compressive stress in brickwork in bending,
p_{st} = permissible tensile stress in steel,
M = bending moment,
Q = shear force,
q = shear stress,
T = tensile force in steel,
f_b = bond stress,
Σ_0 = total perimeters of reinforcing bars.

For equilibrium of forces we have:

compressive force in brickwork = tensile force in steel.

Therefore:

$$\tfrac{1}{2}bd_nf_{bc} = A_{st}f_{st}. \tag{5.1}$$

Since stress is proportional to distance from neutral axis and stress in steel is m times the stress in brickwork, we have:

$$f_{st} = mf_{bc}\frac{d_1 - d_n}{d_n}. \tag{5.2}$$

Substituting in equation (5.1) we get:

$$\tfrac{1}{2}bd_nf_{bc} = mf_{bc}\frac{d_1 - d_n}{d_n}A_{st}$$

and

$$bd_n^2 + 2md_nA_{st} - 2md_1A_{st} = 0. \tag{5.3}$$

The positive root of equation (5.3) gives the solution for d_n. Taking moments about centroid of reinforcement, we get:

$$M = \tfrac{1}{2}bd_nf_{bc}\left(d_1 - \frac{d_n}{3}\right) \tag{5.4}$$

from which f_{bc} can be calculated.

Alternatively, by taking moments about the centroid of compression

$$M = A_{st} f_{st}\left(d_1 - \frac{d_n}{3}\right) \tag{5.5}$$

from which f_{st} can be calculated for a given value of A_{st}.

If A_{st} is not known it is not possible to calculate d_n accurately. However, since the moment at the section depends on the lever arm $d_1 - d_n/3$, a close approximation of the true moment can be obtained by choosing a reasonable value for d_n. Therefore d_n can be calculated on the basis that the steel and brickwork are stressed simultaneously to their permissible values p_{st} and p_{bc} respectively. For this case it can be shown that:

$$n_1 = \frac{1}{1 + (p_{st}/mp_{bc})} \tag{5.6}$$

where $n_1 = d_n/d_1$, and d_n can be calculated.

Now consider an element of brickwork of width b, effective depth d_1 and length dx as shown in Fig. 5.2. The external forces acting on face A of the element are Q and M and on face B, $Q + dQ$ and $M + dM$.

Taking moments of the external forces about face B we have:

$$M + Qdx - (M + dM) = 0 \tag{5.7}$$

or

$$Q = \frac{dM}{dx}. \tag{5.8}$$

At section A, taking moments about the centroid of compression, we have:

$$M = T\left(d_1 - \frac{d_n}{3}\right) = Tl_a$$

where T = tension force in the steel,
l_a = lever arm.

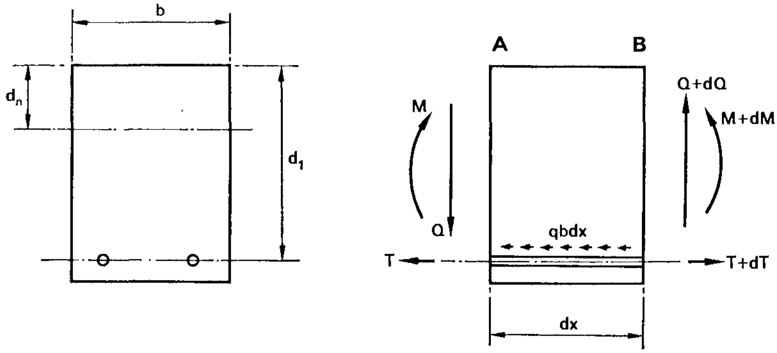


FIG. 5.2. Forces acting on element of brickwork.

Similarly at section *B*, we have:

$$M + dM = (T + dT)l_a$$
$$= Tl_a + dTl_a$$
$$= M + dTl_a$$

or $$dM = l_a dT \tag{5.9}$$

and from equation (5.8):

$$Q = l_a \frac{dT}{dx}. \tag{5.10}$$

The change in tension along the reinforcement sets up shear stresses q in brickwork. At the same time the change in tension can only be brought about by an effective bond existing between the steel and brickwork. We therefore have for horizontal equilibrium at the level of reinforcement:

$$T + qbdx = T + dT$$

or $$\frac{dT}{dx} = qb. \tag{5.11}$$

Substituting in equation (5.10)

$$q = \frac{Q}{l_a b} \tag{5.12}$$

also $$T + f_b \Sigma_0 \, dx = T + dT$$

or $$f_b = \frac{dT}{dx.\Sigma_0} = \frac{Q}{l_a.\Sigma_0} \tag{5.13}$$

where f_b is the bond stress between the steel reinforcement and brickwork and Σ_0 is the total perimeter of the steel bars.

When a reinforced brickwork member is subjected to direct compression as well as bending, the resultant compressive stress in brickwork is obtained by adding the direct compressive stress to the maximum compressive stress due to bending. Although equation (5.3), used for determining the position of neutral axis, strictly no longer applies, it may nevertheless be still used with sufficient accuracy. The resultant tensile stress in steel is obtained by deducting mf_c from the tensile stress due to bending, where f_c is the compressive stress in brickwork due to the direct load and m is the modular ratio.

The following examples illustrate the principles outlined above:

EXAMPLE 5.1

A reinforced brickwork wall is 0·23 m (9 in.) thick, 1·83 m (6·0 ft) wide and carries a load of 150,000 N/m (19,000 lb/ft) at an eccentricity of 0·03 m as shown in Fig. 5.3. The compressive stress due to self-weight of the wall is 1·2 N/mm² (174 lb/in²). Distance between centres of floor slabs is 4·88 m (16 ft).

Assuming a brick strength of 52 N/mm² (1500 lb/in²) and a 1:¼:3 mortar, calculate the maximum compressive stress in the wall and the required area of vertical reinforcement.

Solution

Effective height of wall = ¾ × 4·88 = 3·66 m (12·0 ft).

$$SR = \frac{3{\cdot}66}{0{\cdot}23} = 16$$

$$\text{Eccentricity} = \frac{0{\cdot}03}{0{\cdot}23} = 0{\cdot}13.$$

From Table 4.3 the basic stress for brick strength of 52 N/mm² and 1:¼:3 mortar is 3·5 N/mm² (510 lb/in²).

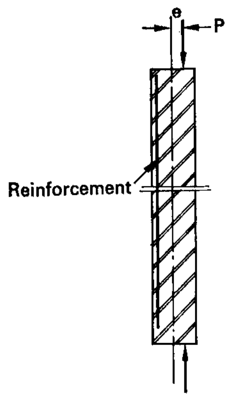


FIG. 5.3. Reinforced brickwork wall with vertical reinforcement.

By interpolation from Table 4.4, stress reduction factor = 0·65. Therefore the maximum permissible direct compressive stress
= 3·5 × 0·65.
= 2·28 N/mm² (330 lb/in²).
Therefore the maximum permissible flexural compressive stress, p_{bc}
= $\frac{4}{3}$ × 2·28 = 3·04 N/mm² (441 lb/in²).
Bending moment in the wall due to imposed loading
= 150,000 × 1·83 × 0·03
= 8250 Nm (6080 lb ft).
From Table 5.1, modular ratio m = 15.
Therefore from equation (5.6):

$$n_1 = \frac{1}{1 + (p_{st}/mp_{bc})} = \frac{1}{1 + 140/(15 \times 3{\cdot}04)} = 0{\cdot}244.$$

Assuming a distance of 40 mm (1·6 in.) from surface of wall to centre of reinforcing bars, the effective depth of wall

$$d_1 = 230 - 40 = 190 \text{ mm } (7{\cdot}48 \text{ in.})$$

and $d_n = n_1\, d_1 = 0{\cdot}244 \times 190 = 46$ mm (1·8 in.)

lever arm $$l_a = d_1 - \frac{d_n}{3}$$

$$= 190 - \frac{46}{3} = 175 \text{ mm } (6{\cdot}9 \text{ in.}).$$

From equation (5·4)

$$f_{bc} = \frac{2M}{bd_n\, l_a}$$

$$= \frac{2 \times 8250 \times 10^3}{1830 \times 46 \times 175} = 1{\cdot}116 \text{ N/mm}^2 \text{ (162 lb/in}^2\text{)}.$$

The average direct compressive stress $f_c = 1{\cdot}2 + \dfrac{150{,}000}{1000 \times 230}$

$$= 1{\cdot}2 + 0{\cdot}652$$

$$= 1{\cdot}852 \text{ N/mm}^2 \text{ (269 lb/in}^2\text{)}$$

which is less than the maximum permissible direct compressive stress of 2·28 N/mm² (330 lb/in²).

Also $f_c + f_{bc} = 1{\cdot}116 + 1{\cdot}852 = 2{\cdot}968$ N/mm² (432 lb/in²) which is less than the maximum permissible flexural compressive stress of 3·04 N/mm² (441 lb/in²).

The Code requirements are therefore met.

From equation (5.5)

$$A_{st} = \frac{M}{f_{st}\, l_a} = \frac{8250 \times 10^3}{140 \times 175} = 337 \text{ mm}^2 \text{ (0·52 in}^2\text{)}.$$

Due to direct compression in the wall, as well as bending, the actual amount of steel required would be less but because of the small amount involved it would probably be justifiable to use the area of steel calculated above. This would mean that the actual stress in steel, instead of being the 140 N/mm² permitted by the Code would now be:

$$140 - mf_c = 140 - 15 \times 1{\cdot}852$$

$$= 140 - 27{\cdot}8 = 112{\cdot}2 \text{ N/mm}^2 \text{ (16,300 lb/in}^2\text{)}.$$

Using 6 mm (¼ in.) diameter bars,

$$\text{Area per bar} = \frac{\pi}{4} \times 6^2 = 28{\cdot}3 \text{ mm}^2 \text{ (0·044 in}^2\text{)}.$$

$$\text{Therefore number of bars required} = \frac{337}{28{\cdot}3} = 11{\cdot}9, \text{ say 12 bars.}$$

The bars could be placed vertically through slotted bricks at 0·34 m ($13\frac{1}{2}$ in.) centres.

Because the bending moment due to eccentricity of loading is constant over the full height of the wall, there will be no shear forces set up and therefore there will be no shear or bond stresses acting.

EXAMPLE 5.2

A non-loadbearing reinforced brickwork panel 3 m (9·85 ft) high and 0·22 m (0·72 ft) thick shown in Fig. 5.4, spans horizontally between

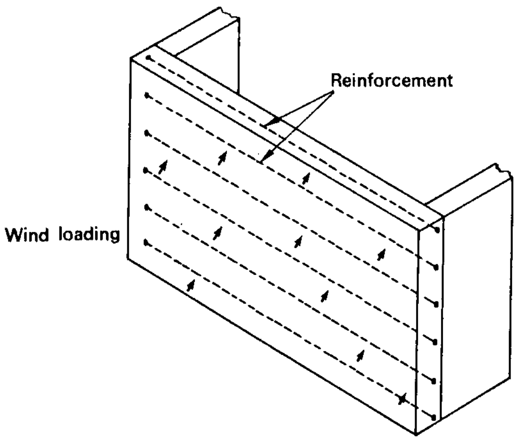


FIG. 5.4. Laterally loaded reinforced brickwork panel with horizontal reinforcement in the bed joints.

intersecting walls spaced at 4 m (13·1 ft) centres. The panel is subjected to a lateral wind load of 2 kN/m^2 (42 lb/ft^2). Assuming a maximum permissible flexural compressive stress of 3·0 N/mm^2 (435 lb/in^2) and a modular ratio of 15, design suitable reinforcement based on maximum bending moment at mid-span of $WL/10$ where W is the total wind load on the panel and L is the horizontal span. Check the actual flexural compressive stress in the panel.

Solution

It has already been pointed out in section 4.9 that CP 111: 1970 does not cover non-loadbearing brickwork panels subjected to lateral forces.

However, the principles of analysis outlined in this section can be applied to this problem to give a satisfactory solution.

Thus, maximum bending moment in panel

$$= \frac{WL}{10} = 2000 \times 3 \times 4 \times 4 = 9600 \text{ Nm (7080 lb ft).}$$

Assuming a distance of 40 mm (1·6 in.) from the internal face of panel to centre of reinforcement,

$$\text{Effective depth of panel section, } d_1 = 220 - 40 = 180 \text{ mm (7·1 in.).}$$

Using equation (5.6)

$$n_1 = \frac{1}{1 + (p_{st}/mp_{bc})} = \frac{1}{1 + 140/(15 \times 3)} = 0{\cdot}244$$

and $$d_n = n_1 d_1 = 0{\cdot}244 \times 180 = 44 \text{ mm (1·73 in.)}$$

lever arm $$l_a = d_1 - \frac{d_n}{3} = 180 - \frac{44}{3}$$

$$= 165 \text{ mm (6·5 in).}$$

From equation (5.5)

$$A_{st} = \frac{M}{f_{st}\, l_a} = \frac{9600 \times 10^3}{140 \times 165} = 415 \text{ mm}^2 \text{ (0·64 in}^2\text{).}$$

Using 6 mm ($\frac{1}{4}$ in.) diameter bars,

$$\text{Area per bar} = \frac{\pi}{4} \times 36 = 28{\cdot}3 \text{ mm}^2 \text{ (0·044 in}^2\text{).}$$

$$\text{Therefore number of bars required} = \frac{415}{28{\cdot}3} = 14{\cdot}7, \text{ say 15.}$$

These bars could be accommodated in the horizontal bed joints of the panel, spaced at regular intervals throughout its height as shown in Fig. 5.4.

From equation (5.4)

$$f_{bc} = \frac{2M}{bd_n\, l_a} = \frac{2 \times 9600 \times 10^3}{3000 \times 44 \times 165}$$

$$= 0{\cdot}88 \text{ N/mm}^2 \text{ (128 lb/in}^2\text{)}.$$

This is well below the permissible value of 3·0 N/mm² (435 lb/in²).

$$\text{Maximum shear in panel, } Q = \frac{W}{2} = \frac{2000 \times 3 \times 4}{2}$$

$$= 12{,}000 \text{ N (2700 lb)}.$$

Therefore, from equation (5.12):

$$q = \frac{Q}{l_a b} = \frac{12{,}000}{165 \times 3000} = 0{\cdot}024 \text{ N/mm}^2 \text{ (3·5 lb/in}^2\text{)}.$$

Also from equation (5.13):

$$f_b = \frac{Q}{l_a \Sigma_0} = \frac{12{,}000}{165 \times 15 \times 6} = 0{\cdot}257 \text{ N/mm}^2 \text{ (37·3 lb/in}^2\text{)}.$$

Both stresses are well below the permissible values quoted in Sections 5.3.1(c) and (d).

Reference

PLUMMER, H. C. and BLUME, J. A. (1953) *Reinforced Brick Masonry and Lateral Force Design*, Structural Clay Products Institute, Washington D.C.

CHAPTER 6

Composite Action of Loadbearing Walls Under Wind Loads

6.1. Parallel Walls

So far we have considered the design of individual brickwork members. In reality a brickwork structure does not act as a number of separate members but as a composite whole. However, so far as the vertical direction of loading is concerned, there is sufficient flexibility in the structure to justify the assumption that each wall or column can act independently of each other. On the other hand, for horizontal forces the situation is different due to the in-plane stiffness of the connecting floor slabs. Here it is more appropriate to assume that a composite action exists which, if one ignores the foreshortening or extension of the connecting floor slabs, makes all lateral displacements of vertical members equal. A complete design method based on composite action in brickwork structures is beyond the scope of this book but the general principles involved in the analysis of such action are outlined below.

Consider a simple loadbearing brickwork structure shown in plan and elevation in Fig. 6.1. The walls must be strong enough to withstand the vertical loads and wind forces acting in any direction. The wind acting in direction *XX* causes a positive pressure on face *AD* and a negative pressure on face *BC*. These pressures are transmitted to walls *AB*, *EF* and *DC* which between them must resist the total wind load in the direction *XX*.

For an approximate solution we assume that the walls *AB*, *EF* and *DC* act as independent vertical cantilevers each one taking a share of the total wind load in the direction *XX*. However, due to the rigidity of the structure and its symmetry, they must all deflect by the same amount. In the case of a vertical cantilever of height *H*, with a uniformly

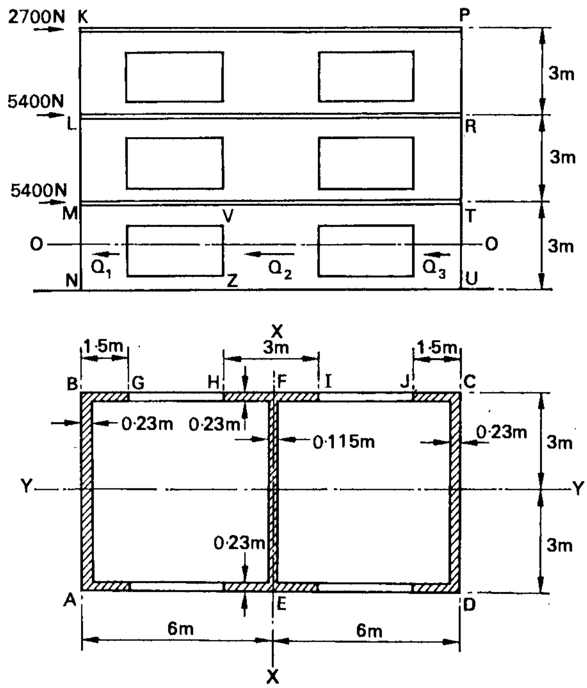


FIG. 6.1. Simple loadbearing brickwork structure.

distributed wind load W acting on it, the maximum deflection at the top will be:

$$\delta = \frac{WH^3}{8EI}$$

where E is the modulus of elasticity and I is the second moment of area of the member about its own axis perpendicular to the wind direction.

In our example we will assume that E is the same for all walls and their height H is also the same. Therefore, since $\delta_{AB} = \delta_{EF} = \delta_{DC}$ we get:

$$\frac{W_{AB}}{I_{AB}} = \frac{W_{EF}}{I_{EF}} = \frac{W_{DC}}{I_{DC}} = k. \tag{6.1}$$

In general we have:

$$
\begin{aligned}
W_1 &= kI_1 \\
W_2 &= kI_2 \\
W_n &= kI_n \\
\hline
\Sigma W_i &= k\Sigma I
\end{aligned}
$$

but $\Sigma W_i = W$, the applied wind load.

Therefore $\quad k = \dfrac{W}{\Sigma I}$ and $W_1 = \dfrac{WI_1}{\Sigma I}$, $\quad W_2 = \dfrac{WI_2}{\Sigma I}$ etc.

In our example, walls *AB* and *DC* are 0·23 m (9 in) thick and wall *EF* is 0·115 m (4½ in.) thick. Assuming a wind loading of 0·5 kN/m², the total wind force in direction *XX* is:

$$0{\cdot}5 \times 9 \times 12 \times 1000 = 54{,}000 \text{ N } (12{,}120 \text{ lb}).$$

$$\text{Also } I_{AB} = I_{CD} = \tfrac{1}{12} \times 0{\cdot}23 \times 6^3 = 4{\cdot}13 \text{ m}^4 \ (477 \text{ ft}^4)$$

$$\text{and } I_{EF} = \tfrac{1}{12} \times 0{\cdot}115 \times 6^3 = 2{\cdot}065 \text{ m}^4 \ (238{\cdot}5 \text{ ft}^4).$$

Therefore

$$\Sigma I = 2 \times 4{\cdot}13 + 2{\cdot}065 = 10{\cdot}325 \text{ m}^4 \ (1192{\cdot}5 \text{ ft}^4)$$

and $\quad W_{AB} = W_{CD} = 54{,}000 \times \dfrac{4{\cdot}13}{10{\cdot}325} = 21{,}600 \text{ N } (4850 \text{ lb})$

and $\quad W_{EF} = 54{,}000 \times \dfrac{2{\cdot}065}{10{\cdot}325} = 10{,}800 \text{ N } (2430 \text{ lb}).$

The bending moment on wall $AB = 21{,}600 \times 4{\cdot}5$
$= 97{,}000 \text{ Nm } (71{,}500 \text{ lb ft}).$

$$\text{Bending stresses in wall } AB = \pm\frac{My}{I} = \pm\frac{97{,}000 \times 3}{4{\cdot}13 \times 10^6}$$

$$= \pm\, 0{\cdot}07 \text{ N/mm}^2 \ (\pm\, 9{\cdot}8 \text{ lb/in}^2).$$

These stresses must be added algebraically to the compressive stresses caused by vertical loads. The same procedure is used to calculate stresses in walls *FG* and *CD*.

6.2. Parallel Walls with Asymmetry

So far we have dealt only with symmetrical walls. When symmetry is lacking either through uneven spacing of walls or uneven distribution of mass then the centre of gravity of the wind force, which coincides with the geometric centre of gravity of the exposed building area, will not coincide with the mass centre of gravity of the wall units. In such cases the building will undergo a rotation as well as a deflection. The calculation of the share of total wind load taken by each wall now becomes a little more involved.

Let us consider a simple case of three parallel walls of equal dimensions as shown in Fig. 6.2. Walls 1 and 2 are spaced 1 unit apart and

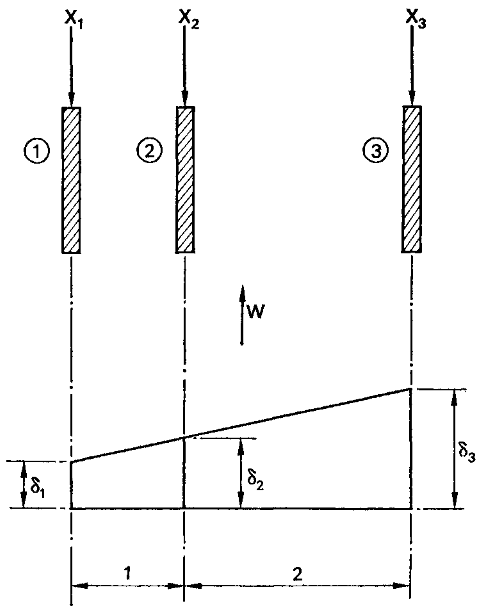


FIG. 6.2. Parallel walls with asymmetry.

walls 2 and 3, 2 units apart. If X_1, X_2 and X_3 are the loads taken by the three walls respectively then from equilibrium of forces:

$$X_1 + X_2 + X_3 = W. \tag{6.2}$$

We assume that plane sections remain plane after deflection and rotation. If δ_1, δ_2 and δ_3 are the deflections of walls 1, 2 and 3 respectively then from geometry we get:

$$\frac{\delta_2 - \delta_1}{1} = \frac{\delta_3 - \delta_2}{2} = \frac{\delta_3 - \delta_1}{3}. \tag{6.3}$$

But $$\delta_1 = \frac{X_1 H^3}{8EI} = CX_1 \quad \text{where} \quad C = \frac{H^3}{8EI}.$$

Since $I_1 = I_2 = I_3$ we have

$$\delta_2 = CX_2 \quad \text{and} \quad \delta_3 = CX_3$$

and equation (6.3) can be written as:

$$\frac{X_2 - X_1}{1} = \frac{X_3 - X_2}{2} = \frac{X_3 - X_1}{3}. \tag{6.4}$$

The final equation is obtained from the equilibrium of moments. Taking moments through the centre of gravity of the wind force which passes through the centre of the building we get:

$$1{\cdot}5X_1 + 0{\cdot}5X_2 - 1{\cdot}5X_3 = 0 \tag{6.5}$$

or

$$X_2 = 3(X_3 - X_1).$$

Solving for X_1, X_2 and X_3 we get:

$$X_1 = \frac{8}{28}\,W, \quad X_2 = \frac{9}{28}\,W, \quad X_3 = \frac{11}{28}\,W.$$

6.3. Loadbearing Walls with Stiffening Beams

Let us return now to the building shown in Fig. 6.1. The wind acting in direction YY will be transmitted to walls AD and BC and due to their symmetry each will take one-half of the total wind force. In wall

BC the piers *BG*, *HI* and *JC* could be designed as free cantilevers fixed at the base, with each taking a share of the total load on the wall in proportion to its second moment of area. Alternatively, each pier can be considered as made up of storey high panels stiffened at each end by stiffening beams running the full length of the building. By using the second alternative we can reduce substantially the bending moments in the piers.

The first step is to determine the load acting on each pier. Assuming again a wind loading of 0·5 kN/m^2 (10 lb/ft^2) the total wind force in the direction $YY = 0{\cdot}5 \times 9 \times 6 \times 1000 = 27{,}000$ N (6060 lb) and wind load on wall $BC = 13{,}500$ N (3030 lb).

Ignoring any changes in length in the connecting floor slabs, piers *BG*, *HI* and *JC* must deflect by the same amount. Therefore the load on any pier is proportional to its second moment of area and since all piers are of the same width, the load on each pier will be in proportion to the cube of its length.

$$\text{Thus load on pier } BG = 13{,}500 \times \frac{1{\cdot}5^3}{1{\cdot}5^3 + 3^3 + 1{\cdot}5^3}$$

$$= 13{,}500 \times \frac{3{\cdot}38}{33{\cdot}75}$$

$$= 1352 \text{ N (304 lb).}$$

$$\text{Similarly load on pier } HI = 13{,}500 \times \frac{27}{33{\cdot}75}$$

$$= 10{,}800 \text{ N (2430 lb)}$$

and load on pier $JC = 1352$ N (304 lb).

The wind force in direction *YY* is transmitted to walls *BC* and *AD* through the roof and floor slabs as concentrated loads at points *K*, *L* and *M* with the force at *K* assumed as one-half of the forces at *L* and *M*. The panel *MN* will be subjected to a shear force Q_1 and taking a section through *O–O* we get from horizontal equilibrium:

$$Q_1 + Q_2 + Q_3 = 13{,}500 \text{ N (3030 lb)}$$

with

$$Q_1 = 1352 \text{ N (304 lb)}$$

$$Q_2 = 10{,}800 \text{ N (2430 lb)}$$

$$Q_3 = 1352 \text{ N (304 lb).}$$

The panel *MN* is assumed to suffer a horizontal displacement with its ends fixed and with a point of contraflexure at its mid-height as shown in Fig. 6.3. It follows that the bending moment at mid-height of the panel is zero. The fixed-end moments can be obtained by taking moments about the mid-point. Thus:

$$Q_1 \times h/2 - M = 0$$

where h = height of panel,
M = fixed-end moments at top and bottom.

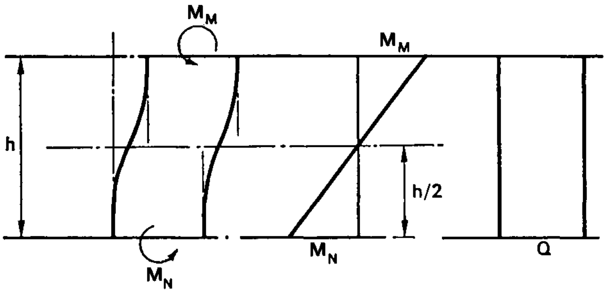


FIG. 6.3. Forces and displacement in panel *MN*.

Therefore fixed-end moments for panel $MN = 1352 \times \frac{3}{2}$

$$= 2030 \text{ Nm (1500 lb ft).}$$

Similarly, fixed-end moments for panel $VZ = 10{,}800 \times \frac{3}{2}$

$$= 16{,}200 \text{ Nm (11,950 lb ft).}$$

Assuming that walls *BC* and *AD* are both 0·23 m (9 in.) thick, we get the following section modulus for panel *VZ*:

$$\text{Section modulus} = \tfrac{1}{6} \times 0{\cdot}23 \times 3^2 = 0{\cdot}345 \text{ m}^3 \text{ (12·2 ft}^3\text{).}$$

Maximum bending stresses in panel *VZ* due to wind in direction *YY* are therefore:

$$\pm \frac{16{,}200}{0{\cdot}345 \times 10^6} = \pm\, 0{\cdot}047 \text{ N/mm}^2 \; (\pm\, 6{\cdot}9 \text{ lb/in}^2).$$

These stresses must be added algebraically to the compressive stresses caused by vertical loads.

The fixed-end moments at the top of panels *MN*, *VZ* and *TU* can only be mobilized providing the stiffening beam is strong enough to resist them as well. Considering the beam *MVT*, the bottom panels transmit clockwise moments to the beam. Additional clockwise moments are transmitted to the beam from the panels above. As an approximation we assume that the moments on the beam are applied at the centre of each panel.

From Fig. 6.1 the shear force in panel *LM*

$$= \frac{Q_1 \times (2700 + 5400)}{(2700 + 5400 + 5400)}$$

$$= 1352 \times \frac{8100}{13{,}500}$$

$$= 813 \text{ N},$$

and the fixed-end moment transmitted to the stiffening beam *MVT* by panel *LM* is 813 × 3/2 = 1218 Nm (900 lb ft).

Therefore, the total clockwise moment applied to beam *MVT* at the centre of panel *MN*

$$= 2030 + 1218 = 3248 \text{ Nm (2400 lb ft)}.$$

Also, total clockwise moment applied to beam *MVT* at the centre of panel *VZ*

$$= 16{,}200 + 16{,}200 \times \frac{8\,100}{13{,}500} = 25{,}940 \text{ Nm (19,130 lb ft)}.$$

Similarly, total clockwise moment applied to beam *MVT* at the centre of panel *TU*

$$= 3248 \text{ Nm (2400 lb ft)}.$$

Figure 6.4 shows the bending moments set up in the stiffening beam *MVT* due to the applied moments.

6.4. Coupled Shear Walls

The analysis of the piers in the preceding sections was based on the assumption that the panels between the stiffening beams suffered only horizontal displacements. This assumption is reasonable for buildings up to three or four storeys high. However, recent years have seen a rapid increase in the number of tall buildings with relatively slender proportions and for such structures the above assumptions would no longer apply as there would now be considerable rotation in the walls as well as horizontal displacements under the action of wind. The individual walls could of course still be designed as separate cantilevers but this would ignore the stiffening effect of the floors which connect

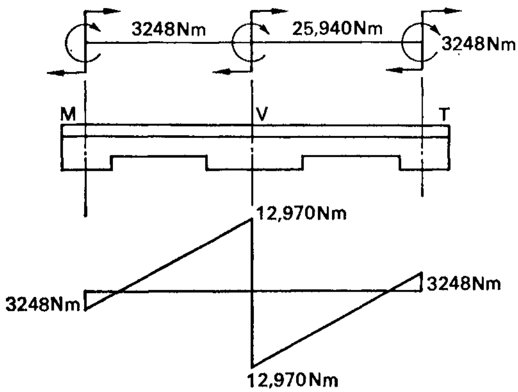


FIG. 6.4. Bending moments in stiffening beam.

the walls. It is therefore of great practical importance to obtain a satisfactory method of analysis of such coupled shear wall structures.

One relatively simple method of analysis has been produced which assumes that the discrete system of connections, formed by the floor slabs or by lintel beams, may be replaced by an equivalent continuous medium. By assuming also that the connecting beams have a point of contraflexure at their midspan caused by the lateral displacement of the coupled walls and that their axial foreshortening under load is negligible, the behaviour of the system can be expressed as a single second order differential equation, enabling a general solution to the problem to be obtained. Experiments on model structures have yielded results in good agreement with the values given by this method. So far as brickwork structures are concerned, recent full scale experimental work has shown that this method of analysis tends to underestimate deflections and that more accurate results are obtained if the distance between the centres of the shear walls is used in the calculations instead of the clear distance between them.

ROSMAN'S THEORY OF COUPLED SHEAR WALLS

NOTATION

A_1, A_2 = cross-sectional areas of the coupled walls,
b = clear span of connecting beams or slabs,
h = floor-to-floor height,
H = height of walls,
x = distance from top of walls,
I_1, I_2 = second moments of areas of the coupled walls,
I_p = reduced second moment of area of connecting beams or slabs,
l = distance between centroids of cross-sections of walls,
m = width of slab that can be considered to act as connecting beam,
t = slab thickness,
T = integral shear force or sum of laminae shears above a given level,
v = shear force per unit height in the laminae,
v_m = maximum value of v,
v_t = value of v at top of the walls,
V = shear force in connecting beams,
w = uniformly distributed wind loading per unit height,
z = distance from top of building to point of maximum shear force per unit height, v_m.

THEORY

In the theory it is assumed that all connecting beams or, in case of slabs, equivalent beams, are equally spaced and of the same second moment of area with the exception of the top connecting beam which

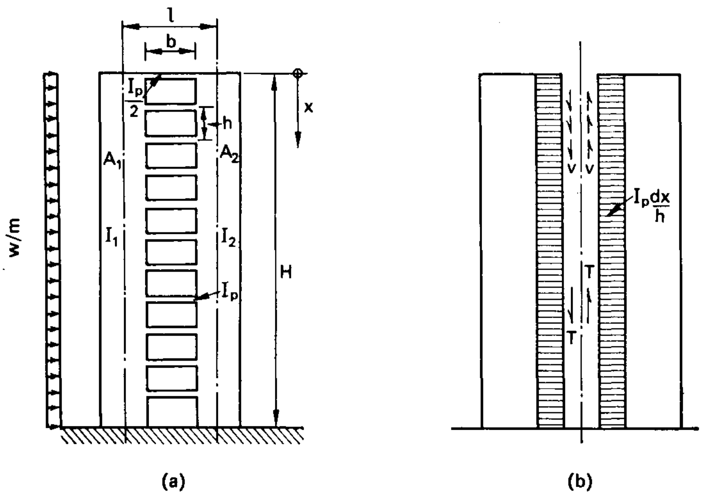


FIG. 6.5. (a) Couple shear walls. (b) Individual connecting beams replaced by a continuous connection.

is assumed to have one-half the second moment of area of the other beams (see Fig. 6.5a). Using this assumption it is possible to replace the discrete beams by a continuous connection, or laminae, of height dx and an equivalent second moment of area of $I_p\, dx/h$ as shown in Fig. 6.5b. Under the uniformly distributed horizontal loading w per unit height the walls will deflect, inducing shear forces in the laminae and bending them, with a point of contraflexure at the centre of their span. If the laminae are now considered cut through their middle (i.e. at their points of contraflexure) where only the shear forces v per unit height are acting then from the consideration of deformation and continuity of the laminae, one can establish a second order differential equation in T where

$$T = \int_0^x v dx.$$

For simplicity consider two identical coupled walls with $A_1 = A_2 = A$ and $I_1 = I_2 = I$. In this case the total wind force w per unit height is equally shared by the walls with w_1 acting on each wall where $2w_1 = w$. At any section of the longitudinally cut walls a lamina suffers a displacement δ_1 due to the slope of the walls, a displacement δ_2 due to bending and shear caused by the shear forces v and a displacement δ_3 due to direct extention and compression of respective walls, again caused by the forces v. The respective displacements are shown in Fig. 6.6a, b and c.

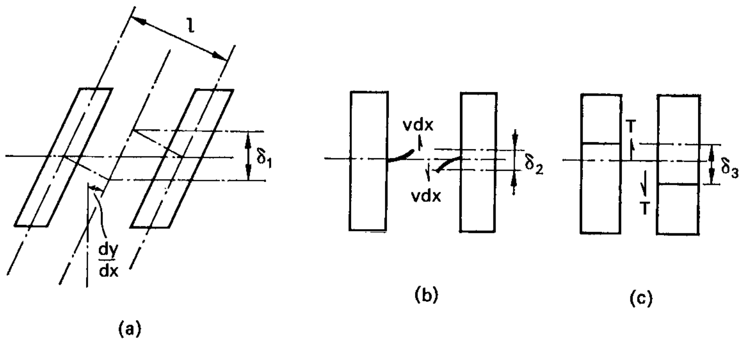


FIG. 6.6. Displacements of longitudinally cut laminae: (a) displacement due to slope, (b) displacement due to bending and shear, (c) displacement due to direct extension and compression.

In the calculation of δ_2 the shear deflection can be neglected provided a reduced second moment of area I_p is used where:

$$I_p = \frac{mt^3}{12[1 + 2{\cdot}4\,(t/b)^2]}$$

where m = width of beam or width of floor slab that can be considered to act as connecting beam,

t = depth of beam or thickness of slab.

From Fig. 6.6a the displacement δ_1 is given by:

$$\delta_1 = l\frac{dy}{dx}$$

where dy/dx is considered small.

To calculate δ_2 we consider the deflection of a lamina of elemental thickness dx and equivalent second moment of area $I_p\, dx/h$ acting as a cantilever of span $b/2$ under a concentrated force vdx. The total displacement δ_2 is given by:

$$\delta_2 = -\frac{hb^3}{12EI_p}v.$$

The displacement δ_3 due to direct extension and compression caused by the forces v per unit height is given by:

$$\delta_3 = -\frac{-2}{EA}\int_0^x\left(\int_0^x vdx\right)dx = \frac{2}{EA}\int_0^x\left(\int_0^x vdx\right)dx.$$

However, due to the continuity of the lamina:

$$\delta_1 + \delta_2 + \delta_3 = 0$$

or

$$l\frac{dy}{dx} - \frac{hb^3}{12EI_p}v + \frac{2}{EA}\int_0^x\left(\int_0^x vdx\right)dx = 0. \qquad (6.6)$$

Differentiate equation (6.6) with respect to x and by rearranging we get:

$$\frac{d^2y}{dx^2} = \frac{hb^3}{12EI_p l}\frac{dv}{dx} - \frac{2}{EAl}\int_0^x vdx. \qquad (6.7)$$

Using the standard beam expression:

$$\frac{EId^2y}{dx^2} = -M$$

and applying it to one of the walls (clockwise moments positive) we get:

$$EI\frac{d^2y}{dx^2} = \frac{l}{2}\int_0^x vdx - \frac{w_1 x^2}{2}. \qquad (6.8)$$

Substituting for d^2y/dx^2 from equation (6.7) we get:

$$EI\left(\frac{hb^3}{12EI_p l}\cdot\frac{dv}{dx}-\frac{2}{EAl}\int_0^x vdx\right)-\frac{l}{2}\int_0^x vdx=-\frac{w_1 x^2}{2}. \quad (6.9)$$

Putting

$$T=\int_0^x vdx$$

and rearranging we finally get the second order differential equation:

$$\frac{d^2T}{dx^2}-\alpha^2 T=-\beta x^2 \quad (6.10)$$

where:
$$\alpha^2=\left(\frac{l^2}{2I}+\frac{2}{A}\right)\cdot\frac{12I_p}{hb^3}$$

and
$$\beta=\frac{wl}{4I}\cdot\frac{12I_p}{hb^3}.$$

When $A_1 \neq A_2$ and $I_1 \neq I_2$

$$\alpha^2=\left(\frac{l^2}{I_1+I_2}+\frac{1}{A_1}+\frac{1}{A_2}\right)\frac{12I_p}{hb^3}$$

$$\beta=\frac{1}{2}\frac{wl}{I_1+I_2}\cdot\frac{12I_p}{hb^3}.$$

For the most common case of a fixed base, solution of equation (6.10) gives:

$$T=C\sinh\alpha x-\frac{2\beta}{\alpha^4}(\cosh\alpha x-1)+\frac{\beta x^2}{\alpha^2} \quad (6.11)$$

where
$$C=\frac{1}{\cosh\alpha H}\cdot\frac{2\beta}{\alpha^3}\left(\frac{\sinh\alpha H}{\alpha}-H\right).$$

Differentiation of equation (6.11) gives v. A typical variation of v and T with height x is shown in Fig. 6.7. From the T-diagram and the moments caused by the wind the bending moments in the walls can be obtained and the stresses computed.

To analyse a system of coupled shear walls using equation (6.11) requires a considerable amount of calculation. Fortunately one can simplify the solution without sacrificing too much of the accuracy in

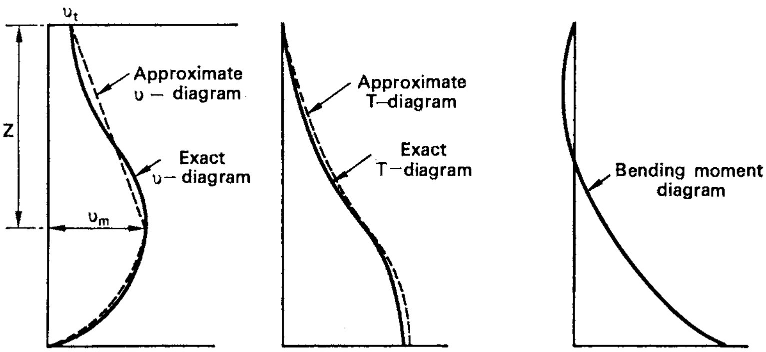


FIG. 6.7. Typical variation of shear forces (V), integral shear forces (T), and bending moment (M).

the following manner. The laminae shear force diagram v shown in Fig. 6.7 can be approximated to a straight line from v_t at the top to the maximum value v_m. The values of v_t and v_m can be deduced fairly easily as well as the position of v_m. The lower part of the v-curve is approximated to a parabola.

The shear force in any connecting beam V is obtained from the difference in the values of T at levels $h/2$ above and below the beam position. Alternatively it can be obtained from the area of the v-diagram between these levels. The shear in the top beam, V_t is equal to the value of T at a distance $h/2$ below the top. V_t is also approximately equal to $v_t \times h/2$. Thus by solving for T at $x = h/2$ from equation (6.11) V_t can be determined.

In most practical problems $\alpha H > 3$ and for these values $\sinh \alpha H \simeq \cosh \alpha H$.

Using this approximation the constant C in equation (6.11) reduces to:

$$C = \frac{2\beta}{\alpha^4}\left(1 - \frac{\alpha H}{\cosh \alpha H}\right).$$

For maximum value of v, $\quad \dfrac{dv}{dx} = 0,$

also since $v = \dfrac{dT}{dx}$, $\quad \dfrac{dv}{dx} = \dfrac{d^2T}{dx^2},$

therefore for v_m, $\quad \dfrac{d^2T}{dx^2} = 0.$

Let z be the distance from the top of the building to the point of maximum laminae shear, v_m and let $\alpha z = y$. Therefore

$$T = C \sinh y - \frac{2\beta}{\alpha^4}\cosh y + \frac{2\beta}{\alpha^4} + \frac{\beta z^2}{\alpha^2} \tag{6.12}$$

$$\frac{dT}{dy} = C \cosh y - \frac{2\beta}{\alpha^4}\sinh y + \frac{2\beta}{\alpha^4} y. \tag{6.13}$$

Again for large values of y (≥ 3), $\cosh y = \sinh y$. Therefore

$$\frac{dT}{dy} = \frac{2\beta}{\alpha^4}\left(1 - \frac{\alpha H}{\cosh \alpha H}\right)\cosh y - \frac{2\beta}{\alpha^4}\sinh y + \frac{2\beta}{\alpha^4} y$$

$$= \frac{2\beta}{\alpha^4}\left(y - \frac{\alpha H}{\cosh \alpha H}\cdot \cosh y\right). \tag{6.14}$$

$$\frac{d^2T}{dy^2} = \frac{2\beta}{\alpha^4}\left(1 - \frac{\alpha H}{\cosh \alpha H}\cdot \sinh y\right) \tag{6.15}$$

$= 0$ for v_m.

Therefore

$$\sinh y = \frac{\cosh \alpha H}{\alpha H} = \cosh y. \tag{6.16}$$

Substituting in equation (6.14) we get:

$$\frac{dT}{dy} = \frac{2\beta}{\alpha^4}(y - 1). \tag{6.17}$$

Therefore
$$\frac{dT}{d(\alpha z)} = \frac{dT}{\alpha dz} = \frac{2\beta}{\alpha^4}(\alpha z - 1)$$

and
$$\frac{dT}{dz} = v_m = \frac{2\beta}{\alpha^3}(\alpha z - 1). \tag{6.18}$$

EXAMPLE ON COUPLED SHEAR WALLS

Figure 6.8 shows the elevation and plan view of a brickwork structure with identical coupled shear walls. The walls are spaced at 6 m

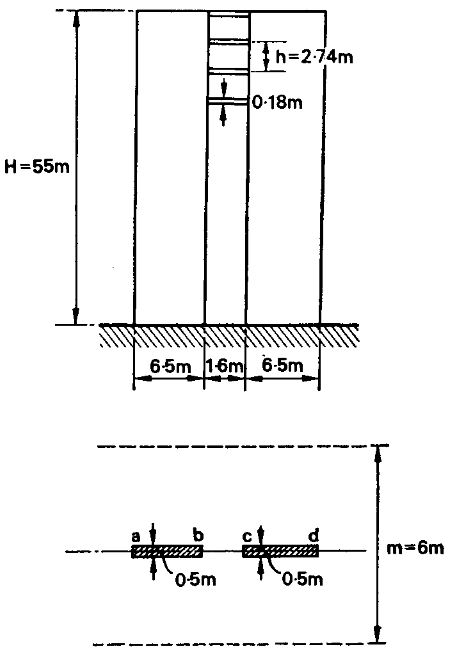


FIG. 6.8. Elevation and plan of coupled shear walls.

(19.7 ft) centres and are connected at each storey height by rigid concrete floor slabs 0·18 m (0·60 ft) thick. The walls are 6·50 m (21·3 ft) by 0·50 m (1·64 ft) in cross-section and are 55 m (180·5 ft) high. The distance between centres of floor slabs is 2·74 m (9·0 ft). Clear distance between the coupled walls is 1·60 m (5·25 ft). The wind loading on the building is 1·90 kN/m² (40 lb/ft²). Roof loading, including dead load, is 2·80 kN/m² (58·0 lb/ft²). Floor loading (dead load plus live load) is 5·74 kN/m² (120·0 lb/ft²). Unit weight of brickwork is 18·8 kN/m³ (120·0 lb/ft³).

Tabulate the distribution of the integral *T*-force and bending moment for each wall. Check also the stresses in the walls 10 m (32·8 ft) and 55 m (180·5 ft) from the top, just below the ground floor level.

Solution

From the information given above we have the following:

$m = 6$ m (19·7 ft) $H = 55$ m (180·5 ft)

$b = 1{\cdot}60$ m (5·25 ft) $h = 2{\cdot}74$ m (9·0 ft)

$t = 0{\cdot}18$ m (0·60 ft) $w = 1{\cdot}90 \times 6 = 11{\cdot}4$ kN/m (780 lb/ft)

Furthermore, the distance between centroids of cross-sections of wall,

$$l = 8{\cdot}10 \text{ m } (26{\cdot}6 \text{ ft})$$

$$A_1 = A_2 = 6{\cdot}50 \times 0{\cdot}50 = 3{\cdot}25 \text{ m}^2 \ (35{\cdot}0 \text{ ft}^2)$$

$$I_1 = I_2 = \tfrac{1}{12} \times 0{\cdot}5 \times (6{\cdot}5)^3 = 11{\cdot}42 \text{ m}^4 \ (1320 \text{ ft}^4).$$

The second moment of area of equivalent beam spanning between the walls, I_p, reduced to take into account the influence of shear forces, is given by:

$$I_p = \frac{mt^3}{12[1 + 2{\cdot}4\,(t/b)^2]} = \frac{6 \times (0{\cdot}18)^3}{12\left[1 + 2{\cdot}4\left(\dfrac{0{\cdot}18}{1{\cdot}60}\right)^2\right]}$$

$$= 0{\cdot}00291 \text{ m}^4 \ (0{\cdot}337 \text{ ft}^4)$$

$$a^2 = \left(\frac{8{\cdot}10^2}{2 \times 11{\cdot}42} + \frac{2}{3{\cdot}25}\right) \times \frac{12 \times 0{\cdot}00291}{2{\cdot}74 \times (1{\cdot}60)^3} = 0{\cdot}01093$$

$$a = 0{\cdot}10461/\text{m } (0{\cdot}03191/\text{ft})$$

$$\beta = \tfrac{1}{2} \times \frac{11{,}400 \times 8{\cdot}10}{2 \times 11{\cdot}42} \times \frac{12 \times 0{\cdot}00291}{2{\cdot}74 \times (1{\cdot}60)^3}$$
$$= 6{\cdot}35 \text{ N/m}^4 \,(0{\cdot}0124 \text{ lb/ft}^4)$$
$$\alpha H = 0{\cdot}1046 \times 55 = 5{\cdot}75$$
$$C = \frac{2 \times 6{\cdot}35}{1{\cdot}195 \times 10^{-4}} \left(1 - \frac{5{\cdot}75}{\cosh 5{\cdot}75}\right) = 104{,}300 \text{ N (23,500 lb).}$$

To get v_t we must first determine V_t which equals T at $x = h/2$ or $x = 1{\cdot}37$ m (4·5 ft).

Therefore at $x = 1{\cdot}37$ m (4·5 ft)

$$\alpha x = 1{\cdot}37 \times 0{\cdot}1046 = 0{\cdot}1432.$$

$$T = 104{,}300 \sinh 0{\cdot}1432 - 106{,}200 (\cosh 0{\cdot}1432 - 1)$$
$$+ \frac{6{\cdot}35 \times (1{\cdot}37)^2}{0{\cdot}01093} = 15{,}000 \text{ N (3370 lb)} = V_t.$$

Therefore $$v_t = \frac{15{,}000}{1{\cdot}37} = 10{,}950 \text{ N/m (745 lb/ft).}$$

From equation (6.16) we have

$$\sinh y = \sinh \alpha z = \frac{\cosh \alpha H}{\alpha H} = \frac{\cosh 5{\cdot}75}{5{\cdot}75} = \frac{315}{5{\cdot}75} = 54{\cdot}8.$$

Therefore $\alpha z = 4{\cdot}68$ and $z = 44{\cdot}7$ m (146·5 ft).

From equation (6·18)

$$v_m = \frac{2\beta}{\alpha^3}(\alpha z - 1) = \frac{2 \times 6{\cdot}35}{1{\cdot}142 \times 10^{-3}} (4{\cdot}68 - 1)$$
$$= 41{,}000 \text{ N/m (2820 lb/ft).}$$

The approximate distribution of v can now be plotted as shown in Fig. 6.9 and the value of T at any level can be readily obtained by finding the area of the v-diagram down to that level. The area of the parabolic portion of the v-diagram is 2/3 of the rectangle enclosing it.

At any section the T-forces produce a couple of magnitude $2 \times Tl/2$

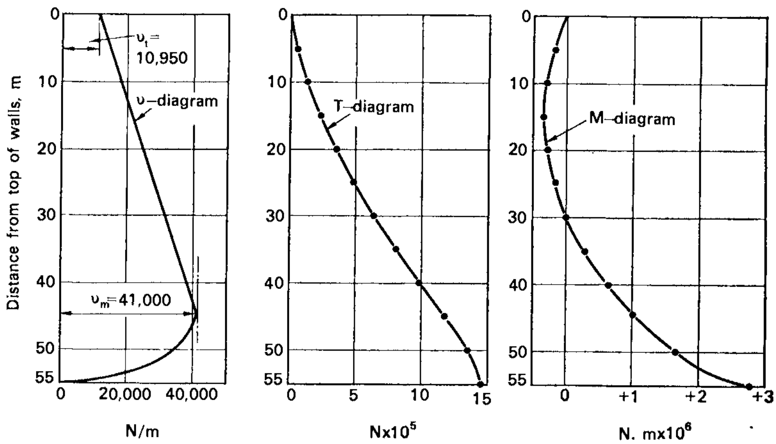


FIG. 6.9. Approximate variation in shear forces (V), integral force (T) and bending moment (M) with height of coupled shear walls.

TABLE 6.1

1 lb = 4·45 N, 1 lb ft = 1·355 N m

X (m)	T ($\times 10^3$N)	$Tl/2$ (Nm) $\times 10^3$	$wx^2/4$ (N m) $\times 10^3$	Wall moment $\times 10^3$ (N m)
0	0	0	0	0
5	63·2	258	71	− 187
10	143·1	580	285	− 295
15	239·9	970	639	− 331
20	353·5	1425	1140	− 285
25	484·0	1962	1782	− 180
30	631·0	2560	2550	− 10
35	795·4	3225	3490	265
40	975·0	3950	4560	610
44·7	1160·0	4700	5700	1000
50	1354·0	5480	7100	1620
55	1441·0	5840	8590	2750

which acts counter to the moment produced by wind. Furthermore, the fraction of the total bending moment taken by each wall is proportional to its second moment of area so that:

$$\text{moment in wall 1} = M_1 = (\tfrac{1}{2}wx^2 - Tl)\frac{I_1}{I_1 + I_2}$$

and

$$\text{moment in wall 2} = M_2 = (\tfrac{1}{2}wx^2 - Tl)\frac{I_2}{I_1 + I_2}.$$

In this example $I_1 = I_2$ and $M_1 = M_2 = \tfrac{1}{4}wx^2 - Tl/2$.

The values of T-forces and wall moments in each wall are tabulated in Table 6.1. The negative values indicate that the bending stresses are compressive on the windward side.

CALCULATION OF STRESSES

The bending stresses at points *a*, *b*, *c* and *d* of the coupled shear walls (see Fig. 6.8), with positive values indicating tension are as follows:

$$\sigma_a = \frac{My}{I} + \frac{T}{A}, \qquad \sigma_b = -\frac{My}{I} + \frac{T}{A},$$

$$\sigma_c = \frac{My}{I} - \frac{T}{A}, \qquad \sigma_d = -\frac{My}{I} - \frac{T}{A}$$

where $y = 6{\cdot}5/2 = 3{\cdot}25$ m (10·67 ft)
$I = 11{\cdot}42\ \text{m}^4$ ($1320\ \text{ft}^4$)
$A = 3{\cdot}25\ \text{m}^2$ ($35{\cdot}0\ \text{ft}^2$).

At $x = 10$ m (32·8 ft) *from top*

$$\frac{My}{I} = -\frac{295 \times 3{\cdot}25}{11{\cdot}42} = -84\ \text{kN/m}^2\ (-1755\ \text{lb/ft}^2),$$

$$\frac{T}{A} = \frac{143{\cdot}1}{3{\cdot}25} = +44{\cdot}1\ \text{kN/m}^2\ (+922\ \text{lb/ft}^2).$$

Therefore:

$$\sigma_a = -84 + 44{\cdot}1 = -\ 39{\cdot}9\ \text{kN/m}^2\ (-835\ \text{lb/ft}^2)$$
$$\sigma_b = +84 + 44{\cdot}1 = +\ 128{\cdot}1\ \text{kN/m}^2\ (+2680\ \text{lb/ft}^2)$$
$$\sigma_c = -84 - 44{\cdot}1 = -\ 128{\cdot}1\ \text{kN/m}^2\ (-2680\ \text{lb/ft}^2)$$
$$\sigma_d = +84 - 44{\cdot}1 = +\ 39{\cdot}9\ \text{kN/m}^2\ (+835\ \text{lb/ft}^2)$$

Vertical loads at 10 m (32·8 ft) *from top*

Area supported by each wall = 6 × 7·3 = 43·8 m² (472 ft²)

Roof	43·8 × 2·80 =	122·7 kN
Floors	3 × 43·8 × 5·74 =	754·0 kN
Wall	10 × 6·5 × 0·5 × 18·8 =	610·0 kN
	Total =	1486·7 kN (334,000 lb)

$$\text{Vertical stress } \sigma_v = -\frac{1486\cdot7}{6\cdot5 \times 0\cdot5} = -457\cdot5 \text{ kN/m}^2 (-9550 \text{ lb/ft}^2)$$

Resultant stresses (with negative sign indicating compression) are:

$$\text{At } a = -39\cdot9 - 457\cdot5 = -497\cdot4 \text{ kN/m}^2 = -0\cdot497 \text{ N/mm}^2 \ (-72 \text{ lb/in}^2)$$

$$\text{at } b = +128\cdot1 - 457\cdot5 = -329\cdot4 \text{ kN/m}^2 = -0\cdot329 \text{ N/mm}^2 \ (-47\cdot7 \text{ lb/in}^2)$$

$$\text{at } c = -128\cdot1 - 457\cdot5 = -585\cdot6 \text{ kN/m}^2 = -0\cdot586 \text{ N/mm}^2 \ (-85\cdot1 \text{ lb/in}^2)$$

$$\text{at } d = +39\cdot9 - 457\cdot5 = -417\cdot6 \text{ kN/m}^2 = -0\cdot418 \text{ N/mm}^2 \ (-60\cdot7 \text{ lb/in}^2)$$

At x = 55 m (180·5 ft) *from top*

$$\frac{My}{I} = \frac{2750 \times 3\cdot25}{11\cdot42} = 782 \text{ kN/m}^2 \ (16{,}340 \text{ lb/ft}^2),$$

$$\frac{T}{A} = \frac{1441}{3\cdot25} = 443 \text{ kN/m}^2 \ (9280 \text{ lb/ft}^2).$$

Therefore:

$$\sigma_a = +782 + 443 = +1225 \text{ kN/m}^2 \ (+25{,}600 \text{ lb/ft}^2)$$
$$\sigma_b = -782 + 443 = -339 \text{ kN/m}^2 \ (-7090 \text{ lb/ft}^2)$$
$$\sigma_c = +782 - 443 = +339 \text{ kN/m}^2 \ (+7090 \text{ lb/ft}^2)$$
$$\sigma_d = -782 - 443 = -1225 \text{ kN/m}^2 \ (-25{,}600 \text{ lb/ft}^2)$$

Vertical loads at 55 m (180·5 ft) *from top*

Roof	43·8 × 2·80 =	123 kN
Floors	20 × 43·8 × 5·74 =	5030 kN
Wall	55 × 6·5 × 0·5 × 18·8 =	3365 kN
	Total =	8518 kN (1,915,000 lb)

$$\text{Vertical stress } \sigma_v = \frac{8518}{6{\cdot}5 \times 0{\cdot}5} = -\ 2620\ \text{kN/m}^2\ (-\ 54{,}750\ \text{lb/ft}^2).$$

Resultant stresses are:

$$\begin{aligned}
\text{At } a &= +\ 1225 - 2620 = -\ 1395\ \text{kN/m}^2 \\
&= -1{\cdot}395\ \text{N/mm}^2\ (-202{\cdot}5\ \text{lb/in}^2) \\
\text{at } b &= -\ 339 - 2620 = -\ 2959\ \text{kN/m}^2 \\
&= -\ 2{\cdot}959\ \text{N/mm}^2\ (-429{\cdot}0\ \text{lb/in}^2) \\
\text{at } c &= +\ 339 - 2620 = -\ 2281\ \text{kN/m}^2 \\
&= -\ 2{\cdot}281\ \text{N/mm}^2\ (-331{\cdot}0\ \text{lb/in}^2) \\
\text{at } d &= -\ 1225 - 2620 = -\ 3845\ \text{kN/m}^2 \\
&= -\ 3{\cdot}845\ \text{N/mm}^2\ (-558{\cdot}0\ \text{lb/in}^2)
\end{aligned}$$

References

Hahn, J. (1966) *Structural Analysis of Beams and Slabs*, Pitman.
Rosman, R. (1964) Approximate Analysis of Shear Walls Subject to Lateral Loads, *Journal A.C.I.*, Vol. 61, No. 6.

CHAPTER 7

Recent Developments in Brickwork Technology

7.1. High-tensile Bond Mortars

One of the limitations of conventional brickwork is that it can take little or no tensile forces. This fact has put severe limitations on the architectural concepts of brickwork design. In an attempt to overcome these restrictions and to eliminate this inherent weakness in brickwork, high-tensile bond mortars have recently been introduced. This development offers the architect and engineer new freedom in their design. Once labelled "the weak link in masonry systems", mortar can now achieve the inherently high strengths of the fired clay units and contribute to monolithic construction with predictable strengths.

The basic ingredients of the adhesive mortars is an epoxy resin of vinylidene chloride with latex and cement additives. When using adhesive mortars the following points required special attention:

Adhesive of this nature is expensive and should be used sparingly. Application is best done by a caulking gun procedure which puts down a thin bead. The adhesive then spreads under the weight of a superimposed brick unit and permits small adjustments for the tolerances. The pot life of the adhesive mortar must be suitable for both hot and cold weather. The setting time must not be delayed. Bricks should be clean and free of dust. They should be fairly dry and have low, if any, shrinkage characteristics. The thinness of the mortar joint requires the bricks or other clay units to be dimensionally accurate.

Structural testing of brickwork with high tensile strength mortar has been going on for some years now. In one series of tests where this type of mortar was used the crushing strength of walls increased by about 60% over that with conventional mortar and the bond and tensile strengths by about four times. In addition there is evidence of improved moisture resistance which reduces water penetration, a necessary attribute for thin-wall construction, reduced efflorescence and elimination of spalling due to successive cycles of freezing and thawing. In nearly ten years of site exposure no erosion of mortar could be detected and therefore maintenance costs are minimised. Finally, high bond mortar can be applied equally well with traditional tools and techniques and in industrially sophisticated systems.

Recommended methods of testing high bond mortars include testing for compressive, tensile and flexural strengths. At present only American specifications are available as this is where these types of mortar have originated.

For compressive strength at least three 50 mm (2 in.) cubes prepared on site should be tested. The specimens shall be cured 24 h on site and then cured an additional 27 days at about 24°C (75°F) and 50% relative humidity. Twenty-eight day compressive strength shall be at least 41·1 N/mm² (6000 lb/in²).

The tensile strength of high bond mortar shall be obtained by testing at least three figure-eight briquettes cured as above. Twenty-eight day tensile strength shall be at least 5·24 N/mm² (760 lb/in²).

To test the compressive strength of brickwork with high bond mortar three bricks should be stacked one above the other with two horizontal joints. Five such specimens should be tested at 28 days and the minimum crushing strength should not be less than 41·4 N/mm² (6000 lb/in²).

To test for flexural strength seven bricks are cast one above the other and tested at 28 days as a beam with third point loading. Modulus of rupture should not be less than 1·73 N/mm² (250 lb/in²).

7.2. Prefabricated Brickwork

The use of prefabricated building units has been growing steadily over the last decade. Although concrete still leads the field, recent years have seen a rapid development in prefabricated brickwork wall panels. There are several important factors which led to this development. Conventional bricklaying costs have been rising steeply and in many parts of the world there is a serious shortage of skilled bricklayers. Conventional bricklaying is also severely affected by adverse weather conditions. Finally, there is a recognition that, with gradually improving living conditions, manual bricklaying on site with all its discomforts may in future become socially unacceptable.

It has been recognized at the outset that, for prefabrication of brickwork units to be economic, use should be made of every labour-saving device and, whenever possible, of unskilled labour. It might be argued that there is little to be gained from prefabricated brickwork if conventional bricks are still used as they do not lend themselves readily to a fully automated production process which is a vital factor in prefabrication. Experience so far has not borne this out. This is due partly to the extent to which, by means of various ingenious devices, automation has actually proceeded in spite of the intrinsic difficulties, and partly to the compensating factors of improved working conditions, steady output independent of weather conditions, and improved quality of the product due to the vastly improved supervision and quality control.

Prefabricated wall panels are conveniently classified as loadbearing and non-loadbearing, the latter used mainly for cladding. For obvious economic reasons emphasis has from the start been placed mainly on the former type of panel. Here use is made of every advancement in brickwork technology including the use of high bond mortar, reinforcement and even prestressing. Of the many systems currently in use most have developed abroad but notable advancements have also been made in this country. Information on many systems of prefabrication is scanty as many are still pending a patent application.

HORIZONTAL CASTING METHODS

Figure 7.1 shows the plan view of a recent invention which enables a brick panel to be made almost completely automatically. The machine consists of a horizontal reciprocating tray in which rows of bricks are successively added as the tray moves backwards and forward, so weaving a panel. A row of bricks is placed at each end of the tray. A simple spacing device is used to ensure their correct location and thickness of vertical joint. The tray then operates horizontally bringing one row against the next, again using a spacer to ensure correct thickness of the bed joint. Mortar is then pumped under pressure into the spaces to form the joints and the tray moves back again to receive a further row of

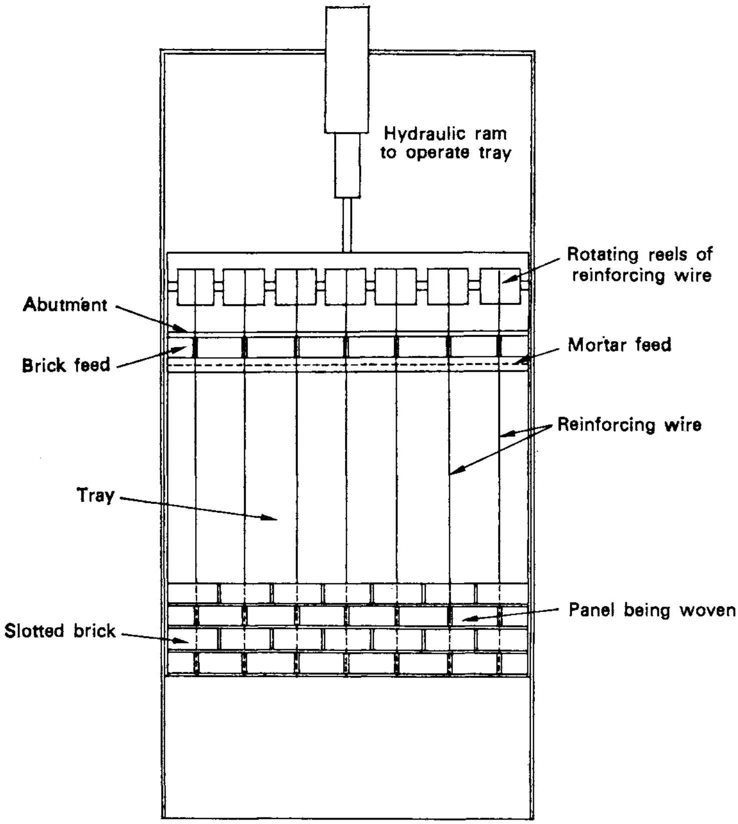


FIG. 7.1. Plan of a horizontal casting machine.

bricks. Reinforcing wires are used to strengthen the panel and slotted bricks are used to accommodate them.

In another horizontal casting method developed in South Africa the casting frame rests on a high-frequency vibrating table. Bricks are placed and clamped in position and mortar or fine concrete is pumped over the vibrating panel. Excess moisture is extracted by vacuum suction. After about 10 min the table is rotated about a hinge to a vertical position and the panel removed. Reinforcing bars, lifting hooks, wall ties, electrical fittings and windows can be incorporated in a panel if required.

The completed panel is pushed by hand on rails to the maturing bay where the edge shutters are removed. In the meantime the table is cleaned with compressed air and returned to the horizontal position ready for the next panel. Panels could be produced at the rate of one every 30 min. The vibrators were operating at approximately 9000 cycles per minute and the total centrifugal force employed was between 1 and 1·5 times the combined weight of the table top and the brick panel. Tests on the prefabricated panels and traditionally built panels showed that the crushing strength of the former was up by a factor of 1·8 and its flexural strength was nearly three times as great as that of the conventional panels. Furthermore, in comparing the resistance to water penetration by horizontal ponding tests, it was found that no dampness appeared on the underside of the prefabricated panels during a test period of 8½ h, while the traditionally built panels became dripping wet within minutes after the water has been poured over them.

VERTICAL CASTING METHODS

In a method developed initially in the United States the wall panels are cast vertically. A series of pins are used to support the respective courses of brickwork. The bricks are then held by friction inside the "box" of the casting machine, the pins are removed and grout is injected to fill the joints. Making due allowances for the suction rate of the bricks it has proved possible to use ordinary cement/lime/sand grouts and to obtain cycles of 1½ h.

THE M-G PLANK—A STOREY HEIGHT UNIT

While bricks and blocks of standard size can readily be assembled into panels in a variety of ways, the production of full-storey height units as a single piece of ceramic has rarely been attempted. The reason for this is that in general heavy clay raw materials present intractable problems in drying and firing large units, certainly those more than 1 m in length.

Recently the British Ceramic Research Association was able to produce in one piece a storey high ceramic unit. In order to do this it was first necessary to solve the ceramic problems of manufacturing to more precise specifications of size and strength. This was achieved by adding to the raw materials certain non-plastic and diluent materials and by modifying the firing and drying processes so that the desirable properties were obtained in the final product.

The M-G planks are 0·30 m wide, 0·10 m thick and 2·6 m long which is the storey height recommended for housing in Great Britain. The planks are made overlong and the ends trimmed after firing to the exact length required. The planks may be joined together to form panels, three or four planks wide. Work is now being carried out on the development of a dry joint in which the sealing material is attached to one side of the M-G plank and the next plank merely butted up. They are particularly suitable as a facing material and are not intended as loadbearing elements although compressive strengths of up to and over 6·89 N/mm^2 (1000 lb/in^2) can be achieved.

All prefabricated brick panels must be strong enough to withstand the stresses imposed on them during demoulding, lifting to storage or later handling on the building site. For normal handling it would be unwise to rely solely on the normal tensile bond between brick and mortar and unless a lifting jig is provided or unless high tensile bond mortar is used, some form of reinforcement should be put in the panel. The actual amount of reinforcement is usually quite small. For a typical 3 m square single leaf panel only three or four vertical bars of 10 mm ($\frac{3}{8}$ in.) diameter would be needed. It is also usual to include some horizontal reinforcement at the base and top of the panel.

Whether the prefabricated panels are loadbearing or not they should in all cases be resistant against rain penetration, frost and sulphates attack and, when applicable, corrosion of reinforcement. As regards the latter, the Code of Practice, CP 111 gives some guidance on the cover required for reinforcement. In no case should the cover be less than 25 mm (1 in.).

7.3. Bricklaying Methods

In spite of the rapid increase in prefabricated brickwork construction manual bricklaying will continue to be the predominant form of construction in the forseeable future. It is useful, therefore, to investigate various possibilities of increasing productivity in this field. Work study techniques have been used in the construction industry with increasing intensity to determine levels of working and finding out more about what is actually done in various operations. As a result of these studies, new methods of working have been proposed. For example, some early work at the Building Research Station suggested that the bricklayer could be helped considerably by certain modifications of his normal method of bricklaying which would minimize his unproductive effort in his capacity as a skilled craftsman. Thus excessive stooping, bending and reaching should be reduced by maintaining the bricks and mortar at convenient working level and using smaller scaffold lifts; eliminating plumbing by using profiles; reducing unnecessary walking by raising the line three courses at a time; bricks should be placed flat, frog up on the platform.

An effective way of increasing the productivity of the bricklayer would be to reduce the time spent on avoidable delays. This means improving the organization on the site and more efficient methods of handling materials. As regards the latter a number of possibilities exist, including the use of hoists, cranes, brick cages and prepacked bricks. Distances over which materials are to be handled should be kept small and, where possible, materials should be handled in bulk.

Other ways of increasing output are to use improved aids to manual bricklaying. Examples of this are a steel framed jig which is designed to conform to the outline of the building. Apart from the direct savings of plumbing and levelling time, other advantages include: a saving in time in erecting lines; removing the need to build up the corners; no need to lay bricks some distance from the line; no need to continually alter the height of the work; and less danger of running out of bricks through bad servicing. Finally, one of the most effective ways of increasing productivity is by ensuring that the bricklayer works for a greater percentage of the time available than he has done in the past. This can be best achieved by suitable incentives, improved supervision and adequate working conditions.

7.4. Prestressed Brickwork

Prestressing techniques are usually associated with concrete but there are numerous examples where it had been successfully applied to brickwork. Records show that it had been used 140 years ago by Marc Isambard Brunel in the construction of brickwork caissons for the Thames Tunnel Project. Since those days developments in prestressed brickwork have been rather slow, due primarily to the extensive use of prestressed concrete. There are signs however that interest in this type of construction is being renewed.

In a recent case it has proved possible to construct 7·34 m (24 ft) high walls in 0·28 m (11 in.) cavity brickwork which would otherwise have had to be of the order of 0·457 m (18 in.) solid construction or would have needed intermediate framing or buttressing. This has been achieved at a labour cost normal for cavity wall construction by post-tensioning the walls. Figure 7.2 shows a section through the post-tensioned wall. High-tensile steel rods were suspended from the high-level fascia beams of the structure and were built into the foundations

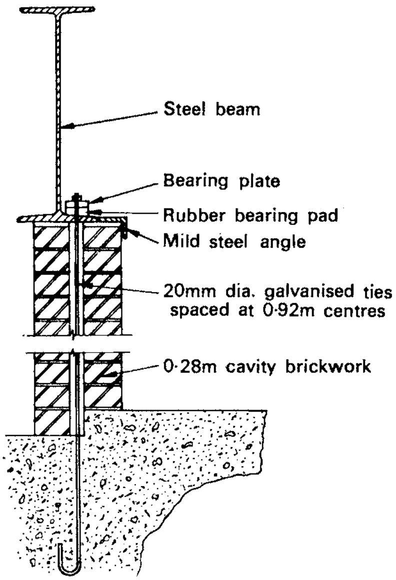


FIG. 7.2. Section through a post-tensioned brickwork wall.

and lightly tensioned. The upper ends of the rods passed through rubber pads and were threaded to receive high-tensile friction grip nuts. As an anti-corrosion measure two coatings of bituminous paint were applied to the stressing rods.

In a still more recent case a static water tank with a capacity of 545·5 m^3 (120,000 gal) was constructed in post-tensioned brickwork. The tank was 10·6 m (40 ft) in diameter, 4·8 m (15·25 ft) high with 0·228 m (9 in.) solid brick walls. The walls were prestressed both vertically and circumferentially against the hoop forces set up by the radial pressure exerted by the water. The method of analysis was that recommended for prestressed concrete as laid down in CP 115 with appropriate modifications being made for creep of brickwork and shrinkage of mortar joints. All stressing operations were carried out when the brickwork was at least 28 days old.

7.5. Design of Brickwork for Accidental Forces

The Ronan Point disaster, in which a gas explosion caused a progressive collapse in a tall block of flats, has focused attention for the need of a new concept which would allow for accidental damage in loadbearing structures. Foremost in ones mind are accidental forces due to gas explosions but other accidental loadings could be also caused by petrol, vehicle impact, earth slip, failure of foundation, earthquake, faulty materials or workmanship and fire.

Recently, full-scale tests were carried out by the British Ceramic Research Association to study the effects of gas explosions on conventionally built brickwork structures. The investigations up-to-date lead to the following conclusions:

The maximum pressure which can be reached in a gas explosion in a closed compartment can be about 0·689 N/mm^2 (100 lb/in^2) but actual pressures occurring in practice are considerably lower, first because the gas/air mixture is not the optimum and only partially fills the compartment and secondly because venting provides relief. The extent of the reduction depends on the amount of venting but it is thought that even under the most severe conditions of venting the actual pressure is unlikely to exceed 0·11 N/mm^2 (16 lb/in^2) and in most cases would fall below 0·014 N/mm^2 (2 lb/in^2). Windows typically fail at 0·002–0·005

N/mm^2 (0·3–0·7 lb/in^2) and a complete door-window cladding at about 0·007 N/mm^2 (1·0 lb/in^2).

The pressure necessary to damage a loadbearing wall depends upon the restraint provided by the superimposed load and its ability to arch horizontally against vertical restraint. In typical cases a 0·114 m (4½ in.) single leaf wall or an 0·28 m (11 in.) cavity wall may withstand a pressure of up to 0·023 N/mm^2 (3·3 lb/in^2).

Following the Ronan Point disaster the Ministry of Housing and Local Government have issued an amendment (known as the Fifth Amendment) to their Building Regulations which specifies the functional requirements designed to minimize and restrict the local damage resulting from accidental forces. Basically, it says that buildings must be constructed so that if any portion of any one essential structural member were to be removed, the consequent structural failure would be limited and localized to the storey of which that portion forms part, the storey next above (if any) and the storey next below (if any). A "portion" of a structural member is defined as that part of a member which is situated or spans between adjacent supports or between a support and the extremity of a member.

An alternative requirement of the Fifth Amendment is that structural members which are not assumed to be removed are capable of withstanding a load of 0·0345 N/mm^2 (5 lb/in^2) with a factor of safety of just over unity.

Some recent tests on axially restrained loadbearing walls have shown that brickwork can, by mobilizing arching action, withstand a lateral pressure of 0·0345 N/mm^2 (5 lb/in^2). However, these results were obtained on cavity walls with the inner leaf 0·18 m (7 in.) thick. It would seem that with normal sized bricks the only practical solution at the present is to design the walls so that any accidental damage can be localized within the limits specified above. It should be noted that the Fifth Amendment is intended to apply only to buildings having five or more storeys.

There are a number of ways in which loadbearing brickwork buildings can be designed so as to localize any accidental damage and thus reduce the risk of a progressive collapse. This can be defined as the spread of local damage to other parts of the structure remote from the point of initial damage, probably affecting the overall stability. One

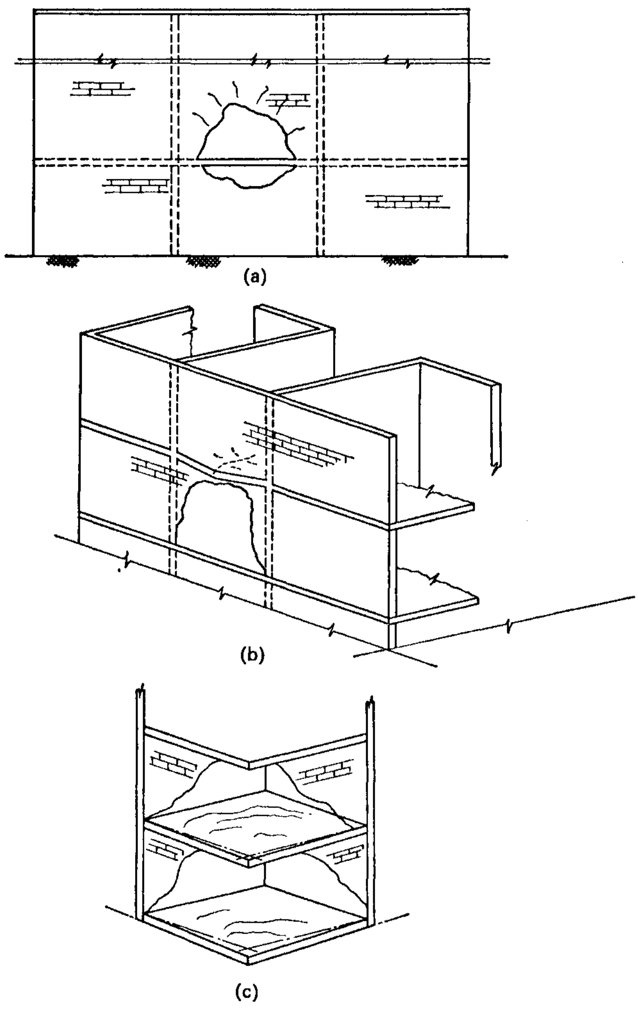


FIG. 7.3. Localized damage due to accidental forces: (a) arching action in brickwork, (b) spanning of floor slab over a damaged section of wall, (c) cantilever action.

way is to choose an overall layout which is inherently stronger than any other. An example of this is shown in Fig. 4.1a in which the supporting walls span in two directions and the floors are monolithic with the structure. For this type of structure the accidental damage to any portion of a loadbearing wall will be localized by either arching action of the wall above, spanning over the damaged zone or by cantilever action of the walls above acting compositely with the built-in floor slabs. These types of action are illustrated in Fig. 7.3a–c.

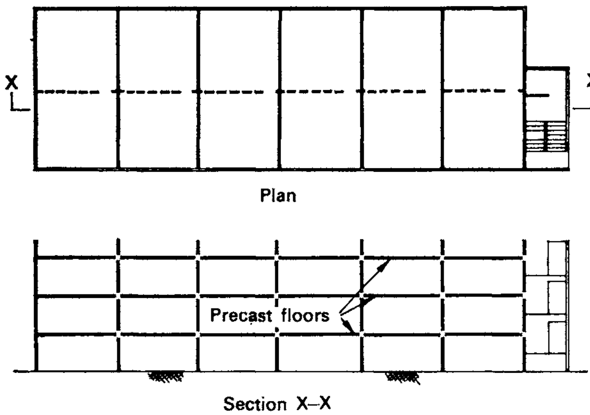


FIG. 7.4. Example of brickwork construction which could lead to a progressive collapse.

At the other end of the scale we have a simple crosswall structure shown in Fig. 7.4. In this there are no return ends on the walls, no spine wall, with the longitudinal stability being provided by a stair tower and the floors are unconnected, simply supported precast units. Removal of a critical length of a wall by accidental force, particularly the gable wall which is the most vulnerable part of the structure, could lead to progressive collapse. One way of improving the stability of such a structure would be to provide a spine wall as shown in dotted lines in Fig. 7.4, thus giving the floor slabs an additional line of support. As a further precaution the end bays could be designed to cantilever using the composite action of walls and floor, or the gable walls could be strengthened with built-in stanchions which would have to be designed for the 0·0345 N/mm^2 (5 lb/in^2) loading, prescribed by the Fifth Amendment, acting on the gable walls.

References

ALLEN, M. H. and WATSTEIN, D. (1970) Structural Performance of Clay Masonry Assemblages Built with High-bond Organic Modified Mortars, Second International Conference on Brick Masonry, Keele University, Stoke-on-Trent, England.

ASTBURY, N. F. (1969) Brickwork and Gas Explosions, *The British Ceramic Research Association Technical Note* No. 146.

FOSTER, D. (1967) Progress in Brickwork Prefabrication—A Comparative Study, *The Brick Bulletin*, Vol. 6, No. 9.

FOSTER, D. (1970) The Design and Construction of a Prestressed Brickwork Water Tank, Second International Conference on Brick Masonry, Keele University, Stoke-on-Trent, England.

HASELTINE, B. A. and THOMAS, K. (1969) Loadbearing Brickwork—Design for Accidental Forces, *C.P.T.B. Technical Note*, Vol. 2, No. 6.

MORTLOCK, D. J. and WHITEHEAD, B. (1970) Productivity in brick and block construction—A literature survey, *Building Science*, Vol. 4, No. 4,

Index

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