Studies Relating To Structural Masonry

by

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Abstract

The thesis comprises of 44 published papers mainly dealing with various aspects of structural masonry. The success of masonry and its use in high-rise buildings and other civil engineering works required better understanding of materials and its structural behaviour. The thesis describes the most comprehensive and systematic research investigation undertaken in area of material science relevant to structure, structural performance and design methodology for plain, reinforced and prestressed masonry. Novel test methods for obtaining tensile strength of individual brick and bi-axial strength of masonry in flexure are described.

Several series of full-scale tests to investigate the behaviours of brickwork subjected to combined compression and shear, multi-storey brick shear wall structure subjected to wind loading, progressive collapse, lateral strength of wall panels with and without precompression, interaction between wall and floor slabs are also described.

Based on the tests of real structures, design methods for multi-storey masonry structures subjected to vertical and wind loading, and to limit the progressive collapse due to accidental loading are recommended. A Coulomb type of equation is suggested to calculate the ultimate shear strength of masonry. A theory to predict the lateral strength of masonry subjected to precompression has also been described.

Methods developed to predict the shear and flexural strengths of reinforced and prestressed masonry are elucidated. A plastic method, similar to concrete, is used for the theoretical prediction of the shear strength of prestressed beams. An analytical technique for calculating the load-deflection and the ultimate moment capacity of reinforced and prestressed sections is also presented. The technique takes into account all the sources of non-linearity; such as non-linear material behaviour, cracking and tension stiffening.

Most of the findings of the research have been incorporated in the national and international standards.

Declaration

a) The 44 publications cited in this thesis are all the result of work done in Edinburgh subsequent to my Ph.D. These publications have not been submitted for any other degree or diploma except publications 5 and 23. Publications 5 and 23 are partially based on data (about 20%) obtained during my Ph.D. and on more recent extensive investigations.

b) I am the sole author of thirteen of the cited papers. As the first or second author in the case of joint authorship with a senior colleague or colleagues from outwith the university, I undertook most of the work from research investigation to writing up the material. My contribution is 80% for the publications 9-11, 13-15, 20, 21, 24, 25, 29 and 32, and 70% for the publications 23, 26, 28 and 36.
In all other cases of joint authorship (publications 2,3, 6-8, 16, 19, 33, 34, 35, 37-39, 41 and 44) in which I am either the first or second author, the work was shared equally between other authors and myself.

B. P. Sinha.
Signature

Studies Relating to Structural Masonry

1. Introduction

Many residential buildings in masonry have been satisfactorily designed and built with empirical rules and practices without the need of special consideration. However, the limits of this approach cannot be extended much beyond the scale of two-storey buildings of conventional construction without using very thick walls, which is very uneconomical. Thus, a major research effort had to be directed towards understanding of, both, the behaviour of material and structure for the application of structural engineering principles to the design of high-rise buildings and other major civil engineering works in plain, reinforced and prestressed masonry and for its economic viability. Therefore, this thesis consists of published papers based on research investigations, mainly concerned with problems related to structural brickwork. The papers are not arranged in chronological order as published, but are divided into four major groups, each dealing with a different aspect of structural brickwork.

1.1 Group 1 - Historical Development: Papers [1 and 2] in this group trace the historical development, past researches and the structural use of plain, reinforced and prestressed masonry. Paper 1 also examines its relevance to a developing country like India.

1.2 Group 2 - Plain Structural Masonry: The papers in this group are divided into four sub-groups as mentioned below.

1.2.1 Properties of mortars, bricks and brickwork: Prior to the publication of the draft BS 5628 Part 1: 1978 dealing with unreinforced masonry, there were no minimum strength requirements for different grades of mortar. Many sands, conforming to the grading limits of BS1200:1955, used successfully prior to the code could not be used as they did not now meet the strength requirements of the mortar.

As the result of the paper [3], the draft was revised according to the suggestions made from the research findings. This work was subsequently extended further to survey all the sands available in Scotland and it was found that 40% of the sands presently used successfully in practice were outside the grading limits of BS 1200: 1955. It was shown that mortar of various grades made from the sands could fulfil the revised strength requirements, hence in the joint paper [4] written with a junior colleague in which the candidate made a substantial contribution; recommendations were made to relax the grading limits set in BS 1200: 1955. This was accepted by the drafting committee and eventually the grading limits were relaxed by identifying sand S and sand G having two different grading limits. The grading of sand not only affects [5] the strength of the mortar, but also the interface bond tension and shear which brickwork is subjected due to lateral loading [5]. This aspect has yet to be recognised by the BS 5628 Part 1.

1.2.2 Properties of bricks and brickwork: Brickwork has normally been used for compression members; such as walls and columns, hence the stress-strain relationship and its compressive strength in bed-joint directions were the subject of investigation for a considerable period of time. This resulted in a standardised test procedure of testing bricks and brickwork strengths perpendicular to the bed joint which served as an index value for the Code.

However, in cases of reinforced and prestressed flexural members; compressive stresses can develop in the other two orthogonal directions. Therefore, investigations [6 and 7] were undertaken to establish the stress-strain curve and the strength of brickwork in three orthogonal directions using prisms of brickwork made of different types of bricks and mortar grades. The prisms were loaded axially. The work was further extended [8] to cover prisms loaded eccentrically at the 'Kern' limit (t/6) to simulate the strain distribution in the bending of a beam and the stress was calculated from the stress-strain relationship established in the axial test. No increase in compressive strength was detected due to strain gradient. The stress-strain relationships and the strength obtained from the prism tests (34 to 39) were used for the first time to predict the behaviour of reinforced and prestressed flexural members

The suggested format for prisms in the paper (7) was incorporated in BS 5628 Part II: 1985 for predicting the strength of reinforced and prestressed brickwork flexural members.

A further extension of the work was to investigate the compressive strengths in three orthogonal directions of brickwork prisms [9] made from perforated and slotted bricks. It appeared from the investigation that the compressive strengths in three orthogonal directions are different and the characteristic compressive strength of brickwork for any grade of mortar was lower than the provisions of the code BS 5628.

Masonry cladding panels supported on three and four sides bend like thin plates under lateral loading and are subjected to bi-axial bending. It is, therefore, necessary to know the strength in two orthogonal directions. Paper [10] describes the tests done to obtain the orthotropic strength. It appeared from the tests that the ratio of the flexural strengths of brickwork parallel and perpendicular to the bed joint was not constant as assumed in B.S. C.P.111. This was recognised in the current limit state code BS 5628, which superseded the CP 111.

The works described in this section were financially sponsored by the Building Research Establishment, the Science and Engineering Research Council and Structural Clay Products Ltd., U.K.

1.2.3 Compressive strength of brickwork strip wall and walls stiffened along their vertical edges: Prior to the Limit State Code, the structural design of brickwork in the U.K. was governed by CP111:1964, which was based on a large number of tests on individual piers and walls and experience gained since 1948 in this field. These tests were carried out on isolated walls or piers which completely ignored the interaction between the various elements in the masonry structures. An investigation [11] was therefore done on walls at 1/6th scale, the validity of which was established earlier, under a realistic situation found in a full-scale building. Generally, the failure of brickwall under compression happens due to lateral tension developed in the bricks, hence *a novel technique* was developed by the candidate to find the tensile strength of bricks and to correlate it with the wall strength. Further, logical steps were taken to extend the work to cover the compressive strength of walls supported along their vertical edges; i.e. I- section or diaphragm wall [12,13]. The investigation revealed that in compression the brick walls with vertical edges stiffened by return or returns or diaphragm walls of various aspect ratios do not show any increase in strength over strip walls at least up to a slenderness ratio of 32. This fact has now been recognised internationally and the suggestion made was incorporated into the international recommendation for the design of masonry structures issued by the Commission W23 of the Council of International Building Research and Documentation.

The research was supported by the Science and Engineering Research Council and the British Ceramic Research Association.

1.2.4 Lateral strength of brickwork panels: The study of the resistance of non-load bearing wall to lateral pressure became an urgent problem due to the upward revision of the design wind pressure (CP 3: Chapter V Part 2: 1972) in the U.K. An earlier lateral load investigation [14] was carried out on cavity-walls with various types of ties, with and without damp proof course and with short return to formulate the design guidance for the Department of the Environment.

Later on in 1978, the code BS 5628 Part 1 was published giving the bending moment coefficients for the design of laterally loaded rectangular panels without precompression based on yield line analysis, developed from the theory applied to reinforced concrete slabs. The brittle nature of unreinforced masonry is directly at odds with a fundamental criterion for yield line, i.e. plasticity. The results of yield line analysis when compared to the test results of panels with and without openings consistently over-estimate [15,16] the failure pressure if the orthogonal ratio for brickwork is interpreted as strength ratio. This is not surprising, since brickwork is brittle material and also possesses stiffness orthotropy. An approximate method [17,18] called fracture line which considers both stiffness and strength orthotropies, was put forward for the design of such panels. The proposed method [17,18] gave very good correlation with the experimental results obtained from testing triangular, octagonal and rectangular panels with and without openings. Though, the method incorporates the stiffness orthotropies, but suffers the same limitations as the yield line method. Any proper mathematical solution of this problem requires the failure criterion of masonry in bi-axial bending. A *novel and pioneering* method [19] was developed to determine the failure criterion of masonry in bi-axial bending. The criterion was incorporated into a finite-element plate bending program, which accurately predicts the strength of masonry cladding panels of various boundary conditions subjected to lateral loading.

The work described in this section was supported by the Property Services Agency, Directorate of Civil Engineering Development and the Building Research Establishment, Department of the Environment.

1.2.5 The Quarry Project: Having carried out research on properties of brickwork and on compressive strength of walls, it was decided to examine the strength and rigidity, and the interaction of the floor/wall junction of a full-scale brickwork large panel multi-storey structure. For this, a disused 'quarry' was developed as a full-scale structural testing station [20]. *The planning, site supervision, the structural design for the loading frames including the test structures, and the development of the quarry was solely carried out by the candidate. The advantage of this unique and pioneering development was that the lateral* loads, to structure up to five storey high, were applied by jacking against the rock face, thus saving the cost of building a large expensive reaction frame. In tests not requiring lateral load, the quarry face provided a rigid support for stabilising structures and a fixed plane of reference for measurements. Several series of full-scale tests were carried out as mentioned below:

i) Shear test on six full scale single storey brick structures: The tests [21] were conducted to study the behaviour of brick structures under combined compression and shear. Six single storey structures with and without openings were tested. It was found that at low precompression, the shear stress increased linearly with precompression and a Mohr-Coulomb type equation was proposed to predict the shear stress at failure. The result of this work formed the basis for the revision of the then British Standard Code CP111:1970.

ii) Behaviour of multi-storey brick structures subjected to wind loading: In high rise buildings, one of the major problems is to ensure adequate strength and rigidity under wind loading. In a brick building, wind loads are resisted by a complex system of shear walls inter-connected with slabs and it was, therefore essential to know the actual behaviour of such structures. For the design [22] of such buildings a simple method is adopted in which lateral moments are apportioned between the shear walls present in proportion to their flexural rigidities. A more refined method takes into account interaction between the shear walls and inter-connecting floor slabs or beams on the assumption of fully rigid connection between the various elements. The applicability of the various design methods was examined by testing 1/6th scale model and a full-scale 5-storey brick cross-wall [23,24] structures subjected to lateral loading. From these tests, it was inferred that the best approximation of the actual behaviour of a brick shear wall structure is obtained by replacing the actual structure by an equivalent frame in which the columns have the same sectional properties as the walls and the interconnecting slabs span between the axes of columns.

iii) Progressive Collapse: The Ronan Point disaster in 1968 aroused interest in the progressive collapse of the multi-storey structures and concern was voiced in press and public on the integrity of similar buildings in brickwork following accidental damage. Full-scale tests [251 were carried out at the 'quarry' in which a major load-bearing element was removed in the ground floor from the experimental five storey building to examine the susceptibility to progressive collapse in the event of accidental damage. Prior to these tests, the stability of the building following the removal of a section of cross-wall was assessed by a method suggested in paper [25] and tests confirmed the theoretical prediction that the structure would remain stable.

iv) Lateral strength of wall panels subjected to precompression: The lateral load design of panels without precompression was dealt with earlier [Section 1.2.4]. It was decided to investigate the strength of panels with precompression in realistic boundary conditions. The experimental 5-storey structure was modified and used for lateral loading tests of panels [26] with and without returns subjected to precompression. As a result of these tests, a simple method based on the principle of an internal three-pin arch [261 was put forward for the design of such panels. The design method has been incorporated in BS 5628 Part 1.

v) **Floor/wall interaction:** The CP111 and its successor BS 5628 does not offer the designer proper guidance for the calculation of eccentricity due to floor loading, the load distribution between two leaves of a cavity wall and inadequate guidance relating to effective height. In a multi-storey building, the effective height of a wall and eccentricity imposed on it depends on type of loading, disposition of the wall in a building, relative stiffness of floor slab and wall and end fixity due to precompression. The assessment of these factors is difficult due to lack of data, hence tests on one 1/6th model [27] and two full-scale [28,29] multi-storey structures were done. From the tests, it was found that the inner-leaf of the cavity wall carries most of the floor loading; the outer-leaf shares the moment according to stiffness, but carries only 7 to 10% of the floor loading. The effective heights and the eccentricities due to floor loadings varied throughout the height of the structures. It seemed from the tests that the multi-storey structure can be idealised as a rigid frame for sophisticated design calculations termed as level 2 design method. This has been adopted for international recommendation for masonry structures issued by Commission W 23 of the Council of international Building Research and Documentation. Most of the papers [13,14 & 21 to 28] in Sections 1.2.3 and 1.2.5 resulted from the tests done by the candidate. The papers have been published jointly with a Senior Colleague, but the candidate made the major contribution in writing them.

The research was sponsored by the Science and Engineering Research council, the British Ceramic Research Association, the Building Research Establishment, the Brick Development Association and the Structural Clay Products Bureau, U.K..

1.3 Group 3 - Reinforced Masonry: The papers [30 to 33] in this group mainly deal with reinforced grouted beams and slabs for which no comprehensive data were available. This form of construction has advantages over ordinary reinforced beam, as the shear and main reinforcement can be accommodated in the cavity before grouting. The effect of variables, such as brick and mortar strength, percentage area of steel and shear arm/effective depth ratio on the behaviour and ultimate strength in bending and shear were considered. A method of predicting the ultimate moment [30] by using the strength and stress-strain relationship derived from prism tests was proposed. The result of this work formed the basis of shear stress provision in the BS 5628 Part II: 1985. Two of the papers [32, 33] in this series have been written jointly with colleagues from outside the University, whose contributions were minimal.

The work was financially sponsored by the Building Research Establishment, Department of Environment, and the Structural Clay Products Ltd., U.K.

1.4 Group 4 - Prestressed Brickwork: Traditionally, brickwork has been used for compression members because of its considerably high compressive strength compared to its tensile strength. Prior to this work, no serious attempt was made to apply the principle of prestressing, like concrete, to brickwork so that it can be used as flexural members to resist the load primarily in bending. A comprehensive investigation [34 to 37] was carried out to examine the effects of variables as mentioned in Group 3 on deflection, cracking and ultimate load behaviour of post-tensioned brickwork beams, so that it can be used in practice. A computer program was developed to predict the flexural behaviour of the beams up to failure using the non-linear [37] stress strain curve obtained from the prism tests [7, 8, 41] and taking tension-stiffening into account. This work was further extended to study the behaviour of partially prestressed [38,39] beams. A brief investigation [40] was also carried out to study the comparative structural behaviour of both prestressed brickwork and concrete beams of similar sectional properties, prestressing force and steel areas. It was found that the structural performance of the beams made of both materials was similar and they failed practically at the same load.

Brickwork is anisotropic, hence the prestressing force could be applied normal or parallel to the bed-joint. For these cases, the flexural and shear strengths will be different. In most of the works described earlier, the prestressing forces were applied parallel to the bed-joint. In the case of retaining walls, the prestress will be applied normal to the bed-joint, hence an investigation was carried out to supplement the data on prestressed brickwork [42]. Due to the lack of data for prestressing normal to bed-joint, the code uses the value obtained from shear tests of walls subjected to precompression. The work described in the paper [43] has shown that this is not correct and suggested amendment based on the test results. The shear strength of prestressed brickwork beams [44] is very closely predicted by plastic failure theory recently developed for concrete beams.

The candidate was the principal investigator for the research done in the field of reinforced and prestressed masonry supported by *the Science and Engineering Research Council, the Building Research Establishment, the Structural Clay Products Ltd and the Brick Development Association* . But, the papers in this group and some in section 1.2.2 have been written jointly with junior research associates, to which the candidate has made major contribution as the sole guide and the originator of the proposal.

2. List of Publications

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6. R.F. Pedreschi and B.P. Sinha, "The Stress-strain Relationship of Brickwork", 6th. Int. Brick Masonry Conf., May 1982, Rome pp. 321-325.
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18. B.P. Sinha, "An Ultimate Load-Analysis of Laterally Loaded Brickwork Panels", The lnt. Jnl. of Masonry Construction, London, 2nd Issue, 1980, pp. 57-61.
19. B. P. Sinha, C. L. Ng and R. F. Pedreschi, "Failure Criterion And Behaviour of Brickwork In Biaxial Bending", Jnl. Mat. in Civ. Engineering, ASCE, May 1997, pp. 70-75.
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Historical Development of Structural Masonry

(Group 1 - Papers 1 & 2)

HISTORICAL DEVELOPMENT OF STRUCTURAL BRICKWORK, ITS POTENTIAL, AND ITS RELEVANCE TO INDIA

B.P. Sinha *

Masonary bearing wall structures have been used for centuries for all types of buildings, from the small simple shelter to some of our magnificant monuments and public buildings.[1] In 1889-91, this type of construction reached its peack in Chicago in the shape of the Monadnock[1,2] Building (Figs 1 and 2) built in brickwork with 182 cm thick walls. The walls were load bearing, but the design was based on rule-of-thumb and not on structural calculations. This type of construction and the design concept had no place in the world of the competitive market.

CAUSES OF DECLINE

The two factors; huge rise in the cost of land and the bricklayer's strike in Chicago, forced William Le Baron Jenny (1893) to develop the structural steel frame as an alternative to brickwork for the construction of the Home Insurance Building. The develoment of the structural frame replaced the structural use of brickwork in multi-storey buildings and limited its use to a curtain wall to support its own weight. The discovery of glass and lightweight partition in the middle of the 20th century, both led further onslaught and replaced brickwork for many modern constructions. Thus the years 1900-1960 can very well be called the era of the structural frame.

REVIVAL OF STRUCTURAL BRICKWORK

In comparison with brickwork buildings, the all glass building is devoid of textural warmth, colour and is most unsatisfactory from the environmental point of view.

During the 1960's, somewhat earlier in Switzerland[2,3] the architects and engineers turned the clock back again and started using brickwork for frameless multi-storey buildings, utilizing its structural strength and aesthetic qualities. The economic advantage of frameless construction coupled with the revolt against the appearance of concrete helped multi-storey brickwork to come back on the scene. These buildings were no longer treated as a piece of traditional craftmanship, but analysed according to the similar techniques as have been used previously for steel, reinforced concrete and timber.

Switzerland, having no governmental bye-laws or Codes, and no indigenous steel industry became the pioneer in the revival of brick masonry for multi-storey construction. Some 1600 wall specimens[5]) were tested at EMPA which helped in the design and construction of 13-storey apartment buildings in Basle (1951-53). Based on the test results in 1957, the tallest load-bearing 18-storey building (Fig. 3) supported on relatively thin walls was built in Switzerland and since then this type of construction has become the norm all over the world as a result of its flexibility, economy and speed of construction.

In the United Kingdom between 1926-34 intensive research[4] was carried out on square brick piers at the Building Research Station which apart from other things established that mortar strength does not significantly affect the brickwork strength (fig. 4). However, the potential of brickwork was not harnessed to any appreciable extent until 1960. The construction of a 12-storey flat at Birmingham, and the Swiss experience had great impact, which resulted in the revision of the 1964 Code paving the way of more extensive use of structural brickwork in the U.K. It would not be out of place to mention that the Indian Code of Practice is largely based on this out-dated Code, which some years ago was withdrawn and replaced by the limit state code. It is unfortunate that no real efforts have been made to develop a Code of Practice on Structural Brickwork in India based on their own tests.

Denmark, like Switzerland has no steel industry of its own, hence its building economy was

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also suited to the development of brickwork. In 1963, a 16-storey building supported on 35.5 cm thick walls was built.

Although experiments were carried out on walls and piers to establish the strength of brickwork as early as 1915 in the U.S.A., nothing very exciting happened until 1950. Between 1944-1951, while construction cost rose by 43%, there was an overall fall in the quality and comfort standards; living room sizes were reduced from 14 sq.m. to 13.5 sq.m., kitchen and dining rooms from 8.4 sq.m. to 6.5 sq.m., and bedrooms from 11.6 sq.m. to 11.2 sq.m. It became apparent that something had to be done to arrest the rising cost of construction without further reduction in the comfort and quality standards expected from houses and homes. Therefore, the Home and Housing Finance Agency sponsored a project with a view to arresting the cost by better planning and design of buildings.

NEW CONCEPT IN DESIGN :

As a result of this investigation, Fisher and Associates[7] suggested that the houses should be built on a cross-wall principle with adequate shear walls, so that wind blowing from any direction can be resisted by all the walls present in a building (Fig. 5). This was quite a new concept compared with the principle on which the Monadnock building was built, where the exterior wall was designed on the middle third rule (Fig. 6) to resist all the wind loading with no co-action between the slabs and walls present in the building. The additional advantage of this new concept was that masonry could be used both in compression and shear. A typical 16-storey brickwork building[3,6] 6) built in the U.S.A. on this principle with 30.5 cm brick walls is shown in Fig. 3.

In the West, attempts were made to arrest the cost by research and development without cutting the comfort and quality standard, whereas in India this has invariably been done by cutting down the specifications and quality in the name of low cost housing. Only the National Building Organisation (NBO) tried to take advantage of this development in the field of structural brickwork; and in the early 70's persuaded some constructional agencies to build experimental[8] 4 to 5-storey houses in 23 cm thick walls (fig. 3).

The concept put forward by Fisher and Associates (7), revolutionised the use of brickwork and research since then moved to examine the strength in conjunction with the interaction of brickwork with other elements of structure.

In the U.K., a 'disused' quarry[9,10] was developed in Edinburgh where full-scale structures up to 5-storeys (fig. 7) could be tested under every aspect of loading. Between 1967-1979 numerous tests were carried out on brick structures to study the effect of wind loading, eccentricity of loading, accidental removal of members and many other factors, all under realistic conditions. The partial collapse of the Ronan Point large panel concrete building due to a gas explosion led the British Ceramic Research Association[9] to examine the problem in brick buildings under realistic situations. Several explosion tests on full-scale buildings were carried out in which rooms were filled with gas. Due to venting, the failure pressure recorded never reached 35 kN/sq.m. as now required for the design. It also demonstrated the ability of brick walls to sustain the lateral loading due to arching between the supports.

All these developments in the field of construction and research were mainly concerned with domestic buildings, schools and dormatories. In recent years, brickwork frameless buildings with cellular wall construction has started to compete with steel and concrete in large scale building such as factory buildings, sports complexes etc. No doubt, brickwork has become a dominant structural material in the West, its potential in the Civil Engineering field has yet to be realised. In the Victorian era, it had been used for retaining walls, dock structures, castles, warehouses and the foundations for all types of buildings. The clock can not be put backward to return to the massive gravity type of structure, because of the cost of labour and material. It would be possible to build slender structures and members which carry loads primarily due to bending by reinforcing or prestressing the brickwork; which may be cheaper alternatives to concrete or steel.

REINFORCED AND PRESTRESSED BRICKWORK

Reinforced brickwork was first used in 1825 and since then has been used in areas subjected to seismic loads. In the 1920s Brebner used reinforced brickwork extensively for slabs, beams and columns in the states of Bihar and Orissa. Some of these had to be replaced within 40 years because of the corrosion of steel and spalling of the brickwork. Because of the short life of these structural members, the use of reinforced brickwork suffered in India in the public sectors although it is still largely being used in the private sector in U.P. Earlier, the reinforcing steels were used in the mortar joints coupled with bricks of high water absorption by Brebner; which led to the corrosion of steel. It would be possible to protect the steel by using[9] Quetta bond or grouted cavity construction or pocket type construction in which the voids or pockets containing the reinforcements can be filled with concrete. This type of construction is suitable for slabs, retaining walls, etc. (fig.8).

The reinforced brickwork retaining walls have proved cheaper[9] and more acceptable than concrete in the U.K. Reinforced brickwork in some situations can fail in shear and it is very difficult to reinforce it against that. This deficiency could be overcome by prestressing. Prestressed brickwork up until now has not been used as extensively as concrete. Some prestressing of brickwork by using threaded bar in the cavity (fig. 8) has been used to enhance the wind resistance of walls in the U.K. A prestressed circular water tank and windmill (figs. 9, 10) were built as demonstration models by Structural Clay products Ltd. of the U. K. Some years ago, a research and development project (11) investigating the behaviour of prestressed brickwork beams was undertaken at the University of Edinburgh. The beams were entirely made of brickwork and only nominal concrete core was used for grouting. These beams were 6m long and tested as simply supported (fig. 11) to develop the theory and design method for their use in practice. A further extension of this work was to undertake a comparative study[12] of the behaviour of similar beams made of brickwork and concrete. It appears from the test results that prestressed beams made of brickwork and concrete fail at similar ultimate moment and their load-deflection relationship is similar (fig. 12). As a result of this work, it would be possible to use prestressed brickwork for flexural members, such as beams, slabs, retaining walls, etc. This development has tremendous scope in any developing country, where there is a shortage of cement and aggregate. In India, reinforced concrete lintels, beams and slabs are used even in the housing sector, it could be replaced by prestressed brickwork, probably a cheaper alternative. Prestressed brickwork sections can be built on site or in a factory.

STRUCTURAL BRICKWORK AND ITS RELEVANCE TO INDIA

India is facing an acute housing shortage (30.5 million units at the beginning of the 7th five year plan) and also facing the scarcity of materials such as steel, cement, and energy. Under the circumstances, it is advisable to use low energy input material wherever possible. Structural brickwork, being labour intensive and low energy input material, is the most appropriate technology for India. Table 1 shows the requirement of energy for production of different construction materials. Although the data is from the west, it may be applicable to India also. In addition,an economic research carried out in the 1980s under the sponsorship of the Overseas Development Agency at Glasgow University, has shown that the use of structural (load bearing) brickwork generates more employment, and at the same time reduces the cost of construction in the housing sector in India.

Table 1

Energy requirement in production of construction materials

Material	Energy Kwh/tonne
Steel	8,600
Aluminium	70,000
Glass	3,400
Cement	2,000
Bricks	700

Recent estimates suggest that the investment of 1 crore rupees in residential building construction generates direct employment on site of 565 man-years of skilled and unskilled labour at 1983-84 wage rate. Although brick is a readily available indigenous material for construction, and is often the mainstay of construction for roads and buildings, its wasteful use is not uncommon because no serious attempts have been made to exploit its potential, structurally and effectively, due to the lack of research and development efforts in this field. The current Indian Standard for the structural design of masonry seems to be based on some overseas standards with all their shortcomings and specification unattainable in India. This situation can only be improved by research and development in this important technological field which suffers due to two major reasons :

1. Lack of Education and Training :
   The education and training imparted by the teaching establishments in India is mostly geared to the use of high energy input prestigious materials such as steel or concrete. The masonry is looked on as inferior and craft based and as a result the investigation on its scientific use suffers.

2. Inability of Manufacturers to sponsor research and development work and production shortage : While in the developed countries the brick industries are highly organised and spend large sums on promotion, scientific and development research to compete with other constructional materials, this is lacking in India. The brick industry in India is organised on a small scale, like most handicraft and small scale industries lacking resources to sponsor development research. Furthermore, the shortage of material has meant that whatever is produced is bought by the consumer, hence the manufacturer does not see the importance of research investigations. This has also led to scant regard to dimensional co-ordination of brick units. As a result, the practice has developed to hide the defects due to bricks in walls behind thick plaster. If the dimensional accuracy of brick is strictly enforced, there will be a saving in cement and the cost of construction, since building and housing construction generally alone account for one-quarter of the total planned expenditure on construction.

The tremendous potential offered by this technology which is being harnessed in the West, can only be utilised in India provided research and development efforts are strengthened in this field. To ameliorate the situation it is strongly suggested that the Government of India may set up an independent Indian Institute of Masonry Research to take up urgent investigation for optimising the use of this indigenous and low energy input material.

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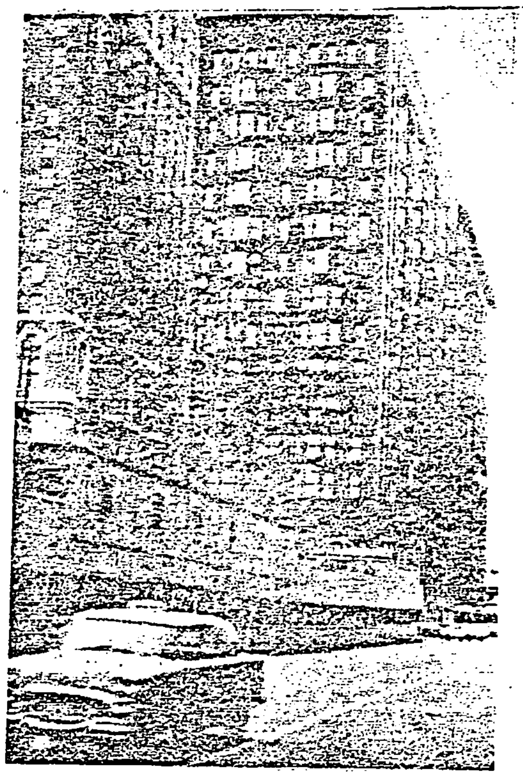


Fig.1 : Monadnock Building in Chicago

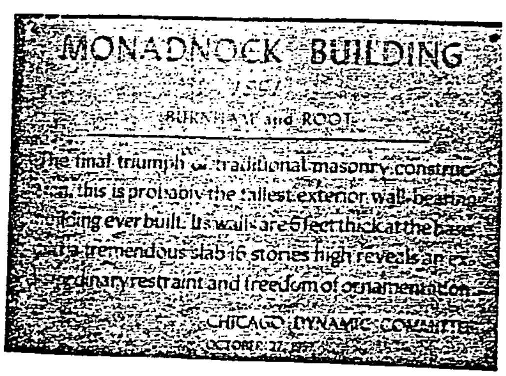
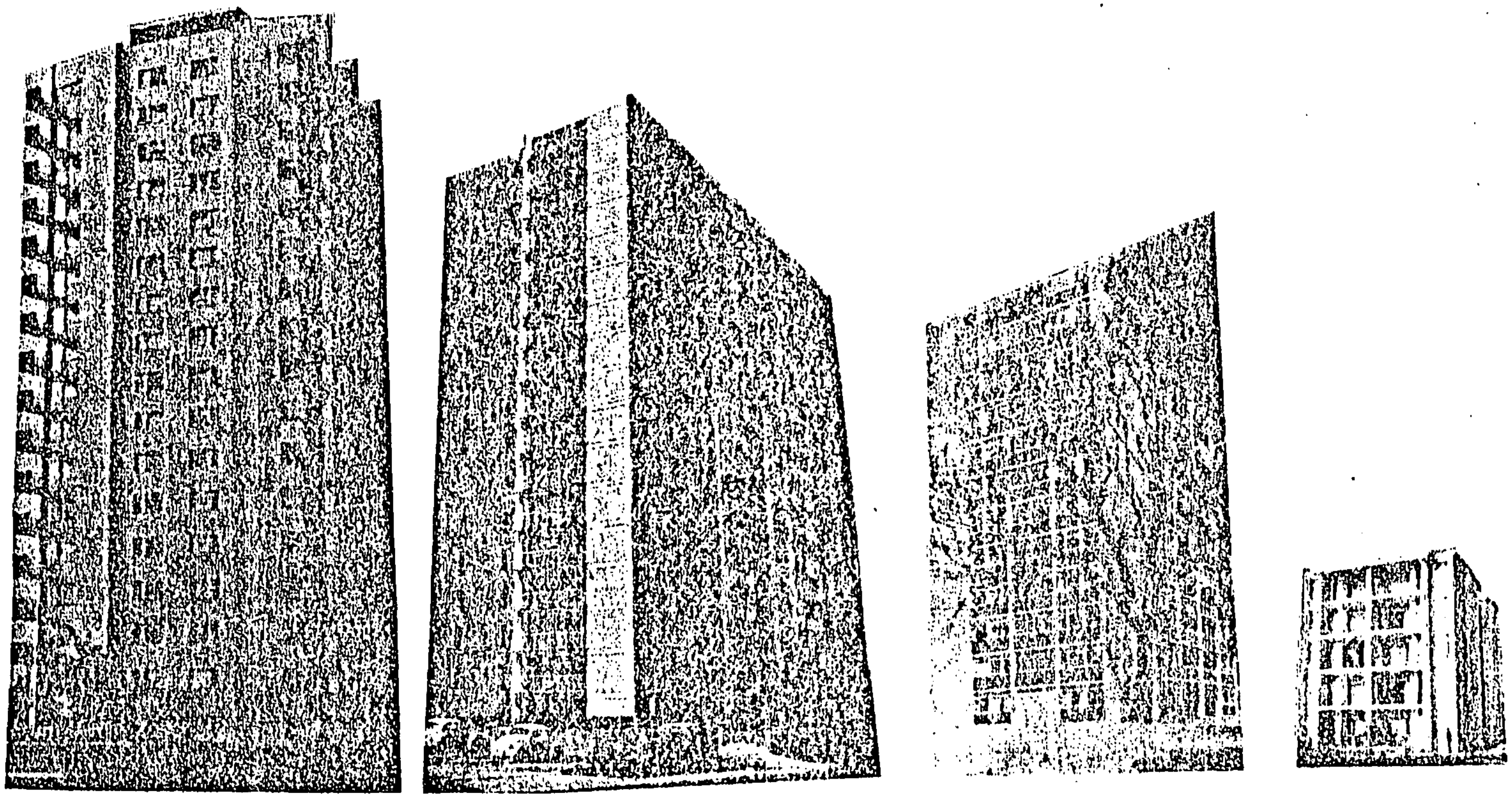


Fig.2 : Plaque put up by Chicago Dynamic Committee



Switzerland, 1957
18 storey building
Wall thickness 15 cms.
(based on test results)

U.S.A. 1965
16 Storey building
Wall thickness = 30.5 cms

U.K. 1961
12 Storey
Wall thickness 22 cms
(designed based on
1948 code)

India 1976
5 storey
wall thickness: 23 cms

Fig. 3 : Shows the multi-storey structures built in Brickwork

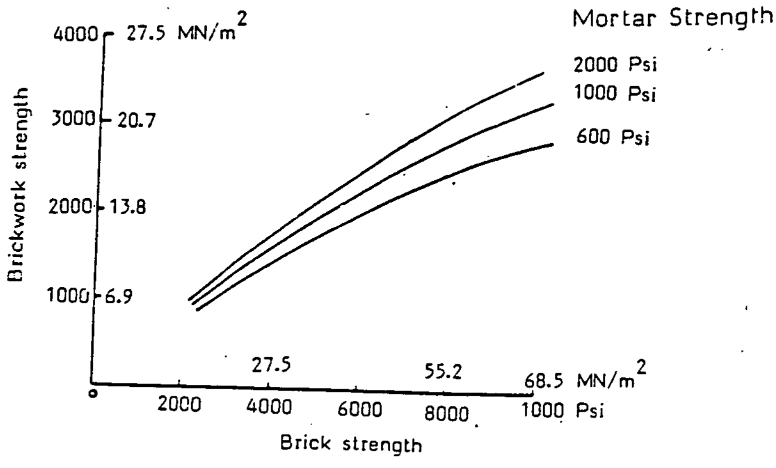


Fig. 4 : Showing the effect of mortar strength on brickwork

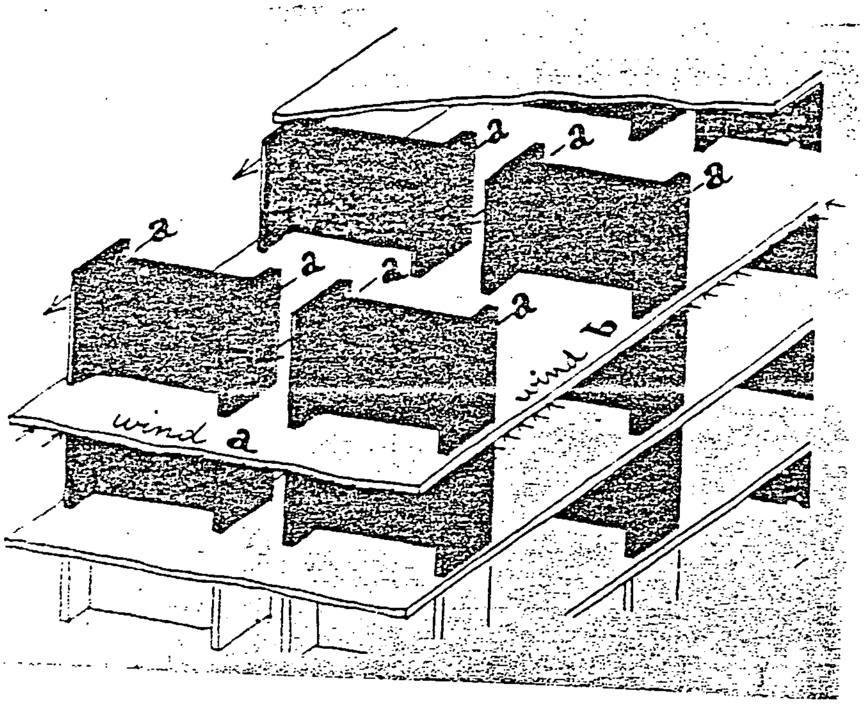


Fig. 5 : Shows the directions of wind and the resisting wall elements

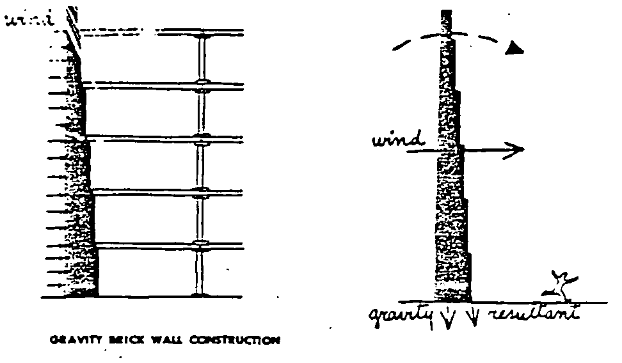


Fig. 6 : Gravity brickwall construction by middle third rule

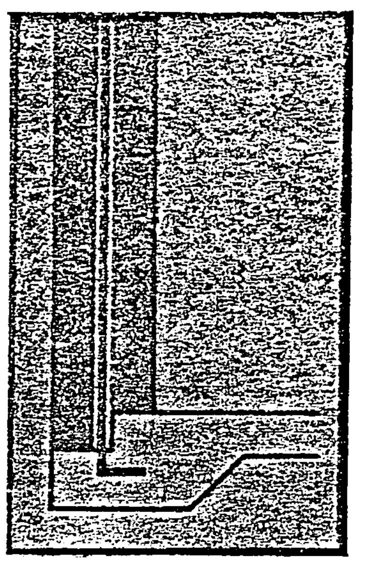
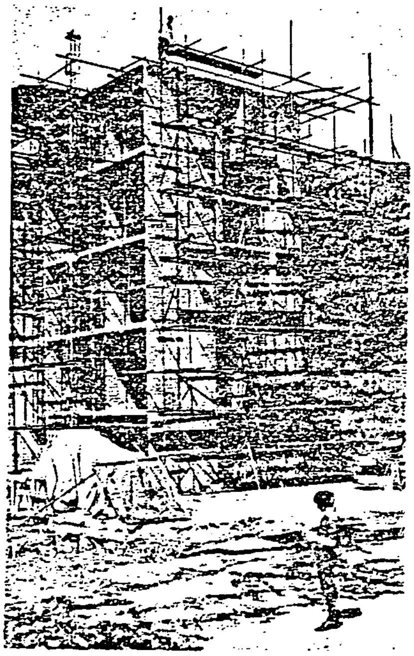


Fig. 8 : Showing simple form of prestressing



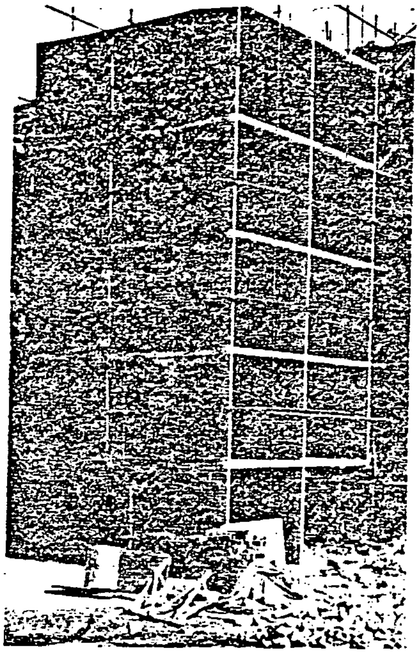


Fig.7 : (a) Test structure in disused' quarry

(b) Test structure after removal of an end load bearing wall in the G.F. (No progressive collapse)

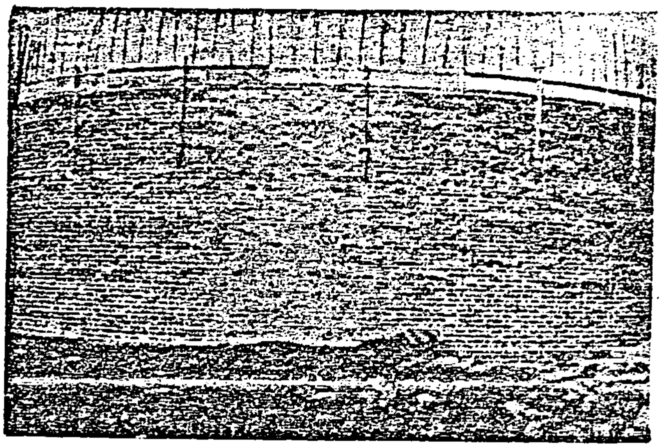


Fig. 9 : Prestressed brickwork water tank

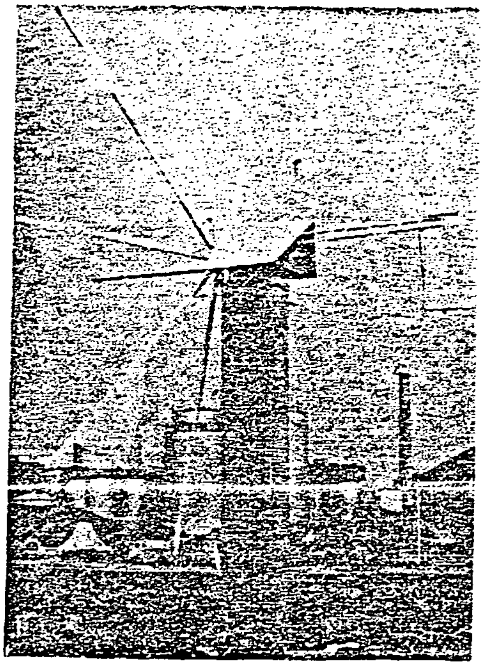


Fig.10 : Prestressed brickwork wind mill tower.

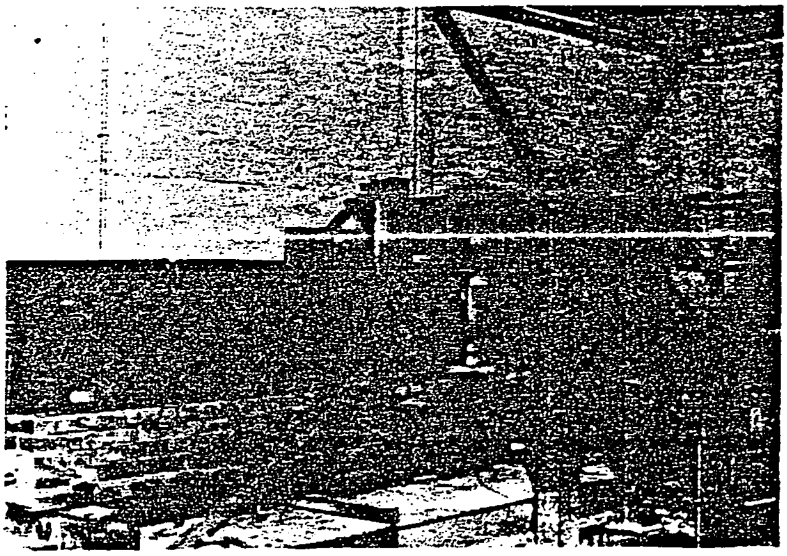


Fig. 11 : Prestressed brickwork beam under test (span 6m)

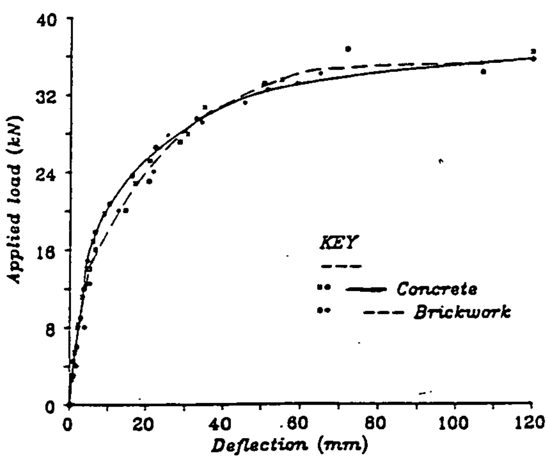


Fig. 12 : Showing the load-deflection relationship of prestressed concrete and brickwork beams.

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Reinforced and Prestressed Brickwork

B P Sinha, *Non-member*

R F Pedreschi, *Non-member*

The paper presents a chronological review of the literature of research work on reinforced and prest ressed brickwork. A brief summary of the most significant contributions is given. A comprehensive list of references is provided at the end of the paper.

INTRODUCTION

Because of the very low tensile strength of brickwork, any structural brickwork element subjected to tension has to be reinforced or prestressed. Brickwork can be reinforced either by placing reinforcements in the mortar joints, pocket or in the cavity as shown in Fig 1. Prestressing of brickwork is achieved by applying precompression to counteract either some or all of the tension that would otherwise develop under service loading.

The use of reinforced brickwork goes back a long way, being first used by Sir Marc Isambard Brunel in the Thames Tunnel[1] in 1825 and the forthcoming section of the paper deals with reinforced brickwork. Prestressed brickwork has developed relatively recently and is considered later. The last section of the paper deals with durability of reinforced and prestressed brickwork.

REINFORCED BRICKWORK

In 1837 Pasley[2] carried out experiments on unreinforced and reinforced brickwork beams, 2 bricks wide and 4-course high, with reinforcement placed in each mortar joint. The load carrying capacity of the brickwork beam was enhanced many times due to reinforcing. He also conducted experiments to show that cantilever staircase can be built in reinforced brickwork. An empirical method was developed for the design of reinforced brickwork elements on the basis of these experiments.

In 1853, reinforced brickwork was used to build a reservoir in Georgetown, USA. R F Tinker (1905) used reinforced brickwork as formwork and facing for the construction of a dam resulting in huge savings in the cost of construction.

Earlier pioneering research work of Brebner (1919) in India and subsequently of Fillippi (1930–32) in the USA and Kanamouri in Japan led reinforced brickwork into the forefront of construction. In 1922, the Public Works Department, Government of India, published the results of 282 tests on beams and slabs conducted by Brebner[3]. The tests confirmed that the behaviour of reinforced brickwork is similar to reinforced concrete and the reinforced concrete design theory can be applied to the design of reinforced brickwork. Later Plummer and Blume[4] arrived at similar conclusions. As a result of Brebner's work, 279 000 m^2 of building were constructed at Patna, the state capital of Bihar.

Brebner also tested axially loaded[3] square, circular and fluted brickwork columns, six-course high, with and without longitudinal and lateral steel. The percentage of the longitudinal steel varied from 0 to 1% and the lateral steel from 0 to 5% for square and from 0 to 6% for circular columns. The load-carrying capacity of the square and circular brickwork column increased by 62% and 36% respectively due to incorporation of lateral steel only. The addition of longitudinal steel with lateral steel showed no significant improvement in the strength of columns. The failure of fluted columns was due to lateral local instability of the longitudinal steel as no stirrups were provided in these columns.

In Japan, RB retaining walls were used for canal, quay walls and harbour entrances. From the investigation on reinforced brickwork, Kanamouri[5] (1930) also concluded that the principle of reinforced concrete design is applicable to brickwork with the modification of the modular ratio to 25 for reinforced brickwork instead of 15 as used for concrete. The results show that the inclusion of 0.3% of steel by volume of brickwork increases the load-carrying capacity of the member by four to five times.

Following Brebner, Fillippi[6] (1932) tested reinforced brickwork slabs and beams. His tests have shown that the measured deflection was less than allowable under the Chicago Building Regulations and the recovery was 80% on removal of an applied load, equal to 1.5 times the design load. Fillipi used reinforced brickwork to build a number of railway bridges and trestles.

Between 1922 and 1935 reinforced brickwork became the subject of research in many educational establishments of North America. Whittemore and Dear[7, 8] (1922) tested 30 slabs of 95.4 mm thickness in 1 : 1 : 6 cement : lime : sand mortar and also compared its behaviour with concrete slab. The initial failure of RB slab happened before attainment of allowable deflection and an ample factor of

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safety against ultimate load was obtained. It was reported that the percentage recovery of deflection past the design load was good. It was also concluded that th behaviour of reinforced brickwork is similar to reinforced concrete and the design can be done on the principle of reinforced concrete, which reconfirmed the earlier works of others[3, 5]

Danforth (1924) tested 22 RB beams, but no tangible conclusions were drawn.

Parsons, Stang and McBurney[9] (1932) investigated the shear strength of 18 RB beams using two types of bricks with percentage areas of the longitudinal steel equal to 1%. They also tested associated small scale specimens to obtain interface tensile and shear bond strength between brick/mortar and pull-out tests to ascertain the bond strength between brickwork and steel reinforcement. The shear strength of the beams varied from 0.45 - 1.1 N/mm^2.

Withey[10] (1933) presented the test results of 25 beams having varying percentage of tensile reinforcements from 0.5 to 2.5 percent and also consisting of variable percentage of steel stirrups. Most of the beams failed due to the yielding of steel. Three beams with high percentage of steel failed in compression. Again the conclusion reached was that the extreme fibre stresses, shear, bond and deflection of reinforced brickwork beams can be calculated according to the formula used for concrete taking into account appropriate constants for brickwork. Thus these American results[7–10] affirmed and reaffirmed the earlier works[3, 5] done in the twenties.

Lyse[11] (1933) reported axial load tests on 33 columns of slenderness ratio of 9.6 with central grout core. The percentage of longitudinal steel varied from 0 to 2 percent for mild steel and 0 to 0.67 percent for high tensile steel. The percentage of steel stirrups varied from 0 to 1% and the stirrups were placed in either the first, second or third course.

Withey[12] (1935) tested 32 brickwork columns of slenderness ratio of 5.8 with and without longitudinal steel and stirrups. The format of the columns were similar to Lyse's. The percentage of the longitudinal steel varied from 0 to 4.1 percent and the stirrups varied from 0 to 1.52%.

Both investigators[11,12] recorded increases in the ultimate load carrying capacity of the RB columns and a significant difference in the mode of failure of unreinforced and reinforced brickwork columns. The failure of columns without reinforcement was brittle. The failure of reinforced columns was due to vertical cracking and spalling of the brickwork. In many cases the stirrups helped in confining the reinforcement in the grout core and the failure was due to peeling off or spalling of surrounding bricks.

As a result of these investigations, by the mid 1930s most buildings in the San Francisco area used reinforced brickwork to resist the earthquake forces.

Hamman and Burridge[13] (1939) reported tests on overreinforced brickwork beams incorporating heavy shear reinforcement to delay premature shear failure. The beams were built from two types of bricks of different physical and mechanical properties. Although, the beams were overreinforced, the failure did not happen in flexural compression preceeded by extensive shear cracking. Their conclusion was that elastic theory can be applied to RB flexural members. However this was not different from methods already postulated by earlier researchers[3, 4]. With high percentage of area of steel shear failure was predominant.

In the Building Research Station, UK, Thomas and Simms[14] (1939) reported a series of comparative tests on reinforced brickwork and concrete beams of similar cross-sectional areas and having similar percentage of tensile reinforcement. Tests were also done with higher percentage of steel and with or without shear reinforcement. No significant difference in the load-deformation behaviour or in the ultimate strength of RB and RC was found for the beams of similar cross-section with low percentage of steel. This is not surprising, since the failure of beams was due to the yielding of steel, hence the strength will depend on the forces developed in steel reinforcement, as the strength of the brickwork exerts only a secondary effect. The shear strength was found to increase as the shear span to depth ratio decreased. With high percentage area of steel, failure was due to shear rather than flexure.

To economise in the use of steel for the construction of single storey factories during the second world war, the use of reinforced brickwork columns was necessary. Tests under eccentric loading were done at the Building Research Station on brickwork columns incorporating four longitudinal bars. The results were reported by Davey and Thomas[15] (1949). Very high tensile and compressive stresses developed in steel and brickwork in these tests.

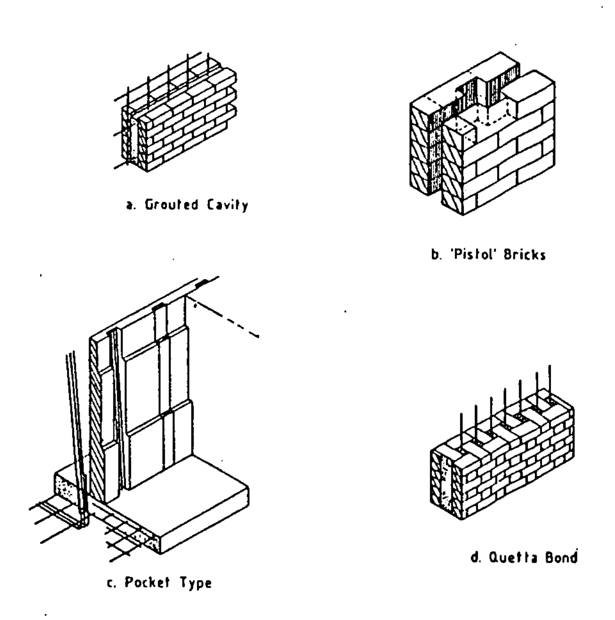

Fig 1 Various methods of reinforcing brickwork

Schneider[16] (1951) described an investigation into RB beams built with common and facing bricks to study the effect of workmanship and two types of mortar mixes (1 : 03 : 4.5 and 1 : 0.16 : 3 cement : lime : sand) on the shear strength. He concluded that the brick strengths have no influence on the shear strength. The recorded deflections of the beams were all less than 1/360 of the span at the ultimate load. The beams of good workmanship built with low grade mortar mix were stiffer and exhibited 27 % higher shear strength (1.55 N/mm^2) than beams with richer mix. A drop of 27 and 33 percent in average shear strength was recorded for poor workmanship for 1 : 0.3 : 4.5 and 1 : 0.16 : 3 mortar mix respectively. The average shear strength of RB beams increased substantially from 1.54 N/mm^2 to 2.8 N/mm^2 with the use of 12.5 mm diameter stirrups spaced at 75 mm centres.

Bradshaw[17] (1963) built a variety of demonstration structures such as slabs, simply supported and cantilever beams and cantilever stair treads, etc designed on linear elastic theory according to the relevant British Code of Practice[18]. Some of these demonstration structures did not fail even after applying twenty times the design load. No doubt, this proved that RB structures can be designed safely but also highlighted the inadequacy of the elastic theory to predict the failure load.

Johnson and Thomson[19] (1967) reported the results of shear tests on 22 RB brickwork beams built with high bond and normal M type mortar. The variables considered were depth and shape of beams and type of mortars. An attempt was made to correlate the shear strength of beams with masonry discs made of similar mortar. The shear strength was found to be much higher for beams made with high bond mortar compared to the normal M type mortar. In their tests, the shear strength ranged from 1.8 to 2 N/mm^2 for high bond mortar and 1.47 to 2.32 N/mm^2 for the normal mortar. The result also confirms that the shear strength increases with decreasing shear span/effective depth ratio and water cement ratio.

Anderson and Hoffman[20] carried out a series of preliminary tests on eccentrically loaded reinforced brickwork columns consisting of central grout core. The columns were of rectangular cross-section (304 × 406 mm) and 3 m high reinforced with four longitudinal bars with 6 mm stirrups at every third course.

The columns were hinged at the top and fixed at the bottom with eccentricities of 0 to 34% of their depth giving a maximum eccentricity of approximately 138 mm. The main purpose of the test was to compare the results with ultimate stress design method of the American Concrete Institute for reinforced concrete column. The test confirmed that the ACI method used for reinforced concrete is applicable to the reinforced brickwork columns. It was suggested that further research should be conducted to define the actual stress-strain curve for the brickwork, the ultimate stress and strain, and the effect of different percentage area of steel on similar columns.

Many of the earlier research investigations mentioned so far were performed either to derive empirical methods of design or to develop design based on permissible stresses. Between 1975 and 1982, the investigations on RB were carried out to establish the values of the characteristic shear strength[21 – 25] and to develop ultimate load analysis using the various types of stress blocks[26, 27, 28] obtained from testing of brickwork prisms. Some research in the UK has been carried out to compare the behaviour of reinforced brickwork with the provisions of the draft British code BS 5628 pt 2.

Suter and Hendry[21] (1975) tested 12 RB beams of shear span/effective depth ratios (1 to 7) built from common frogged bricks having 0.24 and 1.46 percentage of tensile reinforcements. From these investigations and others, it was suggested[22] that a characteristic shear strength of 0.35 N/mm^2 be adopted for shear span/effective depth ratio up to 2. For shear arm/effective depth ratio of less than 2 an increase in characteristic shear stress of 0.8 N/mm^2 was proposed. It was also suggested that no increase should be allowed for increased area of tensile reinforcements for this type of beam.

Suter and Keller[23] (1976) conducted a comparative shear test on RB and reinforced grouted brickwork beams with varying shear span/effective depth ratios of 1 to 7. The percentage areas of tensile reinforcement for RB and reinforced grouted beam were taken as constant at 1.49 and 1.41. The tests confirmed earlier finding that shear strength increases with decreasing shear span/effective depth ratio for both types of the beams. The shear strength of reinforced grouted beams is much higher than reinforced brickwork and less than reinforced concrete. Using the result of reinforced concrete beams[24] it was further suggested that the shear strength of reinforced grouted beams can safely be derived from adding the separate shear strength of the brickwork and grout sections taking into account their relative thicknesses. This may be the case for the beams tested but cannot be applied universally. In a composite section such as reinforced grouted brickwork beam, the load will be carried according to the relative stiffness of the materials in the section. As soon as the capacity of one of the materials is reached, the failure of the composite section will be initiated, hence caution must be exercised in accepting their conclusions.

Maurenbrecher[25] *et al* (1976) reported tests done on grouted and quetta bond reinforced retaining walls at the Building Research Establishment, UK for the Structural Clay Products Ltd. The percentage area of tensile reinforcements used was 0.33, 0.36, 0.47 and 1.78. The failures of the walls were due to the yielding of steel for low percentage of steel with no sign of failure in brickwork. Increasing the percentage area of steel from 0.33 to 1.78 increased the ultimate moment by 4 times, but still caused failure of steel first with some visible sign of the crushing of brickwork. It is difficult in RB to generate a primary compression failure, as found by others[13, 14]. The tests indicated safety factors ranging from 3.41 to 8.4 in bending and 2.3 to 9.6 in shear, when compared with CP111 : 1970. Using the elastic design in conjunction with the compressive strength of brickwork from the prism tests rather than the value given in the code resulted in uniform factor of safety (2.8 to 3.2) in bending. It would appear reasonable to use the prism test for the compressive strength in the design of reinforced brickwork flexural members. Very close prediction of the ultimate load was achieved by using the Whitney stress block.

Sinha[26] (1978) carried out shear tests on 12 grouted cavity brickwork slabs all with 0.88 percentage area of tensile reinforcement. The shear arm/effective depth ratio varied from 2 to 5 in these tests. The ultimate shear stress increased as the shear arm/effective depth ratio decreased. Various methods of analysis using parabolic, rectangular and triangular stress blocks were carried out for predicting the ultiamte moment. A very high factor of safety was obtained, when compared with the permissible stresses based on CP111 : 1970. A method following Zelger[27] was proposed which accurately predicted the ultimate shear strength of these slabs.

Sinha[28] (1981) presented a method of ultimate load analysis of reinforced brickwork flexural members by using the actual failure strain, the stress-strain relationship and the ultimate stress obtained from prism tests (h/t = 4.5). The actual stress-strain curves for low, medium and high strength brickwork was mathematically idealised as a non-dimensional curvilinear form and the characteristics of stress block factors λ_1 and λ_2 were obtained, which were compared with those obtained using cubic parabola. The ultimate moments of the beams and slabs were calculated using both the cubic and curvilinear stress blocks and compared with the test results. It was concluded that the collapse moment can be closely predicted by using cubic or curvilinear stress blocks. The approach was different than Beard[23], who proposed a second degree parabolic stress-strain curve with a falling branch for calculating the flexural strength of RB. However, this hypothesis was not verified experimentally.

Sinha[29] (1982) presented the results of a further series of comprehensive tests on the shear strength of grouted cavity brickwork beams and slabs. The variables considered were : (i) shear arm/effective depth ratios (1.5 to 10) ; (ii) % area of tensile reinforcements (0.88 to 2.54) ; (iii) brick strength (21.55, 59.38 and 88.33 N/mm^2) ; (iv) mortar grade (1 : 1/4 : 3, 1 : 1/2 : 4.1/2 cement : lime : sand) ; and (v) effect of shear reinforcements. Some of the results of the tests are given in Figs 2 and 3. From these figures it appears that the wall sections of thinner depths have a higher shear strength than beams and shear strength increases with decreasing shear arm/effective depth ratio for both sections. The area of tensile steel has a marked effect on the shear strength of the grouted brickwork. The shear strength of grouted beams increased due to shear reinforcement. The characteristic shear strengths for different percentage areas of steel were given. The mortar grade has a slight effect on the grouted brickwork, but the brick strength does not influence the shear strength.

Scrivener[30] (1982) reported results of reinforced brick shear walls in which reinforcement was placed in the collar joint. The tests confirmed that the shear strength depends on the aspect ratio (height/length) of the wall and the percentage area of steel. Based on these results a lower bound value of shear strength equal to 0.7 N/mm^2 was proposed for this type of wall containing more than 0.2% reinforcement.

Edgell[31] *et al* (1982) presented the results of 4 reinforced pocket type brickwork retaining walls with 0.28 and 0.92% area of steel. The primary failure of these walls was due to the yielding of steel. In one wall with high steel percentage, some spalling of brickwork was noticed. By comparing the results with the draft code[32] it was concluded that this type of wall with pocket spacing of 1 m can be designed as a homogeneous cantilever rather than a flanged member.

Appleton and Southcombe[33] (1982) described tests on 6 reinforced grouted cavity brickwork beams. In the same year and at the same Conference Garwood and Tomlinson[34] presented the results of 8 similar beams containing 0.34 to 1.33% tensile reinforcement. Some of the beams were reinforced against shear. In both the above cases, the reinforced brickwork sections used may be difficult to construct. The main purpose of both the investigations appears to be to compare results against the provisions of the Draft Code BS 5628 Part II[32]. It was concluded that the service and ultimate moments were underestimated by the draft code. Appleton *et al* found that the lever arm calculated from the draft did not agree with the measured value, whereas Garwood *et al* suggested a lower partial safety factor for material than proposed in the draft to obtain a

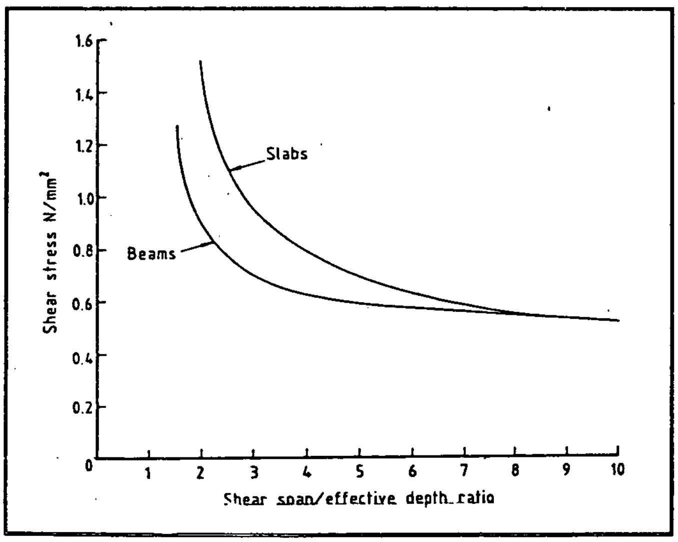

Fig 2 Effect of shear span/effective depth ratio on the shear strength of reinforced brickwork

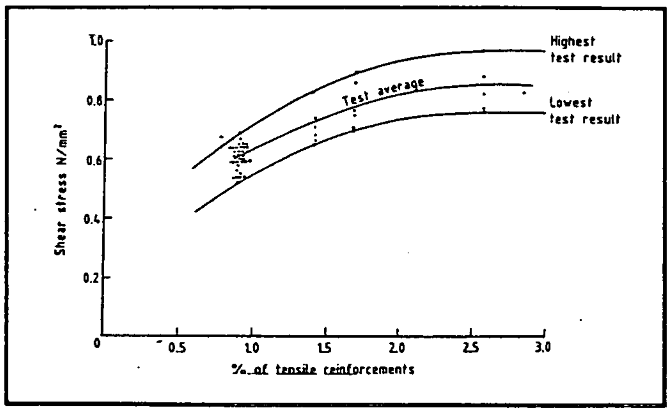

Fig 3 Effect of percentage of tensile reinforcement on the shear strength of reinforced brickwork

realistic design moment for his test beams. However, the anomalies found by both the investigators could have easily been explained theoretically, provided the brickwork material properties from the prism test were used as allowed by the draft Code.

Tellet and Edgell[35] (1983) have described the result of a series of tests on reinforced pocket-type brickwalls having a 0.27 and 1.4 percentage area of tensile reinforcement and with variable shear arm/effective depth ratio. The shear strength increases with a decrease in shear arm/effective depth ratio and with an increase in the percentage of tensile reinforcement. Hendry[36] has shown that their results on shear strength conform to the same pattern of grouted brickwork beams.

Davies and Eltraify[37, 38] (1982, 1984) carried out tests on short half-scale reinforced brickwork columns with a central concrete core subjected to axial load and bi-axial bending. Based on their work they produced a design chart for reinforced brickwork columns subjected to uni-axial and bi-axial bending.

LONG TERM TESTS ON REINFORCED BRICKWORK

Very few tests have been reported on long term flexural performance of reinforced brickwork.

Disch[39] (1949) carried out creep tests on reinforced brickwork beams and came to the conclusion that the creep deformation is less in the reinforced brickwork than in concrete under similar conditions.

Maurenbrecher[40] (1976) studied the behaviour of a 4 m high reinforced pocket-type brick wall on site. He reported the deflection of the wall at top was 24 mm after 517 days ; 8 mm of which was due to slip at the base. The deflection was less than twice the deflection after 20 days ignoring the slip at the base.

Sinha[41] (1979) tested two reinforced grouted cavity brickwork retaining walls (2.519 m high) built back to back and loaded with a water filled bag sandwiched between them. One of the walls had copper damp proof course. The walls were loaded at stages of 1, 1.5 and 2 times the working load. The load-deflection behaviour of both walls was similar. The initial and final deflection of wall with dpc was more than the wall without dpc. The final deflection of wall with dpc at top was 13 mm (1/194 of span) after 400 days, which was twice that of the wall without dpc.

PRESTRESSED BRICKWORK

Although prestressing has been applied extensively to concrete structures[43,44] for many years it has only been applied to brickwork in comparatively recent times.

1959–1979

The first recorded application was in 1952 by F J Samuely[42] who used prestressing to stabilise slender brick piers, over 10 m tall. Some work was also carried out on blockwork silos around this time[52]. After this initial interest prestressed brickwork did not receive further attention until 1963 when Thomas[45] reported tests on two prestressed brickwork beams. The sections are shown in Fig 4(a). High prestressing force (up to 11 N/mm^2) was

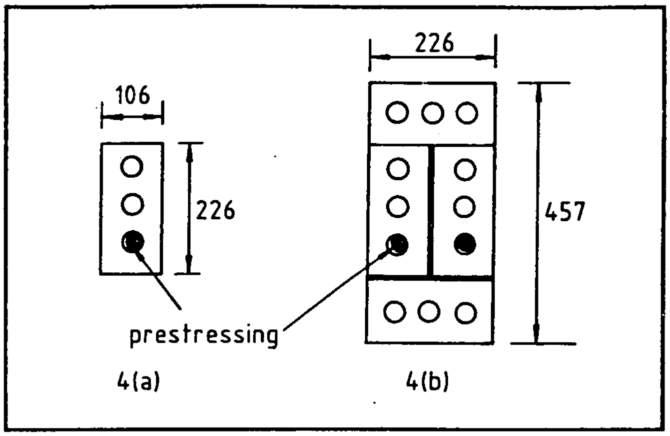

Fig 4 Beams of K Thomas

applied. Failure of the beams indicated that the applied prestress forces were probably excessive for the beam section and brickwork strengths chosen. Plowman[45] continued the work on a section similar to Fig 4(b) with a test programme of 13 beams. The beams were subjected to varying degrees of prestressing and the tendons left unbounded, highlighting one of the difficulties that later researchers would encounter, namely effective grouting of prestressing tendons. The analysis of the results was limited but they did indicate a substantial reserve of moment capacity over the decompression moment, and that the failure mode tended to be in flexure rather than in shear, as occurs more typically in reinforced brickwork.

Some work[45, 46] was carried out on prestressed masonry flooring systems. These systems used extruded clay units (Fig 5) and were pretensioned. Although both systems were patented they do not appear to have had a great deal of commercial success.

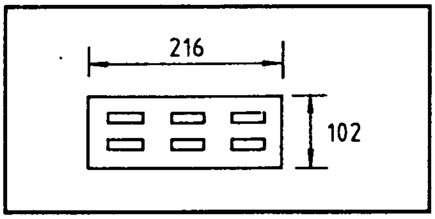

Fig 5 Extruded clay floor unit

In 1970 Mehta and Fincher[47] tested six pretensioned, grouted cavity beams (Fig 6). All beams failed in shear. Throughout this period there were a number of applications of prestressing to walls and other structures.

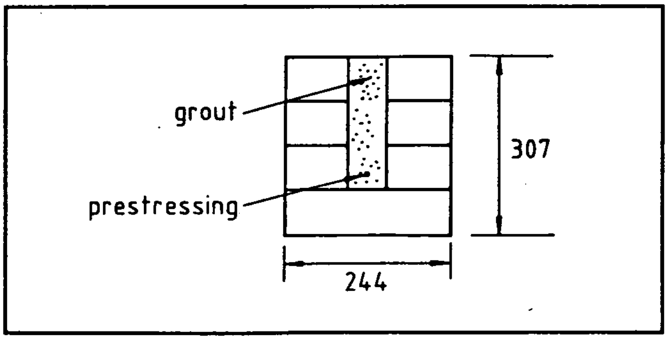

Fig 6 Beam section of Mehta and Fincher

In 1966[42] prestressing was used in conjunction with a steel framed building to increase the lateral loading capacity of a 7 m high wall. The application of the prestress enabled the wall thickness to be kept to 275 mm without piers.

One of the more imaginative uses was in the design of a cylindrical water tank[48], the tank was 12 m in diameter and 5 m high. Both vertical and circumferential prestressing was applied and the tank has been performing satisfactorily ever since.

Prestressing was applied to storey height box beams[49] comprising floor slabs as flanges and brick walls as webs. The application of the prestress was shown to increase the resistance of the webs to diagonal cracking.

1979 Onwards

From 1979 onwards there was a sharp growth in both research and use of prestressed brickwork. This work can be divided into two distinct categoreis :

(i) prestressed walls — prestressing is used to enhance the lateral load capacity of walls of tall, single storey buildings or in retaining walls ; relatively low levels of prestress are applied.

(ii) prestressed beams — a development of reinforced brickwork, with higher levels of prestress.

This paper will consider each category separately.

Prestressed Walls

The most common application of prestressing to date has been in diaphragm walls in which two leaves are separated by a deep rib, forming a large cavity within the wall (Fig 7). A steel tendon runs through the wall cavity from the foundation and through a concrete capping beam on top of the wall. The tendon is stretched against the capping beam either by hydraulic rams or torque wrench.

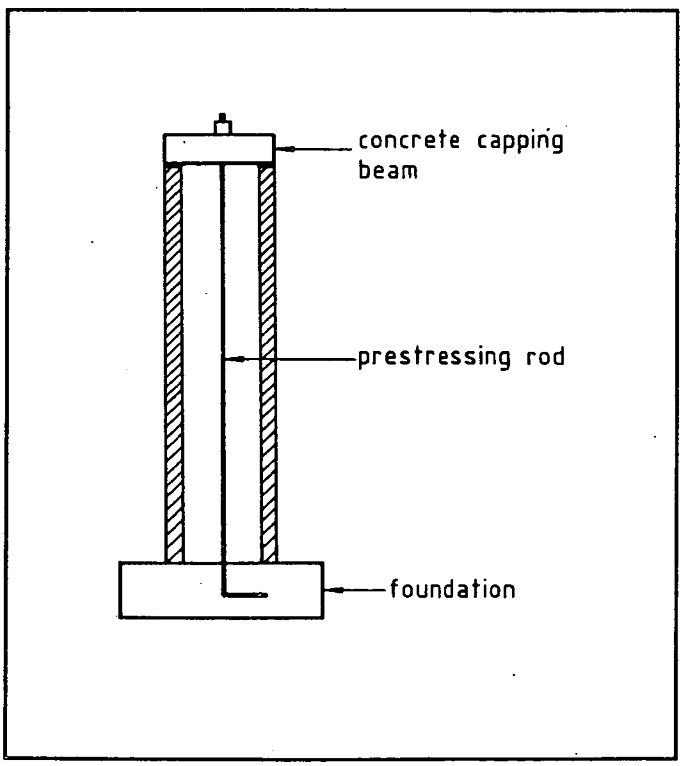

Fig 7 Post tensioned wall

In 1982 Williams and Phipps[50] reported on a study to examine the behaviour of post-tensioned diaphragm walls using six box beams (Fig 8). In such a large cavity the tendons are left unbonded. Consequently as the beam deflects the tendon 'rises' towards the compression flange.

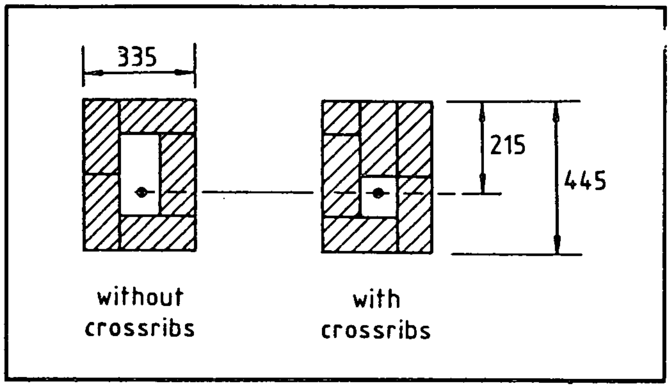

Fig 8 Beams of Williams and Phipps

One aim of this work was to compare the effect of restricting tendon movement by incorporating cross ribs into three of the beams. This was found to increase the moment capacity. Curtin and Phipps[51] reported on two full-scale tests on a 7.25 m high post-tensioned diaphragm wall subjected to various levels of prestress (0-1.8 N/mm^2). Perhaps, not surprisingly, they confirmed that by increasing the precompression the lateral load at which cracking occurred increased. Roumani and Phipps[53], investigating the behaviour of post-tensioned diaphragm walls, carried out tests in 15 I section beams (Fig 9). The influence of shear span/depth ratio, degree of prestress and the depth on the strength of the beams was considered and a relationship between principle tensile strength and shear span/depth ratio was proposed.

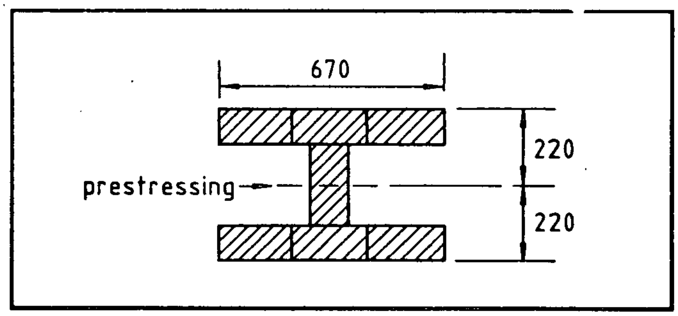

Fig 9 Beams of Roumani and Phipps

A series of 12 tests on concrete blockwork beams[54] with the prestress force applied centrally indicated that post-tensioned walls will suffer a reduction in strength of around 20-30 % when either a vertical damp-proof course or when the bending between flange and web is replaced by metal ties. As previously discovered[50] restricting the tendon movement with cross ribs will increase the moment capacity.

More recently[55, 56, 57] research in post-tensioned walls has tended towards eccentric prestress with triangular loading patterns, simulating pressures due to retained material. Some of this work is repetitive, changing only the form of loading, although higher levels of prestress are now being applied[55].

Sinha[58] has reported on the behaviour of post-tensioned pocket-type retaining walls in which the tendon is fully grouted. This results in improved structural behaviour, which in certain applications may be more effective than diaphragm walls.

In the last decade there has been a number of structures built using post-tensioned masonry. Generally these have been retaining walls[60-62] or tall single storey buildings[59,60] where the walls have been subject to wind loads. In these examples it has been demonstrated that high (up to 8.5 m) free standing walls can be built successfully. In most cases the levels of skill required by the builders was not great and most had never used post-tensioning before. Indeed in agricultural buildings[62] it has been shown that post-tensioned masonry is a technology that the farmer himself may apply with confidence, resulting in buildings that compare favourably with reinforced concrete in terms of cost, performance and construction.

Post-tensioning[63] was used in the construction of a fire station to carry the lateral thrust from a mono pitched roof, which would otherwise have required substantial buttressing or framing.

In situations where buildings may be prone to mining subsidence, post-tensioned diaphragm walls[64] have been used to tie the walls and foundations together and hence minimise damage as the building settled.

Other applications include tall brick piers[60] and channel section walls[65].

Although most applications of post-tensioning have been applied to brick or block walls, it is worth noting that it has also been applied to natural stone[66]. Large cladding panels were prefabricated from slabs of limestone to form deep, narrow beams, spanning over 9 m (Fig 10) carrying its own weight and the glazing above.

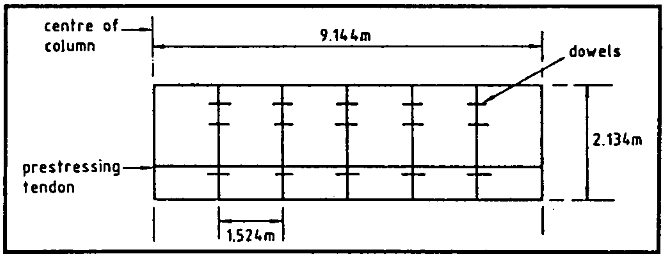

Fig 10 Prestressed limestone walls

PRESTRESSED BRICKWORK BEAMS

The use of prestressing in brickwork beams developed from work on reinforced brickwork in an effort to improve the shear resistance and reduce or eliminate flexural cracking under working loads. In comparison with post-tensioned walls in which the prestress is used to artificially induce precompression in single storey buildings that would otherwise occur due to the self-weight in multi-storey buildings, prestressed brickwork beams are much closer in concept to concrete beams where they act primarily as flexural members.

What distinguishes the last ten years of research in this area from earlier research is the scope and detail of the investigation carried out.

Pedreschi and Sinha[67, 68, 69] have reported the results of an extensive study of prestressed brickwork beams. Fifty-one full-scale tests were carried out. The beam section (Fig 11) was of particular importance as it was developed to utilise traditional bonding patterns, use as much brickwork as possible and hence minimise grouting. The influence of brick strength, mortar grade, percentage area of prestressing steel, prestress force and the shear span/effective depth ratio

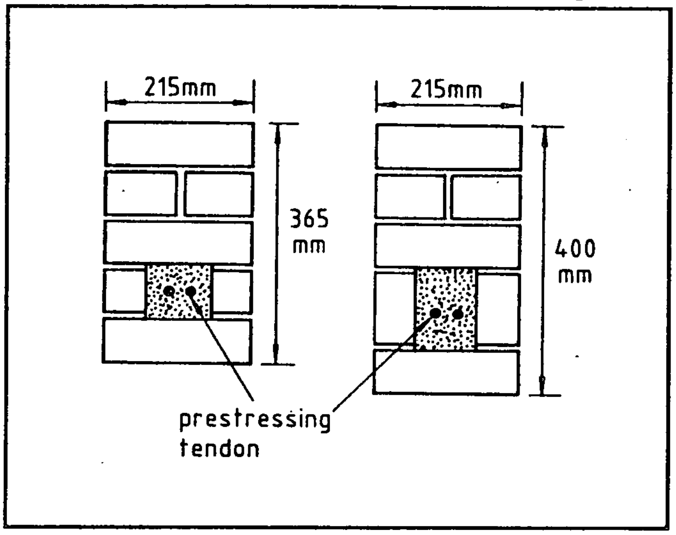

Fig 11 Beam sections of Pedreschi and Sinha

on the strength deflection and cracking behaviour of the beams was studied. It was found that with low percentage area of steel flexural failure was predominant. Brick strength and mortar grade had only a marginal influence. Doubling the area of steel produced over reinforced sections and shear failure occurred. A semi-empirical method was proposed for the prediction of the shear strength. In conjunction with the full-scale programme, a detailed study of the stress/strain characteristics of brickwork prisms loaded parallel to the bed-joint was undertaken. It was found that for a wide range of brick strengths, prism formats and mortar grades, the stress/strain relationship of brickwork in axial compression could be accurately represented using a three-degree polynomial expression.

$$f/f_m = 2.265(\varepsilon/\varepsilon_m) - 2.092\,(\varepsilon/\varepsilon_m)^2 + 0.843\,(\varepsilon/\varepsilon_m)^3$$

An analytical technique that took into account the non-linear material behaviour, cracking and tension stiffening was developed and used to calculate the moment curvature relationship, load/deflection response and crack widths. Excellent agreement with the experimental results was obtained.

The work by Pedreschi and Sinha[71] was subsequently extended to include an investigation into the use of highly perforated bricks (in excess of 20% of gross cross-sectional area perforate). A total of nine beams with three different patterns of holes was tested and it was found that there was no significant reduction in flexural strength when compared to the three hole bricks used in the previous study[67].

Walker and Sinha[72, 73, 74] esxtended the work further with a study of partially prestressed brickwork beams. The study recognised that partial prestressing may offer a satisfactory intermediate alternative to reinforced and fully prestressed brickwork by improving the shear and cracking behaviour of reinforced brickwork and reducing the camber and anchor zone stresses that can occur in fully prestressed beams. An extensive programme involved 41 full-scale tests on beams with a combination of stressed and unstressed reinforcement. The cross-section was very similar to that used by Pedreschi and Sinha (Fig 12). The main areas under consideration were percentage areas of steel, ratio of stressed to unstressed reinforcement, brick strength, mortar grade and cover to non-stressed reinforcement. The study confirmed earlier findings[69,70] that for under reinforced sections the brick strength and mortar grade has little influence on the flexural strength. It was also found that the use of partial prestressing could improve the post-cracking behaviour of prestressed brickwork beams. The analytical techniques were developed further in tension stiffening, prediction of crack widths and in the production of an interactive computer programme for analysis of reinforced and prestressed beams.

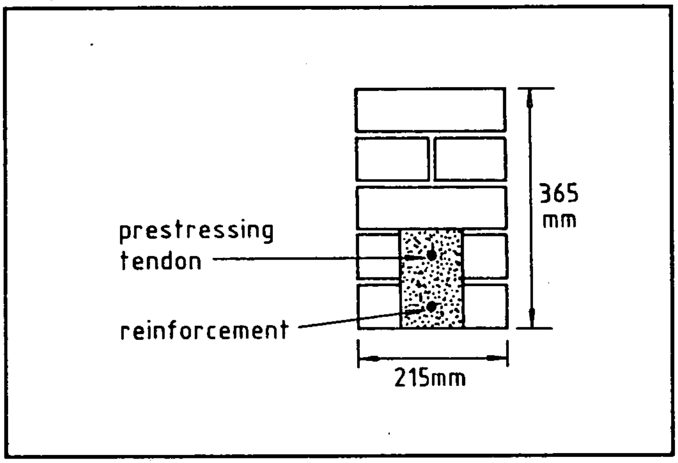

Fig 12 Beam section of Walker and Sinha

The work by Pedreshci and Sinha and Walker and Sinha, has demonstrated quite clearly that the strength, deflection and cracking behaviour can be accurately modelled using small scale tests and that prestressing increases the shear strength of brickwork beams (Fig 13).

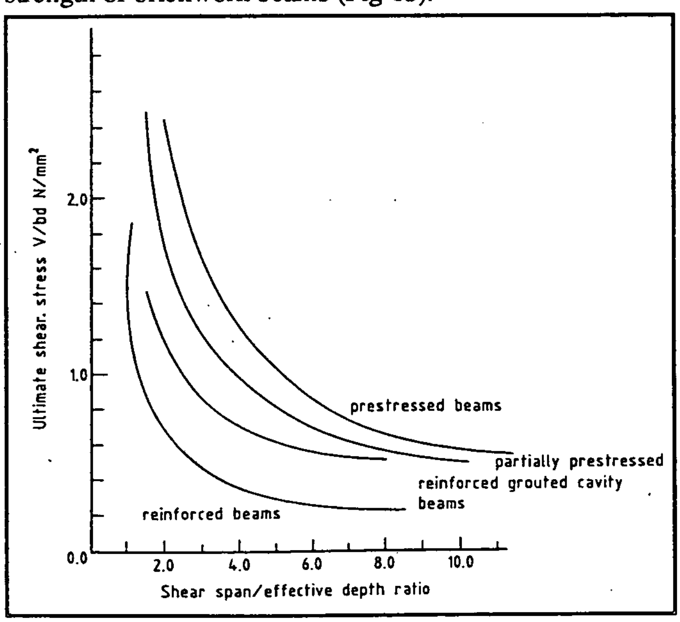

Fig 13 The shear strength of brickwork beams

Robson *et al*[75] reported on a series of tests on unbounded beams with the section shown in Fig 14. 18 beams with two levels of prestress were tested. The section chosen was perhaps unnecessarily complicated from a constructional stand point. The results were compared with the recommendations of the current British Standard and it was found that they substantially under estimated the failure moments and overestimated the deflection.

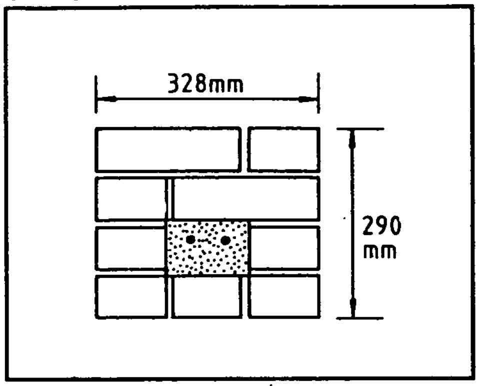

Fig 14 Beam section of Robson, *et al*

Garwood[76] has reported tests of 9 partially prestressed pier bonded beams (Fig 15). The degree of prestress varied from zero to maximum allowable for the stressed reinforcement. The beams with lower prestress forces tended to fail in shear whereas those with higher prestress failed in flexure.

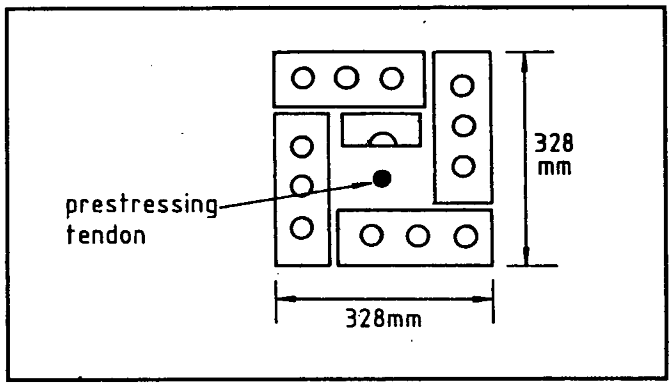

Fig 15 Beam section of Garwood

More recently Uduehi and Sinha[77] compared the behaviour of prestressed brickwork with prestressed concrete beams. The beams had the same cross-sectional dimension, degree of prestress and comparable compressive strengths. It was found that the cracking moment was higher in the concrete beams due to the higher modulus of rupture but the moment-curvature relationship, load/deflection response and ultimate moment were very similar confirming earlier conclusions, Uduehi[78] also studied the shear behaviour of partially prestressed brickwork beams and proposed a method for calculating the shear strength that was shown to produce good agreement with experimental results.

Some research has been carried out on loss of prestress[79, 80] by Lenczner who carried out tests on walls and has proposed a theoretical method to predict the long term losses. This work may not be directly applicable to beams in which the compressive forces develop parallel to the bed-joint.

DURABILITY OF RB AND PRESTRESSED BRICKWORK

In design, the durability of of RB and prestressed brickwork must be considered. It is well understood that the life of RB and prestressed brickwork can be adversely affected by frost damage, sulphate attack and above all the corrosion of steel. The damage to brickwork due to frost depends on degree of exposure to driving rain and temperature. Frost damage can only occur, if 90% of the pore space in brick is filled with water under freezing conditions. Obviously this may not be as important in warmer, drier climates.

All clay bricks contain soluble sulphates; low strength bricks tend to have higher levels than high strength bricks. In wet conditions these sulphates may have a twofold effect. Firstly, in creating an acidic environment around the reinforcement leading to corrosion and, secondly, reacting with the constituents of Portland cement and hydraulic limes to form an expansive compound, tricalcium aluminate, which results in the disintegration of the brickwork. This can be avoided by using bricks with low soluble salt content, sulphate resisting cement and rich mortar mixes (*ie*, greater than 1 : 1 : 6, cement : lime : sand) or replacing lime with a plasticiser.

Out of the three factors which affect the durability, the corrosion of steel is the most potent and lethal, not only for RB but also for RC structures. Some RB roof slabs in India had to be replaced approximately within 35 years due to corrosion of steel. Rangaswami[81] *et al* (1964) reported a corrosion survey on RB and RC buildings built between 7 and 50 years and summarised the causes of the corrosion of steel as :

(i) carbonation of cement mortar;
(ii) corrosive salts in mixing water for concrete and mortar;
(iii) leached out sulphate from brick causing acidic environment around reinforcements;
(iv) steel touching bricks;
(v) poor shedding of water from roof slab due to inadequate slopes;
(vi) Industrial environment with high content of sulphur dioxide causing acidic conditions around the reinforcements.

Foster and Thomas[82] by carefully breaking up reinforced brick structures between two and forty years old were able to investigate the degree of corrosion present in the steel. It was found that even coated reinforcement placed in the bedjoint was heavily corroded in structures between two and five years old. The steel was in a similar condition when placed in the outer leaf of a cavity wall with only 19-38 mm of cover. The steel in a grouted cavity of a ten-year-old structure was unaffected by corrosion. In a forty-year-old structure the steel bars having a cover of 50 mm of mortar and dense brick was in good condition provided the steel did not touch the brick itself. It was concluded that reinforcement in the bedjoint should have a minimum cover of 100 mm from the face of the external wall and 25 mm from the internal face.

Kropp and Hilsdorf (1982)[83] reported 43% reduction in the cross-sectional area of steel due to corrosion after surveying a 27-year-old building with reinforcement in the bedjoint below the window. From their experiments it was suggested that dense brick in 1 : 2.5 or 1 : 3.3 (cement : sand) mortar with water : cement ratio of 0.75 should be used to stop diffusion of carbon dioxide. In case of lightweight or low strength porous building materials, a protective surface finish of 20 mm with similar grade of mortar as mentioned above, should be used to stop diffusion of carbon dioxide, and thus carbonation of mortar. Similarly, epoxy based coatings which have been successfully employed in concrete can be used as special surface coating for very light weight porous materials.

de Vekey[84] (1982) confirmed some of the causes, already mentioned, of corrosion of reinforcements and suggested methods of avoiding them. Three methods have been suggested :

(i) burial at sufficient depth in alkaline concrete of low water : cement ratio, well compacted, using non-porous and well graded aggregates;
(ii) protective coating such as zinc, cadmium, nickel, bitumen, epoxy or thermoplastic; and
(iii) use of austentic steels (containing 18% chromium and 8% nickel) in adverse and hostile environment for RB and lightly prestressed (up to 15% of ultimate strength) brickwork.

The study concludes that all bedjoint reinforcements should be protected as embedment of steel in mortar joint does not offer safeguard against corrosion. In highly stressed prestress member, stress corrosion must be considered. Recently it has come to light that highly stressed galvanised steel fails due to hydrogen embrittlement[85] hence may not be suitable for prestressing.

The problem of corrosion is not confined to RB. Many reinforced concrete structures have also been adversely affected by it. Corrosion can, however, be avoided by proper selection of materials and attention to detailing.

CONCLUSION

Some of the imaginative uses of RB (Figs 16 to 19) can be found in the Third World. But nowadays the use of reinforced brickwork is not as widespread and extensive as concrete. In some parts of India, it is used for floor slabs,

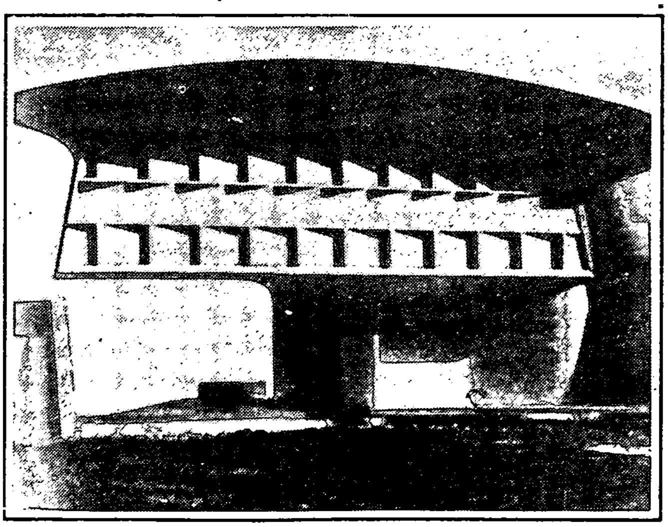

Fig 16 Church in Atlantida, Uruguay

Fig 17 Detail showing wall, Church in Atlantida

lintels and sunshades around windows in the private sector, but not in the public sector projects. In the USA and Canada reinforced grouted masonry wall construction is common for buildings to resist earthquake forces. In Europe its use is increasing. In the UK the construction of post-tensioned walls is now an established technique.

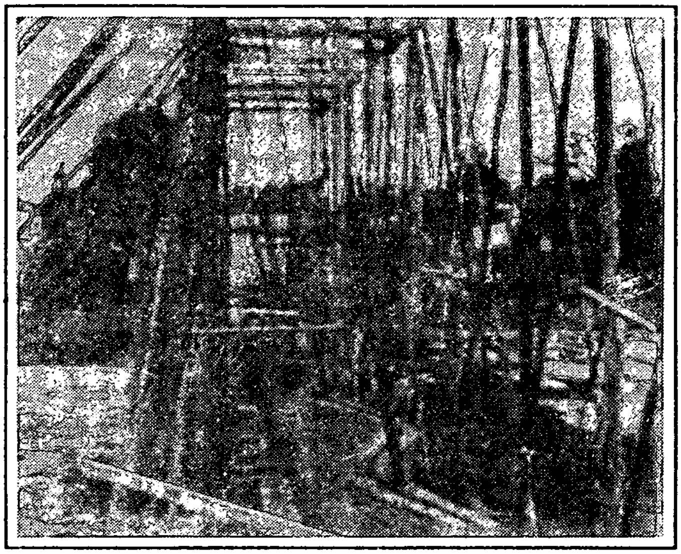

Fig 18 Construction of wall, Church in Allantida

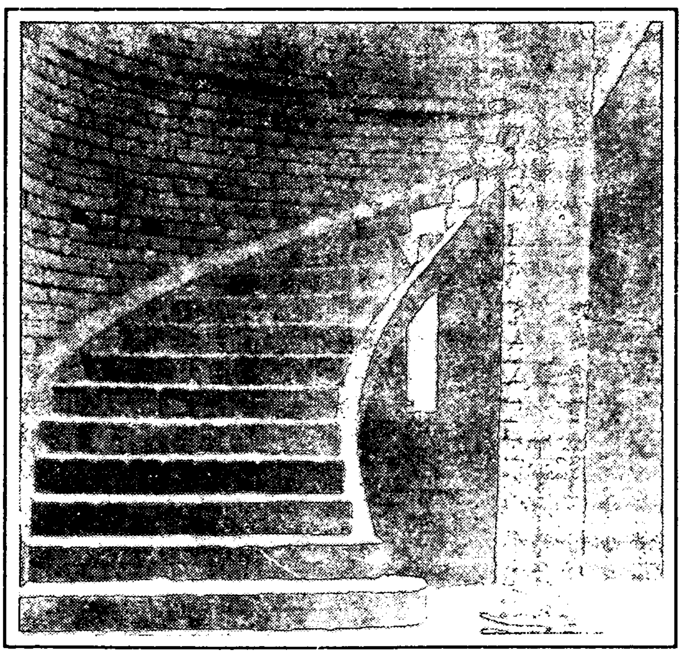

Fig 19 Reinforced brickwork spiral staircase, India c, 1917

Undoubtedly, in many situations both in the developed and developing countries, reinforced and prestressed brickwork can compete and provide an alternative, economic structural solution compared to other construction materials. However, there are still too few engineers who appreciate its potential, and many teaching establishments in the developed and developing countries ignore teaching the structural design of brickwork and until this situation changes the full potential of reinforced and prestressed masonry will not be reached.

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Plain Structural Masonry

Properties of Mortar, Bricks and Brickwork

(Group 2, Section 1.2.1 & 1.2.2 - Papers 3 to 10)

?roc. Instn Civ. Engrs, Part 1, 1976, **60**, Nov., 655–662

TN 142 Strength of mortar for brickwork

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ntroduction

Vhen the draft Code CP 111[1] (Structural recommendations for masonry walls) is ccepted, the various grades of mortar will need to fulfil the strength requirement nvisaged in Table 6 of that document. In some cases the laboratory control ests requirements are difficult to meet, even though the materials conform to the British Standards. The fact has been recognized by the Structural Ceramics dvisory Group of the British Ceramic Research Association and an alternative as been suggested. However, it appears that the requirements are set so high some cases that it may be difficult to get the required strength using local uilding sand conforming to the BS 1200: 1955. Thus, modifications to the nix or a higher grade of mortar will be necessary, resulting in higher construction osts. This adjustment of the mix is not necessary, largely because, within the mits of a particular grade, the compressive strength of the mortar does not reatly affect the brickwork strength. Hence the code requirements may be djusted to values which can be achieved in practice. This may avoid the necesty of the contractor having to seek another sand or the engineer from having redesign the structure using another brick or the next higher grade of mortar.

2. The object of this report is to investigate the strength of three commonly sed mortar mixes for brickwork; this includes the effect of the water/cement atio and the range of the water/cement ratio suitable for brickwork using a ommon building sand conforming to BS 1200: 1955.

nvestigation

laterials

3. The materials used were:

(*a*) *Portland cement and lime*: ordinary Portland cement to BS 12: 1958 and lime to BS 890: 1966 for the mortar cubes.

(*b*) *Sand*: locally available ordinary building sand (dry) conforming to BS 1200: 1955 the grading of which is shown in Fig. 1.

roportioning of mixes and bulk density

4. Although the mixes are specified by volume in the code, the corresponding y weight proportions were used for each constitutent. The bulk densities of e material were found to be:

cement: 1274 kg/m^3
lime: 560 kg/m^3
dry sand: 1410 kg/m^3

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ritten discussion closes 15 January, 1977, for publication in *Proceedings*, Part 1.

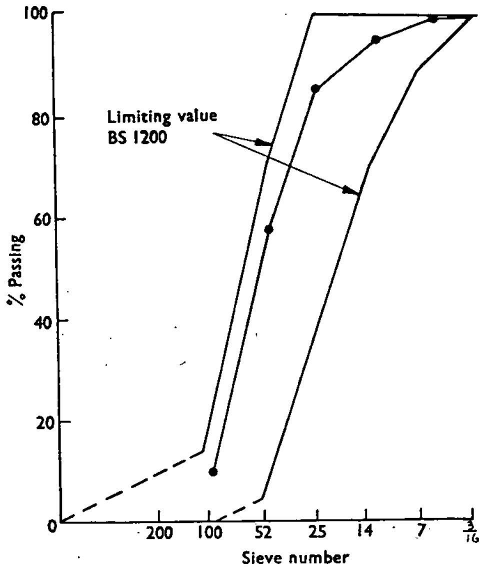


Fig. 1. Grading curve of sand

These are lower than mentioned in BS 4551: 1970, Part I. However, the d…nsit
ratio of sand to cement in both cases appears to be the same.

Preparation of test specimen

5. Sufficient materials for making four cubes were first mixed dry in th mixer before adding any water. Twelve 100 mm cubes and twelve briquette comprising four different water/cement ratios were made according to the coc for testing.

6. The specimens were stored for 24 h at approximately 90% relative hu midity completely covered with Polythene. After 24 h the cubes were remove from moulds and stored in clean water. Six 100 mm cubes were tested at 7 an 28 days immediately on removal from the water.

7. At the time of testing the load was applied within the range of 2 N/mn per min to 6 N/mm² per minute until failure, depending on the strength of morta The compression test results are shown in Table 1 whilst Table 2 shows th results of tensile strength tests.

Discussion

8. As expected, in all three grades of mortar the strength decreased (Table with increasing water/cement ratio (Fig. 2). In the case of mixes with a very lo water/cement ratio, the strength may, however, appear to decrease. No sign ficant increase in strength at low water/cement ratio was observed.

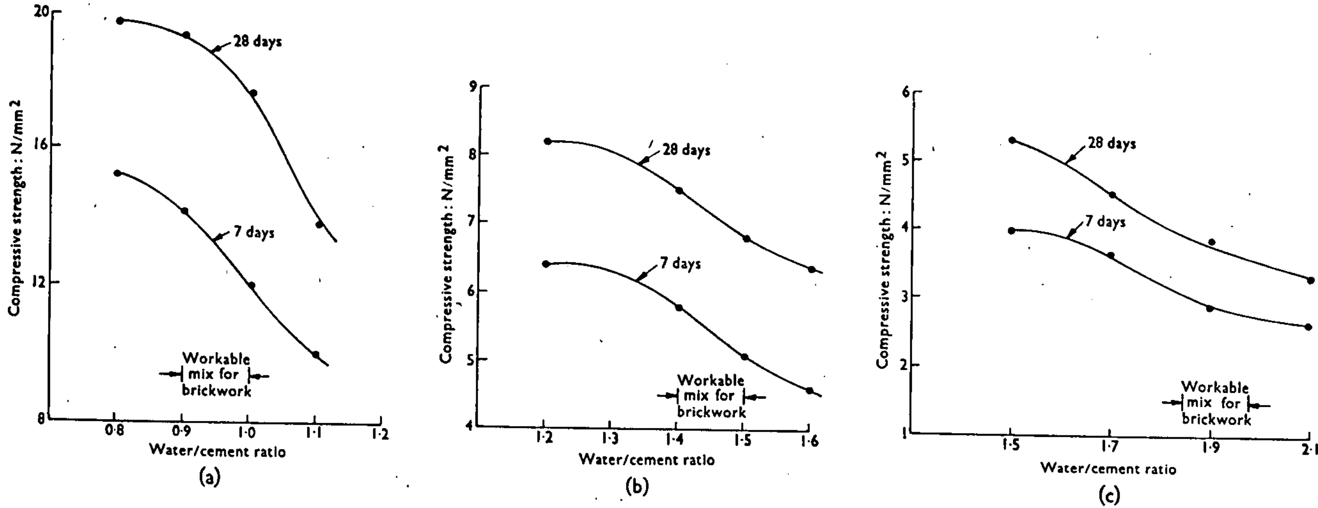


Fig. 2. Effect of water/cement ratio on strength of mortar cubes (a) 1 : ¼ : 3 mix; (b) 1 : ½ : 4½ mix; (c) 1 : 1 : 6 mix

Table 1. Mortar test results (compressive strength)

Mortar	Description	Mix (vol.) cement:lime:sand	Water/cement ratio	Average strength, N/mm² (6 cubes)		Range, N/mm²		Standard deviation, N/mm²		Co-efficient of variation, %	
				7 days	28 days	7 days	28 days	7 days	28 days	7 days	28 days
Grade I	Cement: lime:sand	1:0¼:3	0·8	15·2	19·7	14·5–16·4	18·8–20·3	0·73	0·71	4·8	3·6
			0·9	14·2	19·4	13·3–14·7	17·8–21·0	0·59	1·27	4·15	6·55
			1·0	12·0	17·7	11·6–12·8	16·6–18·6	0·55	0·76	4·58	4·28
			1·1	10·0	13·8	9·6–10·4	13·4–14·7	0·25	0·56	2·5	4·0
Grade II	Cement: lime:sand	1:½:4½	1·2	6·4	8·2	5·4–7·0	7·1–8·9	0·67	0·83	10·45	10·15
			1·4	5·8	7·5	5·7–6·0	7·4–7·7	0·11	0·16	1·94	2·1
			1·5	5·1	6·7	4·8–5·5	6·5–7·0	0·22	0·19	4·29	2·85
			1·6	4·6	6·4	4·3–4·9	5·8–6·9	0·19	0·33	4·15	5·07
Grade III	Cement: lime:sand	1:1:6	1·5	4·0	5·3	3·6–4·1	5·2–5·7	0·20	0·22	5·0	4·13
			1·7	3·7	4·5	3·5–3·8	4·3–4·7	0·12	0·12	3·24	2·62
			1·9	2·9	3·9	2·8–3·0	3·7–3·97	0·08	0·10	2·9	2·53
			2·1	2·7	3·3	2·5–2·8	3·2–3·5	0·13	0·18	4·8	5·45

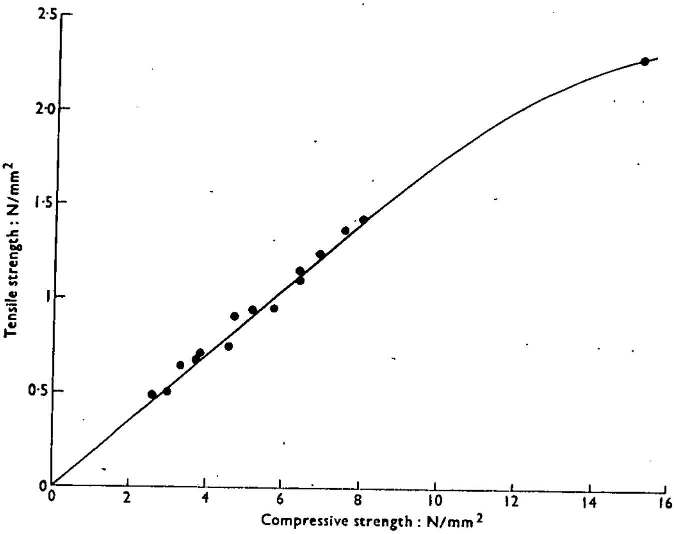
Table 2. Mortar test results (tensile strength)

Mortar	Description	Mix (vol.) cement:lime:sand	Water/cement ratio	Average, N/mm² (6 specimens)		Range, N/mm²		Standard deviation, N/mm²		Co-efficient of variation, %	
				7 days	28 days	7 days	28 days	7 days	28 days	7 days	28 days
Grade I	Cement: lime:sand	1:¼:3	0·8	2·3*	—	1·6–1·6	—	—	—	—	—
Grade II	Cement: lime:sand	1:½:4½	1·2	1·1	1·4	1·0–1·34	1·27–1·6	0·1	0·15	9·29	10·37
			1·4	0·96	1·38	0·86–1·1	1·24–1·48	0·09	0·09	9·3	6·6
			1·5	0·96	1·27	0·86–1·1	1·17–1·41	0·1	0·1	10·71	7·83
			1·6	0·93	1·17	0·79–1·0	1·0–1·31	0·09	0·11	9·66	9·5
Grade III	Cement: lime:sand	1:1:6	1·7	0·69	0·77	0·65–0·72	0·69–0·83	0·02	0·06	3·1	7·33
			1·9	0·52	0·72	0·41–0·65	0·58–0·79	0·09	0·09	6·9	12·4
			2·1	0·48	0·69	0·41–0·62	0·52–0·77	0·08	0·094	16·6	13·69

* Only four specimens tested

Table 3. Minimum average compressive strengths of mortar cubes for brickwork

Grade	Description	Mix (vol.), cement: sand: lime	Model specifications and code recommendation (BCLP/29: 1971) mortar cube strength, N/mm²		Test compressive strength, N/mm²		Revised draft Code BCLP/29: 1974, comparative strength, N/mm²
			7 days	28 days	7 days	28 days	28 days
I	Cement: sand	1:0–¼:3	11·0	16·00	12·0	17·70	16·0
II	Cement: lime: sand	1:½:4½	5·5	8·0	5·0	6·7	6·5
III	Cement: lime: sand	1:1:6	2·75	4·0	2·9	3·6	3·6



j. 3. Relationship between compressive and tensile strength of mortar

9. For grades I, II and III mortars, the water/cement ratios 0·9–1·0, 1·4–1·5, d 1·85–1·95 are satisfactory for laying brickwork, at least for the sand used in s test. It might need a little adjustment if different sand grading is used. As code specifies the minimum strength to be obtained, the higher water/cement io in the above range has been taken for strength consideration and comrison with the requirements. In the case of $1:\frac{1}{4}:3$ grade I mortar, the average ength of the six cubes was more than prescribed by the code or model speciation[2] for load bearing clay brickwork. For grade II mortar, the strength quirement is higher than obtained in the test at 7 or 28 days. If a low water/ nent ratio is taken, the strength at 28 days is still lower than required. For :6 mortar the 28 day strength is again lower than requirement. This is bese of the assumption in the model specification that the strength of grade I ortar is twice the strength of grade II and four times the strength of grade III ortar and the 28 day strength is 1·5 times the 7 day strength on which the ft[1] was based.

0. For batching by weight the strength of cubes will be similar provided the ne density ratio is used; any lower density may give low strength cubes.

1. The draft code specifies a 28 day strength only. For practical work, vever, it would be useful to give the 7 day strength.

2. The tensile strength increases with increasing compressive strength, but y not be directly proportional at higher strengths (Fig. 3). The tensile strength mortar is less sensitive to the water/cement ratio than to the compressive ength.

nclusion

3. The strength requirements for grade II (at 7 and 28 days) and grade III rtar (at 28 days) are high and may be revised in the light of these experiments ble 3). It appears that the recent draft has incorporated the suggestion[3] of

lowering the strength requirement for grade II and III mortars, and if accepte will enable all building sands to be used within the grading limits given i BS 1200: 1955.

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4

Survey of Scottish sands and their characteristics which affect mortar strength

Drew Currie and Braj Sinha

Any sand which is used for masonry mortar in the United Kingdom strictly should conform to the grading limits of BS 1200.[1] The basis of the grading limits seems somewhat impractical however, as many natural sands which have been used successfully in the past, or are being used at present are outside these limits. Previously, as long as the mortar achieved the strength required by the individual designer, no particular attention was paid to the grading of the sand. This situation has changed recently owing to the publication of BS 5628,[2] which incorporates a mandatory strength requirement for the various grades of masonry mortar. Consequently, more attention has to be paid to the quality of the individual mortar constituents, especially the sand, as the cement and lime are manufactured to high standards. This may result in rejection of a sand purely on the basis of grading requirements

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which may not be practical or justified. A resource survey therefore was carried out in Scotland to find out to what extent the sands currently in use are outside the limit of BS 1200. The aim of the survey also was to examine if the mortar made from the natural sands outwith the limit, fulfils the strength requirement of BS 5628. In this paper, we summarise the results of the survey, and in view of the results suggest modification of the current BS 1200.

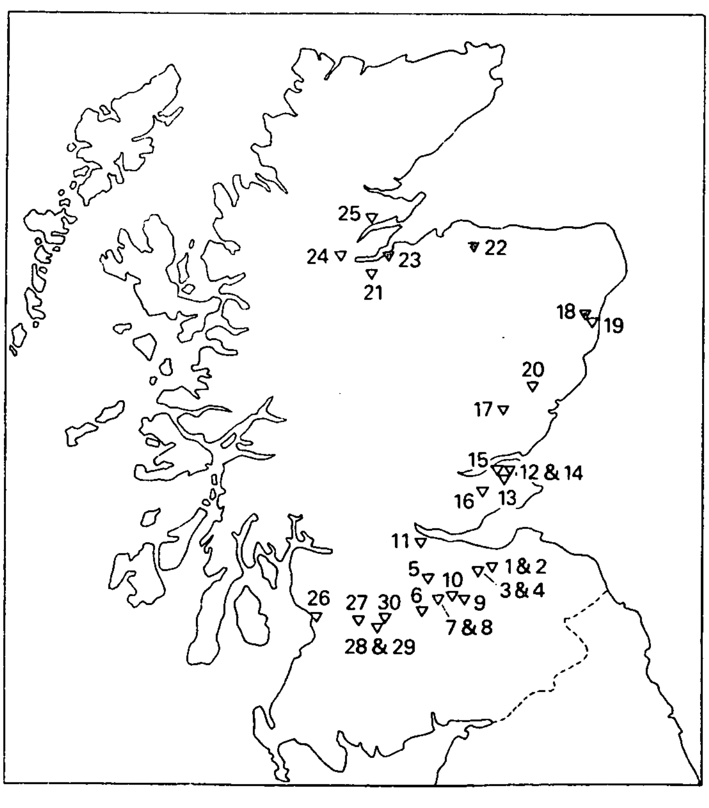
Sources of building sand in Scotland and their physical characteristics

Most of the major sources of supply of sand in the central belt and in the east and north of Scotland were included in the survey (Fig 1). In all, 24 sources were visited, which produced mortar sand, but a number of these used both washing and dry-screening to obtain two types of sand, which increased the samples taken to 30. Great care was taken in the collection of these samples, as the accuracy with which the results of any test can reflect the characteristics of the bulk material is limited by the extent to which the samples are representative. The recommendations of BS 812 (Section 1 Part 4a)[3] were followed and each sample was taken from the fresh material immediately after the washing or screening plant.

Physical characteristics of the sands

Particle size distribution. The particle size distribution of each sand was determined by dry sieve analysis according to BS 812.[3] The results of the individual analyses are presented in triangular graph form in Fig 2. This shows the percentage of material greater than 600µm, between 150 and 600µm and less than 150µm and the grading of a sand can be represented conveniently as a single point. The influence of the 300µm limit is neglected in this diagram and therefore sands which fail on this limit only, such as samples 9 and 26, will appear to be within the grading limits as shown in the diagram. The range of sand gradings obtained in this survey is shown in Fig 3 along with the BS grading limits.[1]

Fig 1 Geographical location of sand samples (filled in triangles denote samples outwith the grading specifications of Table 1, BS 1200; open triangles denote samples within the grading specifications of Table 1, BS1200



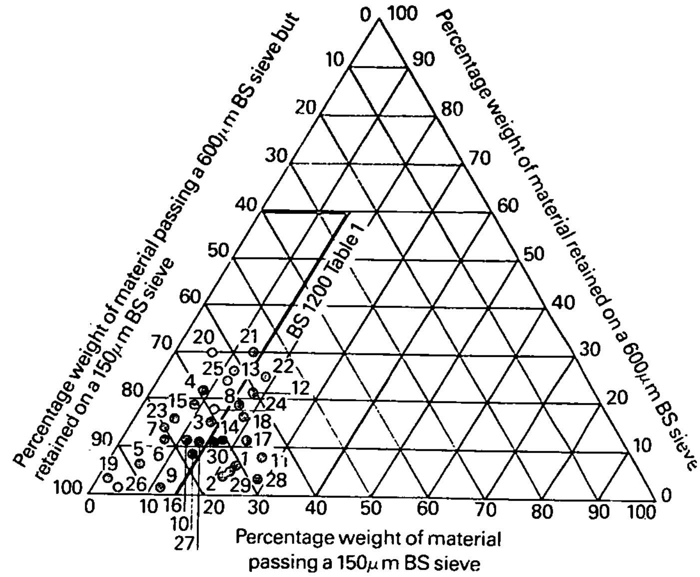


Fig 2 Particle size distribution of Scottish sands

Silt and clay content. The field settling test was used to obtain a rough approximation of the percentage of silt and clay in the sands and a few samples were checked with the more accurate decantation method. As the figures in Table 1 represent both silt and clay, it can be seen that the presence of clay is not really much of a problem in Scottish sands.

Fineness modulus. The fineness modulus is a factor computed from the sieve analysis of the sand and is defined as the cumulative percentages retained on the sieves of the series; 150, 300, 600µm, 1.18, 2.36 and 5.00mm. The coarser the sand, the higher is the value of the finess modulus. This factor however, does not represent the particle size distribution of the sand, as sands with different grading curves can have the same value of fineness modulus as shown by samples 1 and 26 (Table 1).

Fig 3 Range of particle size distributions (broken lines show the zone into which approximately 93 per cent of the grading test results fall (the coarsest and finest results ignored))

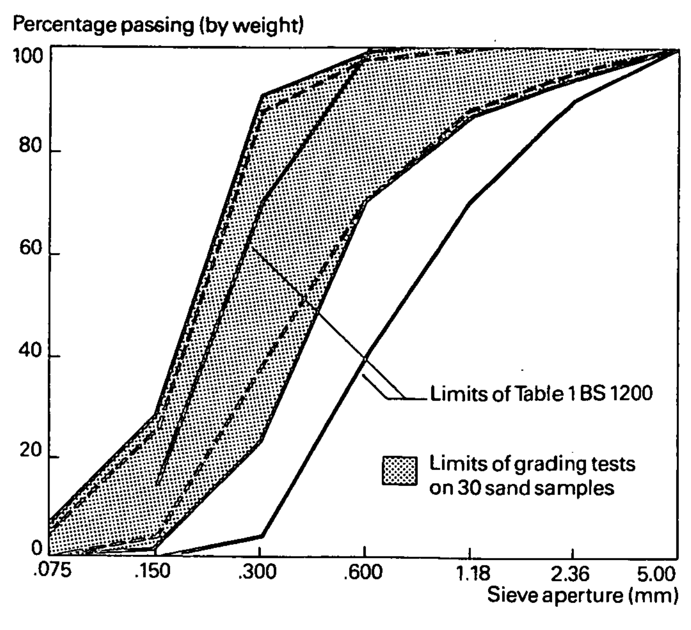


Table 1 **Physical characteristics of sand samples**

Sand no.	Production process	Material finer than 63 microns: Field settling test (per cent)	Material finer than 63 microns: Decant method (per cent)	Bulk densities: Uncompacted (kg/m^3)	Bulk densities: Compacted (kg/m^3)	Void content (per cent)	Fineness modulus	Specific surface measured by nitrogen adsorption (m^2/g)	Lithology (major constituents in bold type)
4	Washed	13	—	1340	1577	40.2	1.73	5.041	**White, brown & red sandstone, basalt, trachyte, dolerite, greywacke, vein quartz, quartzite,** limestone, mudstone
3	Dry-screened	7	—	1374	1594	39.5	1.54	—	
30	Washed	—	—	1484	1628	38.2	1.42	4.502	**Basalt, sandstone,** schist and vein quartz
27	Washed	4	—	1476	1624	38.3	1.38	4.010	**Basalt,** a small proportion of fine grained acid igneous rocks, sandstone and vein quartz
29	Washed	2	—	1519	1671	36.6	1.35	3.914	**Basalt, sandstone** and some vein quartz
2	Dry-screened	10	8.9	1367	1590	39.7	1.03	3.788	**White, brown & red sandstone, basalt, trachyte, dolerite, greywacke, vein quartz, quartzite,** limestone, mudstone
16	Dry-screened		—	1407	1611	38.9	1.52	3.343	**Lower Old Red Sandstone lavas, sandstone,** quartzite, vein quartz and acid igneous
28	Dry-screened	—	—	1426	1617	38.6	0.88	3.312	**Basalt, sandstone** and some vein quartz
11	Washed	6	—	1398	1616	38.7	1.04	3.114	**Sandstone,** some lava and shale
1	Washed	5	4.4	1372	1582	40.0	1.06	2.885	**White, brown & red sandstone, basalt, trachyte, dolerite, greywacke, vein quartz, quartzite,** limestone, mudstone
6	Washed	6	—	1432	1626	38.3	1.59	2.674	**Sandstone, basalt, felsite, greywacke,** vein quartz and schist
10	Washed	6	—	—	—	—	1.30	2.587	**Sandstone, basalt, felsite, greywacke,** vein quartz and schist
7	Washed	2	—	1472	1667	36.7	1.56	2.405	**Sandstone, greywacke,** conglomerate and felsites
8	Washed	7	—	1477	1687	36.0	1.59	—	
17	Dry-screened	5	—	1428	1665	36.8	1.35	2.288	**Quartz quartzite, Lower Old Red Sandstone lavas,** schist and various intrusive igneous rocks
9	Dry-screened	4	—	—	—	—	1.17	2.158	**Sandstone, basalt, felsite, greywacke** and vein quartz
20	Dry-screened	2	1.5	1436	1579	40.1	2.01	2.015	**Quartzite, psammite** (quartz-feldspar-granulites), some sandstone, lavas, a little coarse-grained igneous
15	Dry-screened	8	—	1535	1740	34.0	1.70	0.912	**Impure quartzite, grits, Lower ORS lavas,** some sandstone, conglomerates, quartzite, vein quartz acid igneous
12	Dry-screened	14	7.2	1564	1784	32.3	1.66	0.256	**Impure quartzite, grits, Lower ORS lavas,** some sandstone, conglomerates, quartzite, vein quartz, acid igneous
13	Washed	9	—	1606	1779	32.5	1.84	—	
14	Washed	7	—	1466	1663	36.9	1.30	—	
26	Washed	1	—	1384	1563	40.7	1.06	0.211	Raised beach deposit with some aeolian deposits on top
18	Dry-screened	2	4.6	1422	1610	38.9	1.50	0.211	**Granite, psammite, semipelitic schist, mica-schist,** gabbro
21	Washed	2	—	—	1769	32.9	1.91	0.098	Mainly metamorphic (e.g. **quartzite, psammite,** etc).
23	Dry-screened	4	—	—	1671	36.6	1.73	0.092	**Psammite,** quartzite and granite
24	Washed	—	—	—	—	—	1.70	—	
25	Washed	—	—	—	—	—	1.83	—	
19	Dry-screened	1	—	—	1643	37.6	1.38	0.062	Aeolian deposits
22	Washed	1	4.6	1493	1688	35.9	1.73	0.061	**Psammite,** pelite, sandstone, prophyry and granite
5	Washed	1	—	1464	1615	38.7	1.41	0.030	No information on this sand – manufactured by crushing soft sandstone

Bulk density. For the determination of bulk density, BS 812[3] recognises two degrees of compaction; loose (or uncompacted) and compacted. Both these methods of obtaining the bulk density were used, except that the average of at least four tests was taken and the number of compactive blows per layer was increased to 100 in the case of the compacted bulk density test. Bulk density is affected by the size distribution and shape of the particles and the compactive effort used. This increased compactive effort resulted in a greater degree of packing and the compacted bulk density obtained therefore will be closer to the potential dictated by the particle characteristics.

Void content. The void content shown in Table 1 is related directly to the compacted bulk density and is found by assuming that the sands tested are composed mainly of silica which has a specific gravity of 2.635. The compacted bulk density then can be considered to be a true indication of the voids in the aggregate, i.e., a bulk density of $2635 kg/m^3$ would imply zero void content.

Specific surface and technique used in its determination. Gas adsorption has been employed successfully for the determination of the total surface area of a variety of powders and fine aggregates with a high degree of accuracy. In this investigation it was assumed that the surface texture of the sand particles would influence the water requirement of the mortar and a technique therefore was chosen which would measure both the external surface of the sand particles and the internal surface of any cracks, pores, fissures etc. A dynamic nitrogen adsorption technique[5,6] therefore was employed. Basically, this involves the determination of the quantity of nitrogen necessary to form a mono-molecular layer (monolayer) on the surface to be measured. Then, the number of molecules required to form a monolayer is evaluated and the surface area of the material is calculated from a knowledge of the cross-sectional area of the gas molecules.

Table 2 **Bulk densities of cement, lime and sand**

	Experimental results (kg/m^3)	BS 4551 (kg/m^3)
Cement	1293	1450
Lime	593	575
Sand	1340-1564	1450-1900
Sand:cement ratio	1.04-1.21	1.0-1.31

Influence of characteristics of sand on the strength of mortar

Three grades of mortar were made from the sands to determine the compressive strength and to assess which of the properties of sand affects the mortar strength.

Materials

The materials used were:

- Ordinary Portland Cement in compliance with BS12:1958;
- Hydrated lime in compliance with BS 890: 1966;
- General purpose building sands from different sources in Scotland with varying gradings, not all of which conform to the limits of Table 1, BS 1200.

Proportioning of mixes and bulk densities

Using the specified volumetric proportions for each mortar grade and the dry bulk densities (Table 2), which were determined for each constituent, the mortar mixes were all batched by weight. The experimental values of the bulk densities for the cement and the sands are all lower than the British Standard values. However, the ratios of sand bulk density to cement bulk density are very similar.

Preparation and testing of mortar cube specimens

The mortar was mixed according to BS 4551,[4] and the consistence was determined for each batch using the dropping ball test.[4] Six, 100mm cubes were made from each batch of mortar and stored under polythene for the first 24 hours. The cubes then were removed from the moulds and stored in clean water. Apart from three extra batches, all specimens were tested at an age of 28 days immediately on removal from the water. An Avery testing machine complying to BS 4551 was used, and the rate of loading was kept to within 2 and 6N/mm² until failure. Test results are shown in Table 3 and 4.

Discussion of the results

Grading of sand samples

Of the 30 sands surveyed, 13 fail to satisfy the lower grading limit of Table 1, BS 1200, and the remainder have gradings which are close to this limit. From the range of sand gradings encountered (Fig 3), it can be seen that even when the finest and coarsest grading curves are neglected, to prevent any exaggeration due to an unusual grading, there is still a significant percentage of the range outside the lower limit.

Specific surface

It can be seen from the nitrogen adsorption results in Table 1 that the sands tested have a wide range of specific surfaces. The distribution of the results is shown in the histogram (Fig 4) which suggested that two groups of aggregate might exist. This supposition was further substantiated when the geographical distribution, and geological composition of the samples was examined. The samples represent mainly glacial or fluvio-glacial deposits which closely resemble the bedrock geology of their regions. On the basis of these results, it therefore can be assumed that the specific surface of a sand sample, as measured by nitrogen adsorption, is dependent mainly on the geological composition of that sample, sandstone and basalt producing greater specific surfaces than metamorphic rocks. No relationship could be found between the specific surface and the other sand characteristics.

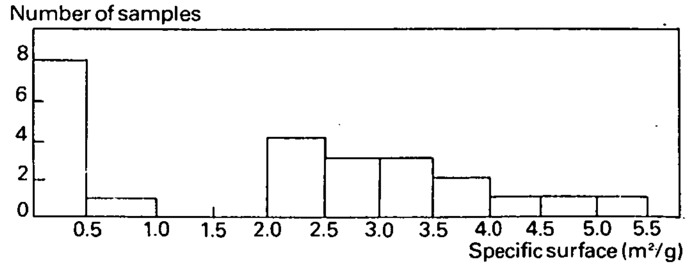
Results of mortar cube tests

In the first series of cube tests, three sands with widely different gradings were used to make three grades of mortar with varying water:cement ratio. The sands used were:

- Sand no. 1, a fine sand, well outside the lower BS grading limit;
- Sand no. 20, a coarse sand, well inside the BS grading limits;
- Sand no. 22, a uniformly graded sand which came between the others but still failed the 150μm limit.

The compressive strength results of the first series of

Fig 4 Frequency of specific surface results



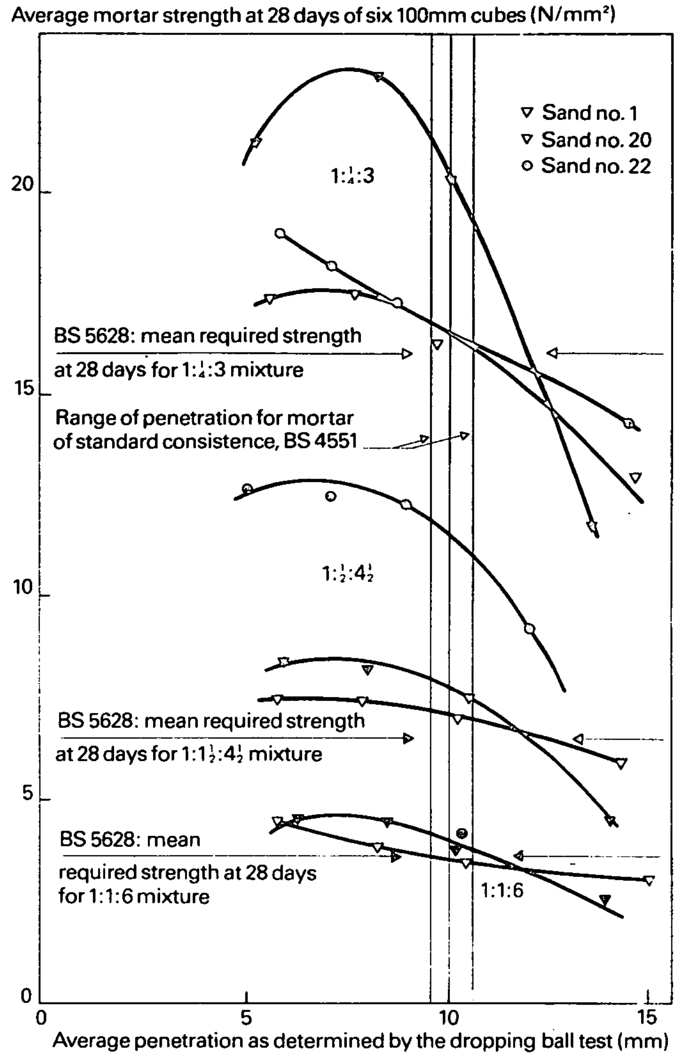


Fig 5 The relation between strength and consistence of mortar

cubes are shown in Table 3 and Fig 5. The coefficient of variation of the results is low except in the case of grade 1 mortar made from sand no. 20. The cause of the variation is unknown. It can be seen that all the sands produce mortar of satisfactory strength for grades 1 and 2 at the standard consistence. For mortar grade 3 the results are closer to the limit and sand no. 1 just fails to achieve the required strength as the standard consistence.

As grade 3 mortar had the strength requirement least likely to be achieved, a second series of tests was carried out in which a large number of sands were used to make grade 3 mortar of standard consistence. Sixteen sands were used in this second test series, nine of which were outside the lower BS grading limit. One batch of grade 3 mortar at the standard consistence was made per sand. The compressive strength results are shown in Table 4 and again show a low coefficient of variation.

As can be seen from Table 4, three sands (nos 1, 2 and 26) fail to produce 1:1:6 mortar of strength greater than the 3.6N/mm² required by BS 5628. For these sands, the mortar proportions were changed from 1:1:6 to 1:1:5 (cement:lime:sand), which is within the allowable proportions of grade 3 mortar. The cubes made were tested at seven days and the results are given in Table 5. These seven-day strengths were similar to the 28-day strengths as envisaged in the Code,[3] except for sand no. 26. Previous research[7] showed an average increase in compressive strength of 35 per cent from 7 to 28 days and the expected 28-day strengths for these cubes also are shown in Table 5. From these results it can be inferred that the mortar using sand no 26 will have a 28-day strength higher than that required by the Code.[2] Therefore, within the allowable proportions for grade 3 mortar, given in Table 1, BS 5628, all the sands used in this test series will produce mortar of satisfactory strength.

Variation of mortar strength with consistence for a particular sand and mortar grade

One thing that is apparent from the results of the first test series is the variation of mortar strength with consistence for a particular sand and particular mortar grade. Comparing the results in Table 3 for a particular grade and sand, it can be seen that the lowest consistence and therefore the lowest water-cement ratio, does not always produce the highest strength. Furthermore, there seems to be a tendency for the strength to decrease as the consistence becomes very low.

In Fig 5 the variation in strength, for a particular mortar, can be seen as the consistence changes from a low value to a high value. Although the water:cement ratios cannot be compared directly, the strength at the standard consistence for masonry mortar can be determined easily. From the diagram, it is alo evident that for most of the mortars, there is an optimum consistence or alternatively an optimum water-cement ratio which produces the maximum compressive strength. This is obviously located at the point where maximum compaction occurs and any further increase in water content merely reduces the density of the mortar mix and results in lower strength.

Effect of void content, fineness modulus and specific surface of the sand on the water-content of mortar

The water requirement and therefore the strength of a mortar is influenced by the shape and texture of the aggregate and these factors are expressed indirectly by the packing of the aggregate, i.e. the percentage of voids in a loose condition. The relationship is shown in Fig 6 and it can be seen that the water requirement of mortar of standard consistence, increases with the void content of the sand used. However, the test results show only 69 per cent correlation. The water-content of the mortars also seemed to vary with the fineness modulus of the sands, as shown in Fig 7, but again there was only 69 per cent correlation.

The specific surface measured by nitrogen adsorption seems to have no relation with the water content of mortar and therefore does not influence the mortar strength. The nitrogen adsorption method gives both the internal and external surfaces of the particles and therefore it is possible that the specific surface has been exaggerated and that only a small proportion of the total surface area of the aggregate, the external surface, has any effect on the water requirement of mortar. This may be due to the formation of a dense

Table 3 **Influence of water:cement ratio on mortar strength: Mortar test results (compressive strength of 100mm cubes)**

Mortar	Description	Mix (vol) cement: lime:sand	Water: cement ratio	Av. strength N/mm² (6 cubes) (28 days)	Range (N/mm²)	Standard deviation (N/mm²)	Coefficient of variation (per cent)	Dropping ball test penetration (mm)
Grade 1	Cement:lime	1:¼:3	0.85	17.30	16.65-18.20	0.44	2.54	5.5
	sand		0.91	17.50	17.20-17.90	0.08	0.46	7.6
(Sand 1)			0.97	16.30	15.80-16.80	0.17	1.04	9.6
			1.10	12.97	12.30-13.20	0.11	0.85	14.6
Grade 2	Cement:lime:	1:½-4½	1.23	7.52	7.10- 7.70	0.05	0.66	5.7
	sand		1.32	7.42	6.85- 8.05	0.24	3.23	7.8
(Sand 1)			1.41	6.96	6.65- 7.10	0.02	0.29	10.2
			1.57	5.90	5.60- 6.15	0.04	0.68	14.2
Grade 3	Cement:lime:	1:1:6	1.60	4.42	4.12- 4.77	0.06	1.36	5.8
	sand		1.74	3.88	3.70- 4.05	0.03	0.77	8.2
(Sand 1)			1.88	3.49	3.37- 3.60	0.01	0.29	10.4
			2.08	2.96	2.88- 3.10	0.01	0.34	15.0
Grade 1	Cement:lime:	1:¼:3	0.74	21.25	19.70-24.30	3.55	16.70	5.2
	sand		0.80	22.88	20.30-25.00	4.05	17.70	8.2
(Sand 20)			0.86	20.23	18.30-22.20	2.69	13.30	10.0
			1.07	11.88	11.50-12.30	0.12	1.01	13.5
Grade 2	Cement:lime:	1:½:4½	1.09	8.39	8.25- 8.60	0.02	0.24	5.9
	sand		1.17	8.28	7.70- 9.01	0.20	2.42	8.0
(Sand 20)			1.27	7.50	7.17- 7.75	0.26	3.47	10.5
			1.55	4.48	4.17- 4.75	0.26	5.80	13.9
Grade 3	Cement:lime:	1:1:6	1.48	4.37	4.00- 4.67	0.29	6.64	6.2
	sand		1.60	4.31	4.15- 4.48	0.11	2.55	8.4
(Sand 20)			1.73	3.72	3.34- 3.96	0.21	5.64	10.2
			2.10	2.54	2.40- 2.80	0.15	5.90	13.8
Grade 1	Cement:lime:	1:¼:3	0.80	19.02	18.20-19.80	0.59	3.10	5.8
	sand		0.82	18.25	17.30-19.30	0.63	3.45	7.0
(Sand 22)			0.88	17.30	16.50-18.30	0.67	3.87	8.6
			1.00	14.35	14.10-14.80	0.27	1.88	14.4
Grade 2	Cement:lime:	1:½:4½	1.00	12.60	12.30-12.80	0.18	1.43	5.0
	sand		1.08	12.60	12.50-12.70	0.11	0.87	7.0
(Sand 22)			1.16	12.27	11.40-13.00	0.54	4.40	8.9
			1.30	9.25	8.70- 9.70	0.38	4.11	11.9
Grade 3	Cement:lime:	1:1:6	1.73	4.13	3.60- 4.40	0.08	1.95	10.2
	sand							
(Sand 22)								

Table 4 **Mortar test results (compressive strength of 100mm cubes). (Grade 3 mortar at standard consistence – second test series)**

Sand no.	Water:cement ratio	Average 28 days strength (6 cubes) (N/mm²)	Range (N/mm²)	Standard deviation (N/mm²)	Coefficient of variation (per cent)
1	1.88	3.49	3.37-3.60	0.01	0.29
2	1.78	3.41	3.20-3.56	0.14	4.05
4	1.62	4.38	4.25-4.55	0.13	3.03
5	1.69	4.72	4.61-4.82	0.09	1.95
6	1.69	4.68	4.58-4.75	0.07	1.57
11	1.80	3.75	3.70-3.80	0.05	1.46
12	1.60	4.29	4.20-4.40	0.07	1.55
16	1.63	4.77	4.52-4.94	0.17	3.48
17	1.66	5.08	4.95-5.30	0.13	2.63
18	2.00	4.14	4.00-4.30	0.13	3.14
20	1.73	3.72	3.34-3.96	0.21	5.64
22	1.73	4.13	3.60-4.40	0.08	1.95
26	1.76	2.48	2.35-2.58	0.09	3.63
27	1.75	3.67	3.62-3.72	0.04	1.13
29	1.66	6.04	5.90-6.20	0.10	1.68
30	1.78	4.42	4.27-4.66	0.14	3.11

*Grade 3 (1:1:6, cement:lime:sand) mortar-consistence measured by the dropping ball test as described in BS4551:1970[4] and kept to the standard10±0.5mm penetration

layer of cement paste on the aggregate surface. although this is not usually thought to prevent the aggregate from becoming saturated. The water requirement also may be affected by surface tension forces preventing the water from penetrating the smaller pores and fissures which were accessible to the nitrogen gas.

Effect of water:cement ratio

It appears that the largest single factor affecting the compressive strength of all the mortar specimens in this study is the water:cement ratio. The cube test results of both series are shown in Fig 8, plotted against the water:cement ratio and show 97 per cent correlation. As a large number of sands have been used in this investigation it would seem that the relationship between compressive strength and water:cement ratio is unaffected by the use of different sands and sand gradings.

Effect of sand grading on the compressive strength of mortar

The physical characteristics of the sands have proved unsuitable for forecasting the mortar compressive strength and the only factor left to check is the actual grading curve. From the results of the first test series in Fig 5 it can be seen that the sand with the finest grading (sand no. 1) always produces the lower strengths in all three grades. It is also one of the three sands in the second test series which fails to produce 1:1:6 mortar of strength greater than the 3.6N/mm² required. In fact, these three sands (nos. 1, 2 and 26) are all outside the lower BS grading limit, but this does not mean that all such sands fail to produce satisfactory mortar strengths. Of the remaining 13 sands which produce mortar strengths in excess of 3.6N/mm², six are outside the lower BS grading limit and indeed the two highest mortar strengths are from two of these sands.

The current BS grading limits are shown in Fig 9 along with the range of sand gradings used in the second test series for grade 3 mortar. If the lower grading limit is modified still further and based on the lower boundary of the grading range which produces satisfactory 1:1:6 mortar, then of the 30 sands sampled in this study, 23 would satisfy the limits, four would be outside by a total of 5 per cent or less and three would be more than 5 per cent outside. This is an improvement on only 17 satisfying the current grading limits.

However, increasing the cement content by changing the mortar proportions to 1:1:5, as allowed in BS 5628, enables all of the sands used in the second test series to produce mortar satisfying the grade 3 strength requirement. Therefore, basing the lower grading limit on the range of these 16 sands, means that all 30 sands used in the study would satisfy the proposed grading requirements.

It is proposed that these limits, shown in Table 6, be adopted for sands for general purpose load-bearing mortar as all the sands within these limits will produce mortar of the re-

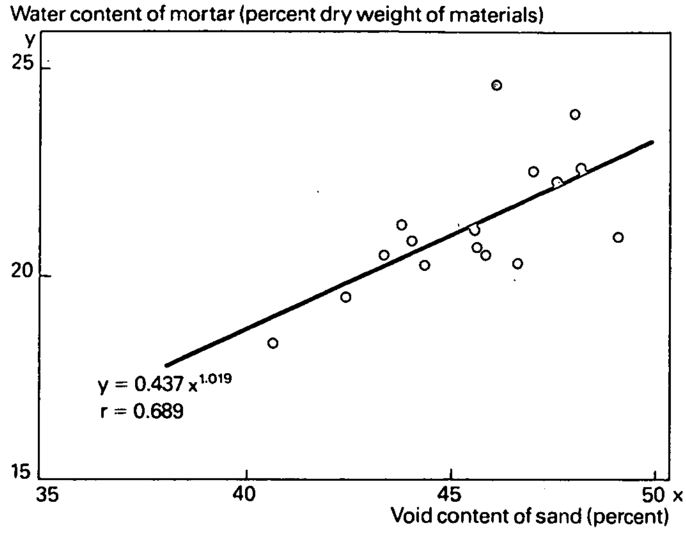
Table 5 **Seven day compressive strengths of 1:1:5 mortar at the standard consistence**

Sand no.	Water:cement ratio	Mean 7-day strength (3 cubes) (N/mm²)	Expected 28-day strength from Sinha's results (N/mm²)
1	1.65	3.64	4.91
2	1.59	3.62	4.89
26	1.53	3.32	4.48

Table 6 **Proposed grading limits for sands for general load-bearing mortar**

Sieve aperture (mm)	5.00	2.36	1.18	600	300	150	75
Percentage passing by weight	100	90-100	70-100	40-100	5-90	0-25	0-10

Fig 6 Relation between void content of sand in a loose condition and the water content of mortar made with the given sand



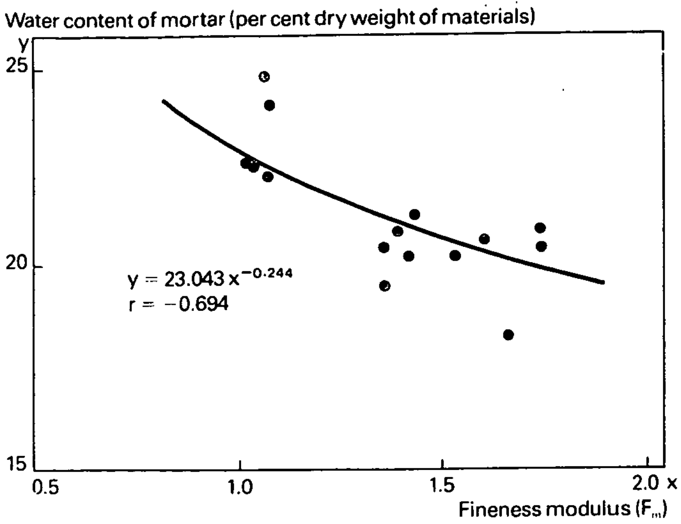


Fig 7 Relation between the fineness modulus of sand and the water content of mortar made from the given sand

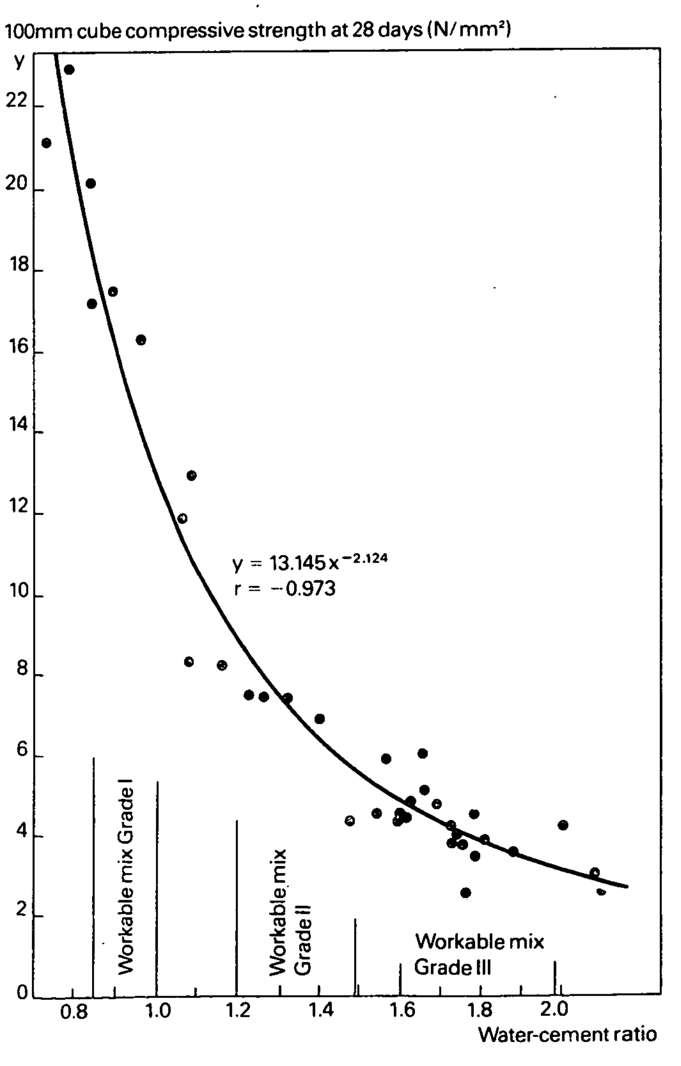
quired strength. From the results of the first series of tests, it can be seen that sands which produce grade 3 mortar of satisfactory strengths, have no difficulty in producing satisfactory grade 1 and 2 mortars.

Conclusions

On the basis of the results obtained in this investigation, we made the following conclusions:

● The water:cement ratio is the most important factor which affects the compressive strength of mortar.

Fig 8 Effect of water:cement ratio on the compressive strength of mortar of grades I, II and III



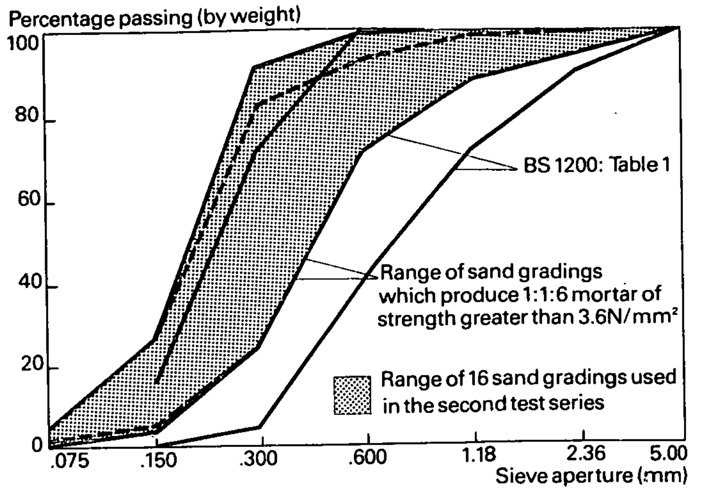


Fig 9 Range of sand gradings used in the second test series compared with current grading limits

● For a certain mortar grade and at a certain degree of compaction there is a maximum mortar compressive strength which occurs at the optimum water content.

● The water requirement of mortar seems to be influenced by the void content of the sand.

● The specific surface of a sand sample, as measured by nitrogen adsorption, is governed mainly by the geological composition of that sample, with sandstone and basalt producing greater specific surfaces than metamorphic rocks. However, no relationship exists between the specific surface as determined by this technique and water requirement of the mortar.

● The new proposed grading limits, based on the availability of the natural sands which produce mortar of satisfactory strength, if accepted by the British Standards Institution, will reduce the wastage of natural resources.

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Factors affecting the brick/mortar interface bond strength

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The paper describes the results of an investigation to assess the effect on brick/mortar interface bond strength of the following variables: i) Moisture content of brick before laying; ii) Load placed on brickwork assembly during curing period; iii) Sand grading. Three different types of bricks having widely different properties and three types of widely different sand gradings were used in the investigation. It appears that the moisture content of bricks before laying and the sand grading both have a marked effect on bond tension and shear strength of brickwork specimens. The bond shear and tensile strengths are not affected by the load placed during the curing period on the brickwork assemblies.

Introduction

One of the primary functions of a mortar is to bind the masonry unit together and thus form a structurally and an environmentally sound wall. The bond or adhesion between mortar is of mechanical nature, and is due to the formation of ettringite crystals. Ettringite crystalises [6] in hexagonal needle-like form with diameter of about 0.05 μm. These crystals must penetrate through the pores of the brick to provide good bond. To understand the fundamental mechanism of bond strength, it is essential to study the formation of these crystals, their dimensions and the pore structure of the bricks. Such studies [6, 14] are expensive, hence less sophisticated experimental method is used to assess the effect of various factors on bond strength. The strength of bond between brick and mortar becomes an important consideration in the design of a brick wall subjected to racking or eccentric load. The strength of the bond between brick and mortar is defined as: i) Bond shear and ii) Bond tension, depending on the way resistance to the load is provided. The bond strength depends on a large number of factors, such as suction rate of bricks [7], moisture content of bricks before laying, water retentivity of mortar [5, 9], consistency of mortar, workmanship, surface [7] characteristics of brick, sand grading and pre-compression [8, 11, 13] etc. The paper describes an experimental investigation to assess the effect of some variables on the bond strength.

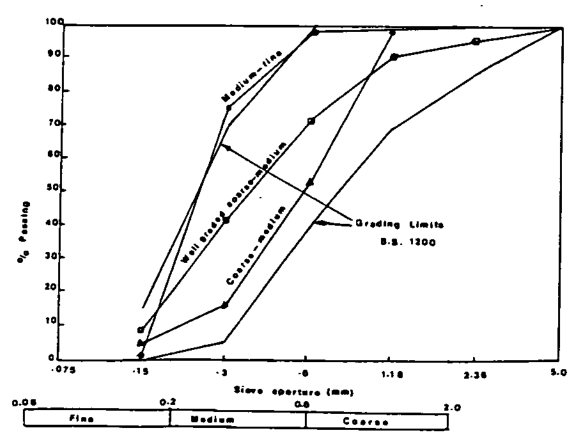

Figure 1. Particle size distribution of sands.

Scope of investigation

The variables affecting the bond shear and bond tension considered in this study were:

i) Moisture content of brick

For a particular type of brick and mortar the bond strength may be affected by the moisture content of bricks at the time of laying. To investigate this, bricks were dipped in water for a period of time ranging from 5 sec. to 2 hours before the test couplets for the tension and shear were made. The moisture content of each brick was determined and the result is shown in Tables 1 and 2.

ii) Loading of fresh brickwork during curing period

Loads were placed on the couplets of brick masonry during the curing period to represent the load occurring in full-size construction resulting from several courses of brickwork laid over. The maximum applied stress of 55 kN/m^2 equivalent to that from a full storey height of brickwork. The result of these tests is given in Tables 3 and 4.

1/6th scale brick in 1:¼:3 mortar was used to study the effect of the variables described above. The water absorption of model bricks as determined by 24hr or 5hr tests were 12.65% and 13.75% respectively. Full-scale bricks in 1:¼:3 mortar were used to study the effect of sand grading [1]. The consistency of the mortar [3] was kept constant in the test.

iii) Sand grading

To study the effect, three different types of bricks (A) 50 N/mm^2 three-hole, (B) 35 N/mm^2 three-hole and (C) 21 N/mm^2 single frog and three building sands having widely different grading curves, were used. It is difficult to classify precisely the sands as the particle size distribution [4] did not rigidly fall within the prescribed boundaries. The sand was identified as coarse-medium, well-graded coarse-medium and medium-fine. The fineness modulus of the various sands were 2.27, 1.95 and 1.23 respectively. The grading curves of the sands used are given in Figure 1. The result of water absorption test [2] for bricks is given in Table 5. These bricks were selected to give a wide variation in the average water absorption value. Table 5 also gives the result of the suction rate of bricks.

Method of test

Bond tension

Cross brick couplet is generally accepted to give quite satisfactory results [5, 9] for bond in tension, hence for full-scale similar specimens were used. In case of 1/6-scale two whole bricks were laid in stackbond, and tensile load with the help

Table 1. Bond tensile strength of brickwork in 1:¼:3 mortar

Condition	Treatment of brick before laying					
	Dry	Dipped in water for 5 sec	Dipped in water for 2 min	Dipped in water for 5 min	Dipped in water for 10 min	Dipped in water for 2 hours
Moisture content %	0.66	3.38	9.91	11.66	11.81	12.17
Bond tensile strength N/mm²	0.14 0.11 0.08 0.48 0.49 0.20 0.13 0.23 0.19	0.39 0.07 0.41 0.17 0.12 0.09 0.69 0.46 0.33	0.59 0.32 0.11 0.23 0.32 0.41 0.45 0.50 0.44 0.46	0.27 0.20 0.05 0.48 0.41 0.48 0.31 0.24	0.05 0.56 0.43 0.28 0.05 0.44 0.30 0.06	0.02 0.048 0.043 0.016 0.040 0.074 0.040 0.016 0.037 0.016
Average	0.23	0.30	0.38	0.31	0.27	0.035

Table 2. Bond shear strength of brickwork in 1:¼:3 mortar

Condition	Treatment of brick before laying					
	Dry	Dipped in water for 5 sec	Dipped in water for 2 min	Dipped in water for 5 min	Dipped in water for 10 min	Dipped in water for 2 hours
Moisture content %	0.66	3.38	9.91	11.66	11.81	12.17
Bond shear strength N/mm²	0.22 0.27 0.41 0.47 0.58 0.52 0.52	0.35 0.32 0.38 0.32 0.67 0.60 0.45 0.59	0.45 0.32 0.32 0.35 0.65 0.60 0.75 0.58 0.32 0.58	0.32 0.27 0.27 0.30 0.61 0.52 0.42 0.67 0.55	0.39 0.32 0.32 0.34 0.15 0.74 0.60 0.36 0.52 0.52	0.20 0.14 0.09 0.15 0.20 0.12 0.06 0.21
Average	0.43	0.46	0.49	0.44	0.43	0.15

of a sand bucket was applied to lower brick as shown in Figure 2. The bucket was filled slowly with sand until failure occurred.

Bond shear test

Full scale shear tests were conducted on 4-brick assembly as shown in Figure 3. In a few samples the load deflection relationship was also obtained. A modified Soil Mechanics shear box was used for the bond shear test (Figure 4) in case of 1/6-scale bricks.

Results

Effect of moisture content of brick

Bond tension and bond shear

From Table 1 and Figure 5 it is clear that the moisture content of the bricks at the time of laying influences the bond strength of brickwork. The results agree with the limited findings of Semenstov, described in Polyakov's [10] work who concluded that the wetting of bricks, before laying with cement mortar substantially increases the bond, but that saturated bricks lead to a large reduction in bond strength.

In this investigation, the optimum bond strength was obtained when the moisture content of the brick was approximately ⅔rd of the value of 24hr water absorption test (24hr moisture absorption 12.65%). The bond strength was the lowest when the bricks were fully saturated. When saturated bricks are laid, they absorb little or no water from the mortar and generally the excess water in the mortar, over and above that required for the hydration of the cement, will remain and could cause the strength of the tensile bond, mortar to brick, to be less than optimum.

When dry bricks are laid they absorb water from the layer of mortar in contact with the brick and there may be insufficient water for the hydration of the cement to take place. Hence, the strength of the tensile bond will be less than optimum.

Bond shear

The moisture content of brick before laying has a marked effect on the bond shear Table 2. A trend similar to bond tensile strength was noticed in this case also.

Effect of load placed on the specimens during curing period

Bond tension and bond shear

The results of tests are given in Tables 3 and 4. From these results there appeared to be no specific relationship between tensile or shear bond strength and applied compression during curing. There was a wide scatter of experimental results indicating the presence of uncontrolled variables such as the surface state of brick etc.

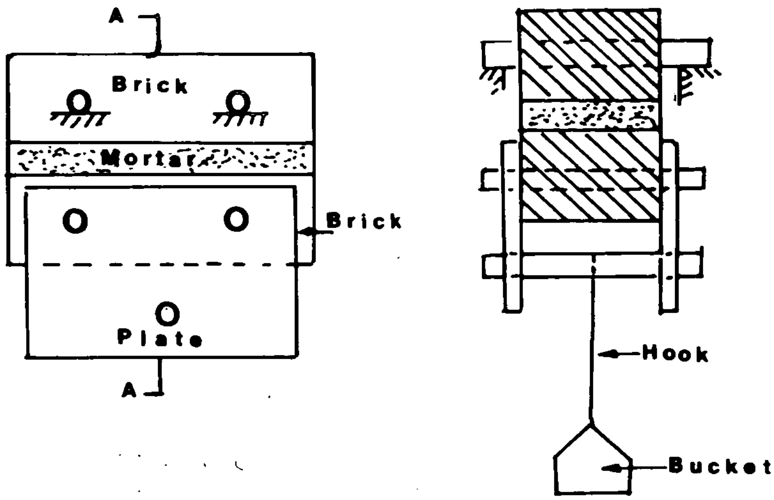

Figure 2. Arrangement for bond-tension test.

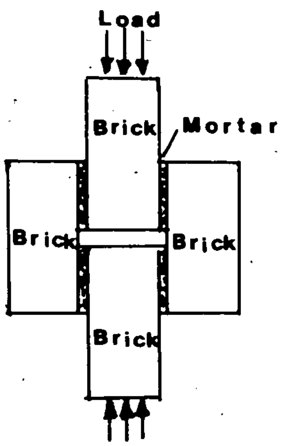

Figure 3. Arrangement for bond shear test for full-scale bricks.

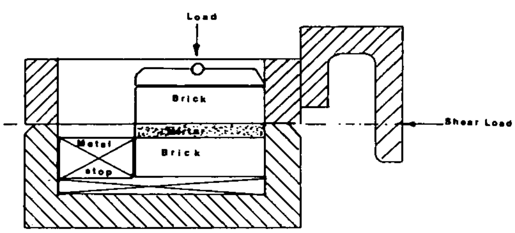

Figure 4. Shear test arrangement for 1/6-scale bricks.

Table 3. Effect of loading during curing period on brickwork bond tension strength

	Stress kN/m²				
	0	13.8	27.6	41.4	55.0
Bond tension strength N/mm²	0.53	0.39	0.22	0.19	0.75
	0.47	0.29	0.41	0.21	0.52
	0.50	0.40	0.57	0.43	0.28
	0.30	0.21	0.42	0.29	0.39
	0.32	0.21	0.28	0.41	0.20
	0.17	0.59	0.31	0.56	0.34
	0.65	0.34	0.20	0.32	0.26
	0.30	0.51	0.41	0.48	0.33
	0.54	0.43	0.24	0.44	0.25
	0.40	0.43	0.22	0.56	0.44
	0.61	0.36	0.30		0.20
Average	0.44	0.38	0.33	0.39	0.36

Table 4. Effect of loading during curing period on brickwork bond shear strength

	Stress kN/m²				
	0	13.8	27.6	41.4	55.0
Bond shear strength N/mm²	0.37	0.60	0.35	0.20	0.15
	0.57	0.25	0.42	0.47	0.25
	0.45	0.45	0.55	0.55	0.50
	0.45	0.22	0.27	0.60	0.35
	0.45	0.57	0.40	0.42	0.40
	0.25	0.65	0.62	0.43	0.69
	0.60	0.22	0.52	0.45	0.45
	0.57	0.33	0.60	0.45	0.40
	0.45	0.40	0.46	0.37	0.55
		0.30	0.50	0.40	0.20
			0.37	0.32	0.51
Average	0.46	0.40	0.46	0.42	0.40

Note: Moisture content of brick before laying = 10.41% for Tables 3 and 4.

Table 5. Properties of full scale bricks

Brick type	Water absorption BS 3491				Suction rate Kg/m²/min			
	Average in %	Range in %	Standard deviation	Coeff. of variation %	Average in %	Range in %	Standard deviation	Coeff. of variation %
A	6.5	5.7– 7.0	0.37	5.69	0.925	0.65–1.05	0.125	13.5
B	15.0	13.4–16.7	1.37	9.13	1.735	1.3 –2.1	0.27	15.5
C	25.6	24.9–26.4	0.44	1.72	2.565	2.1 –3.4	0.42	16.3

Table 6. Effect of sand grading on bond tension strength

Sample no.	Sand	Mortar strength N/mm²	Brick type	Bond tensile strength N/mm²			
				Average	Range	Standard deviation	Coeff. of variation %
1	Coarse -medium	22.1	A	0.178	0.142–0.235	0.037	20.6
			B	0.161	0.093–0.219	0.051	31.7
			C	0.171	0.131–0.226	0.034	19.8
2	Well graded coarse-medium	21.3	A	0.296	0.241–0.35	0.048	16.0
			B	0.222	0.155–0.289	0.049	22.1
			C	0.221	0.122–0.264	0.053	24.1
3	Medium -fine	15.7	A	0.161	0.125–0.185	0.025	15.4
			B	0.142	0.106–0.176	0.023	16.0
			C	0.114	0.075–0.164	0.031	27.1

Sample size: 6 tests each

Table 7. Effect of sand grading on bond shear strength

Sample no.	Sand	Mortar strength N/mm²	Brick type	Shear strength N/mm²			
				Average	Range	Standard deviation	Coeff. of variation %
1	Coarse -medium	22.1	A	0.236	0.190–0.259	0.025	10.6
			B	0.258	0.168–0.309	0.074	28.5
			C	0.339	0.263–0.445	0.066	19.4
2	Well graded coarse-medium	21.3	A	0.479	0.342–0.634	0.102	21.3
			B	0.377	0.345–0.474	0.066	17.4
			C	0.451	0.332–0.547	0.081	18.0
3	Medium -fine	15.7	A	0.162	0.118–0.237	0.043	26.3
			B	0.152	0.129–0.184	0.025	16.2
			C	0.198	0.153–0.271	0.049	25.0

Sample size: 6 tests each

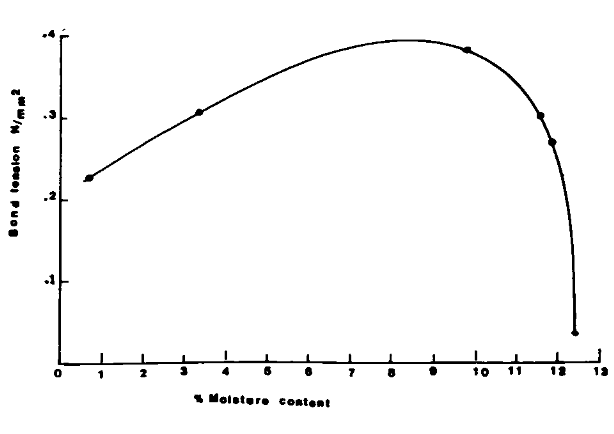

Figure 5. Showing the effect of moisture content of brick on bond tensile strength.

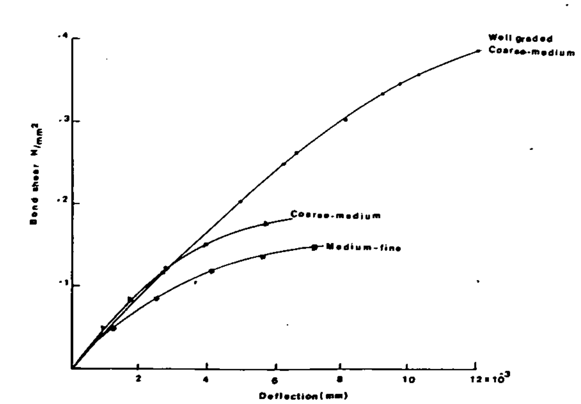

Figure 6. Showing the relationship between shear stress and deflection for three-hole brick (type B).

Effect of sand grading on bond tension and bond shear

From Tables 6 and 7, it is very clear that sand grading affects the mortar/brickwork interface bond strength. The sand defined as well graded coarse-medium (approx. in the centre of the BS grading limits) exhibits a stronger bond both in tension and shear compared to either 'coarse-medium or medium-fine'.

The three types of bricks which had widely divergent properties consistently gave higher interface bond shear and tensile strength for mortar made from medium or well graded sand. In each case the bond shear strength was higher than bond tension strength. Typical load-deflection relationship for the brick type B and the three types of sand gradings is given in Figure 6; and the relationship appears to be non-linear.

Relationship between bond shear and bond tension

From result of the tests, the following general relationship appears to exist between bond tension f_{bt}, and bond shear f_{bs}:

$$f_{bs} = 0.80\, f_{bt}^{0.56} \quad \text{where } f_{bt} \leqslant 0.6 \text{ N/mm}^2 \qquad \text{(i)}$$

Various research workers [8, 10] have suggested a linear relationship, which is true of a particular case, but it is not necessarily generally applicable. Semenstov suggested the following relationship $f_{bs} = 1.7\, f_{bt}$ where $f_{bt} \leqslant 1.8$ Kg/cm^2. The general relationship given in equation (i) agrees reasonably well with others.

Conclusions

The brick/mortar interface bond strength (bond tension and shear) varies considerably with the moisture content of the bricks at the time of laying. For maximum bond strength for a particular cement mortar there would appear to be an optimum value of moisture content, which in the case of the bricks tested is approximately two-thirds of the water absorption of bricks determined by the 24 hour immersion test.

The bond shear and bond tension strengths of the brickwork couplets are not influenced by the load placed on them during the curing period.

Sand grading or the fineness modulus of sand has a mark influence on the brick/mortar interface bond streng (tension and shear) for a variety of bricks of widely differe properties. A well graded coarse-medium sand forms stronger bond than coarse-medium or medium-fine sand.

The bond shear strength was generally higher compar to bond tension strength and the relationship between the appears to be non-linear within the range of the test.

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The Stress/Strain Relationship of Brickwork

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Abstract

Normally the stress/strain relationship of brickwork is determined by testing specimens under compressive loads perpendicular to the bed joint. In flexural members, however the compressive force is usually parallel to the bedjoint. Tests were carried out on brickwork specimens to determine the stress/strain relationship and compressive strength parallel to bedjoint. Four different strengths of brick and two types of prism, built to represent the top course and top three courses of the compression zone of reinforced or prestressed brickwork flexural members were used. The stress/strain relationship for the four different strengths of brick are illustrated and it is shown that the compressive strength of brickwork can vary considerably depending on the type of prism tested.

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1.1 Introduction

It has been recognised for a long time that brickwork is an anisotropic material with different elastic and strength properties in different directions.

In the past (1,2,3) most attention has been given to measuring the properties of brickwork in the direction normal to the bedjoint, mainly to gain insight into the behaviour of load bearing wall systems.

However, with the ongoing development of reinforced and prestressed brickwork as flexural members it becomes increasingly important to consider the stress/strain and strength properties of brickwork loaded in the direction parallel to the bedjoint, as would occur in the compression zone of a reinforced brickwork beam.

During tests on prestressed brickwork beams (4) carried out by the authors it was observed that splitting along the uppermost bed joints occurs in the compression zone prior to failure. This implies that at high levels of load the compressive forces are carried by the brickwork acting as individual courses rather than a monolithic homogenous brickwork section. It was decided, therefor to consider two types of prism in the test programme, one to represent the upper three and the other to represent the top course of the compression zone of a reinforced brickwork beam.

The results of the tests on prisms under axial compression are presented and the stress/strain relationships for four different strengths of brick using the two types of prism are also given. It is shown that the compressive strength of brickwork is influenced by the type of prism used.

A stress/strain relationship is proposed and it is also shown that this represents well the stress/strain relationship for the four strengths of brick.

2.0 Materials and test specimens

2.1 Bricks

Four different strengths of brick were considered. The first three types were three hole bricks of high (82.03 N/mm^2), medium (67.58 N/mm^2) and low (34.18 N/mm^2) strengths. The fourth type was a single frogged brick (22.72 N/mm^2). Compressive tests were carried out on random samples of 10 bricks in each of the three possible directions and the results are given in table 1. All bricks were tested between 6mm thick plywood sheets and in the case of the frogged bricks the frogs were filled with a mortar mix to give a flat surface for testing. The average 24 hours absorption for each type of brick is also given in table 1.

2.2 Mortar

A 1:$\frac{1}{4}$:3 (cement:lime:sand) mix by volume was used throughout. Control cubes were taken and the average compressive strength was 19.1 N/mm^2.

2.3 Test specimens

The two types of prisms used are illustrated in Fig. 1. The three course (type A) prism Fig. 1(a) has a ratio of height to least lateral dimension (h/t) of 2.1 and the single course prism (Type B) has a h/t ratio of 5.0. All specimens were built on wooden battens on the floor of the laboratory by an experienced bricklayer. Each prism was then allowed to cure under polythene for 28 days before testing.

3.0 Experimental procedure

Each prism was capped and levelled using a mix similar to the mortar used in their construction. The prisms were placed in a hydraulic testing machine with a capacity of 2500 kN. Six mm thick plywood sheets were placed between the prisms and the platens of the testing machine to help distribute the load and reduce and platen friction effects.

The strain was measured at six points on the three course prisms and four points on the single course prism using a 'demec gauge' (fig. 1) over a length of 150mm across the central horizontal axis of each prism. The prisms were then tested in axial compression. The load was applied in equal increments the magnitude of which depended on the type of brick and prism being tested. At each increment the load was held constant to enable strain readings to be taken. At between 60 and 70% of the expected failure load of the prisms the load increment was halved. Strain measurements were then taken up to approximately 95% of the ultimate load.

3.1 Experimental Results and Discussion

The experimental stress/strain relationships are presented graphically for both types of prism, in fig. 2 for low strength bricks, fig. 3 frogged brick, fig. 4 medium strength and fig. 5 high strength. The compressive strengths of the prisms are given in table 2. The compressive strengths of the brick on end and on edge (table 1) are considerably lower than the compressive strength in the bedjoint direction.

For all types of brick and prism the stress/strain relationship is initially linear after which the strain increases more rapidly than the stress. During the tests on the three course prisms it was observed that vertical splitting of the bedjoints occurred at approximately 60% of the ultimate load in the case of the high strength brick, 80% for the medium strength, 90% for the low strength and 59% for the frogged brick. Up to the point of splitting the strain readings for each prism has been very uniform indicating that axial strain was occurring, after splitting had occurred there was a redistribution of load indicated by the strain measurements and it appeared that each course was acting individually and carrying a different proportion of the applied load. The strain measurements at particular points on the prisms showed considerable variation with some indicating strains of similar magnitude to those obtained in the single course prisms, as failure approached. Failure usually occurred by crushing of one or two of the courses, simultaneous crushing of all three courses was not observed. Strain readings taken on the single courses indicated axial compression up to higher levels of load. Failure of the single course prisms was initiated by vertical splitting through the brick followed by crushing.

Comparison of the results obtained from both types of prism for the high and medium strength bricks show that although the stress/strain relationships have the same general characteristics the compressive strengths are 40 and 50% higher for the single course prism for the high and medium strengths respectively. The ultimate strains are approximately 40% higher for the single course prisms.

The three course frogged brick prisms showed a slightly greater compressive strength than the single course prisms and the ultimate strains were very similar. The compressive strengths for both types of prism built of the low strength brick are almost identical and only slightly less than the compressive strength of the brick tested on edge (table 1), although much higher strains were experienced in the single course prisms.

It appears therefore, that when splitting occurs in the three course prisms built of the high and medium strength bricks the redistribution of load causes premature failure of one or two of the courses and that if splitting could be prevented the average compressive strength would be considerably higher.

.0 Analysis of Results

To obtain a more general picture of the behaviour of the brickwork the stress/strain results were plotted in non dimensional form for each of the two types of prism (Figs. 6 and 7). A least square approximation using polynomials of degree 2, 3, 4 and 8 were used to obtain relationships to fit the experimental results.

A comparison of the results of the regression analysis are presented in table 3. The area under the curve and the centroid are not greatly influenced by the degree of the polynomial chosen for both types of prism. The standard deviation for both types of prism is less for the three degree polynomial and it would appear that the three degree or cubic equation is best suited, and they are shown in figs. 6 and 7.

The two equations obtained are, for three course prisms

$$\sigma/\sigma_m = -0.00407 + 2.1043\,(\epsilon/\epsilon_m) - 2.0252\,(\epsilon/\epsilon_m)^2 + 0.9234\,(\epsilon/\epsilon_m)^3$$

for the single course prisms,

$$\sigma/\sigma_m = -0.005756 + 2.3961\,(\epsilon/\epsilon_m) - 2.2567\,(\epsilon/\epsilon_m)^2 + 0.8664\,(\epsilon/\epsilon_m)^3$$

.1 Conclusions

1. The use of three course prisms to obtain the compressive strength of brickwork parallel to the bedjoint may result in a reduced estimate of the compressive strength when using high or medium strength bricks.

2. The compressive strength of the low strength and frogged brick is not great influenced, by using one or three course prisms as the compressive strength of the brickwork is very close to the compressive strength of the brick.

3. The stress/strain relationship of brickwork parallel to the bedjoint may be well represented by a 3 degree polynomial based on results obtained from tests on single course prisms.

cknowledgements

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otation

σ stress in brickwork

σ_m compressive strength of brickwork

ϵ strain in brickwork

ϵ_m ultimate strain in brickwork

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TABLE 1 COMPRESSIVE STRENGTH & ABSORPTION OF BRICKS

	HIGH	MEDIUM	LOW	FROGGED
COMPRESSIVE STRENGTH IN BED JOINT				
ave N/mm^2	82.03	67.58	34.18	22.72
RANGE "	64.80-96.70	44.16-91.08	30.70-40.88	15.47-28.84
std. deviation N/mm^2	5.85	12.20	2.79	3.36
coef. of variance %	7.13	18.06	8.16	14.81
Compressive strength on edge				
ave N/mm^2	53.17	26.36	11.48	16.95
Range	33.54-68.03	19.32-39.36	7.51-20.21	12.52-20.21
std. deviation	9.43	5.71	3.54	2.237
coef. of variance	17.73	21.66	30.84	13.20
Compressive strength on end				
ave N/mm^2	40.23	23.23	10.67	15.81
Range	30.00-50.76	10.79-33.80	7.54-18.76	12.62-21.54
std. deviation	6.94	5.90	3.24	2.33
coef. of variance	17.25	25.40	30.37	14.71
24 hrs absorption %	4.17	5.71	7.20	23.85
Range	3.21-4.71	4.62-8.07	6.82-7.74	23.41-24.19
std. deviation	0.461	0.90	0.303	0.244
coef. of variance	11.06	15.80	4.21	1.02

TABLE 2 COMPRESSIVE STRENGTH OF PRISMS

BRICK TYPE	HIGH STRENGTH		MEDIUM STRENGTH		LOW STRENGTH		FROGGED BRICK	
	THREE COURSE	SINGLE COURSE	THREE COURSE	SINGLE COURSE	THREE COURSE	SINGLE COURSE	THREE COURSE	SINGLE COURSE
COMPRESSIVE STRENGTH N/mm^2	22.15	33.17	9.82	19.85	10.05	9.53	8.98	5.56
	19.44	29.30	14.4	18.86	9.26	10.73	8.25	5.82
	18.17	47.63	10.2	29.3	9.75	8.94	9.23	8.07
	22.21	29.70	12.17	28.20	8.25	8.59	14.15	7.87
	20.30	25.92	14.4	25.09	9.23	8.23	13.20	5.80
	20.60	29.58	12.94	21.10	9.64	10.20	12.11	8.35
AVERAGE	20.48	32.56	12.36	23.70	9.36	9.37	10.99	6.91
std. dev.	1.43	7.06	1.82	4.05	0.57	0.88	2.26	1.20
coefficient of variance	6.97	21.69	14.70	17.10	6.06	9.40	20.61	17.30

TABLE 3 Comparison of Regression Analysis

Degree of polynomial		2	3	4	5
Std. deviation	three (A)	0.0943	0.0903	0.0920	0.0917
	single (B)	0.0643	0.0589	0.0609	0.0606
Area under curve	three (A)	0.605	0.604	0.604	0.603
	single (B)	0.660	0.656	0.656	0.658
Centroid	three (A)	0.402	0.374	0.374	0.374
	single (B)	0.381	0.374	0.383	0.382

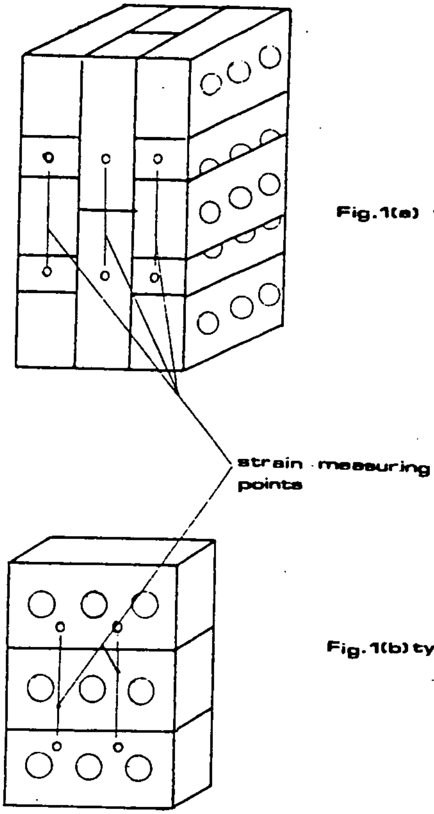
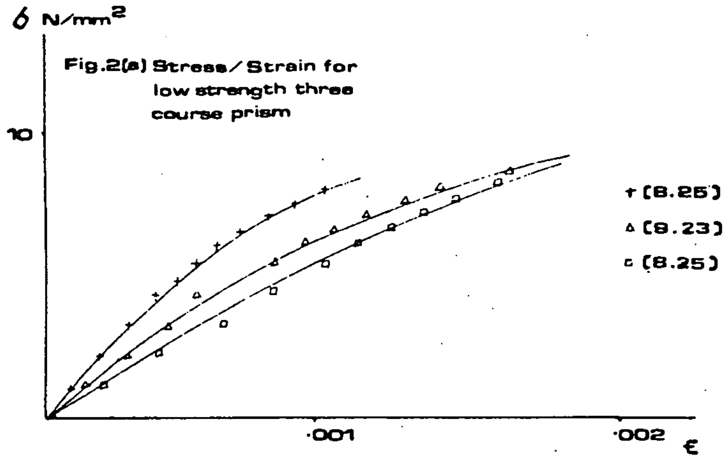
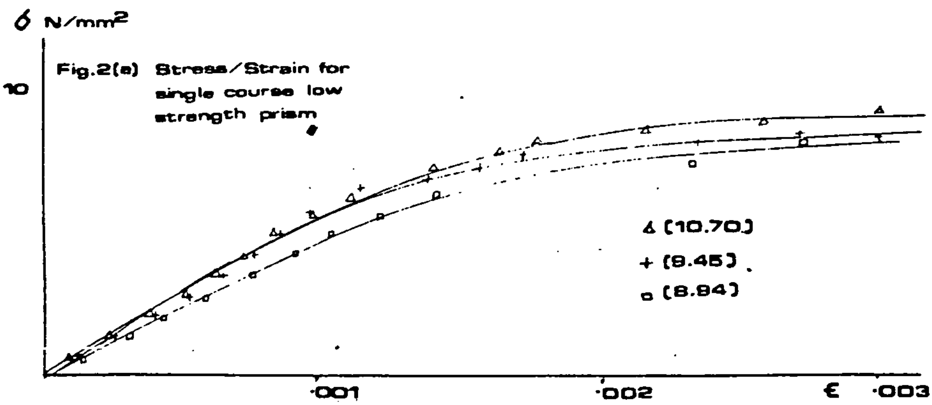


Fig.1(a) type A

Fig.1(b) type (b)

Fig.1 Test Prisms


б N/mm²
Fig.2(a) Stress/Strain for low strength three course prism
10
·001
·002
ε
+ (8.25)
Δ (8.23)
□ (8.25)


б N/mm²
Fig.2(a) Stress/Strain for single course low strength prism
10
·001
·002
ε
·003
Δ (10.70)
+ (8.45)
□ (8.84)

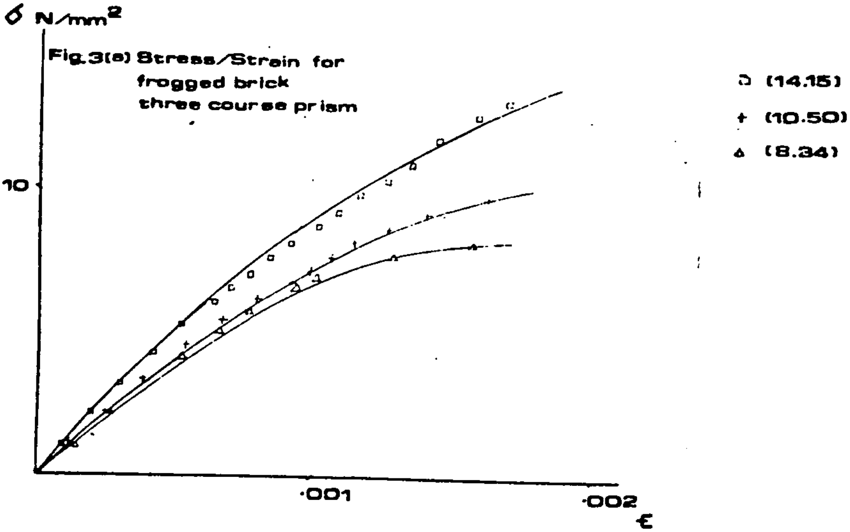


Fig.3(a) Stress/Strain for frogged brick three course prism

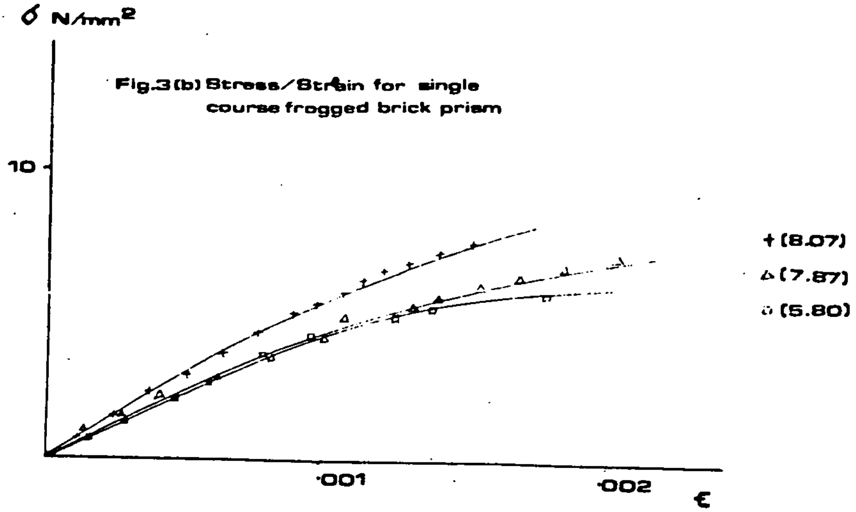


Fig.3(b) Stress/Strain for single course frogged brick prism

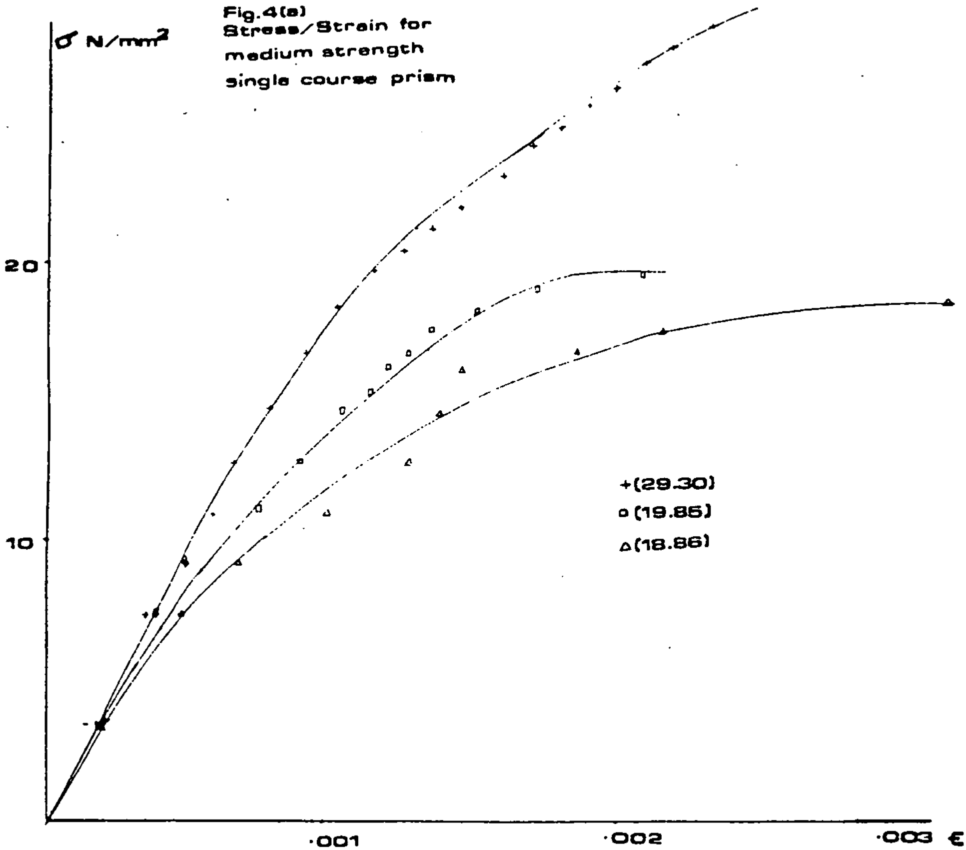


Fig.4(a)
Stress/Strain for
medium strength
single course prism
σ N/mm²
20
10
·001
·002
·003 ε
+(29.30)
▫(19.85)
Δ(18.86)

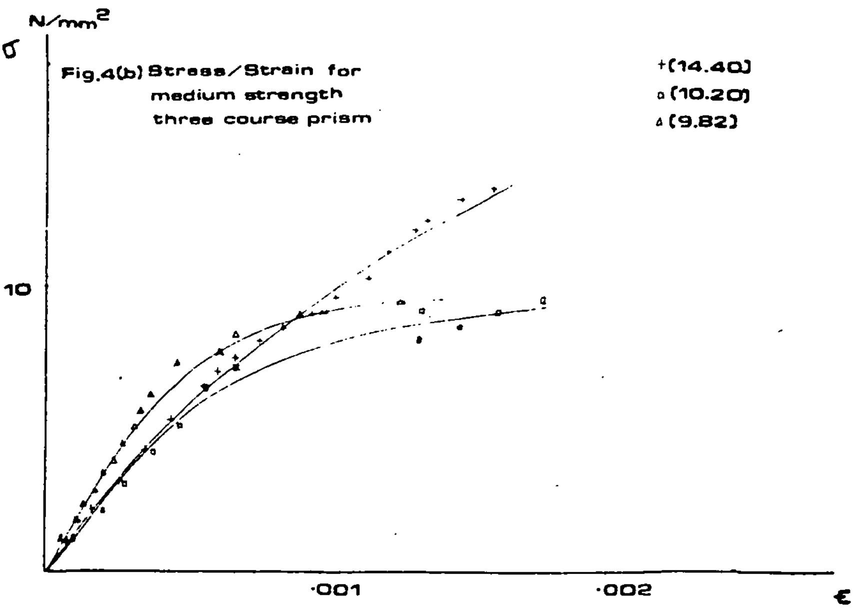


Fig.4(b) Stress/Strain for
medium strength
three course prism
σ N/mm²
10
·001
·002
ε
+(14.40)
▫(10.20)
Δ(9.82)

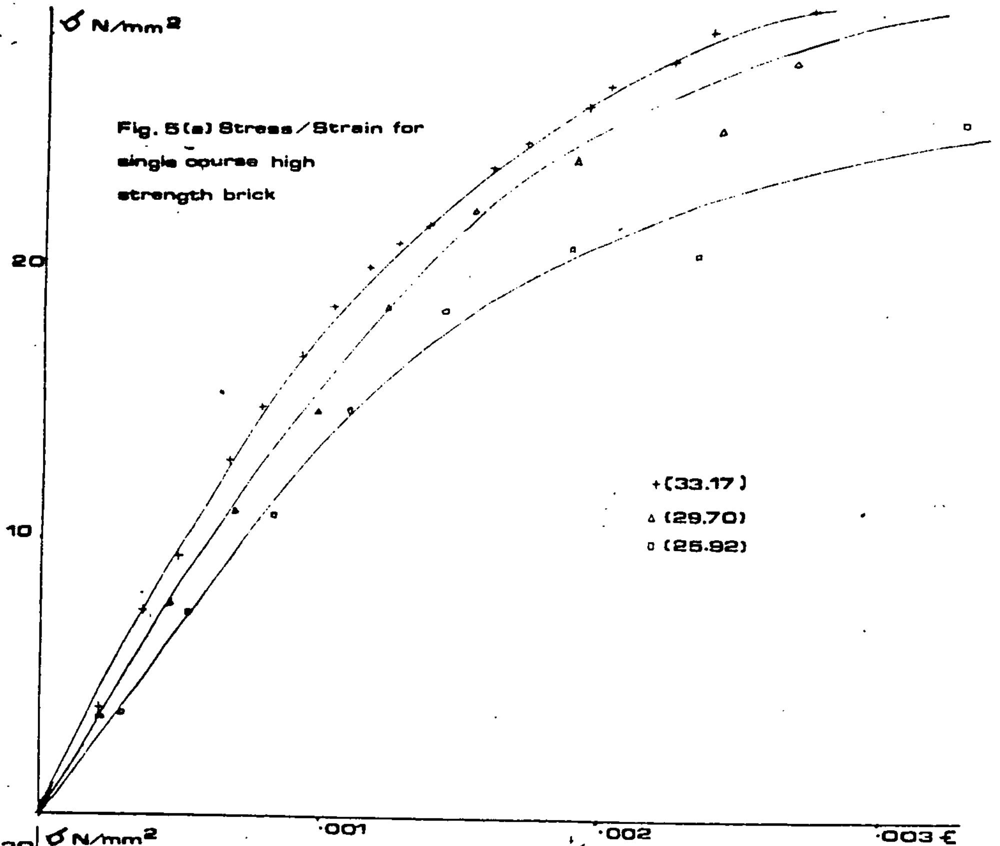


Fig. 5(a) Stress/Strain for single course high strength brick

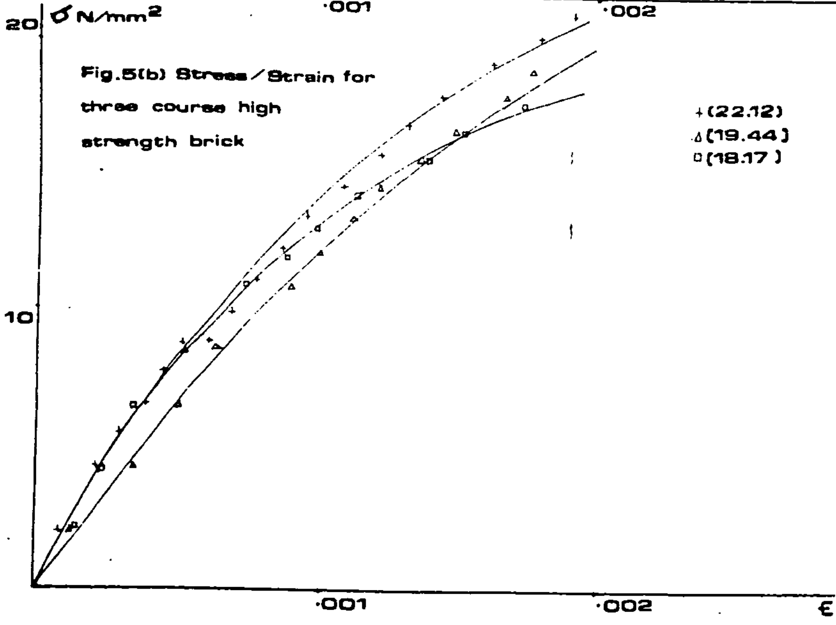


Fig.5(b) Stress/Strain for three course high strength brick

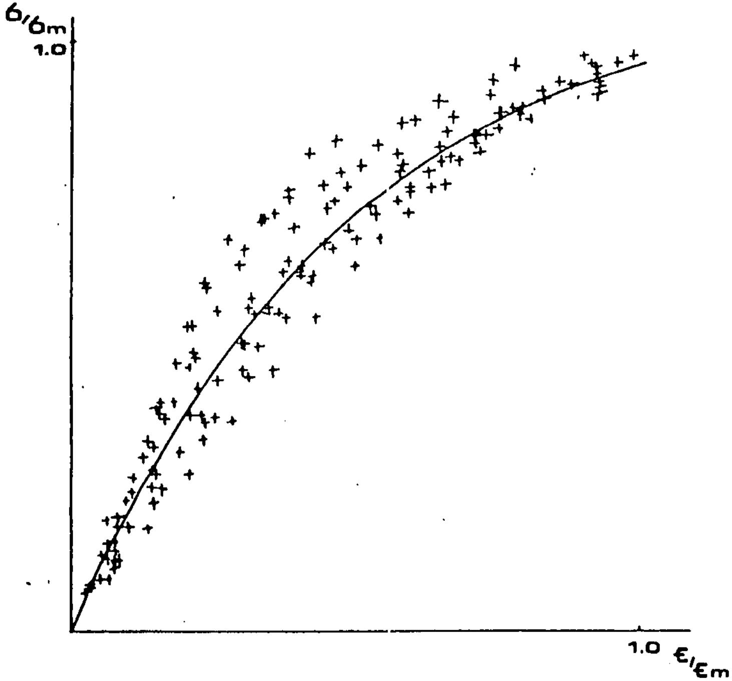


Fig. 6 Non-dimensional stress/
strain relationship based on
single course prisms.

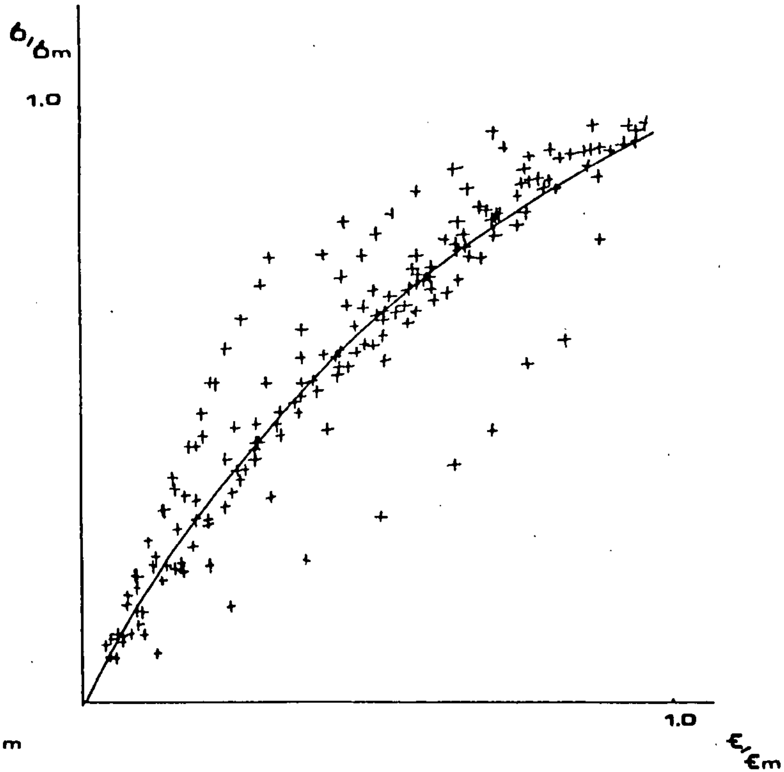


Fig.7 Non-dimensional stress/
strain relationship based
on three course prisms.

Compressive strength and some elastic properties of brickwork

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he paper describes the result of tests on a number of full scale brick and brickwork prisms. Six different types of bricks and rickwork prisms built in 1:¼:3 and 1:½:4½ mortar were subjected to axial compression in different orthogonal directions. he stress-strain relationship of brickwork is presented in non-dimensional form and some characteristics defining the stress lock are given. Relationship between modulus of elasticity and brickwork compressive strength is also given.

IOTATION

m	compressive strength of brickwork	λ_1, λ_2	stress block factors
	compressive stress in brickwork	r	correlation coefficient
n	ultimate strain of brickwork	E	modulus of elasticity of brickwork
	strain in brickwork	n	neutral axis depth

ntroduction

listorically brickwork has been used mainly for compression nembers and hence the stress-strain relationship and its ompressive strength in the bed joint direction were the ubject of investigation for a considerable period of time. his led to the practice of testing bricks and brickwork on ie bed which served as an index value for the code. However is situation has changed to a great extent with the use of einforced and prestressed brickwork as flexural members. 1 this type of situation compressive stresses can develop in rickwork in the other two directions. It is therefore neces-ıry to know the stress-strain relationship and the strength roperties of brick and brickwork in the two orthogonal irections to be able to calculate the ultimate strength and ehaviour of brickwork flexural elements. Very little work as been done previously [1, 2, 3] and hence an investiga-on was carried out on different types of bricks and prisms ıade from them.

laterials

ricks

total of six different types of bricks were used varying om high to low strength, 'perforated and frogged'. Com-ressive strength tests were carried out according to BS 3921 4]. The same method was adopted for determining the ompressive strength in the two other orthogonal directions. he results of these tests and of 24-hour or 5-hour water osorption tests are given in Table 1.

ortar

wo grades of mortar, a 1:¼:3 (cement : lime : sand) grade I nd a 1:½:4¼ (cement : lime : sand) grade II mixes by olume were used. It is highly unlikely that a grade III mortar :1:6) would be used for reinforced or prestressed brick-rk, hence this was not included in the investigation.

ickwork prisms

c different types of prisms were built as shown in Figure 1. ism types A and B represented that section which would subjected to compression in a reinforced brickwork wall beam. Prism types C and D represent the compression ne of prestressed brickwork beams. These prisms were iilt and tested as part of research programmes investigating e behaviour of reinforced and prestressed brickwork xural members.

Some grouted cavity brickwork prisms of single frogged brick in 1:¼:3 mortar were also built (type E and F in Figure 1) to compare with the other prisms.

All specimens were built by an experienced bricklayer and cured under polythene for 28 days in the laboratory.

Experimental procedures

The prisms were capped and levelled using either a rich mortar mix or dental plaster prior to testing. Six-mm thick plywood sheets were placed between the prism and the 'platens' of the machine to help distribute the load evenly. The strains in the brickwork were measured using a 'demec' gauge. These measurements were taken on opposite faces of the prism at two or three different points. At the beginning of the test it was ensured from the strain measurements on the different faces that this load was being applied axially to the specimen. Application of the load was in small, equal increments and held constant at each stage as strain measurements were taken. At between 60 and 70% of the ultimate load the load increment was normally halved. Strain measurements were taken up to between 90 and 95% of the ultimate load.

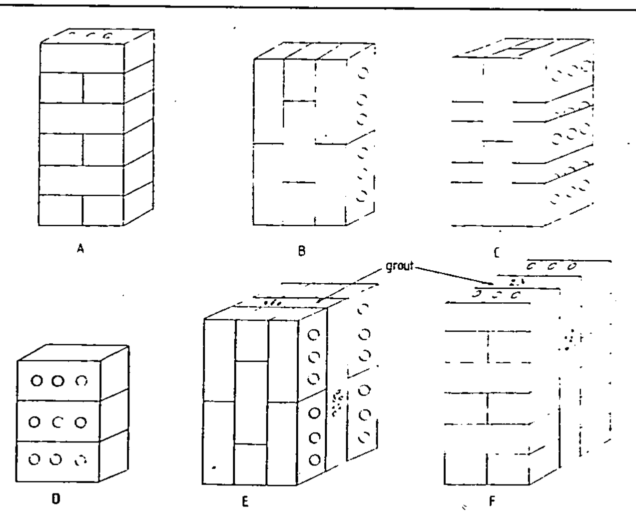

Figure 1. Prism types.

Results

Strength of brick in three orthogonal directions

From Table 1 it appears that the compressive strength of bricks in the three orthogonal directions is different. The highest compressive strength is obtained by testing the bricks on bed and this may be due to the lateral restraint provided by 'platen friction'. The strength of a brick in three orthogonal directions is given in Table 2 as percentage of strength of brick tested flat. No common relationship exists between the strength obtained in three orthogonal directions for different types of bricks. In the case of three hole bricks the compressive strength (Table 2) on edge was the lowest. A similar trend was found by Foster and Lenczner [5] in their test on solid bricks. In the case of three hole bricks the lowest strength was found [5] on end which is different from this work. However, the ratio of the size of the holes in relation to the solid cross-section may be different and greatly influence the results. In general the strength of bricks on edge was between 31 to 78% (Table 2) compared to bed-joint direction and for bricks on end the value varied from 30 to 75% in this investigation.

Strength of prism

The strength of various prisms are given in Table 3. Prism type C, which is bonded, gives a lower compressive strength than type D for same brick and mortar with the only exception being the single deep frogged bricks.

Table 4 compares the strength of prisms in two directio
For convenience, the average strength of the prisms test on end and grouted cavity prisms is expressed as percenta of prisms tested flat. For single and double frog bricks (lo medium strength) it can be seen that higher compressi strength is obtained for prism on end (Type B) compar to prism tested flat (Type A) for both grades of mortar.

This situation is reversed for high strength (three hol bricks which may be due to substantial reduction in bri strength (calculations based on the gross cross-sectional are on end compared with the strength on flat (bed). Howeve the reduction in brickwork strength is only 17 to 19 whereas the reduction in brick strength is 70%. As both typ of prisms were of same height, the 'platen friction' may n affect the result whereas for the brick tested in three orth gonal directions this effect definitely influences the co pressive strength, resulting in a higher value for the bric tested flat.

From the limited tests it appears that the compressi strength (Table 4) of grouted brickwork prisms is high than brickwork prisms built in similar brick mortar co bination tested flat.

Brickwork/brick strength

For all the prisms tested the ratio of brickwork to bri strength varies from 0.2 to 0.6 if the brick strength in t bed joint direction is taken into account. This ratio increas if the brick strength on end is used and varies from 0.3

Table 1. Properties of the bricks used

Type of brick	Compressive strength (N/mm²)			Range (N/mm²)			S.D. (N/mm²)			Co-eff. of var. in %			Water absorption %	
	On bed	On end	On edge	On bed	On end	On edge	On bed	On end	On edge	On bed	On end	On edge	24 hrs.	5 hrs.
1 Single frog	21.55	16.10	16.95	17.77–24.29	11.00–20.00	12.52–20.21	2.14	2.46	2.24	9.94	15.3	13.2	–	19.4
2 Double shallow frog	59.38	31.92	–	56.5–65.0	21.3–43.5	–	3.48	6.18	–	5.86	19.4	–	–	8.9
3 Three holes	88.33	26.4 *30.9	–	74.46–101.48	22.64–40.25	–	10.05	5.9	–	11.37	22.36	–	–	4.34
4 Three holes	82.03	53.17	40.23	64.8–96.70	33.54–68.03	30.0–50.76	5.87	9.43	6.94	7.13	17.73	17.25	4.17	–
5 Three holes	67.58	26.36	23.23	44.16–91.1	19.32–39.36	10.79–33.8	12.2	5.71	5.9	18.06	21.66	25.4	5.71	–
6 Three holes	34.18	11.48	10.67	30.7–40.88	7.51–20.21	7.54–18.76	2.79	3.54	3.21	8.16	30.84	30.37	7.20	–

*Net area.

Table 2. Comparison of strength of bricks in three orthogonal directions

Compressive strength	Single frog	Shallow double frog	Three holes			
			Type 3	Type 4	Type 5	Type 6
Flat	100	100	100	100	100	100
On end	75	54	30 *35	65	39	33
On edge	78	–	–	49	34	31

*Net area is considered.

o 1.12. However, the brickwork strength cannot be greater han the brick strength. The upper limit of the ratio reduces o 0.95 if the net area of brick is used to calculate the compressive strength.

Modulus of elasticity

The modulus of elasticity of brickwork increases with the compressive strength (Figure 2). The relationship may be epresented empirically by the following expression,

$$E = 1180 \cdot \sigma_m^{0.83} \qquad (1)$$

This relationship has been obtained statistically by fitting he 'best' curve. There is quite a wide scatter of results which eflects the variability in bricks, influence of mortar, prism ype and workmanship.

Stress/strain relationship

The general characteristics of the stress/strain relationships for all the prisms tested were essentially the same, initially here was a linear relationship after which the strain increased apidly compared to stress. Typical stress/strain relationships are shown in Figure 3.

Although, the strain measurements were taken up to 95% of the ultimate stress, the peak strain at the time of failure was mathematically extrapolated from the experimental esults. The correlation coefficients for any stress-strain curve was more than 99.5% (Table 3). A similar technique has been used by others [6] for the brickwork loaded in bed joint direction. The strain at ultimate stress is difficult to measure without very sophisticated equipment which has been done n a few cases [3, 7]. Although, in the bed joint direction a falling branch was recorded by Hodgkinson et al [7] no falling branch was detected under similar controlled conditions in case of brickwork loaded in other direction [3] which confirms that maximum strain occurs at peak stress. This is not uncommon for any brittle material.

To obtain a more general appreciation of the behaviour, the stress/strain relationship of each prism was expressed in nondimensional form. A mathematical representation of the experimental stress/strain relationship was then obtained by using a least squares approximation technique in which the nondimensional stress/strain relationship was expressed in terms of a polynomial as:

$$\frac{\sigma}{\sigma_m} = a + b\left(\frac{\epsilon}{\epsilon_m}\right) + c\left(\frac{\epsilon}{\epsilon_m}\right)^2 + d\left(\frac{\epsilon}{\epsilon_m}\right)^3 \qquad (2)$$

By combining the nondimensional stress/strain relationships for all prism types irrespective of brick or mortar strength the following average relationship was obtained:

$$\frac{\sigma}{\sigma_m} = -0.0061 + 2.265\left(\frac{\epsilon}{\epsilon_m}\right) - 2.092\left(\frac{\epsilon}{\epsilon_m}\right)^2 + 0.834\left(\frac{\epsilon}{\epsilon_m}\right)^3 \qquad (3)$$

which is based on over 1200 data points, Figure 4. Equation (3) indicates that there is a small value assigned to the term 'a' which suggests that the curve does not pass through the origin, this is not physically correct and it is due to the statistical nature of the analysis. However the value of 'a' is so small that it may be ignored as it has a negligible effect on the stress/strain relationships.

Stress blocks

For the calculation of the ultimate strength of reinforced or prestressed brickwork four parameters of the stress block relating to the brickwork are required. The first two of these σ_m and ϵ_m relate to the compressive strength and ultimate strain of the brickwork respectively. The second two para-

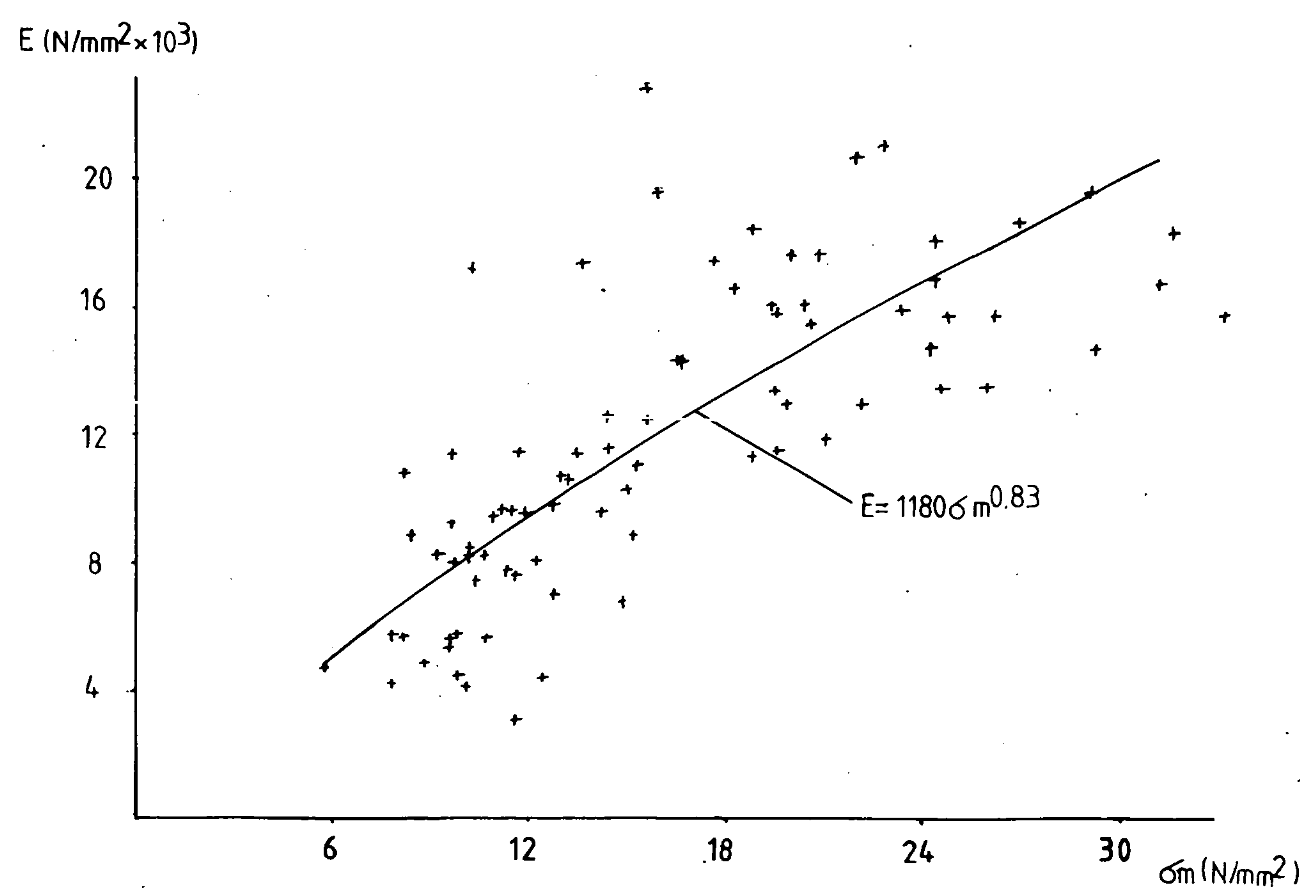

Figure 2. Relationship between compressive strength and elastic modulus.

Table 3. Prism test results

Brick strength N/mm²	Mortar grade	Prism type	Compressive strength σ_m N/mm²	Ultimate strain $\epsilon_m \times 10^5$	Stress block factors λ_1	λ_2	Correlation coefficient r
88.33	1:¼:3	B	20.8	173	0.617	0.361	1.000
,,	,,	B	24.53	236	0.604	0.364	0.999
,,	,,	B	20.47	177	0.598	0.389	0.999
,,	,,	B	24.19	205	0.610	0.369	0.999
,,	,,	B	20.37	156	0.586	0.353	0.999
,,	,,	B	21.90	179	0.652	0.376	0.999
,,	,,	B	18.80	220	0.725	0.397	0.997
,,	,,	B	17.20	255	0.780	0.429	0.998
88.33	1:¼:3	A	31.62	319	0.661	0.386	1.000
,,	,,	A	31.43	265	0.629	0.373	1.000
,,	,,	A	26.20	205	0.549	0.347	0.998
88.33	1:½:4½	B	20.06	180	0.653	0.366	0.999
,,	,,	B	18.79	149	0.605	0.371	0.999
,,	,,	B	24.28	219	0.627	0.372	0.999
88.33	1:½:4½	A	27.00	201	0.594	0.406	1.000
,,	,,	A	24.79	201	0.554	0.345	0.999
,,	,,	A	24.40	209	0.611	0.365	0.997
59.38	1:¼:3	B	15.71	200	0.643	0.375	0.999
,,	,,	B	19.40	173	0.633	0.359	1.000
,,	,,	B	17.64	232	0.644	0.372	0.996
,,	,,	B	23.45	158	0.598	0.365	1.000
,,	,,	B	22.85	176	0.582	0.349	0.999
,,	,,	B	19.38	168	0.624	0.366	1.000
,,	,,	B	16.60	280	0.707	0.388	0.998
,,	,,	B	13.80	245	0.774	0.409	0.999
59.38	1:¼:3	A	12.71	153	0.546	0.339	0.999
,,	,,	A	11.52	155	0.503	0.323	1.000
,,	,,	A	11.43	159	0.578	0.354	1.000
59.38	1:½:4½	B	15.68	116	0.661	0.368	0.999
,,	,,	B	17.59	170	0.669	0.386	0.998
,,	,,	B	15.95	134	0.614	0.364	0.998
59.38	1:½:4½	A	9.58	111	0.597	0.358	0.998
,,	,,	A	11.05	149	0.589	0.359	0.998
,,	,,	A	13.59	221	0.696	0.390	0.998
21.55	1:¼:3	B	15.00	345	0.657	0.399	0.997
,,	,,	B	16.56	180	0.623	0.368	0.999
,,	,,	B	10.00	361	0.587	0.363	0.999
,,	,,	B	11.64	346	0.521	0.347	0.998
,,	,,	B	11.32	289	0.591	0.378	0.997
,,	,,	B	15.10	286	0.657	0.388	0.999
,,	,,	B	12.26	369	0.619	0.373	0.995
,,	,,	B	10.95	165	0.566	0.354	0.991
21.55	1:¼:3	B	13.00	252	0.745	0.402	0.999
,,	,,	B	13.40	235	0.761	0.404	0.996
21.55	1:¼:3	A	12.83	269	0.573	0.364	0.996
,,	,,	A	7.78	268	0.687	0.375	0.999
,,	,,	A	12.26	276	0.632	0.380	0.999
,,	,,	A	9.43	272	0.658	0.377	0.999
21.55	1:½:4½	B	11.75	105	0.523	0.343	1.000
,,	,,	B	11.93	165	0.582	0.357	1.000
,,	,,	B	13.01	234	0.677	0.384	0.999
21.55	1:½:4½	A	9.76	278	0.744	0.396	0.996
,,	,,	A	10.45	224	0.643	0.375	1.000
,,	,,	A	10.13	245	0.703	0.390	0.999
21.55	1:¼:3	D	8.07	174	0.585	0.350	1.000
,,	,,	D	7.87	258	0.618	0.369	0.996
,,	,,	D	5.80	175	0.633	0.363	0.999

able 3. Prism test results *(continued)*

rick strength N/mm²	Mortar grade	Prism type	Compressive strength σ_m N/mm²	Ultimate strain $\epsilon_m \times 10^5$	Stress block factors λ_1	λ_2	Correlation coefficient r
21.55	1:¼:3	C	10.49	171	0.583	0.330	0.999
"	"	C	8.33	161	0.654	0.372	0.998
"	"	C	14.15	201	0.607	0.362	0.998
82.03	1:¼:3	D	33.17	307	0.606	0.371	0.995
"	"	D	29.30	318	0.654	0.372	0.998
"	"	D	25.92	353	0.667	0.370	0.998
82.03	1:¼:3	C	22.12	224	0.616	0.371	0.999
"	"	C	19.44	201	0.580	0.354	0.999
"	"	C	18.17	190	0.634	0.378	0.999
67.58	1:¼:3	D	19.85	230	0.695	0.388	0.997
"	"	D	18.86	306	0.705	0.390	0.994
"	"	D	29.30	253	0.626	0.371	0.999
67.58	1:¼:3	C	14.40	165	0.367	0.357	0.999
"	"	C	9.82	203	0.723	0.399	0.992
"	"	C	10.20	147	0.717	0.399	0.995
67.58	1:½:4½	D	21.73	286	0.663	0.380	0.998
"	"	D	14.38	351	0.775	0.407	0.999
"	"	D	16.61	267	0.698	0.400	0.999
67.58	1:½:4½	C	15.21	240	0.627	0.363	0.996
"	"	C	10.35	318	0.749	0.410	0.999
34.18	1:¼:3	D	9.53	402	0.724	0.402	0.996
"	"	D	10.73	450	0.700	0.403	0.998
"	"	D	8.96	448	0.686	0.395	0.996
34.18	1:¼:3	C	8.26	108	0.596	0.357	0.999
"	"	C	9.24	154	0.588	0.359	0.998
"	"	C	9.64	193	0.530	0.338	0.998
21.55	1:¼:3	E	21.40	252	0.550	0.347	0.999
"	"	E	13.69				
"	"	E	18.57				
"	"	F	13.13	225	0.511	0.348	1.000
"	"	F	13.46	411	0.603	0.366	0.999

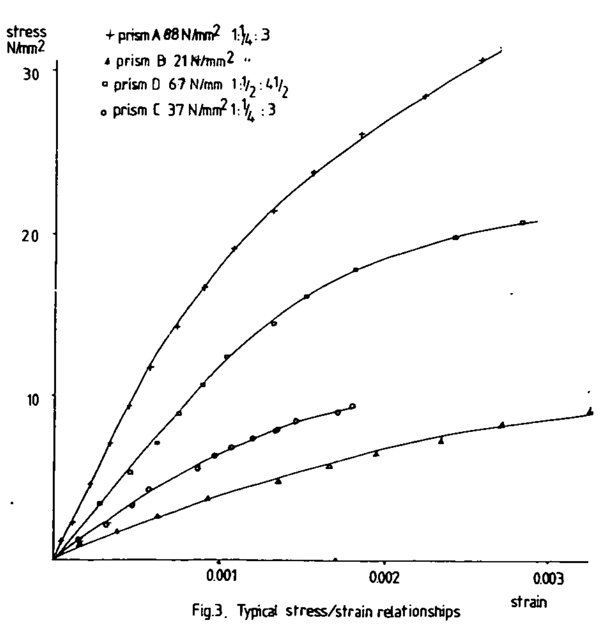

Figure 3. Typical stress/strain relationships.

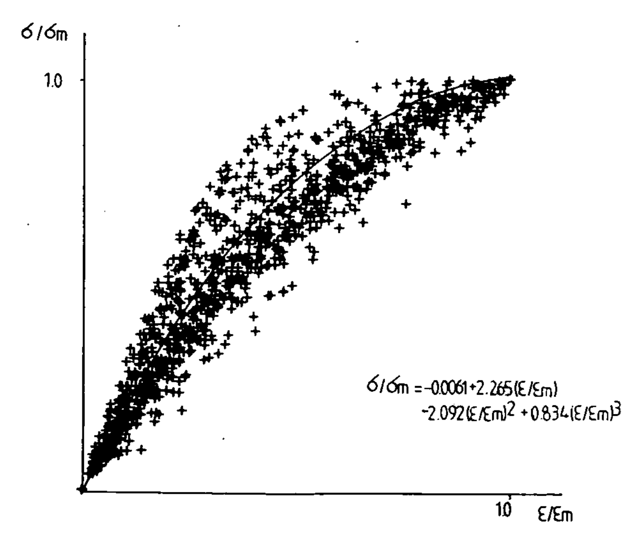

Figure 4. Nondimensional stress/strain relationships.

meters refer to the distribution of stress in the compression zone, Figure 5. λ_1 is the ratio of the compressive strength to the average compressive stress in the compression zone and is equal to the ratio of the area under the non-dimensional stress/strain curve to the enclosing rectangle. λ_2 is the ratio of the position of the resultant thrust in the compression zone to the depth in compression and is equal to the centroid of the area under the nondimensional stress strain curve. The values of all four stress block parameters are presented in Table 3 for each individual prism. The last column of this table lists the correlation coefficient which is a measure of the deviation of the experimental data points to the fitted equation such that for perfect correlation the coefficient is 1.0.

From Table 3 it can be seen that the value of λ_1 may vary from 0.503 to 0.780 while λ_2 varies from 0.323 to 0.429. Figures 6 and 7 show the distribution of λ_1 and λ_2 respectively. 78% of the λ_1 values lies between 0.56 and 0.71 the average being 0.64. From Figure 7, 88% of the results for λ_2 lie between 0.35 and 0.41 the average being 0.373. It has been suggested [2] that the stress block factors may be obtained by assuming that the stress/strain relationship takes the form of a cubic parabolic relationship such that λ_1 = 0.75 and λ_2 = 0.40. It can be seen from Figure 4 that this represents the upper limit of the results.

Although for analytical and research purposes it would be more accurate to have the exact mechanical and material properties it may be advisable to use the average stress block factors λ_1 = 0.640 and λ_2 = 0.380 in the design of brickwork flexural members. These values are obtained from the nondimensional stress/strain relationship (equation 3) and are very similar to the values obtained by using the second degree parabola [6] more commonly associated with the idealised behaviour of concrete, neglecting the portion of the curve past the peak compressive stress. Hence the ultimate strength of brickwork flexural members would not be significantly affected by assuming that the stress/strain behaviour is the same as concrete up to the peak stress.

The limited tests on grouted cavity brickwork prisms (types E and F) result in λ_1 = 0.550 and λ_2 = 0.345 which falls within the lower limits of the nondimensional stress/strain envelope (Figure 4).

Ultimate brickwork strain

The ultimate strain of brickwork is important in the design of brickwork flexural members. There is a considerable variation in the ultimate strain ranging from 0.001 to 0.0045 depending on brick, mortar strength, and prism types. Comparing prism types C and D, higher ultimate strains are experienced in type D for the same brickwork. Considering the high strength brick greater ultimate strains are obtained from Type A in comparison with Type B. But there is not the same significant difference between prisms for medium and low strength bricks. It also appears that where there is a difference in ultimate strains for two types of prism of the same brickwork the compressive strengths are likewise quite

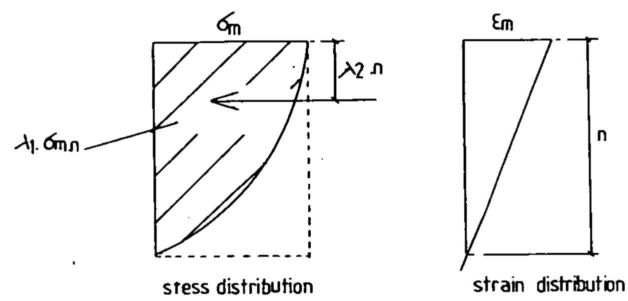

Figure 5. Stress block characteristics

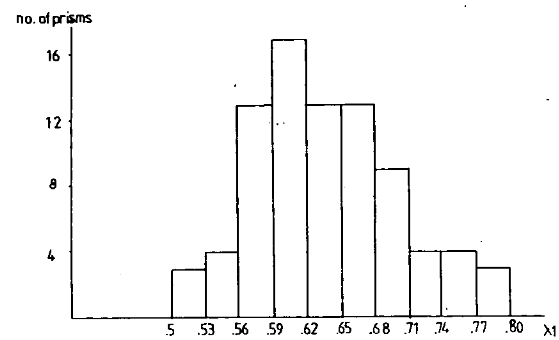

Figure 6. Distribution of λ_1

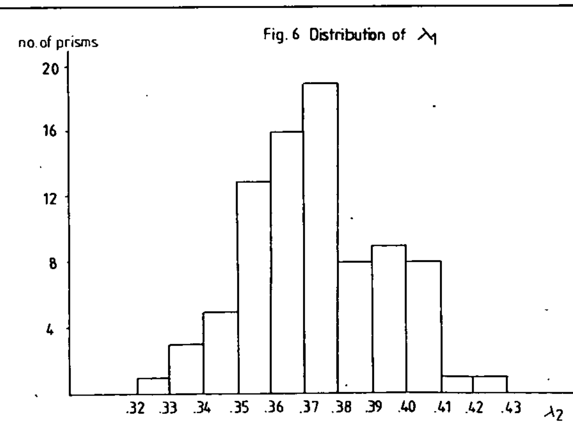

Figure 7. Distribution of λ_2

Table 4. Comparison of prism strength in two directions

Condition	Compressive strength as a % of prism tested flat					
	Type of brick					
	Single frog		Double frog		Three hole	
	1:¼:3	1:½:4½	1:¼:3	1:½:4½	1:¼:3	1:½:4½
Prism tested flat (load normal to bedjoint)	100	100	100	100	100	100
Prism tested on end (load parallel to bedjoint)	103	120	156	142	71	83
Prism tested on edge (Prism D)	68					
Prism tested, grouted cavity (F)	125					
Prism tested grouted on edge (E)	169					

different, the higher compressive strengths corresponding to the higher ultimate strains. Further work is in progress to correlate the prism tests with full scale tests on reinforced and prestressed brickwork beams and thereby ascertain which prism type most accurately predicts the flexural behaviour.

Conclusions

The compressive strength of bricks in three orthogonal directions is different. The compressive strength on edge varied from 31 to 78% of the strength tested flat and on end it was 30 to 75%. The reduction in strength was greater in three hole bricks compared to single frog or double frog bricks due to presence of perforations.

For 1:¼:3 and 1:½:4½ mortar, the compressive strength of prisms tested on end was higher compared to prisms tested flat for single and double frog bricks. In case of three hole bricks the reverse was true. However the reduction in brickwork strength was not as prominent as in the case of individual bricks.

For a particular brick and mortar combination, the value of ultimate brickwork compressive strain is different and is significantly affected by the type of test prism.

The stress/strain relationship of brickwork parallel to the bedjoint in nondimensional form can truly be idealised by a three degree polynomial irrespective of brick strength, mortar or prism type.

The modulus of elasticity of brickwork increases with the increase in compressive strength and the relationship can be expressed as

$$E = 1180\,\sigma_m^{0.83}$$

In this investigation, the stress block factors λ_1 and λ_2 range from 0.503–0.780 and 0.323–0.429 respectively. The majority lie within much closer limits. Therefore the average stress block factors $\lambda_1 = 0.64$ and $\lambda_2 = 0.38$ may be taken for all types of brick from the nondimensional stress/strain relationship.

ACKNOWLEDGEMENTS

The work described in this paper forms a part of a research project investigating the behaviour of reinforced grouted cavity and prestressed brickwork beams. The research projects were sponsored by the Building Research Establishment, Department of the Environment, Science & Engineering Research Council and Structural Clay Products Ltd., Herts.

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Compressive Strength of Brickwork on Edge under Axial and Eccentric Loading

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The paper summarises the results of tests in 74 brickwork prisms subjected to axial and eccentric loading in the direction other than bed-joint. Two grades of mortar and three different bricks were used. The stress-strain relationships obtained under axial compression are used to determine the magnitude and distribution of stress under eccentric loading. The experimental results are compared with BS 5628 and with other investigations.

INTRODUCTION

The load carrying capacity of eccentrically loaded brickwork in the direction of the bed-joint has been studied by various research workers (1,2,3,4). These investigations centred on establishing the magnification factor, k, defined as the ratio of apparent maximum compressive strength under eccentric loading to axial compressive strength. The maximum compressive strength under combined bending and direct stress was obtained by either considering the brickwork as a linear elastic material with no tensile strength or by equating the load-carrying capacity using conventional stress blocks such as rectangular or parabolic. No attempt has been made to establish the strain gradient to actual failure stress under eccentric loading, which is very important for the design of flexural members. In addition, in reinforced and prestressed brickwork, the stress may be applied in directions other than bed-joint direction, for which no data is available. Therefore an investigation was undertaken to study the behaviour of brickwork under axial and eccentric loading applied in a direction other than normal to the bed-joint.

SCOPE OF INVESTIGATION

In this investigation, the following variables were considered:

(i) Brick strength: low, medium and high strength brick

(ii) Mortar grade: Grade 1 and Grade 2

(iii) Axial and eccentric loading: the eccentricity was limited to t/6 only.

The height of the prism was kept low ($^h/_d$ =5.08) so that the secondary effect of loading on strength was negligible. The stress/strain relationship was obtained from axially loaded prisms. The relationship thus obtained was used to derive the magnitude and distribution of stress along the width of the prism using the measured strains.

MATERIALS

Bricks

Three different types of 3-hole perforated bricks were used throughout, varying from high to low strength. Compressive strength and water absorption tests were carried out in accordance with BS 3921 (6); the results are presented in Table 1.

Mortar

Grade I, 1:1/4:3 (cement:lime:sand) and Grade II, 1/½/4½ (cement:lime:sand) mortar mixes were used. The average compressive strength of the mortars for each prism is given in Tables 2 and 3.

Brickwork Prisms

The brickwork test specimen used in this investigation is shown in Figure 1 with nominal dimensions 335 x 215 x 65mm. All test specimens are built by an experienced bricklayer and cured under polythene for 28 days prior to testing.

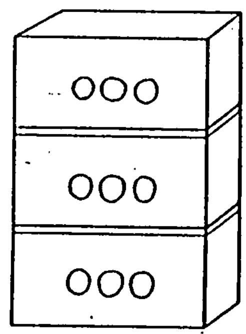

Figure 1. Brickwork Test Specimens

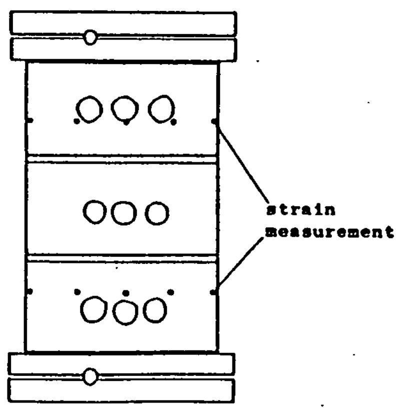

Figure 2. Test Set-up for Eccentrically Loaded Prisms

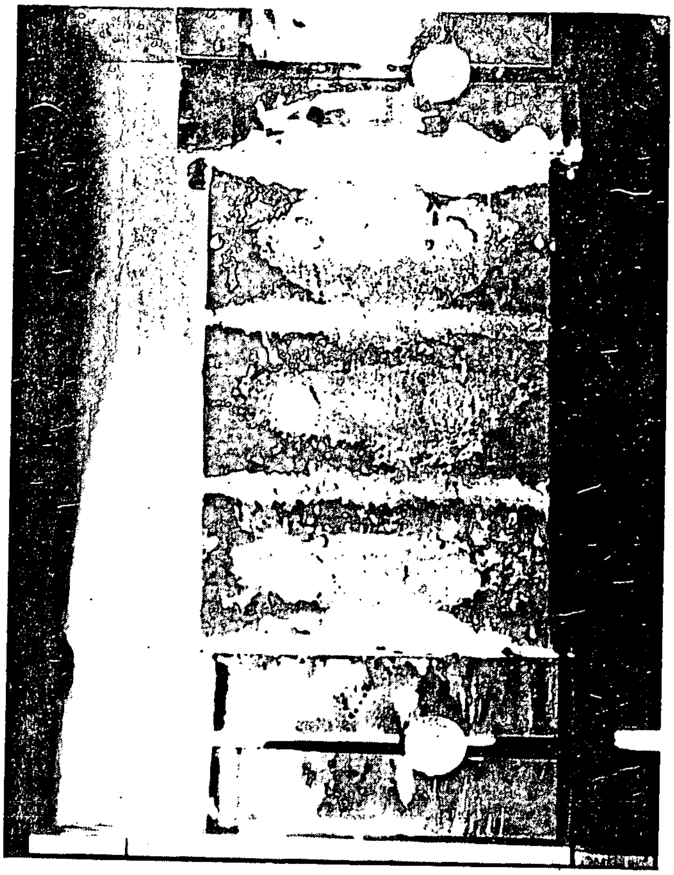

Figure 3 - Test Set-up of Eccentrically Loaded Prisms

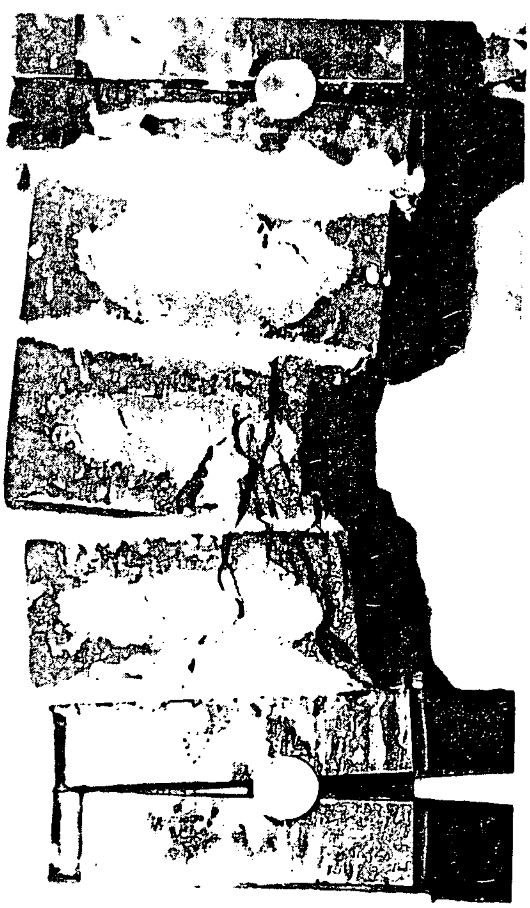

Figure 4 - Typical Failure of Eccentrically Loaded Prism Series DE

Test Procedure

Prior to testing, the axially loaded prisms were capped and levelled using a rich mortar mix. To ensure an even distribution of applied load 3mm plyboard sheets were also placed between the test specimen and loading platens. The set up for the eccentrically loaded prisms is shown in Figs. 2 and 3. The set up was so arranged that the line of action of the load was at an eccentricity of $t/6$.

Strain measurements were taken at positions across the width of the section using a 'Demec' strain gauge at regular intervals of loading. Tables 2 and 3 contain a summary of the test results for ultimate compressive strain.

EXPERIMENTAL RESULTS AND DISCUSSION

Mode of Failure

Vertical splitting of the bricks occurred at between 85-95% of the ultimate load in all of the axially loaded prisms and in the eccentrically loaded prisms built with Class I mortar. Splitting of the bricks was at the centre of the axially loaded prisms and along the line of action of the load in the eccentrically loaded brickwork.

Failure of the eccentrically loaded brickwork prisms built with Class II mortar was preceeded by crushing of the brickwork on the compression face and tensile splitting along the brick/mortar interface at the opposite side. Eventual collapse was caused by explosive spalling of the brickwork on the more heavily loaded face (Fig. 4).

Strain Measurements

Strains were measured at various stages of loading up to 88-95% of the failure load for both axial and eccentrically loaded prisms. The values for ultimate strain (Tables 2 and 3) were mathematically extrapolated from the experimental load (stress)/strain relationships since it was not possible to measure the strain at failure.

The experimental stress/strain relationships for the axially loaded prisms were mathematically idealised in the form of a non-dimensional third degree polynominal (Figs 5-8), such that:

$$f/f_m = X_1\,(\varepsilon/\varepsilon_m) - X_2\,(\varepsilon/\varepsilon_m)^2 + X_3\,(\varepsilon/\varepsilon_m)^3 \quad (1)$$

Values for X_1, X_2 and X_3 for each prism type are given in Table 4. Except for low strength brick, the value of the three constants were very similar to those derived by Pedreschi and Sinha (7).

Before cracking, in all eccentrically loaded prisms, the strain distribution was linear starting with maximum towards the loaded face reducing to zero on furthest face (Figs 9-12); a characteristic of loading at $t/6$.

Upon cracking of the brickwork, the distribution of strain across the width was no longer represented by a single strain gradient: the gradient from the extreme loaded face towards the loaded end changed. Thus the relationship became bi-linear.

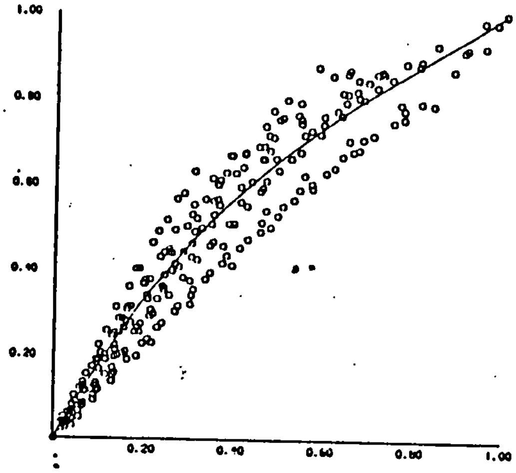

Figure 5. Non-Dimensional Stress/Strain Relationship, Prism Series A

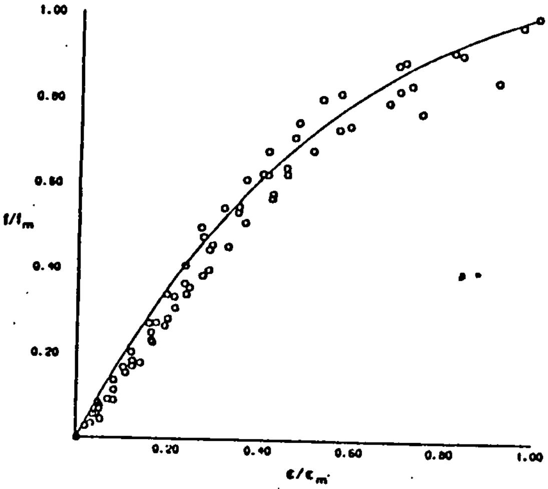

Figure 6. Non-Dimensional Stress/Strain Relationship, Prism Series B

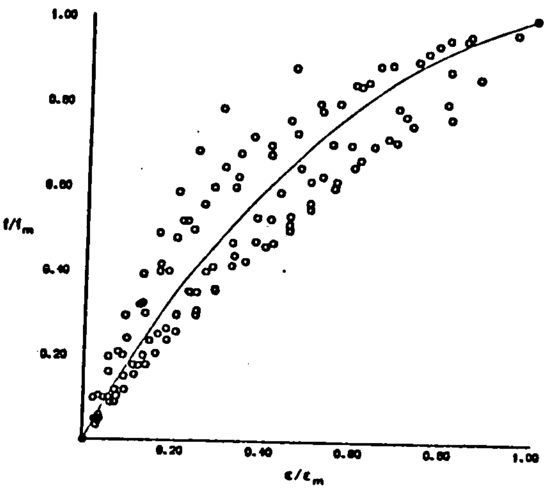

Figure 7. Non -Dimensional Stress/Strain Relationship, Prism Series C

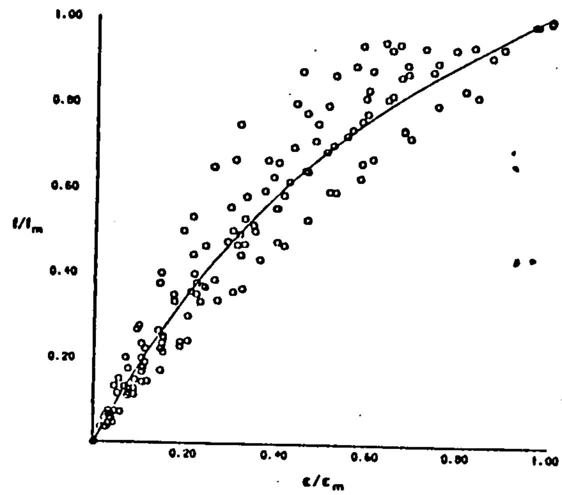

Figure 8. Non-Dimensional Stress/Strain Relationship, Prism Series D

In some prisms, the spalling of brickwork on heavily stressed side led to the reduction of the cross sectional area, which resulted in the line of action of the load acting outside the 'Kern' ($^t/_6$) causing tension in opposite face thus inducing flexural cracking and failure of brickwork (Fig 13).

In case of prisms built with Grade II mortar, the spalling of mortar on the heavily loaded face commenced at approximately 75% of the failure load. With increasing load, the flexural cracking reduces the cross-sectional area, thus further causing crushing and spalling of brickwork on the loaded face. As a result, a very ihigh apparent strain (ϵ_{me} = 0.00694) was recorded.

The magnitude of ultimate strain for prism series and B were equal under both axial and eccentric oading. For prism C the ultimate strain under eccentric loading was 37% lower than for the xially loaded case. Unlike prism series AE nd BE, where splitting occurred prior to failre, cracking of the brickwork in the prisms of ow strength was at failure. The mathematical rediction of ϵ_{me} was underestimated in this ase.

STRESS DISTRIBUTION PRIOR TO AND AT ULTIMATE LOAD

Using the experimentally derived stress/strain relationship for the axially loaded brickwork prisms and from the experimental strain gradient, the compressive stress distributions at ultimate and before (55% of ultimate) for eccentrically loaded prisms were obtained (Table 5 and Figs 14-16). The stress blocks (Figs 14-16) represent the best least-square fit for the result of each prism series. There appears to be very good agreement between experimental and calculated load both prior to and at ultimate load for all prisms, but for the low strength brickwork at ultimate. This is due to under-

tensile region of the series AE and BE prisms have been ignored at ultimate because of the small width (less than 5mm) over which tensile strain developed. The failure load will be overestimated by 0.2% due to this.

Earlier researchers (3) have concluded that the maximum compressive strength of eccentrically loaded brickwork is 10-20% higher compared to axially loaded prism. This apparent increase in maximum stress has been calculated from the experimental failure load assuming rectangular or parabolic stress blocks. From Tables 2 and 3 it is clear that the maximum compressive stress developed in the eccentrically loaded prism is equal to the ultimate strength (f_m) derived under axial loading. Therefore, 10-20% increase proposed in stress may be attributed to the inaccuracy in the assumption rather than real increase in the failure stress.

From the stress block for the Grade II mortar, eccentrically loaded prism (Fig 16) it is clear that at failure only 80% of the section is resisting the compressive load. Crushing of the brickwork has reduced the overall effective width of the section and thereby changing the eccentricity.

By taking moments about the line of action of the load (centre of gravity of the stress block) it was possible to determine the width of the cross-section that had crushed, 5mm, the effective width of the section 210mm, and therefore the eccentricity of the applied load, $^e/_t = ^1/_{5.1}$.

COMPARISON OF EXPERIMENTAL RESULTS WITH OTHER TEST RESULTS AND WITH BS 5628

In Figure 17 the reduction in capacity of eccentrically loaded prisms in terms of axial load has been plotted for various eccentricity ratios ($e/_t$) using the stress block derived experimentally.

Although the present test was limited to $^t/_6$, the capacity reduction was obtained from the non-linear stress block neglecting the portion of the prism in tension for other eccentricities which agrees well with the results of other investigators (1,2,8).

The results of this and all other investigations were compared in Figure 17 with BS 5628 and it appears that the Code overestimates the value of capacity reduction factor thus allowing higher load for eccentrically loaded prisms than obtained in the experiments. BS 5628 assumes that for $^e/_t$ = 0 to 0.05 the capacity reduction factor remains unchanged, which means that the failure load for axially and eccentrically loaded prisms of low slenderness ratio (up to 8) will be unaffected by eccentricity. A rectangular stress block with a constant stress of $1.1f_k$ under ultimate load condition has been assumed giving the value of capacity reduction factor as:

$$\beta = 1.1\ [1-2e_m/t] \qquad (2)$$

The above equation represents a straight line (Fig 17) giving a value of 1.1 at zero eccentricity and zero at $^e/_t = \frac{1}{2}$. From Fig 17 it appears that there is no justification for using the factor 1.1. However, the stress blocks shown in Figs 14 and 16 can be replaced by a simplified equivalent rectangular stress block

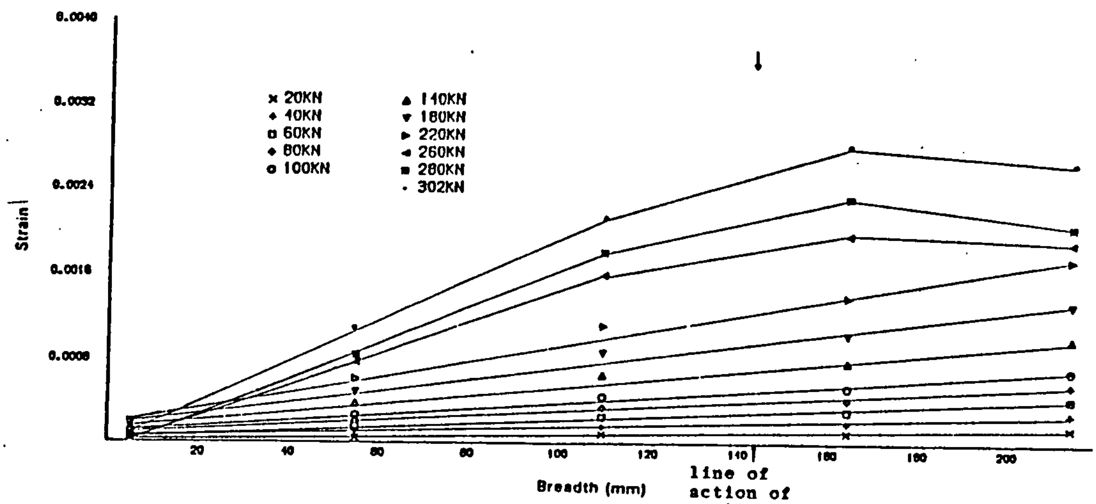

Figure 9. Typical Strain Distribution for Prism Series AE

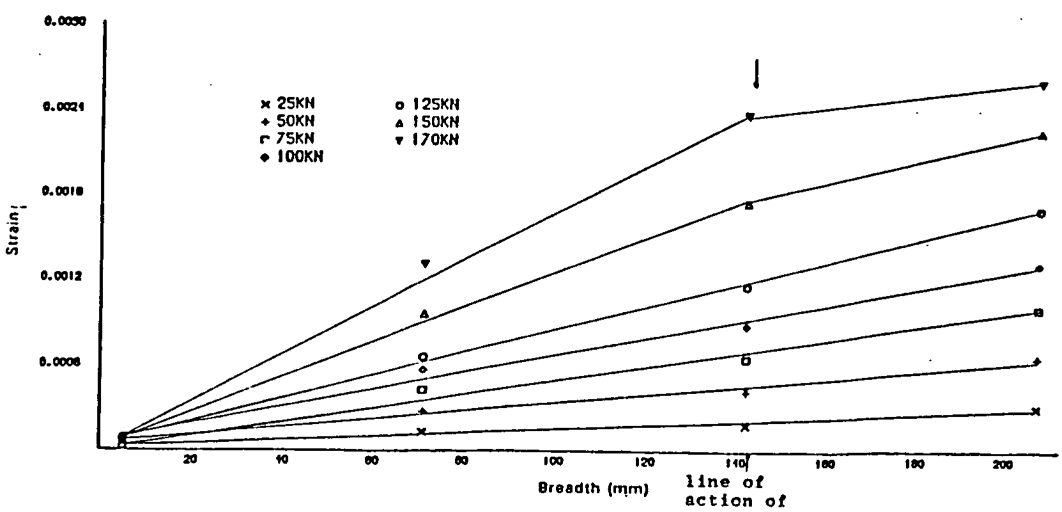

Figure 10. Typical Strain Distribution for Prism Series BE

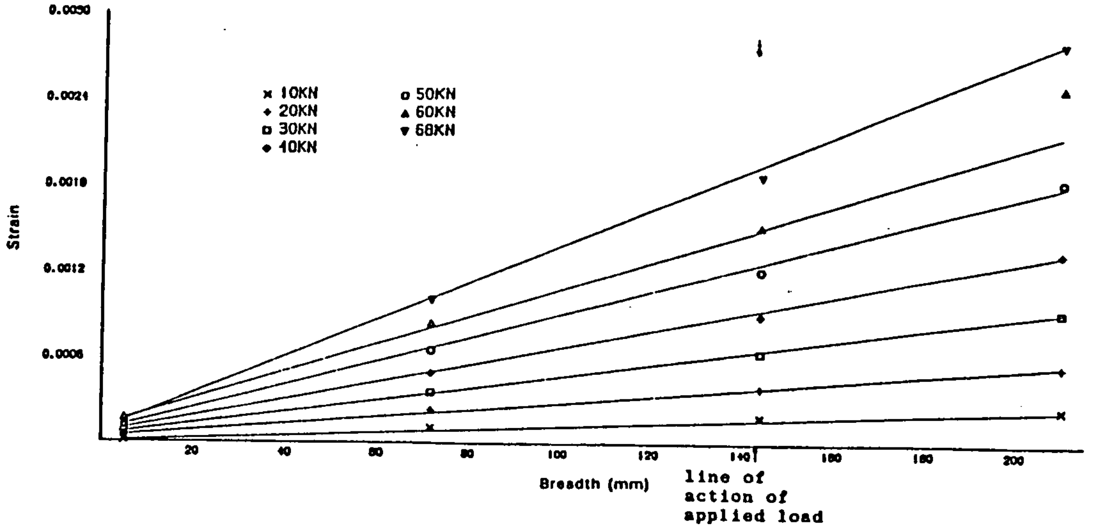

Figure 11. Typical Strain Distribution for Prism Series CE

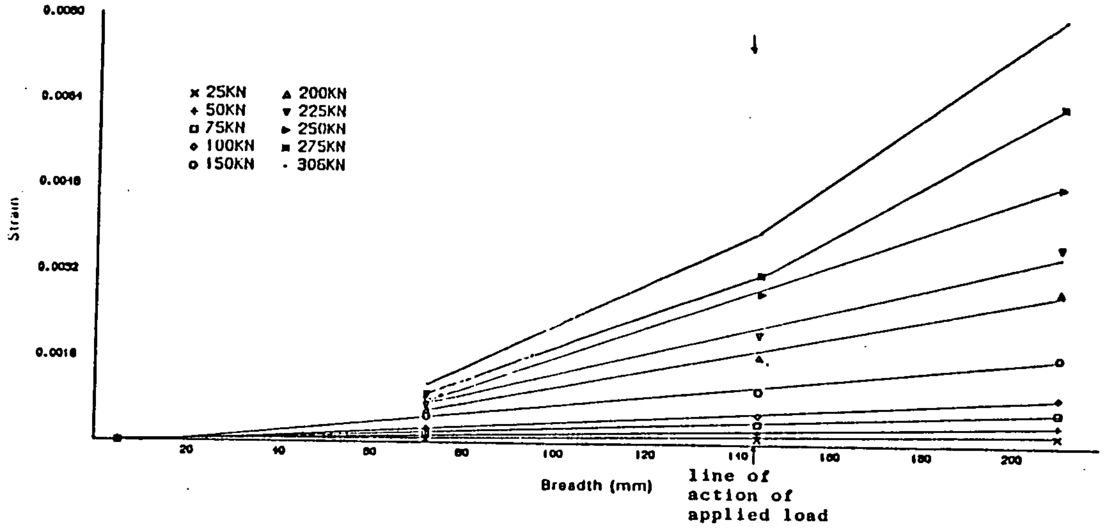

Figure 12. Typical Strain Distribution for Prism Series DC

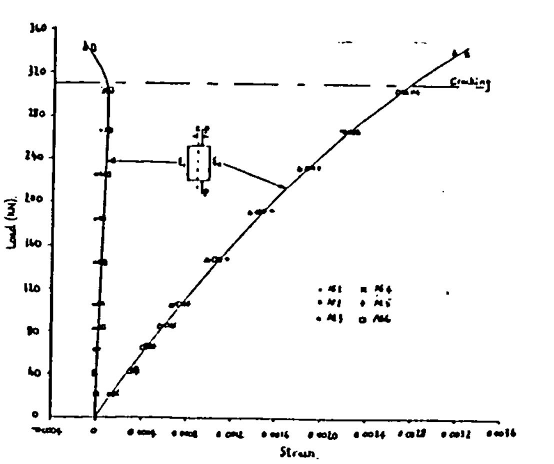

Figure 13. Typical Load-strain Relationships for Eccentrically Loaded Prism

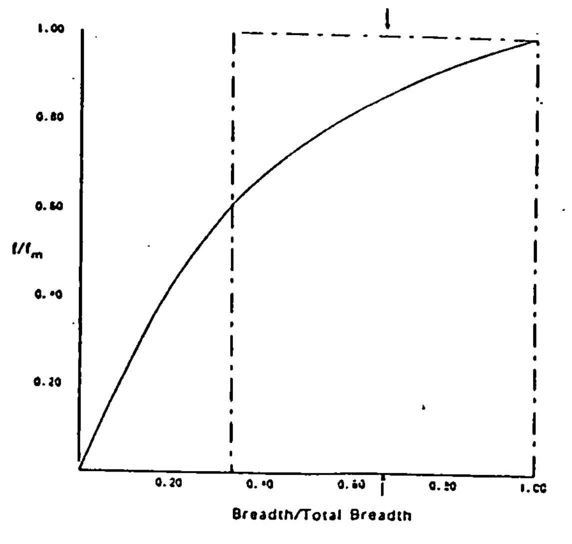

Figure 14. Average Stress Block at Failure for Prism Series AE and BE

with a constant value of ultimate compressive strength which gives the values of:

$$\beta = 1 \times [1-2^{e}/_{t}] \qquad (3)$$

The result of this modification is shown in Fig 17. A very good correlation is obtained between the experimental results and the proposed modification. This modification gives the same linear relationship between capacity reduction factor and $^{e}/_{t}$, which was obtained by stress block obtained from strain gradient.

CONCLUSIONS

1. The load carrying capacity or stress prior to or at ultimate for eccentrically loaded brickwork prisms can be predicted by assuming a linear strain distribution along the width of the section; and by using the actual non-linear stress-strain relationship obtained under axial loading.
2. The maximum stress developed at the time of failure of eccentrically loaded brickwork prisms stressed in directions other than normal to the bed-joint and built with different grades of mortar and brick strength appears to be the same as the ultimate stress in axial compression
3. The British Standard Code of Practice (BS 5628) overestimates the capacity reduction factor due to the use of a rectangular stress block with constant stress multiplied by a factor of 1.1. Although the shape of the actual stress block is different, the simplified rectangular stress block may be used for design provided that the multiplication factor for stress is modified to unity as proposed in this paper.

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NOTATION

e	eccentricity of applied load
f	compressive stress
f_m	axial compressive strength of brickwork
f_{me}	eccentric compressive strenght of brickwork
P	compressive load
P_o	axial compressive load at failure
P_e	eccentric compressive load at failure
t	width of brickwork section
t'	width in compression for loading outside Kern
X_1, X_2, X_3	coefficients of stress/strain relationship
ε	compressive strain
ε_m	axial ult. compressive strain
ε_{me}	eccentric ult. compressive strain
ε_1	maximum compressive strain in eccentrically loaded brickwork at loading P
ε_2	minimum compressive or maximum tensile strain in eccentrically loaded brickwork at loading P

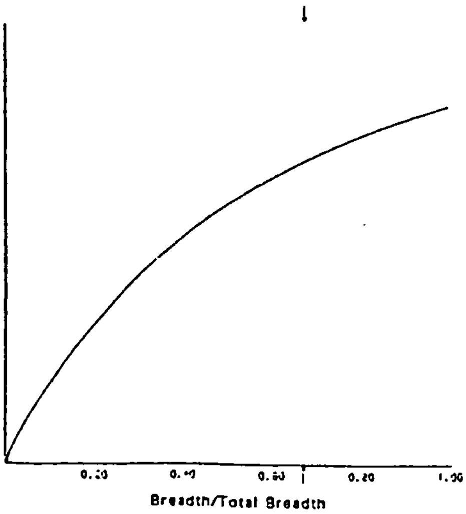

Figure 15. Stress Block at Failure for Prism Series CE

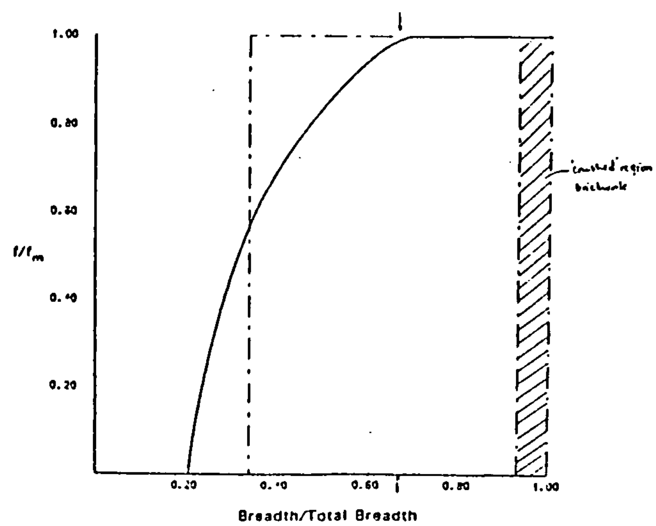

Figure 16. Stress Block at Failure for Prism Series DE

Table 1 Properties of Bricks

Brick Type	Compressive Strength			Absorption
	Loading Direction	Average (N/mm^2)	Coeff. Var. %	% by wt. (5hr.)
High	Bed	96.58	11.8	
	Edge	53.52	10.1	4.40
Medium	Bed	72.25	13.8	
	Edge	23.51	11.3	8.77
Low	Bed	19.69	15.1	
	Edge	7.05	24.7	26.34

TABLE 2

Results of Axially Loaded Prism Tests

Brick Strength N/mm^2	Prism No.	Mortar Str. N/mm^2	Ult. Load P_o kN	Compr. Str. f_m N/mm2	Ult Strain ε_m
19.7	C1	19.9	87.5	6.17	0.00411
	C2	17.5	101	7.12	0.00651
	C3	19.9	105	7.38	0.00492
	C4	17.2	88	6.21	0.00429
	C5	25.2	97	6.86	0.00481
	C6	19.5	105	7.40	0.00482
	C7	19.5	125	8.81	0.00574
	C8	20.5	117	8.23	0.00273
	C9	20.5	103	7.26	0.00493
Average		20.0	103	7.27	0.00476
Coeff. of Variation		11.5%		11.8%	22.1%
96.6	D1	7.3	472	33.18	0.00306
	D2	8.6	427	30.02	0.00211
	D3	6.6	283	19.90	0.00225
	D4	7.8	450	31.61	0.00294
	D5	7.8	439	30.77	0.00218
	D6	7.8	420	29.50	0.00236
	D7	7.8	405	28.45	0.00288
	D8	7.8	450	31.61	0.00383
	D9	7.8	400	28.10	0.00470
	D10	7.5	405	28.48	0.00320
	D11	7.5	431	30.27	0.00301
	D12	7.5	380	26.73	0.00328
	D13	7.5	382	26.83	0.00470
	D14	7.5	438	30.77	0.00380
	D15	7.5	428	30.05	0.00328
Average		7.6	417	29.31	0.00318
Coeff. of Variation		5.5%		10.5%	25.4%

Brick Strength N/mm^2	Prism No.	Mortar Str. N/mm^2	Ult. Load P_o kN	Compr. Str. f_m N/mm2	Ult Strain ε_m
96.6	A1	27.8	427	30.02	0.00313
	A2	16.9	412	28.97	0.00373
	A3	19.8	453	31.83	0.00370
	A4	16.3	427	29.86	0.00315
	A5	19.5	406	28.49	0.00384
	A6	17.9	500	35.12	0.00327
	A7	20.1	403	28.32	0.00472
	A8	19.2	597	41.90	0.00376
	A9	19.9	674	47.31	0.00314
	A10	16.4	636	44.67	0.00347
	A11	19.1	377	26.48	0.00314
	A12	16.1	500	35.14	0.00410
	A13	21.4	452	31.76	0.00380
	A14	21.4	487	34.19	0.00250
	A15	19.0	467	32.78	-
	A16	19.0	550	38.62	0.00365
	A17	19.0	515	36.18	0.00340
	A18	19.3	351	24.67	0.00379
	A19	19.3	392	27.51	0.00396
	A20	19.3	516	36.23	0.00340
Average		19.3	477	33.50	0.00356
Coeff. of Variation		12.9%		18.1%	13.3%
72.3	B1	19.1	274	19.91	0.00260
	B2	19.6	344	24.99	0.00280
	B3	21.3	217	15.77	0.00310
	B4	18.9	289	21.01	0.00308
	B5	20.5	259	18.78	0.00330
	B6	20.5	239	17.33	0.00262
Average		19.9	271	19.63	0.00292
Coeff. of Variation		4.7%		16.4%	9.8%

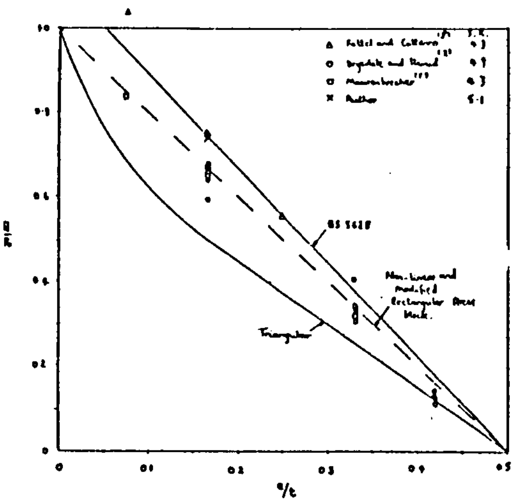

Figure 17. Effect of Eccentricity on Ultimate Load Capacity of Brickwork Prisms

TABLE 3

Results of Eccentrically Loaded Prism Tests

Brick Strength N/mm²	Prism No.	Mortar Strength N/mm²	Ult. Load P_e kN	Ult. Strain ε_{me}	Compr. Strength f_{me} N/mm²
96.6	AE1	19.3	294	0.00412	32.50
	AE2		302	0.00264	28.29
	AE3		318	0.00329	32.96
	AE4		336	0.00304	30.75
	AE5		376	0.00393	33.50
	AE6		309	0.00351	33.50
	Average Coeff. of Variation		323 9.4%	0.00342 15.6%	32.58
72.3	BE1	20.5	155	0.00272	19.13
	BE2		174	0.00324	19.63
	BE3		180	0.00253	18.64
	BE4		208	0.00398	19.83
	BE5		170	0.00256	18.92
	BE6		170	0.00253	18.85
	Average Coeff. of Variation		181 7.9%	0.00294 17.7%	19.53
19.7	CE1	20.5	85.5	0.00364	6.23
	CE2		68.5	0.00231	5.34
	CE3		77.0	0.00256	5.55
	CE4		88.5	0.00316	5.94
	CE5		68.0	0.00279	5.71
	CE6		90.0	0.00362	6.21
	Average Coeff. of Variation		78.0 10.9%	0.00301 18.3%	5.83 6.2%
96.6	DE1	7.5	276	0.00695	29.31
	DE2		296	0.00660	29.31
	DE3		273	0.00685	29.31
	DE4		306	0.00820	29.31
	DE5		276	0.00661	29.31
	DE6		269	0.00540	29.31
	Average Coeff. of Variation		283 5.2%	0.00694 9.4%	29.31

Table 4 Properties of Stress/Strain Relationships

Brick Type	Mortar Grade	X_1	X_2	X_3
High	1:1/4:3	1.958	1.596	0.636
Medium	1:1/4:3	2.094	1.556	0.466
Low	1:1/4:3	2.868	3.665	1.804
High	1:½:4½	2.005	1.566	0.565

Table 5 Comparison of Experimental and Predicted Loading of Eccentrically Loaded Prisms

Prism Type	Ultimate Load			0.55 x Ultimate Load	
	Expt. Load (kN)	Non-Linear Stress Block(kN)	BS 5628 (kN)	Expt. Load (kN)	Non-Linear Stress Block(kN)
AE	323	308	343	180	175.4
BE	181	194	201	100	103.1
CE	78	58	75	40	37.9
DE	283	295	300	150	165.3

A Study of the Compressive Strength in Three Orthogonal Directions of Brickwork Prisms Built with Perforated Bricks

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ABSTRACT
The paper gives test results for 315 brickwork prisms of five different types, built with four types of perforated bricks in designations (i), (ii), and (iii) mortars. The prisms were tested in axial compression to simulate the condition of loading in a wall or a section of reinforced or prestressed brickwork. The strengths of the bricks in three orthogonal directions have also been determined. The characteristic strength of brickwork normal to bed joint given in BS 5628: Part 1 is not strictly applicable to highly perforated bricks. However, the characteristic compressive strength for brickwork other than normal to bed joint can be estimated from the Code, provided the brick strength in corresponding directions is used. Typical stress/strain curves for perforated brickwork are also given.

1. INTRODUCTION

The compressive strength of brickwork is given in BS 5628: Part 1[1] for use in walls under vertical loading so the bricks are usually tested for compressive strength only in the direction perpendicular to the bed-joint. In reinforced and prestressed members, the compressive force develops in directions other than perpendicular to the bed-joint, but BS 5628: Part 2[2] gives the characteristic strength based on Part 1 which may not be applicable. This fact is recognised by the code and the designer is permitted to take the prism test value for characteristic compressive strength, though in many cases this might not be possible and the code values would be adopted. A value of 0·33 f_k has been suggested which may or may not be realistic and may cause problems in the case of flexural members. Some compressive test results[3,4] on small specimens are available, and the second reference[4] examined the effect of 3, 14 and 23 hole perforations on the compressive strength of brickwork in three orthogonal directions. This investigation was carried out to further clarify the assumption of the code and to provide data for design engineers.

2. EXPERIMENTAL

2.1 Scope of the investigation

The uni-axial compressive strength of brickwork in any direction depends on the brick strength, mortar grade, slenderness ratio and the workmanship. In this investigation the same bricklayer was used throughout, hence the workmanship may be assumed to be non-variable, while other variables considered were:

(i) **Types of Bricks:** Four types of perforated bricks were used:- B_{14} (14-hole), B_{10} (10-hole), B_3 (3-hole) and B_5 (slotted) (Figure 1). Brick B_3 was an engineering brick with 12·2% perforation by volume, the remainder had > 20%.

(ii) **Mortar:** The mortar compositions designations (i) $1:\frac{1}{4}:3$; (ii) $1:\frac{1}{2}:4\frac{1}{2}$; (iii) 1:1:6, cement:lime:sand were used for all test specimens. Designation (iv) 1:2:9 mortar was used for Type E prisms made from B_3 and B_{10} bricks.

(iii) **Types of Prism:** Five different types of prisms (Figure 2) were tested in axial compression. In prisms A, D and E the compressive load applied was normal to the bed joint. In prisms B and C the compressive load was applied to the header and stretcher faces respectively.

(iv) **Slenderness ratio (h/t):** This was constant at 4·6 for prisms A to D and reduced to 2·3 for E.

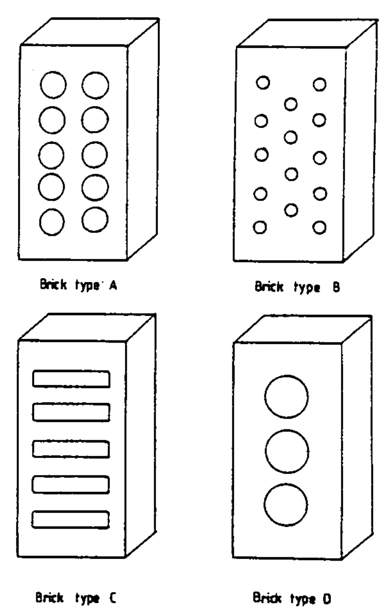

Figure 1—Brick types.

Table 1
Properties of bricks

Brick	*Vol. % perforations*	*24 h water absorption %*	*Compressive strength* Bed (prism A, D, E) Mean N/mm²	c.v. %	Header (Prism B) Mean N/mm²	c.v. %	Stretcher (Prism C) Mean N/mm²	c.v. %
B_{14}	21·3	3·9	74·3	10·1	10·4	19·2	26·2	13·4
B_{10}	23·1	5·4	70·2	8·1	21·7	19·0	29·5	14·7
B_3	12·2	4·2	82·0	7·1	40·2	17·3	53·2	17·7
B_5	20·0	3·4	64·1	17·5	13·8	22·3	51·8	13·6

Table 2A
Compressive strength of brickwork prisms
1:¼:3, 1:½:4½ mortar

Brick	Test nos.	A		B		C		D		E	
		Strength N/mm²									
		Mean mortar	*Brickwork*	*Mean mortar*	*Brickwork*	*Mean mortar*	*Brickwork*	*Mean mortar*	*Brickwork*	*Mean mortar*	*Brickwork*
1:¼:3											
	1	31·5	35·6	31·5	13·7	31·5	8·1	34·4	28·6	29·0	36·7
	2	25·5	24·5	30·4	13·9	31·5	7·3	35·8	34·0	29·0	31·8
B_{14}	3	25·5	25·8	30·4	15·3	23·8	9·1	31·6	22·9	29·7	34·4
	4	25·8	29·1	31·9	14·4	23·8	9·3	30·0	21·5	29·7	29·3
	5	25·8	29·5	31·9	15·9	23·8	8·6	29·5	26·4	30·0	25·8
	Mean		28·9		14·6		8·5		26·7		31·6
	1	31·5	25·0	31·5	18·6	31·5	15·6	35·8	22·5	29·7	28·6
	2	25·5	23·6	30·4	20·6	31·5	11·2	34·4	21·9	29·7	22·4
	3	25·5	20·5	30·4	19·8	24·0	16·1	29·5	22·7	30·0	28·3
B_{10}	4	25·8	22·5	31·9	20·5	24·0	14·2	29·5	25·7	29·0	28·5
	5	25·8	18·8	31·9	20·9	24·0	15·2	30·0	27·1	29·0	29·7
	6					21·0	14·2R	21·0	23·4R	21·0	22·8R
	7					21·0	16·3R	21·0	22·7R	21·0	25·0R
	8					21·0	17·5R				
	Mean		22·0		20·0		15·0		23·7		26·5
	1R	21·0	35·3							21·0	36·3
	2R	21·0	32·6							21·0	36·3
	1	31·5	44·5	31·5	19·4	31·5	22·5	35·8	26·0	29·7	44·9
B_1	2	25·5	35·0	30·4	20·0	31·5	26·4	34·4	27·3	29·7	37·0
	3	25·5	44·6	30·4	22·0		33·2	31·6	28·2	29·0	39·6
	4	25·8	34·3	31·9	22·2		29·3	29·5	26·8	29·0	33·1
	5	25·8	37·0	31·9	25·4		47·6	30·0	30·1	30·0	34·0
							29·7			30·0	33·1
							25·9			30·0	37·1
							29·6				
	Mean		37·6		21·8		30·5		27·7		36·8
	1	31·5	33·0	31·5	12·9	31·5	31·0	34·4	29·0	29·7	34·0
	2	25·5	37·5	31·9	15·1	31·5	30·7	34·4	33·9	29·7	35·9
B_5	3	25·5	32·8	30·4	12·8	22·8	28·4	31·6	24·6	29·0	35·4
	4	25·8	34·3	31·9	13·0	22·8	27·3	30·0	30·2	29·0	35·1
	5	30·0	32·9	31·9	15·8	22·8	27·9	29·5	29·5	25·8	36·5
										25·8	40·4
	Mean		34·1		13·9		29·0		29·4		36·2
1:½:4½											
	1		13·0		11·0		6·6		14·5		17·2
	2		15·9		10·9		7·9		15·9		17·1
B_{14}	3	7·0	15·9	8·0	9·6	8·5	8·4	7·0	16·7	8·0	21·9
	4		13·5		9·4		6·1		13·6		21·9
	5		15·4		8·4		6·6		14·0		20·7
	Mean		14·7		9·9		7·1		14·9		19·8
	1		17·0		17·7		16·0		19·7		22·4
	2		17·6		16·5		16·0		15·9		23·3
B_{10}	3	8·0	14·2	8·0	15·9	8·7	15·2	10·0	13·7	7·8	19·0
	4		13·0		16·6		16·3		16·5		21·4
	5		11·8		17·5		16·9		15·5		22·6
	6R	8·6	14·0			8·6	15·7	8·6	16·9		
	7R		18·9				14·3		17·9		
	Mean		15·2		16·8		15·8		16·6		21·7
	1		19·7		15·7		19·5		19·6		23·7
	2		20·2		16·4		18·6		17·0		25·9
B_1	3	8·8	19·4	6·0	16·4	8·7	19·0	7·6	18·6	7·8	23·3
	4		19·6		18·2		19·5		21·8		18·2
	5		22·4		17·7		21·0		18·5		23·7
	Mean		20·3		16·9		19·5		19·1		23·0
	1		16·3		9·0		23·4		18·9		20·8
	2		19·4		9·2		23·1		19·9		25·1
B_5	3	8·7	16·9	6·0	10·3	8·5	25·4	9·5	18·9	8·0	23·4
	4		15·0		10·4		23·4		21·2		22·7
	5		18·0		9·9		23·6	8·6	14·4		
	6R	8·6	20·8						18·0		
	7R		19·6						16·0		
	Mean		18·0		9·8		23·8		18·2		22·8

Table 2B
Compressive strength of brickwork prisms
1:1:6, 1:2:9 mortars

Brick	*Test nos.*	*A*		*D*		*E*	
		Prism type					
		Strength N/mm²					
		Mean mortar	*Brickwork*	*Mean mortar*	*Brickwork*	*Mean mortar*	*Brickwork*
1:1:6							
	1		11·5		14·9		11·8
	2		10·6		12·7		11·1
B_{14}	3	3·5	11·0	4·5	11·5	3·7	11·1
	4		10·8		12·9		11·7
	5		9·4		10·7		11·6
		Mean	10·7		12·5		11·5
	1		11·7		9·4		16·9
	2		11·9		11·1		12·0
B_{10}	3	5·4	11·3	5·6	11·1	3·6	15·0
	4		13·5		11·1		12·9
	5		12·3		11·1		13·5
		Mean	12·0		11·1		14·0
	1		15·8		15·3		18·1
	2		15·7		14·7		18·8
B_3	3	3·5	15·1	5·8	17·2	3·6	17·7
	4		15·7		15·6		19·0
	5		16·5		16·2		16·4
		Mean	15·8		15·8		18·0
	1		15·3		23·3		16·6
	2		14·0		19·1		12·8
	3	5·4	15·0	4·5	12·2	3·7	25·7
B_5	4		13·2		14·0		19·1
	5		15·3		13·0		24·9
	6R	3·9	11·9	3·9	15·6		
	7R		13·2		14·5		
		Mean	14·0		16·0		19·8
1:2:9							
	1						13·9
	2					2·57	10·7
B_{10}	3						9·3
	4						16·1
	5						8·2
		Mean					11·6
	1						16·4
	2						16·1
B_3	3					2·57	15·9
	4						14·4
	5						13·3
		Mean					15·2

2.2 Experimental details

2.2.1 Bricks

Compressive strength tests in accordance with BS 3921[5] were carried out in three orthogonal directions. The results together with the 24 h water absorption are given in Table 1.

2.2.2 Mortar

Ordinary Portland cement to BS 12,[6] lime to BS 890[7] and building sand to the grading limits of BS 1199 and 1200 were used. The mortars were proportioned by volume using gauging boxes and the water/cement ratio for each mix was kept constant. Dry sand was always used for the mortar. Three 100 mm cubes were made for each batch and tested the same day as the brickwork prisms at 28 d. The results are given in Tables 2A and 2B.

The overall mean compressive strength of designation (i), (ii) and (iii) mortars for all batches was 29·3, 8·14 and 4·5 N/mm² and the density was 2203·2, 2131·7 and 2127·7 kg/m³ respectively.

2.2.3 Test procedures

The prisms were capped and levelled properly with mortar of the same designation while being built. Plywood sheets 6 mm thick were placed between the test specimen and the platen of the machine to distribute the load evenly. Surface strains in the brickwork were measured with 200 and 150 mm Demec gauges. These measurements were taken on opposite faces of the prisms at two different points (Figure 2). Strain measurements were taken on the different faces at the beginning of the test to ensure that the load was being applied axially to the

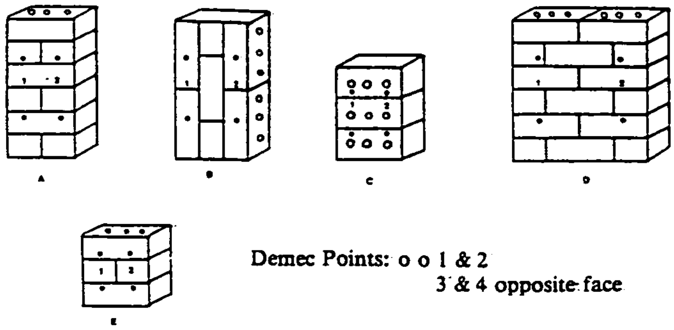

Figure 2—Prisms types.

prisms and continued at various stages of the loading to as near to the failure load as possible.

At least five prisms of each type were tested (Table 2). In some cases tests were done on more than five types to clarify certain trends and these are designated with suffix R in Table 2.

3. RESULTS AND DISCUSSION

3.1 Brick strength in three orthogonal directions

The results of the tests (Table 1) show that the strength of the bricks in three orthogonal directions is different. The highest is recorded in the direction of bed-face and the lowest between headers for all four types of perforated bricks. On examination of the slotted bricks it is clear that in the stretcher direction six short columns between the slots may be resisting the load, while in the header direction two columns longer in length were effectively carrying the load. This may explain the difference in the load-carrying capacity and thus the strength of the bricks in these two directions. The platen effect and the orientation of voids with respect to loading direction may have a significant effect. The platen restraint will be highest on bed than either of the other two directions. The strength of each brick in three orthogonal directions is given in Table 3 as a percentage of brick strength tested on bed. No common relationship exists between them.

Table 3
Compressive strength of bricks as a percentage of the strength between the bed faces

Compressive strength	B_{14}	B_{10}	B_3	B_S
Bed	100	100	100	100
Header	14	31	49	22
Stretcher	35	42	65	81

Table 4
Compressive strength of prisms as a percentage of strength tested on bed

	Prism type and mortar			
	Tested on header		*Tested on stretcher*	
Brick	*1:¼:3*	*1:½:4½*	*1:¼:3*	*1:½:4½*
B_{14}	51	67	29	48
B_{10}	91	114	66	109
B_3	56	83	78	96
B_S	41	57	85	139

Table 5
Compressive strength of bricks in the header direction with perforations filled and unfilled

	Compressive strength			
	Unfilled		*Holes filled*	
Brick	*Mean*	*c.v.*	*Mean*	*c.v.*
B_{14}	10·4	19·3	22·2	13·3
B_{10}	21·7	19·0	24·2	20·2
B_S	13·8	22·3	30·2	12·6

3.2 Strength of prisms

3.2.1 General

The strengths of the prisms are given in Table 2. They are affected by the strength of mortar. The highest compressive strength is obtained with designation (i) mortar and lowest with designation (iv). The compressive load in the case of prisms A, D and E is applied normal to the bed face, while in prisms B and C it is parallel to the bed face.

3.2.2 Comparison of prisms A, B and C

Table 4 shows the strength of prisms B and C as a percentage of the strength of prism A. It varies from 41% to 114% for prisms Type B and 29% to 139% for prisms Type C. The percentage invariably increases as the mortar strength decreases. The values obtained for brickwork are much higher than the respective brick strength percentages in Table 3. It seems the compressive strength of brick on bed cannot be used as an indicator of the strength of brickwork in the two other directions.

In the case of prism B made from 14-hole, 10-hole and slotted bricks using 1:¼:3 mortar (Table 2), the brickwork strength was higher, or equal to the strength of brick which was unusual, though FOSTER and LENCZNER have shown a similar effect[4].

Generally, the brickwork strength is a fraction of the brick strength. This needed clarification so the test on bricks was repeated with the perforations filled with mortar to provide similar conditions to those in the prisms. The brick strength increased significantly (Table 5) so that the strength of brickwork was now lower than the brick strength.

From Table 2 it is clear that, with one exception, the brickwork strength in other directions is higher than one third of the strength in the bed-joint direction. Thus to obtain a realistic strength of brickwork as a function of brick strength, the test on bricks must be done to reflect the orientation of compressive loading and the actual state of the perforations in practice, that is filled or unfilled holes.

3.2.3 Comparison of prisms A and D

With two exceptions, the strength of prism A (one brick wide) and prism D (two bricks wide) was similar for all the three grades of mortar. It seems that no advantage can be gained by testing the larger prisms. Further, such prisms cannot be tested on standard universal testing machines without a major modification and few test houses in the U.K. will be able to test these as routine.

3.2.4 Effect of slenderness ratio

Prism E with a slenderness ratio of 2·1 is stronger than Prism A (SR 4·3). Contrary to the trend the average strength of prism E, made from a 3-hole brick, was found to be lower because of two very high failure stresses obtained for prism A. The test was repeated on two specimens of each using the same mortar, which then showed E to be stronger.

3.2.5 Stress strain relationship

Typical stress-strain curves are given in Figures 3, 4 and 5. The stress-strain relationship for all prisms tested were essentially the same, initially there was a linear relationship after which the strain increased rapidly. As would be expected the strain increases with decreasing strength of mortar. Much higher strains, were recorded near failure for the 1:1:6 mortar prism. However, in all three prisms cracking and crushing took place before ultimate failure and it is very difficult to avoid the measurement of crack or crushing with Demec gauges of overall length of 200 mm or 150 mm.

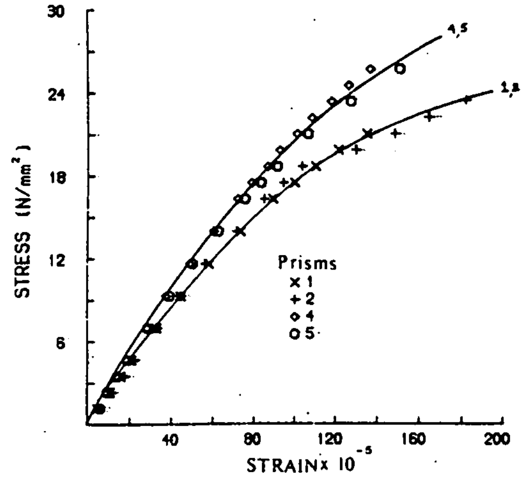

Figure 3—Stress/strain curve for prism type A, B_{14}, 1:¼:3.

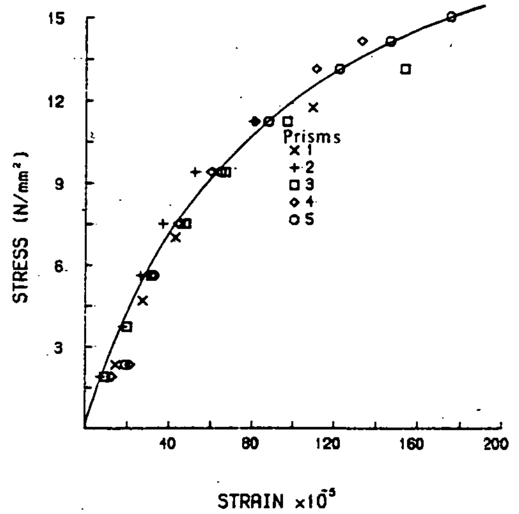

Figure 4—Stress/strain curve for prism type A, B_{14}, 1:½:4½.

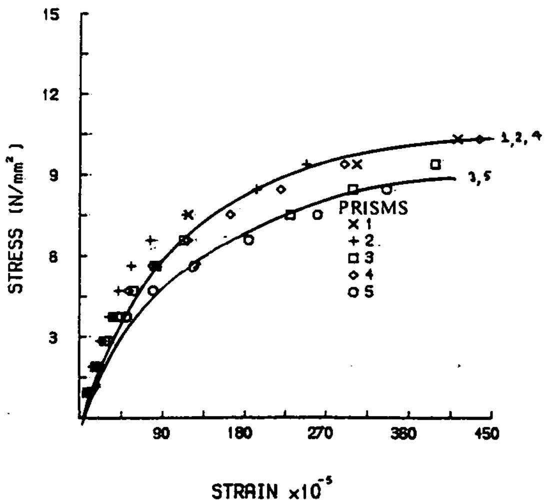

Figure 5—Stress/strain curve for prism type A, B_{14}, 1:1:6.

3.2.6 Comparison between test characteristic strength and Code

Tables 6 and 7 show the characteristic strengths obtained from prism tests and from Figure 1 of BS 5628: Part 1: 1978[1]. The characteristic strength was obtained from prism tests by assuming log-normal distribution. From Table 6 it can be seen that the value given in the Code[1] for the normally loaded direction will overestimate the characteristic compressive strength of brickwork made for 14-hole and 10-hole bricks for all grades of mortar. With a few exceptions, the characteristic strength obtained from prism D was slightly lower than prism A. In the case of 3-hole and slotted bricks, the Code overestimates the characteristic compressive strength of designation (iii) mortar. It may be reasonable to say that the characteristic strength of brickwork, made from highly perforated bricks and loaded in the usual direction (normal to bed-joint), should be obtained from the prism tests rather than the values given in the Code[1].

The characteristic compressive strength of perforated brickwork in the other two directions is much higher than one third of the characteristic compressive strength in the normal direction (Table 4), Table 7 compares the test results of prisms which were loaded in directions other than normal to bed joint with the Code[1]. To obtain the Code[1] characteristic compressive strength, the brick strength in respective directions were used rather than compressive strength on bed. Except for two cases, the Code[1] underestimates the test results. The value of characteristic compressive strength, given in the Code, can safely be used for perforated bricks in other directions provided the brick strength in the corresponding direction is used. Further work needs to be done on other types of highly perforated bricks to confirm this.

Table 6
Comparison of test results with characteristic strength of brickwork from the code

		Characteristic compressive strength f_k N/mm²					
Brick	*Prism type*	*1:¼:3*		*1:½:4¼*		*1:1:6*	
		Test	*Code*	*Test*	*Code*	*Test*	*Code*
B_{14}	A	20·4	20·0	11·6	15·5	8·9	13·5
	D	17·1		12·2		9·3	
B_{10}	A	16·6	19·0	10·1	15·0	10·3	13·0
	D	20·2		13·1		8·9	
B_3	A	29·5	22·5	17·5	16·4	14·7	14·8
	D	24·1		15·4		13·5	
B_5	A	30·4	18·0	14·3	14·0	11·6	12·2
		22·3		13·7		9·7	

Table 7
Comparison of test results with characteristic strength of brickwork other than normal to bed joint from the code

		Characteristic compressive strength f_k N/mm²			
Prism type	*Brick strength N/mm²*	*1:¼:3*		*1:½:4½*	
		Test	*Code*	*Test*	*Code*
B*	22·2	12·7	8·0	7·4	7·0
B	24·2	18·1	8·8	15·5	7·5
B	40·2	17·1	12·5	14·6	10·5
B	30·2	10·9	10·0	8·2	8·5
C	26·2	6·7	9·0	5·2	7·5
C	29·5	11·4	10·0	14·2	8·5
C	53·2	18·9	15·5	17·6	12·5
C	51·8	25·6	15·0	21·8	12·0

*Holes filled with mortar (Table 5)

4. CONCLUSIONS

1. The compressive strength of bricks in three orthogonal directions is different. The compressive strength on header varied from 14 to 49% of strength tested flat and on stretcher it was 35 to 81%. The reduction in strength was greater in 14-hole bricks. However, the reduction of brickwork strength was not as prominent as in the case of individual bricks. The percentage reduction of brickwork strength in other directions compared to strength normal to bed joint decreases as mortar strength decreases.

2. Generally, the characteristic compressive strength obtained from prism D (two bricks wide) is slightly lower than prisms A for all types of mortar (designations (i) to (iii)).
3. The compressive strength of brickwork decreases with the decrease in mortar strength.
4. The compressive strength of the three course (E Type) prism is higher than the six course prisms (A Type) for three designations of mortar due to a lower slenderness ratio.
5. The stress-strain relationship of brickwork is non-linear. The deformation or resulting compressive strain depends on the type of test prisms and is significantly affected by the grade of mortar. The deformation increases with decreasing mortar strength or grade.
6. For any grade of mortar, the characteristic compressive strength of brickwork normal to the bed-joint in the Code[1] may not be applicable to highly perforated bricks (14-hole and 10-hole). The Code also over-estimates the strength of brickwork made from slotted and 3-hole bricks in 1:1:6 mortar.
7. For highly perforated bricks, the characteristic compressive strength of brickwork other than normal to bed-joint can be safely obtained from the Code[1], provided the brick strength in corresponding directions is used, rather than a blanket provision of one third of the strength in the normal direction.

ACKNOWLEDGMENT

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10

Tensile Strength of Brickwork Specimens

By

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ABSTRACT

Tensile and shear tests done on small brickwork specimens showed that the tensile strength of brickwork normal to the bed joint is very variable. The ratio of the flexural tensile strengths of brickwork parallel and perpendicular to the bed joint is not a constant as assumed in B.S. C.P. 111, but rather the relationship between the two is non-linear. It is suggested that a six- or eight-course wallette may be adopted for quality control tests of brickwork.

1. INTRODUCTION

Non-loadbearing panels rely on their tensile strength to resist wind loading. The allowable tensile strength according to B.S. C.P. 111: Part 2 1970[1] perpendicular and parallel to bed joints in brickwork is low, hence it would be difficult for normal storey-height single-leaf or cavity walls to fulfil the design criteria now required by the upward revision of the design wind pressure.[2] However, due to interaction and arching action[3] the non-loadbearing panel can resist wind pressures far greater than those stipulated by the Code.

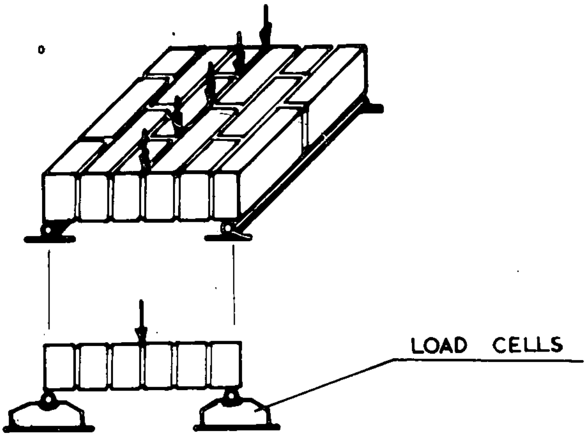
The Codes of various countries[4] have fixed the ratio of the tensile strengths parallel and perpendicular to bed joints at 2, which does not appear to be realistic. The tensile strength parallel to the bed joint may depend on the tensile strength of brick and is generally far greater than suggested by the arbitrarily fixed ratio of 2. Furthermore these codes do not recognize the difference between the axial and the flexural tensile strength. This paper briefly describes exploratory tests done on small full-scale specimens subjected to different loading conditions to highlight these points.

2. EXPERIMENTAL

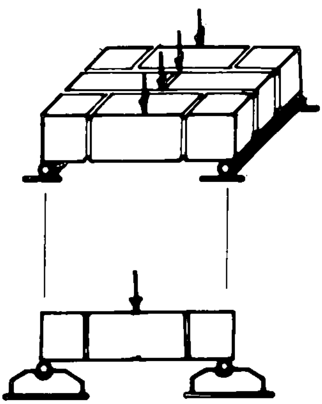
2.1 Materials

Deep-frog Fletton common bricks of compressive strength 26·2 N/mm^2 were used for the tests. The average suction rate of the bricks was 1·64 kg/m^2/min having a standard deviation of 0·62 kg/m^2/min and a coefficient of variation of 38·2%.

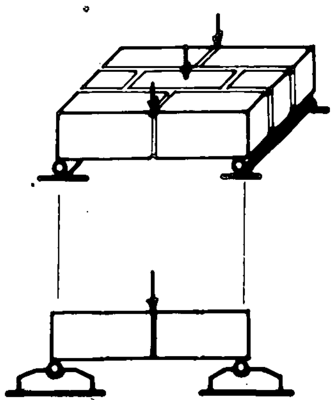
A 1:1:6 mix by volume of rapid-hardening Portland cement : hydrated lime : sand (ordinary building sand) was used for all tests. The materials complied with the relevant British Standards.



TEST ·1·[A]



TEST · 1· [B]



TEST ·1·[C]

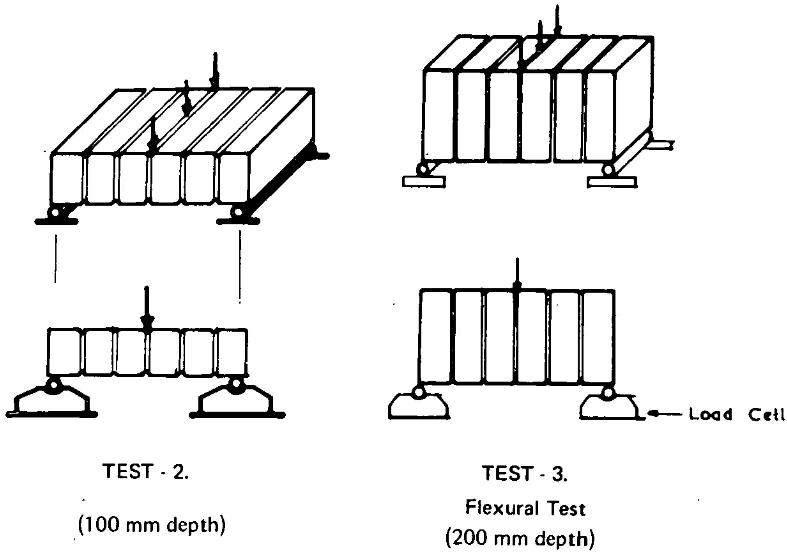


FIGURE 1. Flexural tests to determine the strengths of brickwork in two orthogonal directions.

2.2 Test Specimens

Six-course-high, two-brick-long wallettes (Figure 1, Test 1a) and six-brick-high prisms (Figure 1, Test 2) were made for the flexural tests to obtain the tensile strength perpendicular to the bed joint. The prisms were tested as beams with nominal depths of 100 mm and 200 mm as shown in Figure 1, Test 2 and Figure 1, Test 3. The age at which the specimens were tested is shown in the tables.

Editor's footnote:

The brickwork specimens were tested for flexural strength both parallel to and perpendicular to the bed-joint. When tested parallel to the bed joint the break occurs in that bed joint (Figure 1, Test 1a) while when tested normal to this the break occurs in perpends and through the bricks (Figure 1, Test 1b or 1c). Unfortunately confusion occurs in the literature because testing parallel to the bed-joint is regarded as giving the strength perpendicular to the bed-joint and similarly testing normal to the bed-joint gives the strength parallel to it. In this paper all references are to the strength and thus the parallel direction gives the higher value.

After testing the wallettes to failure, the two broken sections were tested as shown in Figure 1, Tests 1b and 1c to obtain the tensile strength parallel to the bed-joints. This helped not only in cutting down the number of test specimens, but also in obtaining a realistic relationship between the tensile strength perpendicular and parallel to the bed joints as all the factors affecting them remain the same.

Central-line loading was applied to the flexural specimens by a hand-jack and pump, and load cells recorded the load. It is considered that third-point loading would not have changed the results significantly, but in any case, centre line loading was used to maintain the shear arm constant.

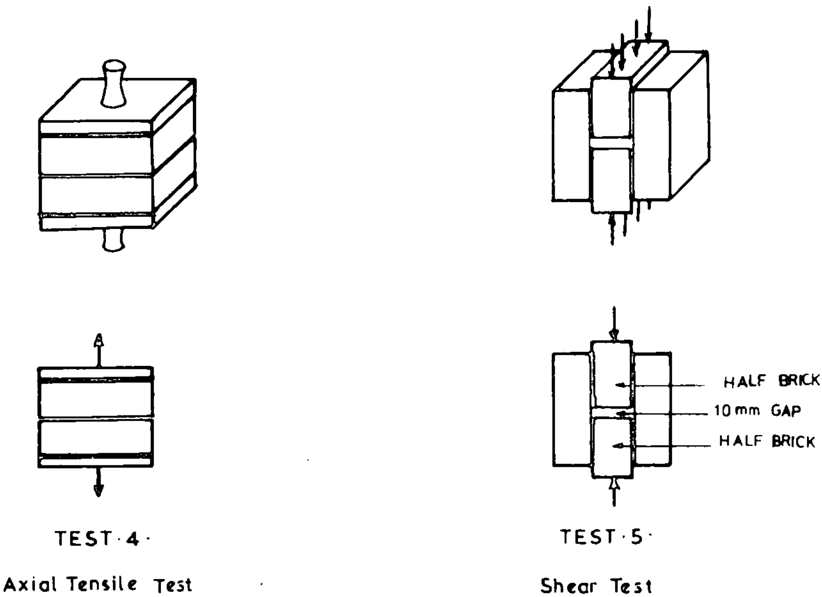


FIGURE 2. Axial tensile test and shear test specimens.

Brick couplets were used to determine the bond tensile strength (Figure 2, Test 4). Two steel plates were fixed with epoxy glue to the surfaces of the specimen and pulled apart in an Instron testing machine. The specimens were kept central in a jig while glueing the plates to ensure concentric loading during testing.

Shear tests were done on three brick assemblies as shown in Figure 2, Test 5.

3. RESULTS

The results of those wallettes tested to give first the strength perpendicular to the bed-joint followed by testing of the two halves to give the strengths parallel to the bed-joint are given in Table 1. The relationship between the strengths in the two directions as given by Tests 1a and 1c is shown in Figure 3. On this are also plotted results obtained by Satti and Hendry[4] which include Fletton bricks as in the present tests, but also wirecut and stiff plastic bricks.

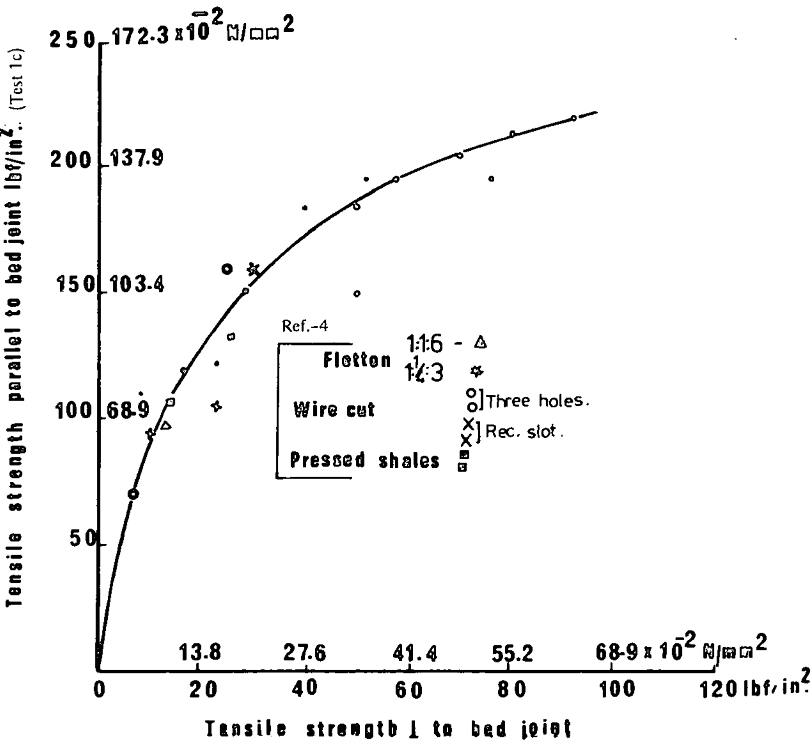


FIGURE 3. Relationship between the tensile strengths in plane parallel and perpendicular to bed joint.

Some wallettes were also tested to give only the strength perpendicular to the bed-joint, and these, together with the results of Tests 2, 3, 4 and 5 are given in Table 2.

TABLE 1.

Relationship between flexural tensile strengths perpendicular and parallel to bed joint carried out on the same specimens.

Mortar: Age days	Mortar: Compressive Strength (N/mm^2)	Flexural tensile strength perpendicular to bed joint (Test 1(a)) (N/mm^2)	Flexural tensile strength parallel to bed joint (Test 1(b)) (N/mm^2)	Flexural tensile strength parallel to bed joint (Test 1(c)) (N/mm^2)	Ratio $\frac{1(b)}{1(a)}$	Ratio $\frac{1(c)}{1(a)}$
22	7.8	0.63	1.99	1.52	3.15	2.39
		0.55	2.32	1.48	4.50	2.69
24	7.6	0.34	1.65	1.27	4.8	3.70
		0.34	1.79	1.03	5.2	3.0
		0.48	2.20	1.41	4.57	2.93
		0.27	2.20	1.27	8.0	4.63
10	6.8	0.39	1.92	1.35	4.88	3.44
		0.36	1.92	1.35	5.36	3.77
		0.52	1.90	1.32	3.63	2.53
	8.2	0.20	1.32*	1.18*	6.62	5.93
		0.11	1.43*	0.83*	12.94	7.50
		0.16	1.07*	0.84*	6.74	5.30

*Slip failure. No cracking of bricks.

TABLE 2.

Strength of brickwork obtained from different specimens

Age days	Mean Compressive strength of mortar N/mm^2	Strength of individual brickwork specimen N/mm^2				
		Flexural			Axial Tensile	Shear
		Test 1a	Test 2	Test 3	Test 4	Test 5
7	3.6					0.45 0.40
13	4.2	0.36 0.30 0.42		0.23 0.17	0.11 0.12 0.09	0.21 0.21 0.29
14	4·9	0.40 0.34 0.31 0.41 0.62	0.52	0.22 0.34 0.62 0.45	0.14 0.16 0.12 0.09 0.09 0.08	0.52 0.45 0.34 0.55 0.41
15	5.2	0.37 0.46		0.43 0.34	0.19 0.08 0.21	0.48
15 •	6.5					0.12 0.24
10	6.8	0.39. 0.36 0.52				0.23 0.19 0.19 0.21 0.39
24	7.6	0.34 0.34 0.48 0.27	0.30 0.26 0.49 0.33 0.27 0.37		0.12 0.10 0.09	0.32 0.20 0.30
22	7.8	0.63 0.55 0.45		0.39 0.41		0.31 0.45 0.41
	8.2†	0.20* 0.11* 0.16*	0.31 0.29			
	Mean S.D. C. of V%	0.38 0.105 27.6	0.35 0.094 26.9	0.36 0.131 36.3	0.12 0.036 29.9	0.33 0.117 35.7

* low strength of this group

† Mortar cubes air cured in Laboratory in this test only; remainder water cured.

The wallettes and prisms in Tests 1, 2 and 3 failed at the brick/mortar interface. In Test 1b, where the tension was resisted by two bricks, three types of failure were observed:—

(i) failure of both bricks in tension, this giving the highest ultimate load.
(ii) failure by a combination of a break of bond and splitting of one brick.
(iii) bond slip failure without any sign of failure in the brick.

In Test 1c the failure was invariably due to tensile failure of the central brick except in a few specimens when bond slip failure occurred. The ultimate strength in the case of bond slip failure was lower than usual.

4. DISCUSSION

Considering all the test results (Tests 1, 2 and 3) the flexural tensile strength appears to be about three times that of the axial tensile strength (Test 4). This is not surprising as a similar trend was noticed with bricks,[3] where the ratio was of the order 2·1 to 2·6 for high- and low-strength bricks respectively. In the axial tensile strength test it is difficult to apply concentric loading, which may result to some extent in low strength.

The wallettes and prisms gave almost similar values for the flexural tensile strength perpendicular to the bed-joint. The ultimate shear stress was also nearly the same as the flexural tensile strength of the beam with 200 mm nominal depth (Table 2). However, in these two tests the coefficient of variation was 36%, which is very high compared to the other tests. It shows that these tests are in no way better than wallette tests for getting the information required. The wallette is more useful than the prism when tested as a beam with a nominal depth of 100 mm because a further test can be done to get the strength parallel to the bed-joint. This makes it more economical for quality control for three- or four-side-supported non-loadbearing panels. In the test specimen 1c failure usually took place by development of a crack passing through the central brick. Thus it represents nearly the maximum possible value of the tensile strength of the wallette parallel to the bed-joint. The mean tensile strength of a single brick calculated from these tests was 3·0 N/mm^2 from Test 1b and 4·0 N/mm^2 from Test 1c. Modulus of rupture tests on individual bricks gave a mean of 4·0 N/mm^2. In test specimen 1b, in which maximum tension in the centre was resisted by two bricks with a perpend joint between, in some cases the failure occurred by bond slip without cracking of either brick, and hence the maximum possible value of the tensile strength parallel to the bed-joint could not be attained.

In one set of tests the flexural tensile strength was very low in both directions. This may be due to several factors: workmanship, surface characteristics of the brick and high initial rate of absorption. The last appears to be the most important factor. The coefficient of variation for the suction rate was 38%, with three bricks out of ten having the very high suction values of 1·97; 2·47 and 2·92 kg/m^2/min. These bricks were also lighter than the rest, but at present there is not much evidence to suggest any relationship between the density of the brick and initial suction rate. The mortar strength 8·2 N/mm^2 was quite high and it cannot be the cause of low strength. Although the strength perpendicular to the bed-joint was comparatively very low, the drop in strength parallel to the bed-joint was not very significant. This may be why the German and Swiss codes allow tension parallel to the bed joint with no reliance on tensile strength perpendicular to it.

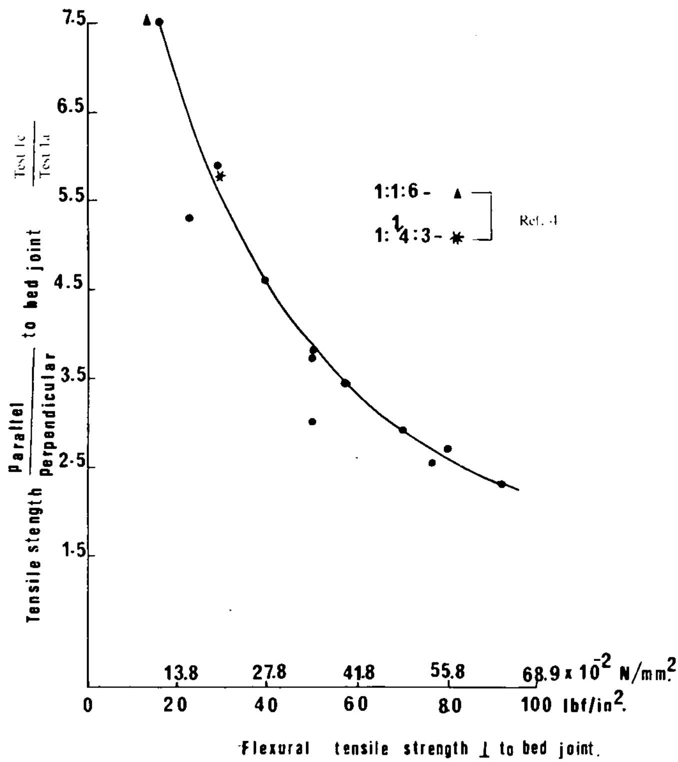


FIGURE 4. Tensile strength of brickwork in two orthogonal directions.

From Figure 4, there appears to be strong evidence to suggest that the ratio of strength parallel and perpendicular to the bed-joint depends upon the value of the latter for a particular brick strength. The factor is not constant as assumed in C. P. 111.[1] The factor varies from 2·4 to 7·5 for this test (Test 1, c & a). The factors reported by Satti and Hendry[2] were 5·4 for a 1:¼:3 mortar and 7·54 for a 1:1:6 mortar both using similar bricks. From Figure 4 it appears that these high factors are not solely due to the use of different mortar mix proportions but are the result of the low tensile strength of the brickwork perpendicular to the bed-joint.

Wallette tests are not in themselves sufficient to determine the ultimate load behaviour of panels with three- or four-sided support, but taken in conjunction with full-scale tests on storey-height panels of brickwork they will help in formulating realistic design requirements.

6. CONCLUSIONS

1. The tensile properties of brickwork can be estimated by testing wallettes of six or eight courses high which may also be adopted for quality control.
2. The flexural tensile strength perpendicular to the bed-joint was several times higher than the axial tensile strength.
3. The tensile strength of brickwork perpendicular to the bed-joint was very variable, however measured, with a coefficient of variation in the range of 27% to 36%. The strength parallel to the bed-joint was more consistent with a coefficient of variation of the order of 10%.
4. The relationship between flexural tensile strength parallel to and perpendicular to the bed-joints was non-linear.
5. The ratio of the flexural tensile strength parallel to the bed-joints to that at right angles to the bed-joints depends on the latter. In the present results mortar strength appeared to have no effect.

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Compressive Strength of Brickwork Walls

(Group 2, Section 1.2.3 - papers 11, 12, & 13)

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3.—Further Tests on Model Brick Walls and Piers

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ABSTRACT

The paper describes the results of tests on a number of one-sixth-scale brick piers and walls. In the case of equivalent 9-in. walls and piers, the variables considered were the brick strengths in compression, axial and flexural tension. The effect of number of courses on the ultimate strength of piers was also investigated. Relationship between masonry strength and brick strength in compression and axial tension are also given. The effect of slenderness ratio on the ultimate strength of 4½-in. (equivalent) walls was also investigated to a limited extent and the strength reductions so found were compared with C.P.111: 1964 and American tests.

1. INTRODUCTION

This paper describes the results of tests on a number of one-sixth-scale brick walls and piers carried out to examine the following matters in an exploratory manner:

(1) The relationship between brickwork strength in compression and brick crushing and tensile strengths.

(2) The relationship between the strength of 9 × 9 in. (equivalent) piers having different numbers of courses.

(3) The effect of slenderness on the strength of 4½-in. (equivalent) walls.

Whilst the tests are too few in number to be conclusive, experience has shown that such model tests give a reliable guide to brickwork behaviour and are of considerable value as a preliminary to full-scale testing. The results presented should therefore be considered from this point of view.

2. MATERIALS USED IN TESTS

2.1 Bricks

One-sixth-scale solid wirecut bricks having the properties set out in Table 1 were used for tests simulating 9-in. walls and piers. For tests on simulated 4½-in. walls, model bricks having a crushing strength of 3885 lbf/in² were used. Compressive tests were carried out in accordance with B.S. 3921:1965. To measure the tensile strength of the model bricks, a number of these were embedded in 1:1 mortar in a standard cement briquette mould. Before embedding, the bricks were kept in water for 2 min in order to develop maximum bond strength.[1] The bricks were set axially in the briquette by fitting them through a $1 \times 1 \times \frac{1}{4}$ in. collar set across the neck of the mould. The collar also had the effect of dividing the mortar into two parts so that only the ceramic material resisted the applied tensile force at the critical section of the test-piece Figure 1. The tests were carried out in an Instron testing machine equipped with suitable grips. Flexural tensile (modulus of rutpure) tests were carried out on simply supported model bricks in the same machine.

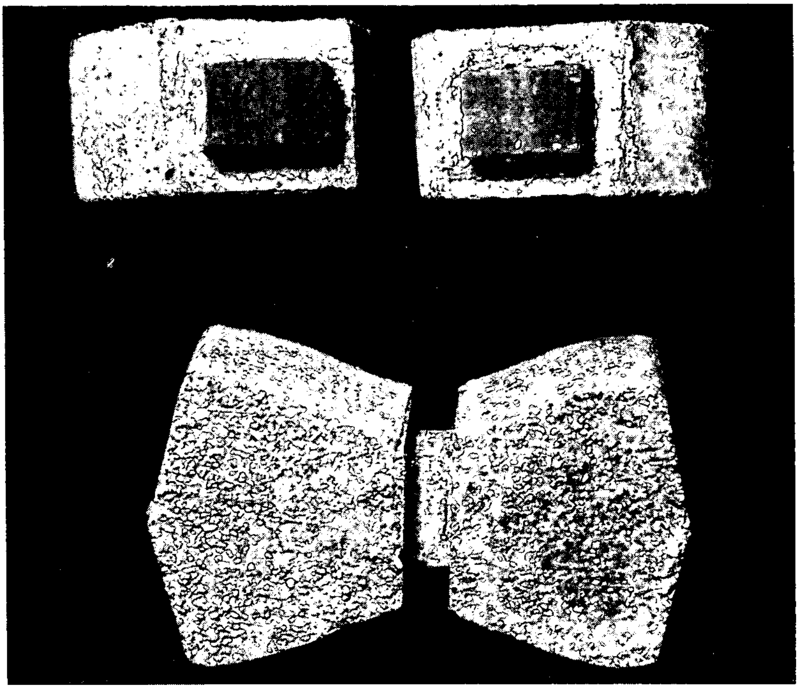

FIGURE 1—Test specimen, and the typical failure of brick in axial tension.

The relationship between compressive strength and axial tensile and flexural strengths are shown in Figure 2.

2.2 Cement and Sand

Ferrocrete to B.S. 12:1958 was used for all the tests. The sand used was dry Leighton Buzzard No. 19; this material conformed to B.S. 200:1955.

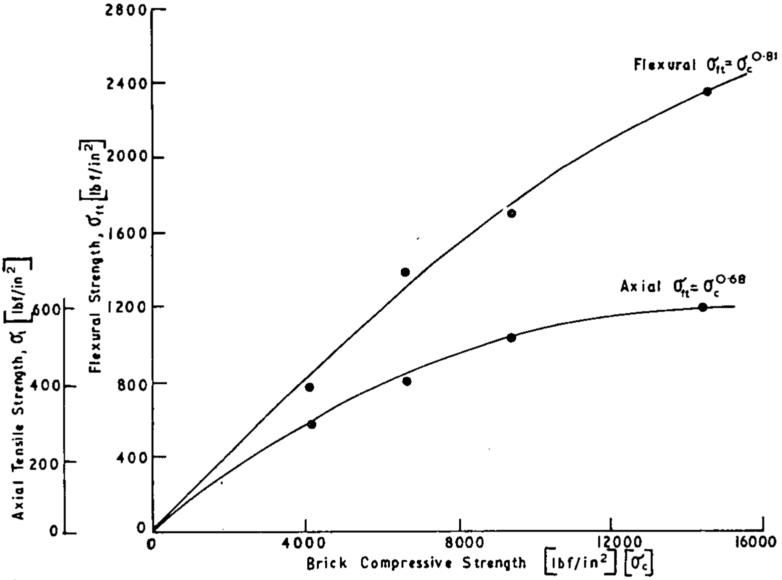

FIGURE 2—Relationship between axial, flexural tensile and compressive strength of brick.

Table 1
Properties of Model Bricks Used in 9-in. Model Wall and Pier Tests

	Test Number			
	1 and 2	*3 and 4*	*5 and 6*	*2 to 9**
Compressive strength (lbf/in²)				
Mean	14 651	9434	6738	4227
Range	13 179–16 410	8271–10 819	5878–7682	3711–4489
S.D.	1104	658	402	294
Coefficient of variation (%)	7·53	6·98	5·97	4·85
Flexural tensile strength (lbf/in²)	2347	1708	1406	773
Axial tensile strength (lbf/in²)				
Mean	598	524	408	294
Range	461–709	440–629	305–522	358–235
S.D.	91	72	64	47
Coefficient of variation (%)	15·17	13·76	15·67	15·97
Water absorption (%) (24-h soak)	11·6	13·4	13·9	12·31

* See earlier tests.[2]

2.3 Mortar

A mortar mix of 1:4 cement/sand (1:3 by volume) was used. The water:cement ratio of 0·91 was kept constant for all the walls. Mortar crushing strengths on 1-in. cubes are reported with corresponding wall tests.

3. TESTS ON 9-in. (EQUIVALENT) WALLS

3.1 Method of Construction and Test

The walls tested were 16-in. high, 6-in. wide and 1·5-in. nominal thickness. They were built in jigs, as previously reported[1] and 7 days before testing, a thin bed of mortar was applied to the top and bottom of the wall. Plywood packing, ¼-in. thick, was used to distribute the load evenly.

The walls were loaded to failure in an Avery universal testing machine under axial load. A summary of the test results is presented in Table 2.

Table 2
Summary of Test Results for Model 9-in. Walls

Test Number	*Brick strength (lbf/in²)*	*Mortar strength (1-in. cubes) (lbf/in²)*	*Ult. load (tons)*	*Ult. stress (lbf/in²)*	*Average stress (lbf/in²)*	*Initial tangent modulus × 10^{-5} (800 lbf/in²)*	*Brickwork strength / Brick strength*
1 2	14 651	1848 1777	21·10 19·85	5383 5064	5224	14·00	0·367 0·346
3 4	9434	2083 3450	14·20 16·50	3600 4157	3878	12·50	0·381 0·441
5 6	6738	2150 1964	16·36 15·80	4113 3973	4043	10·75	0·610 0·590
e.t.2* e.t.3		1940 1380	8·8 11·2	2150 2740	2445	5·92 8·20	0·508 0·648
e.t.4 e.t.5	4227	2350 1120	8·8 10·6	2150 2600	2375	5·42 5·10	0·508 0·615
e.t.6 e.t.7		1904 1315	8·8 11·5	2150 2820	2485	5·50 5·35	0·508 0·667
e.t.8 e.t.9		1926 815	8·8 10·6	2150 2600	2375	6·40 —	0·508 0·615

* See earlier tests.[2]

3.2 Discussion of 9-in. Wall Test Results

As has been found by many previous investigators, the strength of brickwork increases with the compressive strength of the bricks used, but not in direct proportion. Figure 3 shows the relationship between which is typical. The ratio of brickwork to brick strength is thus variable, being higher for lower strength bricks. It is clear therefore brickwork strength and brick strength obtained in the present tests, that comparison of tests on brickwork built of different bricks based on this ratio can be misleading.

Figure 4 shows brickwork strength related to brick tensile strength and in this case there is a reasonably good linear relationship, which is not surprising considering that the brickwork fails by tensile splitting.

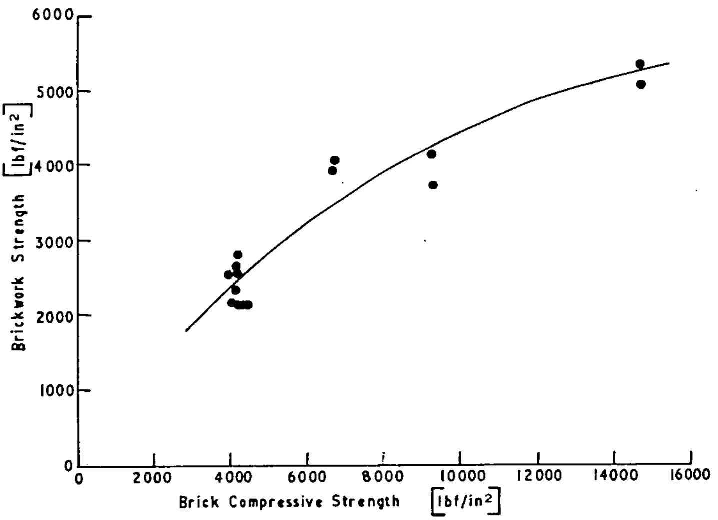

FIGURE 3—Relationship between brick strength and brickwork strength.

Strain measurements made on the model walls are shown in Figure 5 and in Figure 6 the relationship between initial tangent modulus of elasticity of the brickwork and brick strength. Again there is a non-linear variation with compressive strength and a linear relationship to tensile strength.

The main conclusion from these results is that a tensile test on bricks would appear to form a more suitable index to their behaviour in brickwork. Work is now in hand on a suitable tensile test for full-size bricks. Initial results suggest that a convenient test can be devised.

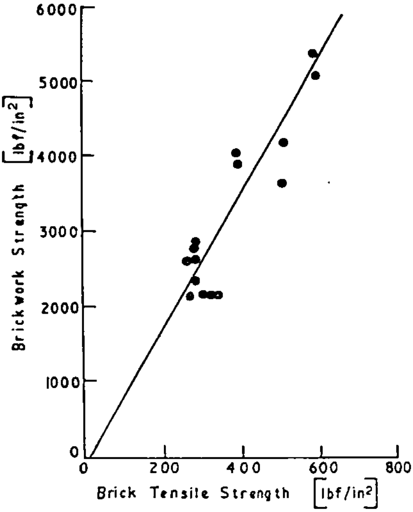

FIGURE 4—Relationship between brick tensile strength and brickwork compressive strength.

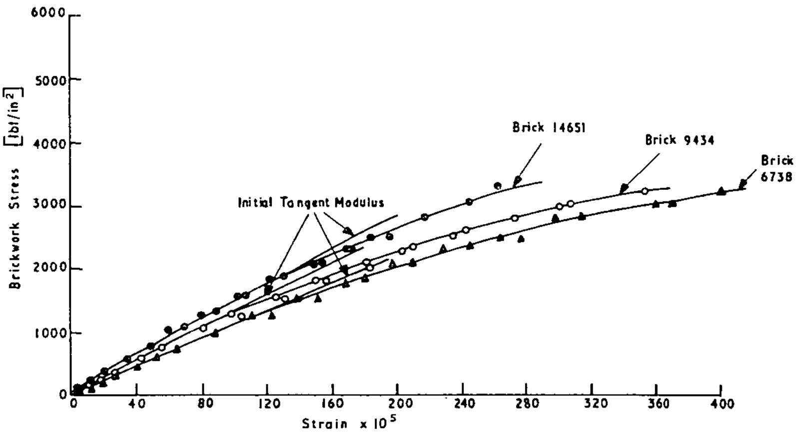

FIGURE 5—Relationship between stress and strain.

4. TESTS ON 9 × 9 in. (EQUIVALENT) PIERS

4.1 Method of Test

The model piers were built in jigs to ensure even coursing and dimensional regularity. Each pier was capped with cement mortar top and bottom and tested in an Avery universal testing machine between pieces of $\frac{1}{8}$-in. plywood.

4.2 Discussion of Test Results

The results of tests on eighty 9 × 9 in. (equivalent) piers, built in four different brick strengths between two and six courses in height, are set out in Table 3. As will be observed from these results and from Figure 7, the pier strength decreases with the increase in number of courses. The effect is most marked with the high-strength bricks. After four to five courses, the strength levels off and may be considered constant.

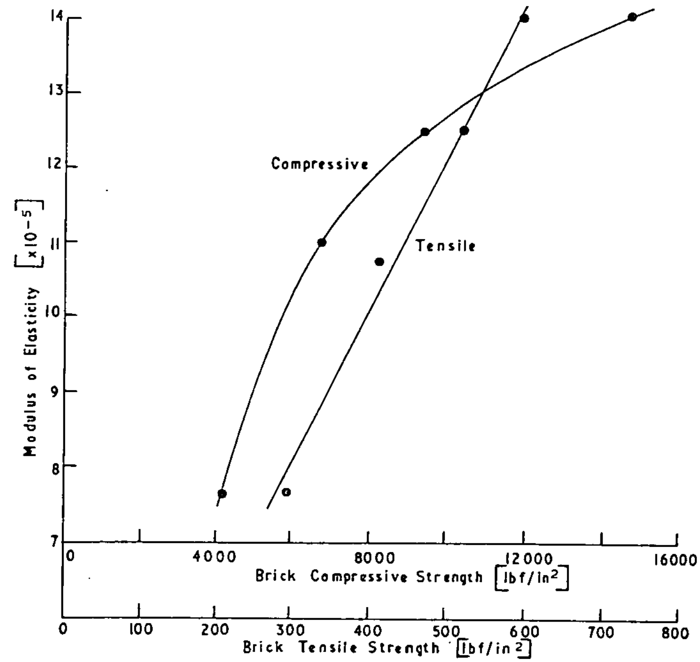

FIGURE 6—Relationship between brick strength and modulus of elasticity of brickwork.

Figure 8 shows the compressive strength of piers, two and three courses high, plotted against the flexural strength (modulus of rupture) and the axial tensile strength. The relationship is non-linear in the case of the flexural strength so that there seems no advantage in using this type of test as compared with a crushing test. There is, however, a reasonably good linear correlation between pier strength and axial tensile strength, as was found in the case of the wall tests.

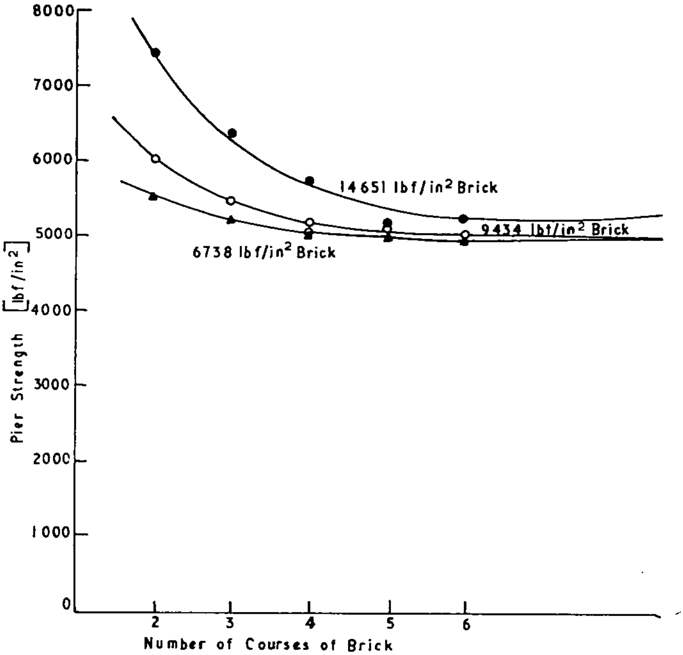

FIGURE 7—Relationship between number of brick courses in pier and brickwork pier strength.

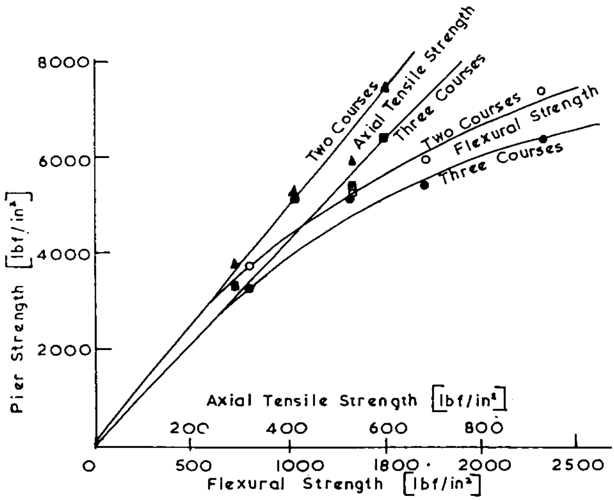

FIGURE 8—Relationship between pier strength andaxial tensile and flexural strengths of brick.

Table 3
Brick, Pier and Wall Strengths

Brick strength (lbf/in²)	*Wall strength / 1-in. mortar strength*	*Average wall strength (lbf/in²)*	*Pier strength (lbf/in²)* Two courses high	Three courses high	Four courses high	Five courses high	Six courses high	*Piers 1-in. mortar cube-strength (lbf/in²)*	*Strength of 3-course piers / Wall-strength*
14 651	5383/1848 5064/1777	5224	7828 7175 7885 6740 7664 Av. 7458	4892 5979 7284 6034 7828 Av. 6403	7175 5327 5806 5240 5164 Av. 5742	5110 5762 4620 4718 — Av. 5052	5979 4674 5327 5240 5218 Av. 5287	2262	1·2
9434	3600/2083 4157/3450	3878	5790 6736 5673 5886 6119 Av. 6040	5865 5503 5153 5280 5684 Av. 5497	5737 5100 5200 4515 4462 Av. 5004	4887 5121 5865 5312 4834 Av. 5203	5567 3782 4672 5312 5312 Av. 4929	1986	1·4
6738	4113/2150 3973/1964	4043	5836 5158 5529 — — Av. 5324	4501 4703 6059 5137 6165 Av. 5313	5540 4894 5646 4840 5349 Av. 5253	5540 5444 4841 5349 — Av. 5293	4343 5550 5127 4597 4789 Av. 4797	2576	1·3
4227	2420*/815–2350	2420	3770 3700 3920 Av. 3770	2940 3380 3480 Av. 3270	3290 2300 2270 Av. 2620	—	—	1367	1·35

* See earlier tests[2] (walls 2 to 9).

4.3 Correlation of Strength of Piers and Walls

As 9-in. brickwork cubes (piers three courses high) have been used for site control purposes and are recommended for this purpose in the Model Specification for Load-bearing Clay Brickwork,[3] the correlation between the strength of model 9-in. cubes and corresponding 9-in. walls may be considered. The ratio of the average of 9-in. cube strength to wall strength varies in these tests from 1·2 to 1·4, with an average of 1·3. Figure 9 shows brickwork strength plotted against brick strength for 9-in. cubes and 9-in. walls.

The ratio of cube strength to wall strength may not be the same for full-size $4\frac{1}{2}$-in. walls for which average ratios of 1·28 for stiff plastic single-frog bricks and 1·65 for perforated wirecut bricks have been reported.[4]

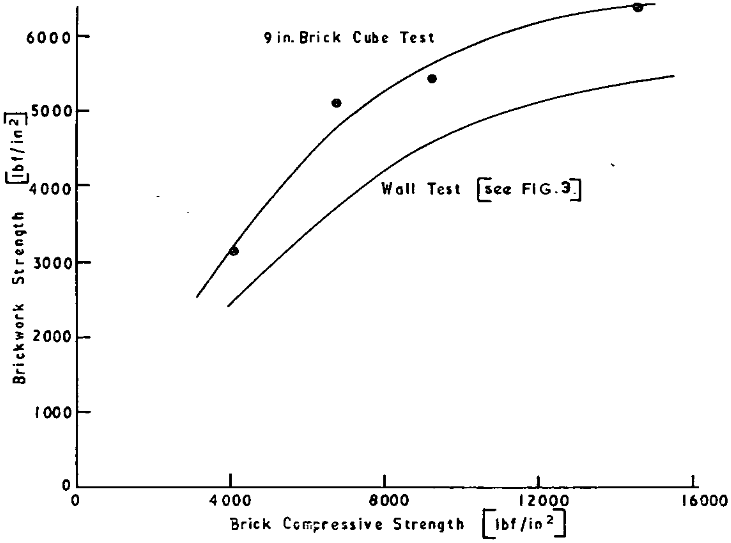

FIGURE 9.—Relationship between brick-cube and brick strength.

5. TESTS ON $4\frac{1}{2}$-in. WALLS TO EXAMINE THE EFFECT OF SLENDERNESS

5.1 Purpose of Tests

The purpose of these tests was to examine the effect of slenderness ratio on the ultimate strength of $4\frac{1}{2}$-in. model brick walls and to compare the strength reductions so found with those implied by the factors given in Table 4 of C.P. 111:1964.

5.2 Details of Tests

Five walls were tested, each 16¾-in. wide by 0·692-in. thick. The walls were built in 1¼-in. base channels for ease of handling. The load was applied through a 2 × 2 in. square steel bar resting on a strip of ⅛-in. plywood which had previously been bedded in mortar on top of the wall to ensure an even distribution of load to the wall. The tests were carried out in an Avery universal testing machine. Strains were measured on both faces with 2-in. and 8-in. Demec gauges; these measurements did not reveal any abnormal loading conditions and as they are not relevant to the main purpose of the tests they are not included in this paper.

5.3 Results of Tests and Discussion

The results of the tests are summarized in Table 4 and Figure 10. The strength of the 4½-in. model walls decreased with increasing slenderness ratio but not to the extent implied by the reduction factors given in C.P. 111:1964. As the number of walls tested is limited, it would not be appropriate to suggest any modification to these factors, but taking into account also a series of full-scale tests carried out in the United States,[5] there is evidence for a reassessment of the code reduction factors in the case of slender 4½-in. walls.

American tests on full-size 9-in. walls (see Figure 10) indicate that the reduction factors for these walls may be different, as may be those for piers upon which, it is understood, a proposed revision of C.P. 111:1964 is based.

The situation as regards reduction factors for slenderness is therefore somewhat confused and further work appears to be necessary in order to obtain satisfactory values for slender elements of various types.

6. GENERAL CONCLUSIONS

The work described in this paper is to be regarded as exploratory and therefore only the following tentative conclusions, suggesting directions for further research, are put forward:

(1) Measurement of the tensile strength of bricks appears to offer a more suitable index test for the performance of bricks in brickwork than the conventional crushing test. Work on a suitable test is proceeding.

(2) Compressive tests on brickwork piers generally give higher strengths than corresponding walls. Furthermore, the ratio of pier strength to wall strength varies with brick strength and type and with wall thickness so that prediction of wall strength from pier tests

Table 4
Ultimate Strength of 4½-in. Wall of Varying Slenderness Ratio

Slenderness ratio	*Compressive strength (lbf/in²)*	*Reduction factor*
7·6	2649	0·98
12	2361	0·87
15·6	2709	1·00
18	2183	0·81
21	2131	0·79

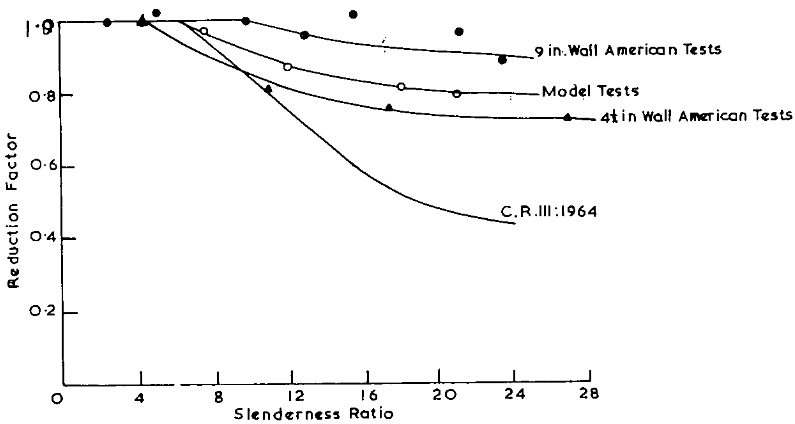

FIGURE 10—The effect of slenderness ratio on ultimate strength of brick wall.

would appear to be difficult. Further work might be of value or it may be decided to regard pier tests as quality control rather than as a means of predicting brickwork strength.

(3) The results of tests on 4½-in. walls with different slenderness ratios are consistent with similar full-scale tests carried out in the United States. Both series of tests indicate that the reduction factors given in C.P. 111:1964 are unduly conservative and should be reappraised on the basis of further wall tests.

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Compressive Strength of Axially Loaded Diaphragm Walls and Walls Restrained on their Vertical Edges

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ABSTRACT
The load carrying capacity of axially loaded brick diaphragm walls and walls supported on their vertical edges is compared with similar strip walls and also with the Code, BS 5628. Walls supported on their vertical edges are not stronger than a similar strip wall and thus the code provisions are not substantiated by these experiments, at least for slenderness ratios up to 32.

1. INTRODUCTION

In any typical masonry structure, the load-bearing walls may be restrained at the top or bottom against in-plane displacement and in addition, on their vertical edges by returns, Figure 1B. In a building where a high unsupported wall is required to carry the load a diaphragm wall is sometimes used.[1] This is essentially a wide cavity-wall connected by webs at regular intervals, Figure 2. Although no comprehensive tests of such masonry walls stiffened along their vertical edges or cellular in plan exist, various Codes[2,3] allow higher compressive stresses in such walls compared to a strip wall, supported at top and bottom, Figure 1A. BS 5628[3] recommends increase in the effective thickness of walls stiffened along their vertical edges by the use of the stiffening coefficients. Thus, the effective slenderness ratio is decreased, which results in a numerically higher value of the capacity reduction factor so increasing

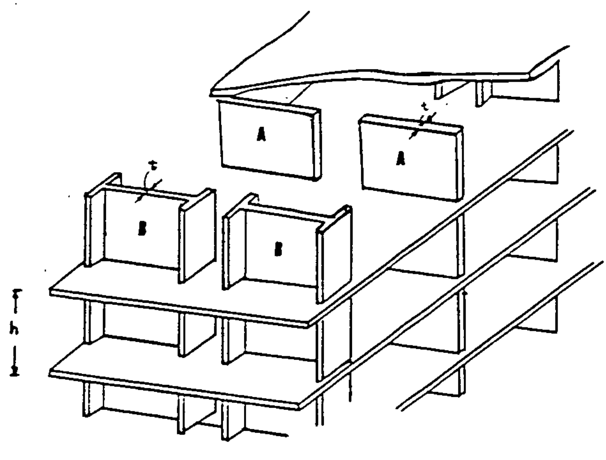

FIGURE 1—Typical masonry structure.

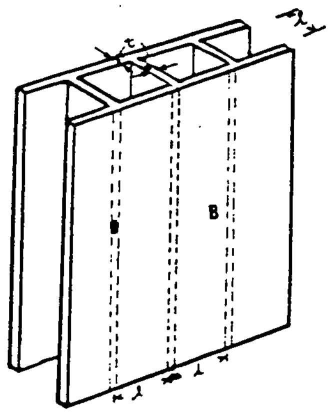

FIGURE 2—Diaphragm wall.

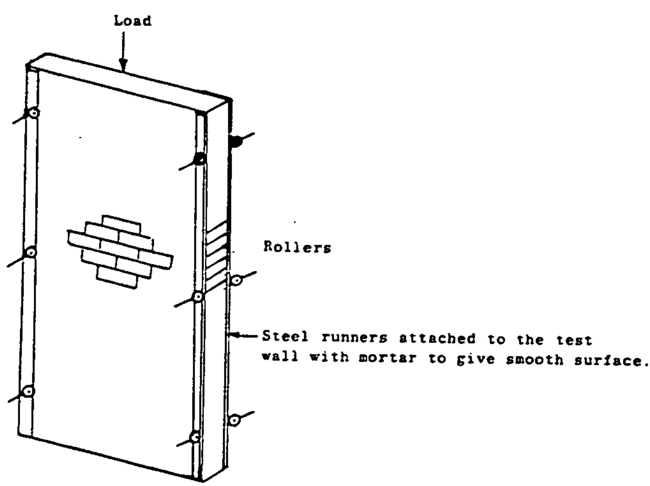

FIGURE 3—Wall laterally restrained by rollers.

the vertical load resistance of the stiffened wall as compared to a similar strip wall. The basis of this comparative increase in the strength of such a wall is somewhat obscure.

An earlier test programme[4] did not substantiate the increase in strength so the investigation was extended to include walls stiffened by returns or simply supported on their vertical edges by rollers and sections of diaphragm walls of various slenderness ratios. Figures 1 to 3 gives details of the various walls. In the earlier investigation[4], where only the web of a wall with returns was loaded, the returns separated at an early stage of the test thus destroying the vertical restraint before failure. It was therefore decided to test walls restrained on their vertical edges by rollers representing simple supports. In practice, this test would represent a load-bearing wall not bonded to returns but simply connected to them perhaps by flexible ties not capable of transferring axial load to the returns by shear, but capable of preventing first mode buckling.

2. EXPERIMENTAL

All test walls and small specimens were built in 1:¼:3 (cement:lime:sand) mortar using half scale single frog bricks of 31·9 N/mm² crushing strength. The average strength at 28 days of 70mm mortar cubes was 15·3 N/mm² and the coefficient of variation was 9·5%. The mean compressive strength of similar cubes for diaphragm walls was 19 N/mm², since these walls were tested approximately three months after construction. All the mortar cubes were tested on the same day as the corresponding test walls.

Eight-course high two bricks long wallettes capped with mortar all around were tested in two orthogonal directions under compressive loading to determine the moduli of elasticity. The axial strains at four points were measured on both faces with a Demec gauge. The moduli of elasticity as obtained from the stress-strain curves are given in Table 1.

Table 1—Modulus of elasticity of brickwork in two orthogonal directions

No.	E_x *parallel to bed joint* N/mm^2	E_y *normal to bed joint* N/mm^2
1	10130	8060
2	9870	8000
3	11800	9500
4	10300	9020
Means	10525	8645

All walls were tested in an Avery Universal testing machine. The diaphragm walls and walls with vertical edges supported by rollers were tested in special rigs which provided the appropriate reactive forces without hindrance to the vertical compression of the wall. The load from the machine was evenly distributed through 152mm armour plate to the web and flanges of the walls stiffened by returns. A concrete slab 152mm thick, heavily reinforced against both shear and flexure distributed the load from the machine to the top of the diaphragm walls. At the beginning of each test care was taken to apply the uniform load axially as far as practicable by monitoring the strains on the different faces of the test walls. When measured strains on the different faces were not exactly similar, flexible packing pieces were introduced between the loading jacks and the platten. The testing was continued only after the measured strains on different faces were similar. The strain was measured with a Demec gauge. The load was applied to the walls in stages up to failure. The crack patterns developed in the various walls are shown in Figures 4, 5 and 6.

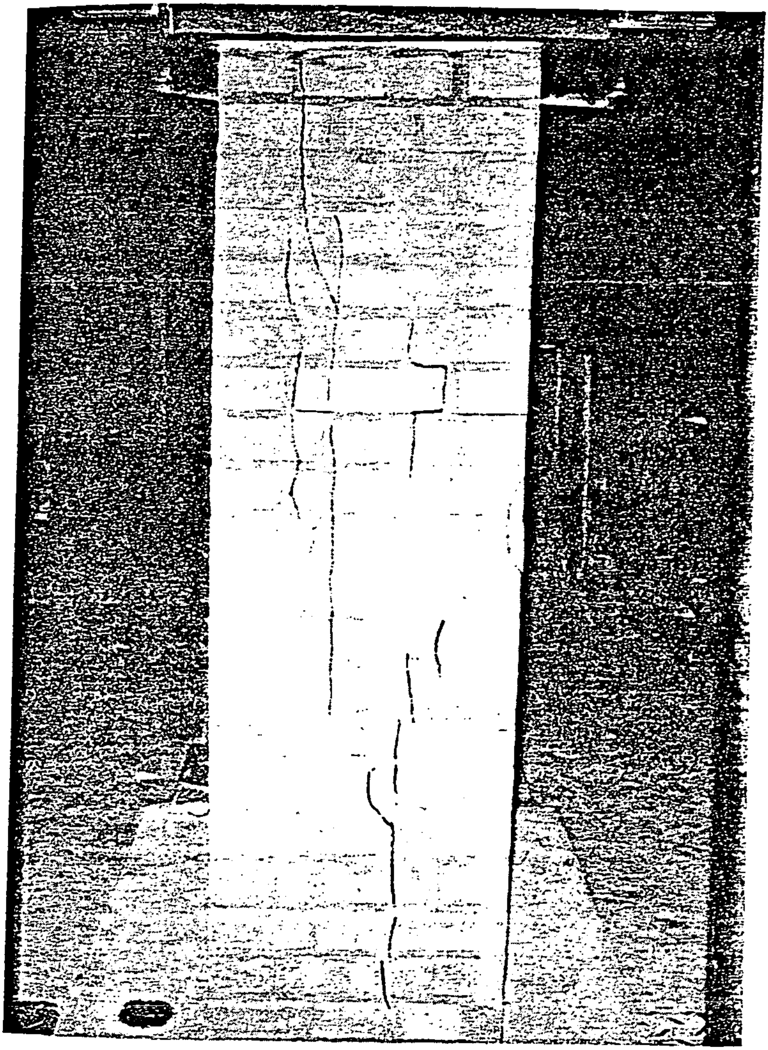

FIGURE 4—Failure of return.

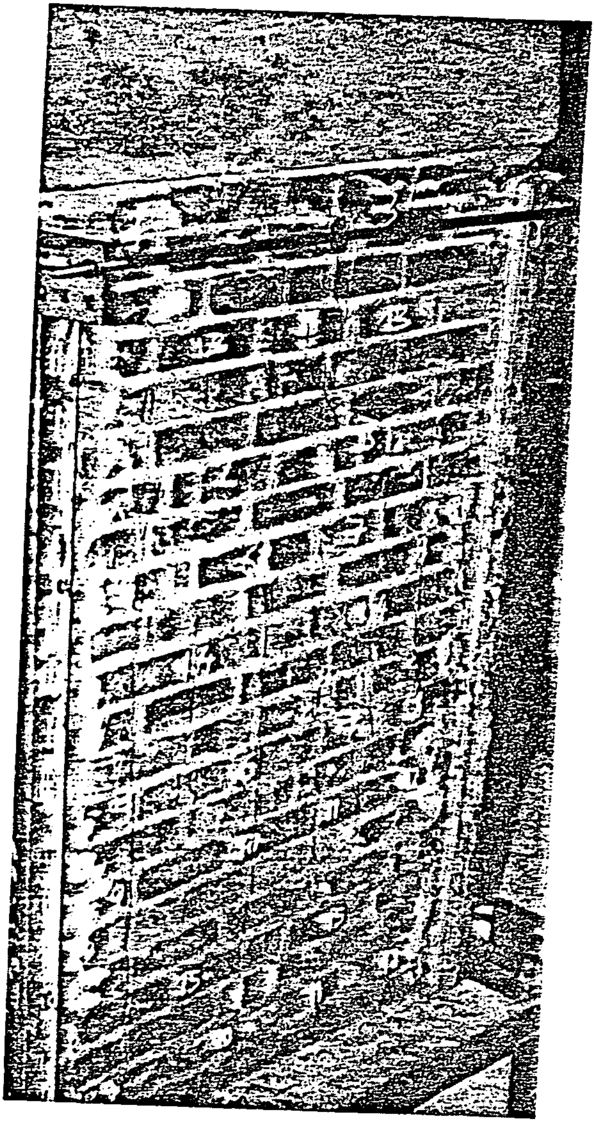

FIGURE 5—Test arrangement and failure of diaphragm wall.

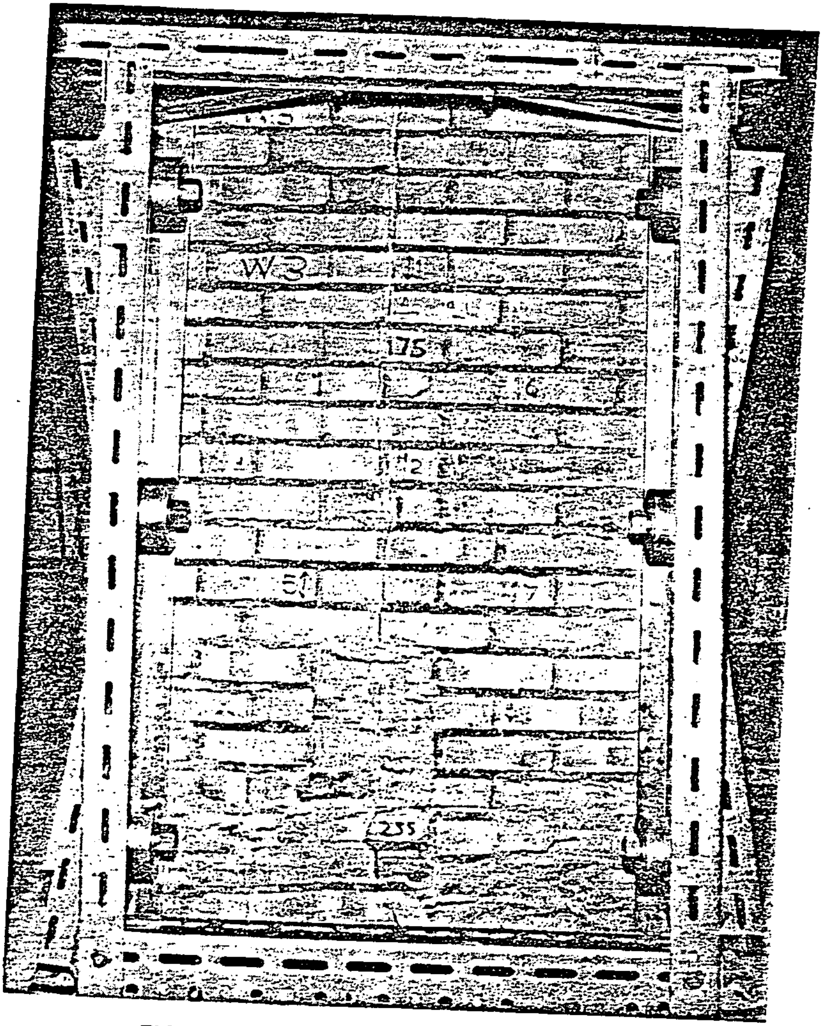

FIGURE 6—Typical failure of simply supported wall.

3. RESULTS AND DISCUSSION

3.1 Wall strengths

The results of the tests are summarised in Table 2. The failure stress of walls with returns, diaphragm walls and walls simply supported along their vertical edges is not higher than a similar strip wall. Under axial loading, all these walls with vertical edge restraint bend in both axes like a plate. The tension developed in the horizontal direction due to plate bending causes the vertical cracking and separation of the

Table 2—Comparison of strip walls and walls with vertical edge support

h/l	h/t	Failure stress: Strip wall N/mm²	Failure stress: Supported wall N/mm²	Ratio of strengths supported/strip
Walls with returns				
0·8	8	11·2	11·0	0·98
0·8	8		10·56	0·94
1·4	8	10·84 } 11·2*	10·24	0·91
1·4	8	11·55 }	9·8	0·87
1·6	16	11·15	9·3	0·83
1·6	16		9·72	0·87
2·8	16	11·15 } 11·15*	10·7	0·96
2·8	16	11·15 }	10·7	0·96
3·12	32	9·35	9·2	0·98
3·12	32		7·9	0·84
5·6	32	9·07 } 9·35*	9·89	1·05
5·6	32	9·62 }	8·50	0·91
Simply supported				
1·6	16	11·15	11·65	1·04 } 0·97*
1·6	16		11·40	1·02 }
1·8	16	11·15	10·62	0·95 }
1·8	16		9·80	0·87 }
3·2	32	9·35	10·63	1·13
3·2	32		13·53	1·40
*Diaphragm walls**				
2·8	16	11·15	10·2	0·91
2·8	16		11·59	1·03
5·6	32	9·35	10·57	1·13
5·6	32		8·87	0·95

Length refers to both cavity leaves and rib. (Figure 2)
*Means of bracketed numbers

returns or webs from the wall flange much earlier than its ultimate bearing capacity. This separation happens in brickwork because it is very weak in tension and shear.

In the case of the walls with returns, these cracks appear in both flanges on either side of the web, thus dividing each flange into two sections as shown in Figure 4. With increasing load these cracks extended throughout the height of the test wall, thus destroying the stiffening effect and resulting in a load-carrying capacity no higher than a strip wall. At between 54% and 76% of the failure stress, similar cracks developed in the diaphragm wall (Figure 5) separating the ribs and flanges and again resulting in no increase in the load capacity. In the case of walls with simply supported vertical edges, tensile cracks appeared at the centre, (Figure 6) at between 40% and 70% of the ultimate stress, thus again removing the effect of stiffening. After the appearance of these vertical cracks, the various separate sections of the wall continued to resist increased loading until failure, which occurred as a result of the development of further cracks accompanied by spalling of bricks.

The walls with simple support along the vertical edges showed some increase in failure stress for the wall with a height/thickness ratio of 32 which cannot be explained by normal scatter of the experimental results. These walls were therefore at just about the point above which the failure under axial loading would be primarily by buckling rather than tensile splitting as happened in these tests. Stiffening of the vertical edges may be effective in increasing the load carrying capacity of a very slender brickwork wall where the failure would occur by buckling at between 40% and 70% of ultimate stress. If buckling stress of a slender strip wall is higher than the stress at which separation of the ribs or flanges occurs in a wall with returns or in a diaphragm wall, the load carrying capacity will remain the same for stiffened or unstiffened walls. Further work on such walls with slenderness ratio (h/t) greater than 32 is continuing to confirm this.

3.2 Tensile strain

The average tensile strain in the horizontal direction ranged between $1{\cdot}8 \times 10^{-4}$ and $2{\cdot}6 \times 10^{-4}$ for the diaphragm walls at the onset of cracks in the flanges. Ignoring the effect of Poisson's ratio, the horizontal cracking stress can be obtained by multiplying the strain by the value of the elastic modulus E_X given in Table 1. The elastic modulus in compression has been used since it can easily be derived and is similar to those obtained from flexural tests[5]. The cracking stress separating the ribs varied from $1{\cdot}84$ N/mm^2 to $2{\cdot}69$ N/mm^2 for these walls with an average of $2{\cdot}2$ N/mm^2. The average tensile strain and stress at cracking was $2{\cdot}0 \times 10^{-4}$ and $2{\cdot}2$ N/mm^2 respectively for the walls simply supported on their vertical edges.

3.3 Comparison of test results with BS 5628

The test results for the wall simply supported and the diaphragm wall cannot be compared with BS. 5628 because their design is not covered under any specific clause. Figure 7, however, shows the comparison for walls with returns and strip walls. The calculation according to BS 5628 is given in Appendix A.

The effective height from the deflection results of test walls[4] appears to be about 0·9h and hence this has been used for the calculations of slenderness ratio for plotting the test results and calculation of the capacity reduction factor. It appears

Table 3—Capacity Reduction Factors

h/t	*Slenderness Ratio Effective height/t*	*B.S.5628*	*Experimental wall type:* *Strip*	*Returns*	*Supported*	*Diaphragm*
8	7·2	1	1	1	—	—
16	14·4	0·83	0·99	0·97	0·97	0·97
32	28·8	0·4*	0·83	0·85	1·07	0·86

*Slenderness higher than allowable, S.R. 27 used.

from Figure 7 that the BS. 5628[3] provision allowing higher load capacity to walls with returns is not substantiated. Table 3 shows a summary of capacity reduction factors obtained in these tests for various walls. These were obtained by dividing the average strength for various walls by the test value of a similar wall with h/t ratio equal to 8. Where data for a wall of similar cross-section for h/t ratio equal to 8 was not available, the average strength of a strip wall of h/t ratio of 8 was used. The corresponding reduction factors given in BS. 5628 are shown for comparison. Except for one, all the test values are very similar, although much less than BS 5628.

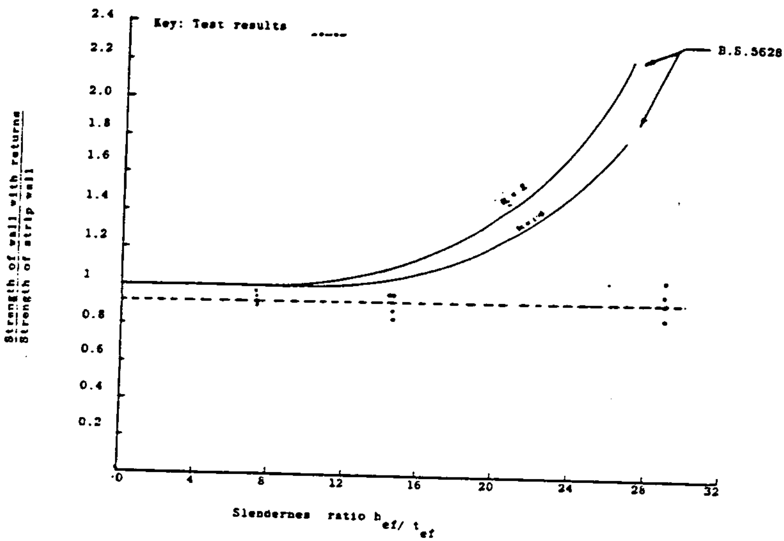

FIGURE 7—Comparison of experimental results for walls with returns and code calculations.

4. CONCLUSIONS

Brick walls with vertical edges or stiffened by returns or diaphragm walls under axial compressive loading do not show any increase in strength over strip walls at least up to a slenderness ratio of 32. It seems reasonable therefore that, up to this limit, no increase in the bearing capacity should be permitted and BS 5628 should be modified.

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ACKNOWLEDGMENTS

The bricks were donated by the London Brick Company Limited.

APPENDIX A

Comparison between design vertical load resistance of a strip wall and a wall with two returns

The design vertical resistance of a wall/unit length

$$= \frac{\beta_t f_k}{\gamma_m}, \text{ Clause (32.2.1)} \qquad \text{(i)}$$

where,

β reduction factor for slenderness ratio
t thickness
f_k characteristic strength of brickwork
γ_m Partial safety factor for material

Consider: A strip wall and walls with return ($h/_L$ = 2·8 and 1·6) with

height/thickness = 16

$$\text{Slenderness ratio} = \frac{\text{Effective height}}{\text{Effective thickness}} \qquad \text{(ii)}$$

$$\text{Hence for (a) a strip wall, S.R.} = \frac{\text{Eff. height}}{\text{Eff. thickness}} = h/t = 16 \qquad \text{(iii)}$$

$\therefore \beta = 0.83$ (Table 7)

(b) wall with two returns ($h/_L$ = 2·8), S.R.

$$= \frac{\text{Eff.ht.}}{Kt} = h/Kt = \tfrac{1}{2}.16 \qquad \text{(iv)}$$

(Tables 5 & 7)

$= 8$

$\beta = 1.0$

(c) wall with two returns ($h/_L$ = 1·6), S.R.

$$= 1/_K.h/_t = \frac{16}{1.4} \quad \text{(Tables 5 \& 7)}$$

$$= 11{\cdot}4$$

$$\therefore \beta = 0.942 \quad \text{(v)}$$

From (i) and (iv)

$$\frac{\text{Design vertical resistance of a test wall (type b) with returns}}{\text{Design vertical resistance of a similar strip wall}}$$

$$= \frac{1.t.f_k}{0.83.t.f_k} = 1.2 \quad \text{(vi)}$$

Similarly, for $\frac{\text{wall with returns type (c)}}{\text{strip wall (a)}}$

$$= \frac{0.942.t.f_k}{0.83.t.f_k} = 1.13 \quad \text{(vii)}$$

These ratios obtained from the code are plotted in Figure 7.

IV–5. Compressive Strength of Axially Loaded Brick Walls Stiffened Along Their Vertical Edges

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ABSTRACT

The paper summarises comparative tests carried out to determine the load carrying capacity of axially loaded brick walls unstiffened and stiffened along their vertical edges, and of various slenderness and aspect ratios. It appears that such walls (stiffened along the vertical edges by return walls) behave like stiffened plates until the appearance of vertical cracks between the returns and the main wall. These cracks neutralise the effect of stiffening, and, as a result, the ultimate strength of the wall is similar to that of an unstiffened or strip wall. This behaviour has been confirmed up to the slenderness ($^h/_t$) ratio of 32. Design codes, as for example BS 5628 (1978) allow higher stresses in walls stiffened along their vertical edges compared to an unstiffened or strip wall, but this is not substantiated by these experiments, at least for axially loaded walls having slenderness ratios up to 32.

INTRODUCTION

The walls in any load-bearing masonry structure may be of two types: i) Unstiffened at the vertical edges; (ii) Stiffened at one or both vertical edges by a return wall. The strength of walls of the first category has been the subject of systematic investigation with a view to establishing the permissible or characteristic strength for the design; no comprehensive tests have ever been carried out on walls of the second category. Although, there is complete lack of data, various codes[1,2,3] have attempted to utilize the stiffening effect of such walls placed at right angles, by allowing heavier loading compared to an unstiffened wall. To remedy this situation, an extensive programme of comparative testing to determine the load-carrying capacity of unstiffened or strip walls and walls stiffened on two vertical edges has been undertaken. The walls were of different slenderness (8 to 32) and aspect ratios and subjected to various loading conditions as shown in Table 1.

MATERIALS

Bricks

Full-scale bricks of 34.3 N/mm², ½-scale bricks of 31.9 N/mm² and ⅓-scale bricks of 30.7 N/mm² compressive strengths were used for the construction of walls.

Mortar

1:¼:3 (cement:lime:sand) Mortar was used for the construction of all the test walls. 100-mm cubes were made for full-scale wall. The average strength of mortar cubes for the full-scale walls were 19.6 N/mm² with a coefficient of variation of 20%. The strength of 70-mm cubes for ½-scale walls were 15.3 N/mm² and the coefficient of variation was 9.5%. 25.4-mm cubes were used for ⅓-scale wall. Their average strength was 27.0 N/mm² and the coefficient of variation was also 20%. All the mortar cubes were tested on the same day as the corresponding test walls.

TEST ARRANGEMENTS

Full and ⅓-scale walls were tested in specially designed test rigs. The distributed load was applied by several hydraulic jacks operated by a single pump. The load was measured by load cells connected to a digital voltmeter and pen-chart recorder. In a few full-scale walls the unloaded flanges of the I-sections were supported at the bottom on a number of load cells to measure the transfer of load from the loaded webs.

Half-scale walls were tested in an "Avery" Universal (Fig. 1) testing machine. The load from the machine was distributed through 152mm armour plate to the web and flanges of the stiffened wall—I in section. Care was taken to see that the load was applied axially as far as possible.

The deflections of the walls were measured by 0.002mm dial gauges. A "Demec" gauge was used to measure the strain.

In some cases, as shown in Table 1, only the main wall, stiffened at both vertical edges, was loaded. In the others both flanges and web were axially loaded, Table 1, equally at all stages till failure.

DISCUSSION OF TEST RESULTS

Wall Strengths

The test results, which are summarized in Table 1, indicate that in both the loading cases the walls with returns do not show increased strength as compared to strip walls. Initially, walls stiffened along the vertical edges by return walls behave like stiffened plates and as a result bend on both axes until the appearance of vertical cracks between the returns (flanges) and the main walls (webs).

In cases where the main wall or the web only was loaded, the cracks appear at the intersection of the flanges (returns) and the main wall (web) as can be seen in Fig. 2. In walls in which both the flanges and web were equally loaded, the cracks appeared in both flanges on either side of the web, thus dividing it into two sections as in Fig. 3. With increased loading, these cracks extended throughout the height of the test wall, thus neutralising the effect of stiffening, and as a result, the ultimate load-carrying capacity of these walls is similar to that of an unstiffened or strip wall.

Deflection

Typical central deflection results for very slender strip walls and walls with returns are given in Fig. 4. The deflection of the wall with returns prior to cracking of the returns was much smaller than in the case of the corresponding strip walls. This indicates that the stiffening effect was evident before onset of the cracks separating the returns with main wall. Also, it can be seen from Fig. 4, that as the distance between the returns increases, i.e. with the increases in aspect ratio, the central deflection also increases. In other words, before cracking the stiffening effect decreases with the increasing aspect ratio.

Strains

Fig. 5 gives the typical strains in the central cross-section of a very slender wall with returns and the corresponding strip wall. The stress-strain curve was linear up to 90% of the failure load. The strains in the wall with returns were smaller than those in a similar strip wall, which confirms the evidence of initial stiffening effect before cracking. Again, the effect is much less with increase in the aspect ratio.

Although great care was taken to apply axial loading, it appears from the strain readings on both faces of the mid cross-section of the strip wall that this was not fully achieved. However, the resulting eccentricity due to deflection and other causes was equal to 0.04 t (t - thickness) or 0.0013 l_e (height) at 88% of the ultimate load; which is minimal. Both faces of walls with returns showed very nearly the same strain, thus indicating that the loading was axial.

Load Distribution and Vertical Shear Stress

In case of walls with returns, where only the web was loaded, some load was transferred to the flanges or returns. It appears that 5.8 to 6.7% of the total applied load was carried by each of the returns before its separation from the web. The average ultimate vertical shear stress which destroyed the bond completely was in the order of 0.345 N/mm^2 to 0.68 N/mm^2 (calculated on an area equal to the height × thickness of the main wall). In these tests returns were bonded to the main wall by alternate brick courses, for any other bonding arrangement the ultimate vertical shear stress may be different.

Effect of Slenderness Ratio

Table 2 shows a summary of capacity reduction factors found for the various walls, taking as a datum the average strength of walls having a slenderness ratio of 8. The corresponding reduction factors given in BS 5628:1978 are shown for comparison. It will be noted that there is no practical difference as between walls with and without returns and that the strength reduction with increasing slenderness ratio is very much less than suggested by the British Code.

The upper slenderness limit of the tests was, however, just about the point at which buckling failure of axially loaded walls would be expected and it is possible that with very slender walls the stiffening effect of returns, which the deflection measurements reveal at lower loads, would prevent buckling failure at a load corresponding to the overall height/thickness ratio. This is now being investigated experimentally.

CONCLUSIONS

1. Walls stiffened along their vertical edges by returns did now show increased strength as compared with strip walls up to a slenderness ratio of 32. It would appear therefore, that up to this limit, no increase in the bearing capacity of axially loaded walls should be made on account of their edges being stiffened by bonded returns. This conclusion holds whether or not the returns are loaded to the same extent as the main wall.
2. Before cracking and separation of returns, the central deflections of walls with returns are smaller than those of corresponding strip walls which indicates effective stiffening up to this point. This stiffening effect decreases with increasing aspect ratio.
3. As returns provide effective stiffening at low axial loads, they may be effective in increasing the strength of very slender walls which may be expected to show buckling rather than strength failures. Further research is proceeding on this aspect.

ACKNOWLEDGEMENT

The work described in this paper was carried out with financial support from the British Ceramic Research Association and the Brick Development Association. Bricks were donated by the London Brick Company.

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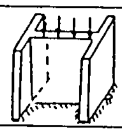
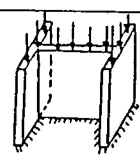
TABLE 1—Test Results of Axially Loaded Strip Walls and Walls Stiffened Along Their Vertical Edges by Way of Returns

Test No.	Ht/Length $L_{e/a}$	S.R. $l_{e/te}$	(a) Failure stress for strip wall N/mm²		(b) Failure stress for walls with returns N/mm²	Strength of walls with returns / Strength of strip walls $^b/_a$
1	1.3	24	10.8	9.2	9.9	1.1
2	1.3	24	7.65		10.9	1.2
3	1.0	24	8.13		8.86	1.08
4	1.0	24			8.86	1.0
5	1.4	8	10.84	11.12	10.24	0.91
6	1.4	8	11.55		9.8	0.87
7	0.8	8	11.2		11.0	0.98
8	Sc10.8	8			10.56	0.94
9	2.8	16	11.15	11.15	10.7	0.96
10	2.8	16	11.15		10.7	0.96
11	1.6	16	11.15		9.30	0.83
12	1.6	16			9.72	0.87
13	5.6	32	9.07	9.35	9.89	1.05
14	5.6	32	9.62		8.50	0.91
15	3.12	32	9.35		7.9	0.85
16	3.12	32			9.2	0.98
17	2.06	32	9.35		7.24	0.77
18	2.06	32			9.08	0.97
19	1.3	24	12.0	14.5	10.6	0.73
20	1.3	24	17.0		9.4	0.65
21	1.0	24	20.3	16.6	10.0	0.60
22	1.0	24	12.94		15.0	0.90
23	1.0	24			11.8	0.71
24	1.0	24			17.7	1.10
25	1.0	24			15.0	0.90

Note 1 to 4 Walls built with ⅓rd scale brick
17 to 18 Walls built with ½ scale brick
19 to 23 Full scale wall



5 to 16 Wall built with ½ scale brick
24 to 25 Full scale wall



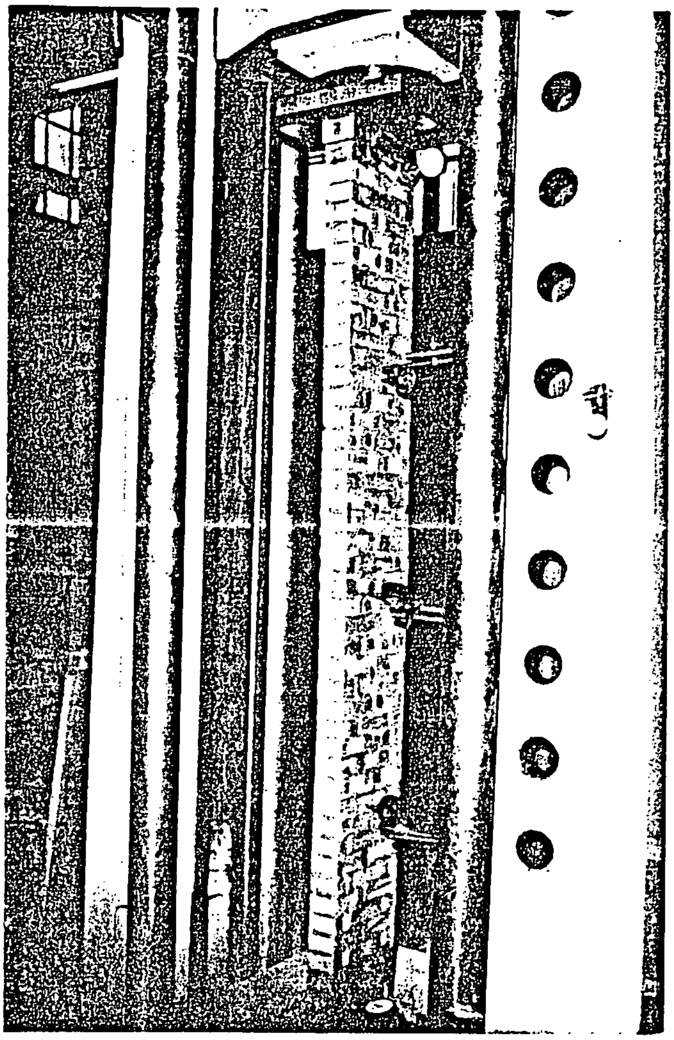


Figure 1. Wall test arrangement

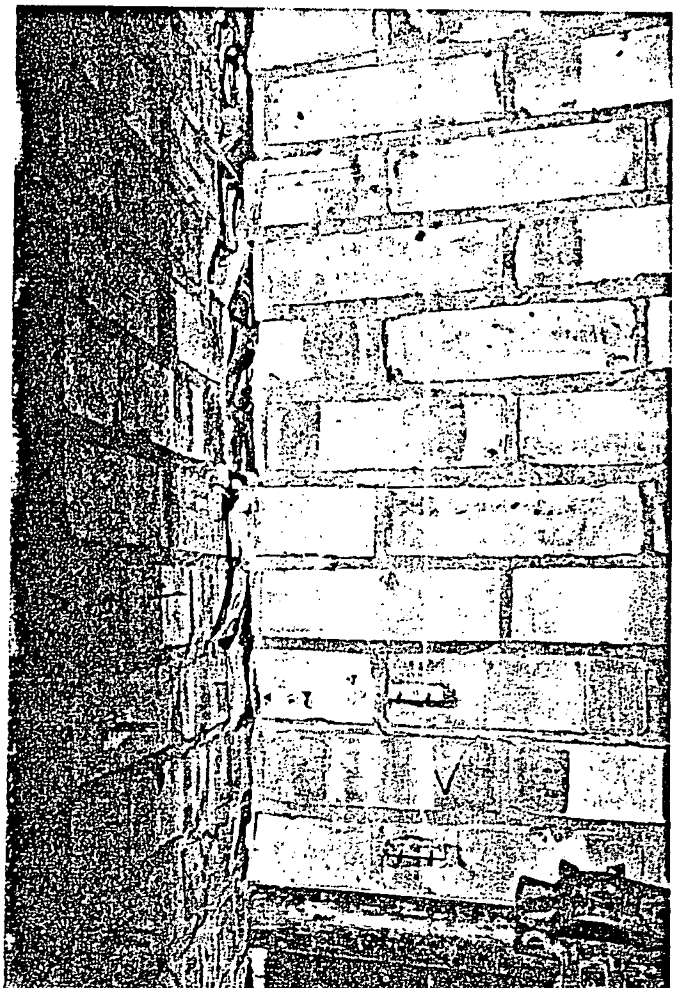


Figure 2. Showing the cracks at the intersection of main wall and return.

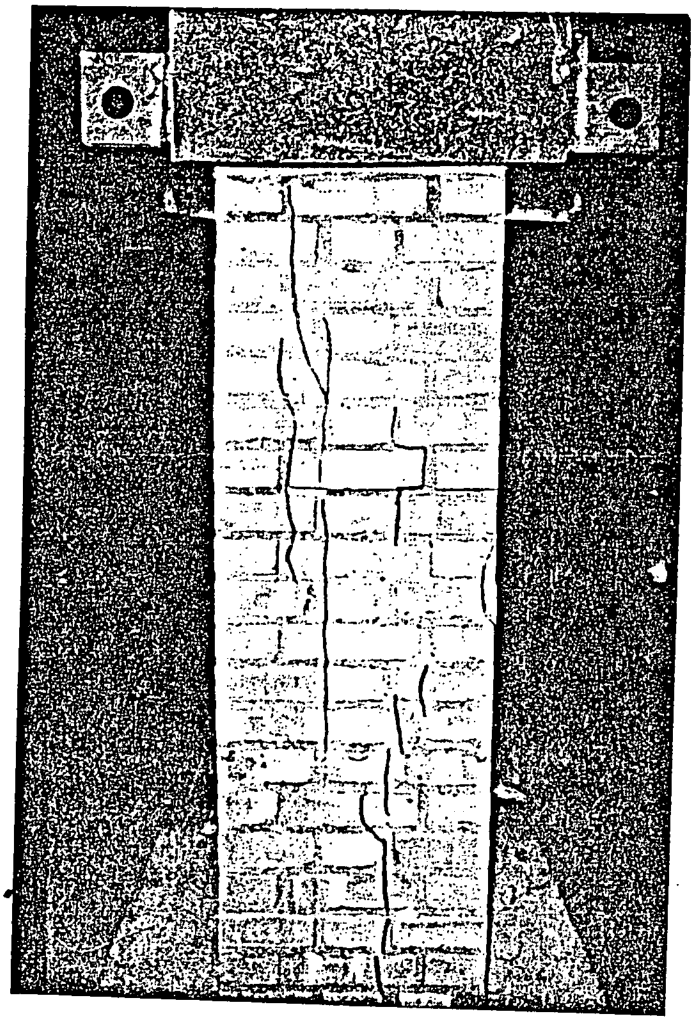


Figure 3. Showing the cracks in the flange on either side of the web.

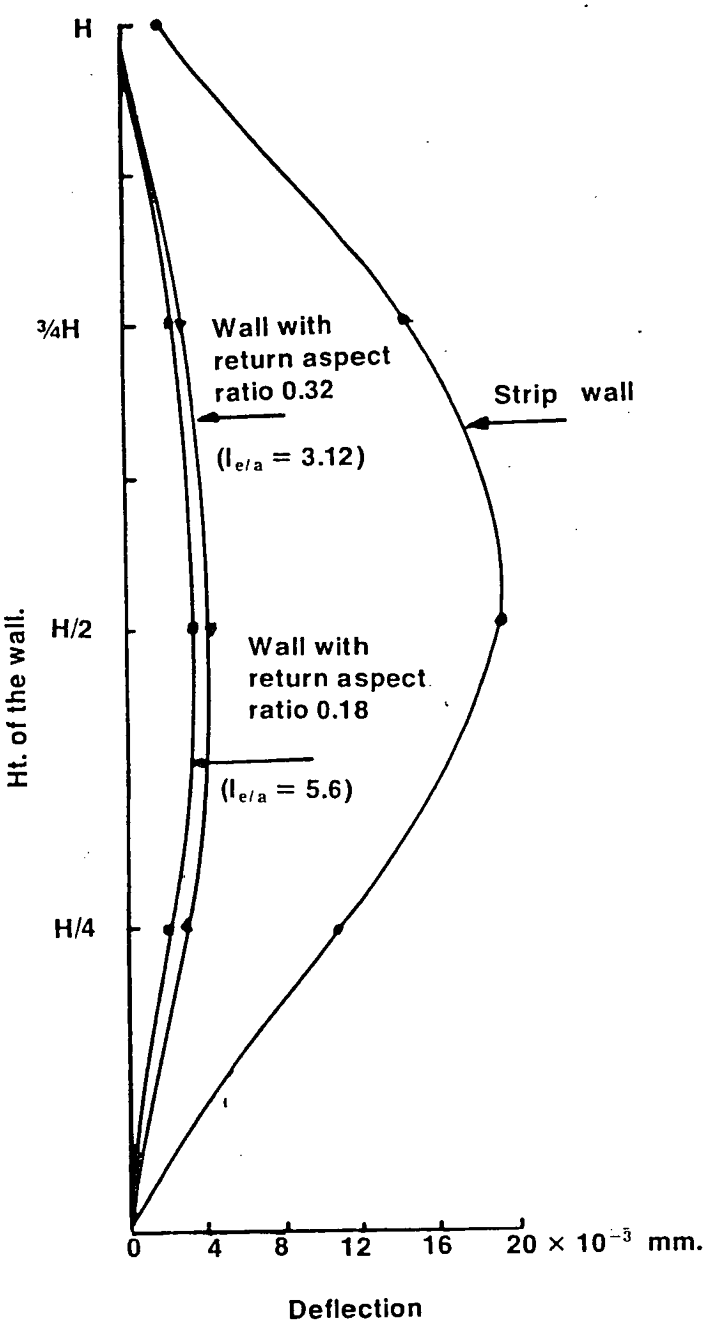


Figure 4. Deflection of walls (S/R = 32) at 6.75 N/mm² stress.

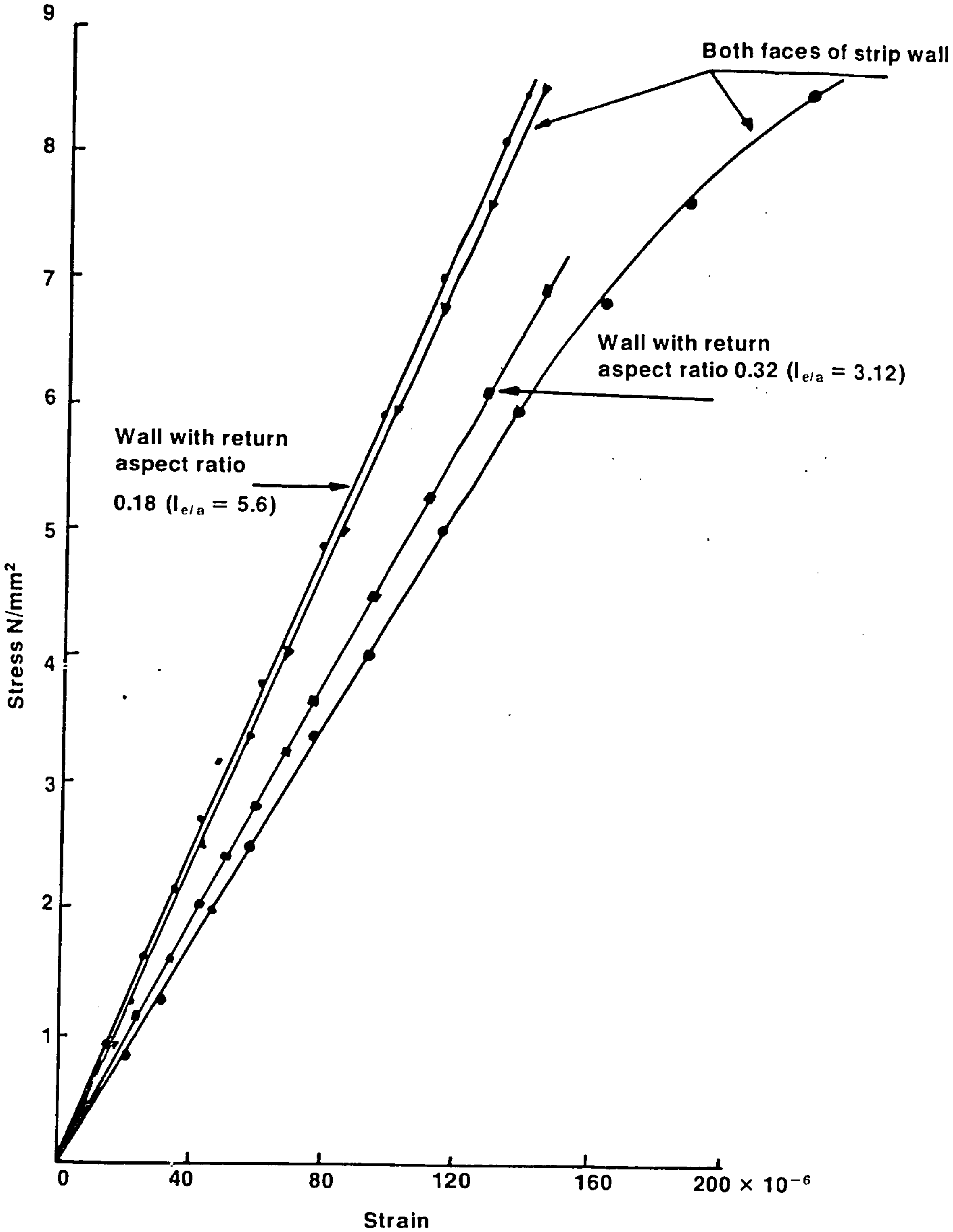


Fig 5 Showing stress-strain relationship

TABLE 2—Capacity Reduction Factor for Slenderness Ratio

$l_{e/t}$	strip walls	Walls with two returns	Capacity Reduction factor BS 5628
8	1	1	1
16	0.99	0.97	0.83
32	0.83	0.83	0.4 (for 27) (32 Not Permitted)

Lateral Strength Of Brickwork Panels

(Group 2, Section 1.2.4 - Papers 15 to 19)

II INTERNATIONAL SYMPOSIUM ON
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WARSAW, SEPTEMBER 1975

HENDRY A.W.
SINHA B.P.

TESTS ON CAVITY-WALLS SUBJECTED TO LATERAL LOADING

/TEST ON FULL-SCALE CAVITY WALLS UNDER LATERAL LOADING/

CIB International Council for Building Research, Studies and Documentation

TEST ON FULL-SCALE CAVITY WALLS UNDER LATERAL LOADING

by

B.P.Sinha, A.W.Hendry
UNIVERSITY OF EDINBURGH

1. Introduction

Simply supported non-loadbearing brickwork panels can resist only a very small lateral pressure due to the low tensile strength of the material. As the result of upward revision of the design wind pressure in the United Kingdom (CP3: Chapter V: Part 2: 1972) the study of the resistance of non-load bearing cavity wall to lateral pressure became an urgent problem. The paper presents the result of a series(1) of tests on 279mm cavity walls. The main objectives of the lateral load test were:

1) to ascertain the strength of a cavity wall with ties at standard spacing i.e. 900mm horizontally and 450mm vertically staggered.
2) the effect of a damp-proof-course on the strength of a cavity wall with standard ties.
3) the influence of the type and number of wall ties and their disposition: i) ties spaced 400mm horizontally and 300mm vertically; ii) with standard ties together with inclined ties.
4) to compare the strength of the cavity-wall with a 102mm single leaf wall
5) the effect of a short return on the strength of the cavity walls: Tests similar to those in No. 2 but with the main cavity wall of aspect ratios (α) l/h = 0.63, 1 and 1.2 all with a short return at one end.

2. Description of Tests:

2.1 General: The details of the test panels are given in Fig 1. The walls were built from a common brick using 1:1:6 (cement: lime: sand) mortar. The materials conformed to the relevant British Standard. The average crushing strength of bricks was 26 MN/m^2. The average strength of 102mm cubes for various tests is given in Table 1. The cubes were cured in water and also left in the laboratory near the test walls to be cured in air. Galvanised steel vertical twist type wallties to B.S. 1243 were used. The type and spacing of the inclined ties are shown in Fig. 1. A bituminous felt damp-proof-course two brick courses above from the bottom of the wall was used for the test walls.

2.2 Test on Small Specimen: In order to obtain the tensile strength of brickwork in two orthogonal directions, a large number of six-courses high; two bricks long wallettes as shown in fig. 2 was tested(2). The wallettes were built with the corresponding walls and tested on the same day to obtain the flexural tensile strength perpendicular to the bedjoint and then after failure, the halves of the same specimen were tested to obtain strength in the other direction; thus all variables affecting the tensile strength remained the same. The average tensile strength perpendicular to the bedjoint was 0.35 N/mm^2 with a coefficient of variation of 27.6%. The average tensile strength of brickwork parallel to the bedjoint was 1.53 N/mm^2 thus giving an orthotropy of 4.36. Some of the results are shown in Table 2. A few specimens were tested with damp-proof-course. The average tensile strength of the specimen with a damp-proof course was 0.04 N/mm^2.

2.3 Wall Tests: The walls were loaded by means of an airbag which reacted against a steel box frame bolted down to the strong floor of the laboratory (fig. 3). The top support for the test walls was also provided by this frame. The loading arrangement and the boundary conditions were very similar to those obtained in practice.

The deflections of walls were measured by means of dial gauges. Pressure was applied by inflating the airbag by means of a compressor, the air pressure being measured with a water manometer. The pressure was applied in small increments until failure.

3. Test Results: The summary of the test results is given in Table 1. The central deflections of strip walls are shown in fig. 4. The failure of the strip wall was always noticed at the mortar and brick interface. The failure of the wall with one return was initiated at the free edge due to the development of a horizontal crack at the interface which changed to diagonal cracks stepping down through the vertical and horizontal mortar joints. The pattern was to some extent similar to yield lines in a reinforced concrete slab. However, the vertical crack or negative yield line at the restrained end did not develop due to premature failure of the return in shear. Cracks were also noticed near the top and bottom supports of the test walls.

4. Analysis of the test results: Because of the nature of the material, there is some doubt as to whether yield line analysis can be applied to a brickwork panel. However, earlier work[3] has suggested that the collapse load can reasonably be estimated by yield line methods. Before any analysis for the three sided cavity wall panels could be attempted it was necessary to establish the relationship experimentally between 102mm wall and a cavity wall with standard ties under similar test conditions. The empirical relationship established from such test is shown in Table 1.

The theoretical ultimate load for 102mm walls were calculated by work method[4] assuming 60% (including the effect of dead weight) and 25% of positive moment for bottom and top support respectively. In case of wall with damp-proof-course, only 27% moment has been allowed for the bottom taki...

into account the dead weight stress and tensile strength at damp-proof-course obtained from the small scale samples.

The results of similar analyses for the three side supported panels are shown in Table 2. Since the fixing moment at top and bottom is not different, the failure pressure for three side supported cavity wall panels with damp-proof-course has been calculated approximately from the equation,

$$m = \frac{\omega \alpha^2 h^2}{12} \left[\frac{3\beta - 2\beta^2}{1{\cdot}26\alpha^2 + \beta^2\mu} \right] \div 1{\cdot}67 \qquad \text{— (i)}$$

where,

$$\beta = \frac{\alpha}{\mu} \sqrt{0{\cdot}7056\,\alpha^2 + 1{\cdot}26\,\mu} - \frac{0{\cdot}84\,\alpha^2}{\mu}$$

The figures 5 and 6 show the effect of the type and number of ties and the effect of return on the strength of a cavity wall.

5. Discussion of the results: The test results and the analysis shows that the yield-line method predicts the strength of the cavity wall fairly accurately and may be used for design. The ultimate load is slightly overestimated in some cases but this could be accounted for by allowing a slightly higher factor of safety in the code.

Contrary to the generally held view, the strength of the cavity wall with standard ties was not twice the strength of a single-leaf under the test conditions. Special ties or an additional number of normal ties increase the stiffness and the strength of an ordinary cavity wall (Fig. 3 and Table1) because of increased co-action between the two leaves. A cavity wall may be treated as a solid wall for design provided the empirical relationship is used for obtaining the equivalent thickness. The inclusion of damp-proof-course (bituminous felt) does not necessarily decrease the strength since reduction in the fixing moment at the base is counterbalanced by the reduction in the span.

The failure of walls under lateral loading was generally due to breakdown of the bond, hence the mortar strength has no direct effect on the ultimate strength.

6. Conclusions:

1. Within the range of the tests the strength of mortar exerted no influence on the ultimate strength of brick walls under lateral loading.
2. Provision of the damp proof course had no detrimental effect on the strength of the test walls subjected to lateral loading.
3. The strength of a cavity wall with standard ties was increased by the use of

additional ties straight or inclined.

4. The strength of a cavity wall with standard ties was not equal to twice the strength of a singl-leaf wall.

5. Provision of a short return substantially increased the strength of the strip cavity wall of aspect ratio (l/h) 0.63, 1 and 1.2. Although the return was quite effective in increasing the strength of walls under test, it would be necessary to examine the effectiveness for aspect ratio 2 or more before any general conclusion could be drawn.

6. Yield line analysis in conjunction with the moments of resistance obtained from small scale specimens predicted the strength of walls under lateral loading.

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Acknowledgement:

The wall tests referred to in this paper were carried out at the request of the Department of the Environment, Property Services Agency, Directorate of Civil Engineering Development, U.K.

The authors are grateful to the P.S.A. of D.O.E. for their permission to publish the results.

Table 1 - Summary of Test Results for strip walls.

Test No.		Mortar strength N/mm²	Ultimate Pressure kN/m²	Average	Cavity wall strength / 102mm solid wall
1 Standard Cavity Wall		6.5 7.8 5.2/6.6* 4.9/6.4*	0.88 1.18 1.08 1.17	1.08	1.48
2 s above but with dpc		3.3 3.3 3.0 3.0	1.29 1.26 1.08 1.27	1.22	1.67
ies spaced at Omm x 300mm	(i)	3.6 5.2/6.6* 4.9/5.4*	1.67 1.76 1.27	1.57	2.14
3 inclined ties	(ii)	5.5 3.6 4.9/5.4* 4.9/6.4*	2.25 2.06 2.06 2.73	2.27	3.0
4 2 mm solid wall		6.9 8.2	0.88 0.59	0.73	1

Table 2 - Comparison of the test results with theoretical failure pressure

Test No.	Support Condition	Aspect Ratio l/h	Mortar Strength N/mm²	Tensile strength (small specimen test) N/mm²	Failure Pressure kN/m²		Theoretical failure pressure kN/m²
4	Two sides supported	- -	6.9 8.2	0.42 0.297	0.88 0.59		0.90 0.62
2	Two sides supported	- - - -	3.3 3.3 3.0 3.0	0.35	1.29 1.26 1.08 1.27	1.22	1.25
5	3 sides supported	0.63 0.63	4.5/5.6* 4.1	0.35 μ = 4.36	4.5 4.4	4.45	4.65
	" "	1 1	7.6/5.6* 5.6/7.4*	" "	3.43 3.21	3.32	3.31
	" "	1.2 1.2	4.4/5.5* 5.1/6.4*	" "	3.23 3.42	3.32	2.98

* Mortar cubes cured in air near the test walls.

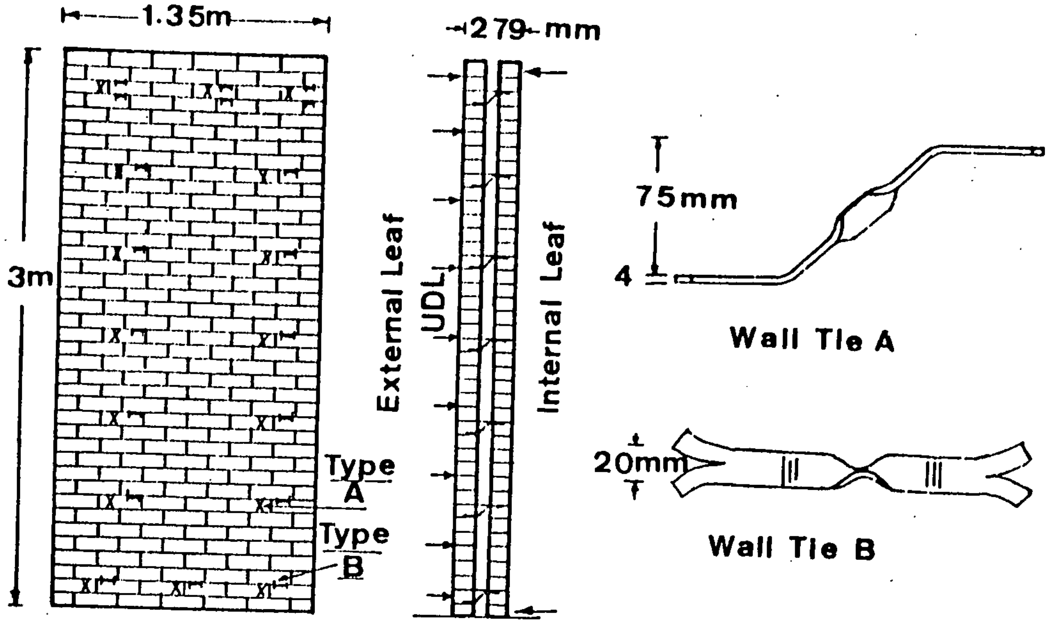


Fig.1- Showing the position and the distribution of the ties.

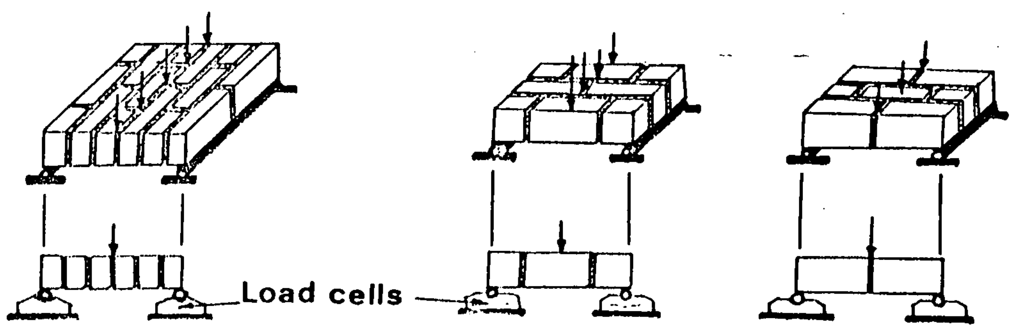


Fig.2 - Test arrangement for flexural strength.

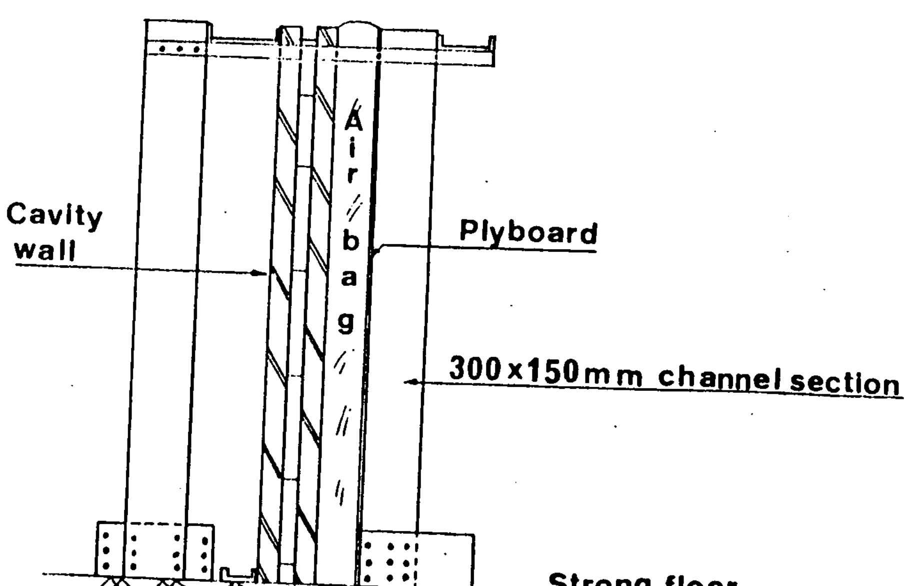


Fig.3 - Test arrangement.

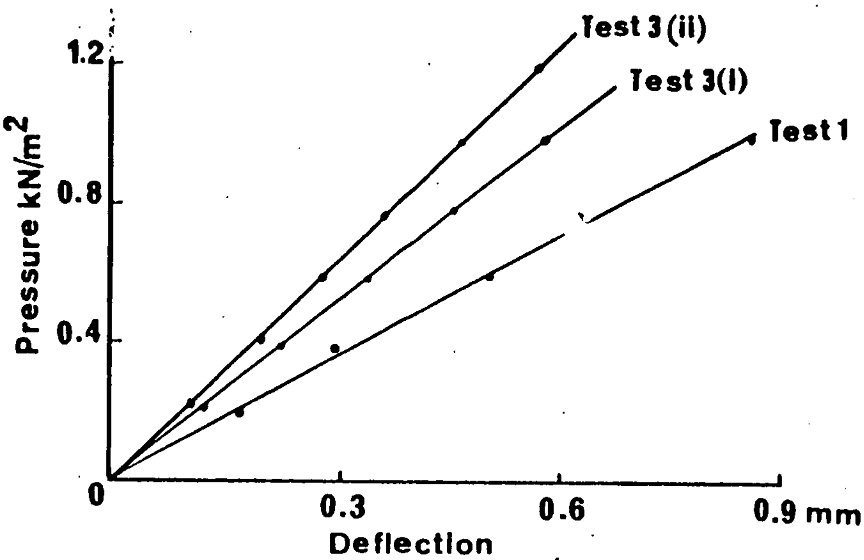
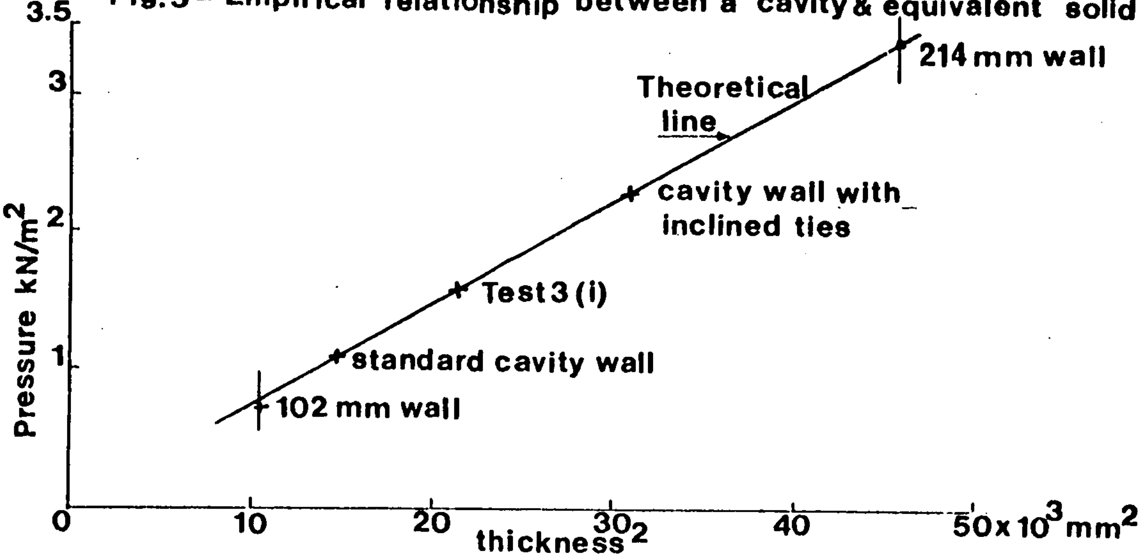


Fig.4 – Relationship between pressure & central deflection of the walls.

Fig.5 – Empirical relationship between a cavity & equivalent solid wall.



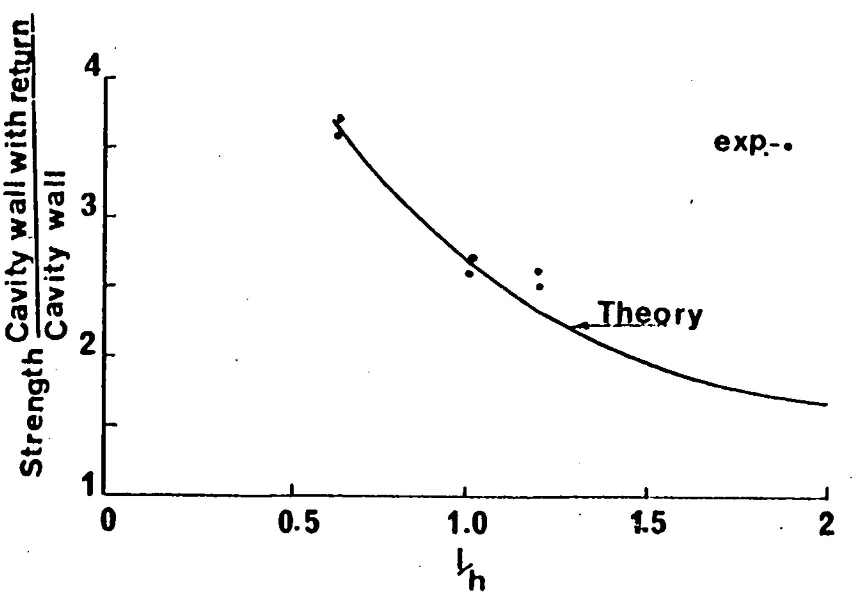


Fig.6 - Effect of return on the strength of strip cavity wall.

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TN 213

Lateral strength of model brickwork panels

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The Paper presents the result of tests carried out on model brickwork panels of different aspect ratios ($L/H = 0{\cdot}5$–2), simply supported on top and bottom and continuous on one or both ends, continuity being provided by short return or returns. The test results are compared with various design methods. The Paper also presents the results of an investigation of elastic properties which confirms that brickwork as a material is orthotropic in nature. A limited investigation was carried out on the effect of unfilled vertical joints in brickwork; it appears that vertical joint filling may contribute up to 44% to the flexural strength of brickwork.

Notation

E_x, E_y	modulus of elasticity in x and y directions respectively
f_{tn}	flexural strength normal to bed joint
f_{tp}	flexural strength parallel to bed joint
H	height
L	length
μ	strength orthotropy, f_{tp}/f_{tn}
ν_x, ν_y	Poisson's ratio in x and y directions respectively

Introduction

The growing interest in the strength of brickwork panels subjected to lateral loading has been enhanced by two factors—the increase in the design wind pressures specified in British Standard CP 3[1] and the introduction of an amendment to the Building Regulations[2] aimed at preventing progressive collapse—both of which require knowledge of the lateral strength of brickwork elements. There are two categories of wall as regards lateral strength: wall panels whose resistance is derived chiefly from the action of in-plane forces and those whose resistance depends on the flexural strength of the brickwork. Panels in the first category with realistic boundary conditions have been investigated with and without returns; a simple theory based on the principle of an internal three-pin arch was proposed by Hendry *et al.*[3] The experimental part of the work of Hendry *et al.* was complementary to a laboratory investigation by West *et al.*[4] on

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panels subjected to in-plane forces. Panels which derive their resistance from flexural strengths have been investigated by several workers[5-10] and empirical, elastic (isotropic)[5] and yield line[6,8,9] methods have been put forward to explain experimental results. The boundary conditions of the test panels used in these earlier investigations were similar to those of the infilled panels[7] of a framed structure or to those of panels simply supported on three or four sides.[5,9,10] The brickwork test panels in the present investigation were simply supported on the top and bottom and bonded at one or both ends to return wall or walls.

Experimental details

2. All the test walls were built with bricks one third of the normal size and a rapid-hardening cement–sand (1:3) mortar mix. The traditional method of bricklaying was used.

3. Small control specimens were used to determine the strength and elastic constant of the brickwork.

4. At present there is no standard test for determining the flexural strength of brickwork. There were certain advantages[11] in using eight-course and six-course high wallettes two bricks long (Fig. 1) to determine flexural strength[5] in two orthogonal directions.

5. When each test wall was built, two wallettes were also built which corresponded to each test wall and they were tested at the same age to obtain the flexural strength in two directions. The results of the tests are given in Table 1.

6. The effect of unfilled perpendicular joints on the compressive strength of brickwork is widely known, but not in relation to flexural strength, and so some wallettes with unfilled perpendicular joints were built in addition to wallettes in which the perpendicular joints were filled. The unfilled joints represent an

Table 1. Relationship between flexural strengths in two orthogonal directions as determined from tests on wallettes

Corresponding test wall	Age at test, days	f_{tn},* N/mm²	f_{tp},† N/mm²	μ
Six-course wallettes				
A1	7	0·35	1·04	2·97
A2	7	0·48	1·24	2·60
B1	7	0·33	1·17	3·58
B2	7	0·53	1·54	2·90
C1	7	0·39	1·02	2·61
D1	7	0·26	0·95	3·73
D2	7	0·39	1·22	3·13
Eight-course wallettes				
E1	12	0·51	1·62	3·18
E2	12	0·53	1·62	3·05
F1	12	0·43	1·13	2·63
F2	12	0·63	1·34	2·13
G1	12	0·56	1·90	3·39
G2	12	0·88	2·02	2·29

* Average of two results.
† Average of four results.

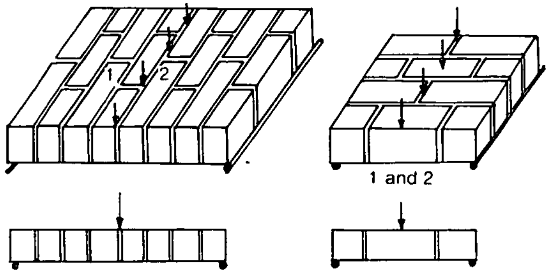

Fig. 1. Test arrangement for determination of flexural strengths in two directions

extreme case which is highly unlikely to occur in practice. Both types of wallette were tested at the same age under central line loading as shown in Fig. 1. The results of the tests (Table 2) for the wallettes with unfilled joints show an average of 44% drop in strength, but the orthotropy remained similar to that of the normal wallettes. Although the same bricks were used throughout the investigation, Table 2 shows three different sets of results because there was a considerable time lag between the building of both types of wallette.

Modulus of elasticity and Poisson's ratio

7. To obtain the values of Young's modulus and Poisson's ratios in two orthogonal directions, eight-course high wallettes were built in addition to flexural wallettes. These were capped with mortar round their four edges to eliminate any unevenness. They were tested at the same age as a wall under uniform compression in each direction and the resulting strains in two orthogonal directions were measured by means of 63·5 mm vibrating wire gauges to obtain the moduli of elasticity and Poisson's ratios. The test results are given in Table 3. The moduli of elasticity in the two directions were different; the average ratio (parallel to the bed/perpendicular to the bed) appears to be 1·4. The values of Poisson's ratio (Table 3) in the vertical and horizontal directions are also different, their ratio also being about 1·4.

Table 2. Effect of unfilled perpendicular joints on the flexural strengths of wallettes

f_{tn}, N/mm²			f_{tp}, N/mm²		
Filled joints	Unfilled perpendicular joints	Drop in strength, %	Filled joints	Unfilled perpendicular joints	Drop in strength, %
0·59*	0·33‡	44	1·6*	0·89‡	44
0·96†	0·55†	43	2·18§	1·31§	40
0·98†	0·55†	44	2·17§	1·16§	47

* Average of wallettes corresponding to walls E1, E2, F1, F2, G1 and G2.
† Average of three results.
‡ Average of two results.
§ Average of six results.

Table 3. Relationship between moduli of elasticity and Poisson's ratio

Number of specimens	Corresponding wall	E_y, N/mm²	E_x, N/mm²	ν_x	ν_y
1	E1	7900	11 100	0·16	0·11
2	E2	9000	13 500	0·13	0·09
3	F1	7100	10 700	0·15	0·10
4	F2	8400	10 100	0·14	0·12
5	G1	8600	12 100	0·17	0·12
6	G2	9900	12 900	0·17	0·13
Mean		8483	11 733	0·15	0·11
Standard deviation		954	1 323	0·016	0·0147
Coefficient of variation, %		11·2	11·3	10·9	13·0

Wall tests and loading arrangement

8. It was felt that the test panels must have well-defined boundary conditions as far as was practicable. Hence they were all built vertically on a strong floor of the laboratory with the lowermost course of brick resting on, but not bonded to, a polythene membrane between the floor and each test wall. There was thus negligible restraint at the base due to interface frictional resistance. All the panels were simply supported laterally along the top and bottom; this eliminated any indeterminate restraint. The support reactions were sustained by an independent frame fixed to the strong floor of the laboratory.

9. A two-brick long return was built at one end of the panels supported on three sides and at both ends of the panels supported on four sides. The returns were post-tensioned in both cases to avoid premature failure. The maximum restraining moment at the return is therefore determined by the flexural strength of the brickwork in the plane parallel to the bed joint, which itself was established from the corresponding wallette test. Throughout the testing the post-tension load was monitored by a load cell; no increase or decrease was recorded. The lateral load was applied to the test wall by means of an air bag sandwiched between the wall and a rigid reacting frame. The pressure was applied in small increments by pumping air into the bag using a small air compressor. The pressure in the bag was measured by means of a water or mercury manometer. Wallettes corresponding to each test wall were tested at the same age to obtain the strength and elastic constants in two directions.

Results compared with design methods

10. The results of the tests are summarized in Tables 4 and 5.

11. The yield line[12] method as applied to reinforced concrete slabs was used to calculate the failure pressure. In most cases, as can be seen from Tables 4 and 5, the yield line analysis overestimates the failure pressure. This is understandable, as it is difficult to imagine any form of yield behaviour in brittle material like brickwork after cracks have appeared and, once cracked, the material can carry little or no moment.

12. An elastic isotropic plate analysis was not attempted, because small scale

tests had already confirmed that a brickwork panel can only be treated as orthotropic. The problem is more complicated because the average strength orthotropy varied from 2·13 to 3·73 and the element stiffness orthotropy (i.e. the ratio of flexural rigidity in two directions) was about 1·4. The strength orthotropy was similar to that obtained in the earlier test on full-scale[11] brickwork wallettes.

13. As moduli of elasticity in two directions were available from the wallette tests corresponding to panels supported on four sides, the Grashoff–Rankine method of mid-span deflexion compatibility was used to calculate the failure pressure using the stiffness orthotropy. The results are given in Table 4. Failure was assumed when either the bending moment in the horizontal or vertical direction reached the corresponding moment of resistance calculated from the wallette test, regardless of the reserve strength in other directions. Not surprisingly, this elastic method based on orthotropy (Table 4) underestimates the failure pressure of the panels.

14. An idealized fracture line analysis[13] using both strength and stiffness orthotropy was done for panels supported on four sides; the results are given in Table 4. The analysis predicts very closely the experimental pressure, the

Table 4. Failure pressure for wall panels 796 mm high with two sides simply supported and two sides continuous

Wall panel	Aspect ratio L/H	Ultimate pressure, $N/mm^2 \times 10^{-3}$			
		Experimental	Yield line analysis	Orthotropic Grashoff–Rankine method	Fracture line analysis
E1	1·0	10·7	13·9	8·2	10·78
E2	1·1*	11·6	15·1	8·9	11·40
F1	1·5	4·7	5·8	2·9	4·60
F2	1·5	6·0	7·4	3·8	5·96
G1	2·0	5·2	6·2	2·3	5·16
G2	2·0	6·7	7·8	3·5	6·60

* $H = 708$ mm.

Table 5. Failure pressure for panels 760 mm high simply supported on two sides and continuous on one side

Panel	Aspect ratio, L/H	Failure pressure, $N/mm^2 \times 10^{-3}$	
		Experimental	Yield line analysis
A1	0·54	7·45	7·28
A2	0·54	10·2	9·20
B1	1·06	2·55	4·50
B2	1·06	5·89	6·45
C1	1·5	2·06	4·30
D1	2·14	1·37	3·10
D2	2·14	2·55	4·20

maximum error only being 2%. Further work on panels of different shapes and types (i.e. octagonal, triangular and with openings) is under way to check the validity of the theory.

Conclusions

15. The flexural strengths of brickwork specimens are different in the directions parallel to and normal to bed joints; in the present tests the average ratio of strength parallel to bed joints to strength normal to bed joints varied from 2·13 to 3·73.

16. Moduli of elasticity and Poisson's ratios were also different in two orthogonal directions, and their ratio for the bricks tested was about 1·4.

17. There are insufficient results to establish properly the reduction in flexural strength due to unfilled perpendicular joints. However, in the tests described a 44% drop in strength in both directions could be attributed to this factor, i.e. the filling of vertical joints contributes up to 44% of the flexural strength of the brickwork.

18. Comparisons made between the experimental results and the theoretical failure pressures showed that the elastic theory based on orthotropy underestimates the failure pressure in lateral loading, and the yield line method overestimates them.

19. There is good agreement between the experimental and the analytical results derived by a modified method (fracture line) for panels supported on four sides. This method may be used for panel design.

Acknowledgements

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Lateral strength of brickwork panels with openings

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Paper 9884

Written discussion closes 15 January 1993

The Paper presents the results of an experimental investigation into the behaviour of brickwork panels subjected to lateral pressure. Twelve panels with window openings built with half-scale bricks were tested to failure. The variables considered were aspect ratios and boundary conditions. The experimental failure pressures were compared with those obtained by the yield-line and elastic analysis.

Notation

q	applied external pressure
m	ultimate moment per unit length along a yield-line
μ	strength orthotropy
ϕ_x and ϕ_y	rotation of the yield-line along the x and y axes
L_x and L_y	projection of the yield-line over the x and y axes

Introduction

Brickwork cladding panels are subjected to wind loading. These panels often contain openings. They resist load on account of plate bending, and their load-carrying capacities depend on the flexural tensile strengths normal and perpendicular to the bed-joint. For the design of panels without openings, BS 5628[1] gives the values of the bending moment coefficients similar to those that can be obtained by yield-line analysis applied to under-reinforced concrete slabs.[2] Strictly speaking, the application of the yield-line analysis to a brittle material such as unreinforced brickwork is questionable. However, the code gives these coefficients based on some test results as design guidance without explicitly acknowledging the sources for these coefficients. No such guidance is available for the design of panels with openings. The suggestion is made in the code,[1] Appendix D, to divide the panels into sub-panels and then to design each part either in accordance with the rules given in clause 36 or by the yield-line or elastic analysis. Some test results of lateral strengths of brickwork panels containing openings are available.[3,4] These have ignored the line loading which develops naturally at the edges of a window opening as a result of wind pressure. Also, no definitive mathematical solution is available at present for panels with window openings subjected to wind loading. Hence, an experimental investigation was carried out on panels with window openings to study the behaviour under lateral pressure. The variables considered in this study were: aspect ratio (h/l); the boundary conditions. The window was positioned in the centre of the panels in every case.

Experimental procedure

Panel details

2. Half-scale bricks were used to build the 12 test walls in 1 : 3 (rapid-hardening cement : sand) mortar. The average cube strength of the mortar varied from 10–18 N/mm², with the characteristic strength of 10·8 N/mm² at 14 days. The dimensions of, and the positions of openings in the test walls are shown in Fig. 1. A plyboard sheet was used to represent the closed window which transferred the wind pressure to the edges of the window opening. It was found that, owing to the different deformation properties of brickwork and the plyboard sheet, the load was transferred as point loads at the corner of the opening. Hence, in order to improve the modelling for the theoretical analysis, it was decided to transfer the pressure from the plyboard as four equal corner loads through four wooden studs fixed at the corner of the test wall, which gave the exact determinate values of the reactions.

3. The lateral loading in steps of 0·4 kN/m² was applied until failure by an air-bag sandwiched between the test wall and the loading frame. The pressure was measured by the water manometer. The deflections at various points were measured by dial gauges. The points at which the deflections were measured are given in Fig. 1.

Determination of flexural tensile strengths and elastic properties

4. The flexural tensile strengths normal and perpendicular to bed joints were obtained by testing wallettes, as shown in Fig. 2. These wallettes were built along with the test walls. In addition, wallettes were extracted from the undamaged portion of the failed walls for obtaining the flexural tensile strengths normal and perpendicular to the bed joint. These wallettes were tested to identify any differences in the strengths compared to the wallettes built along with the test walls.

5. The moduli of elasticity and the Poisson's ratios were obtained by testing wall-

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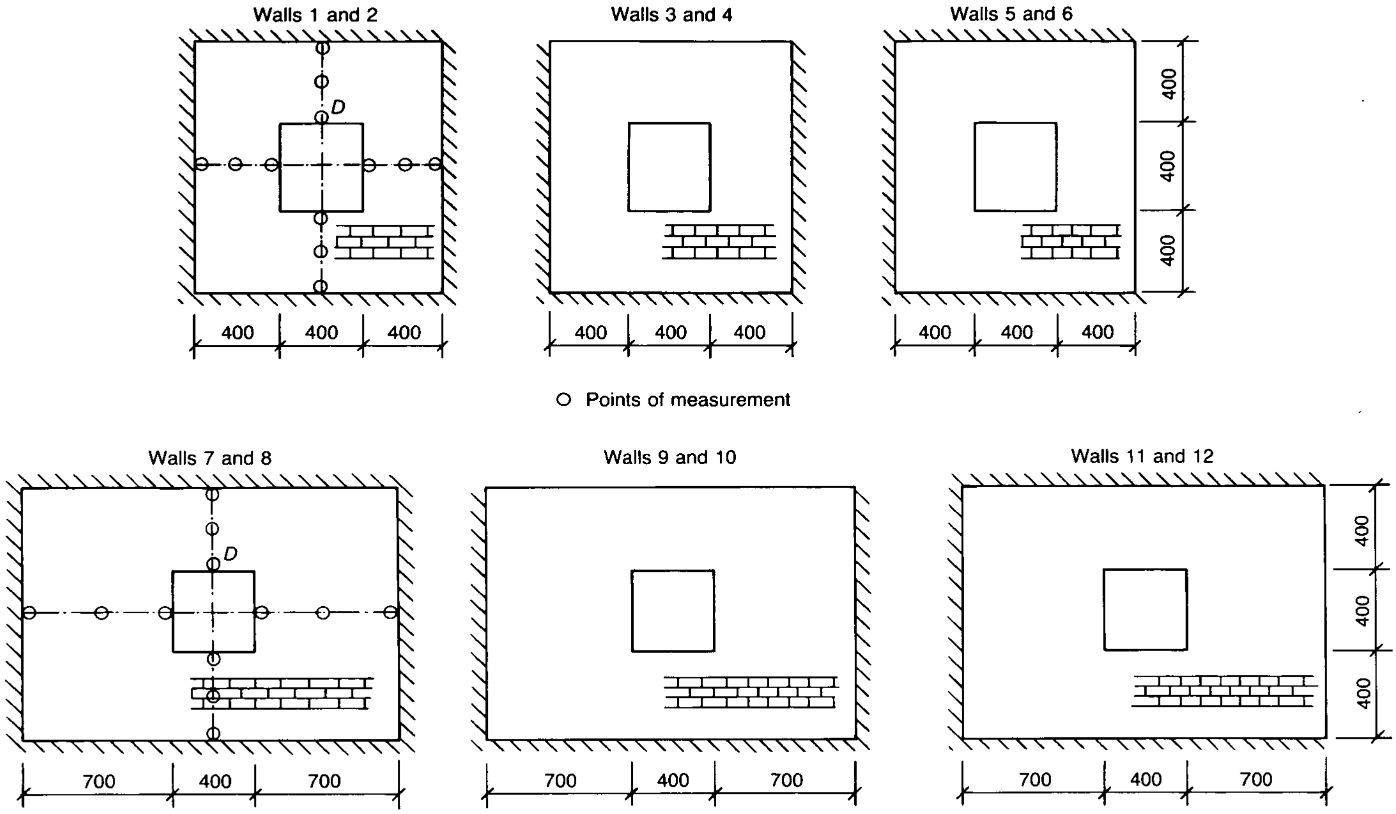


Fig. 1. Wall configurations (all dimensions in mm)

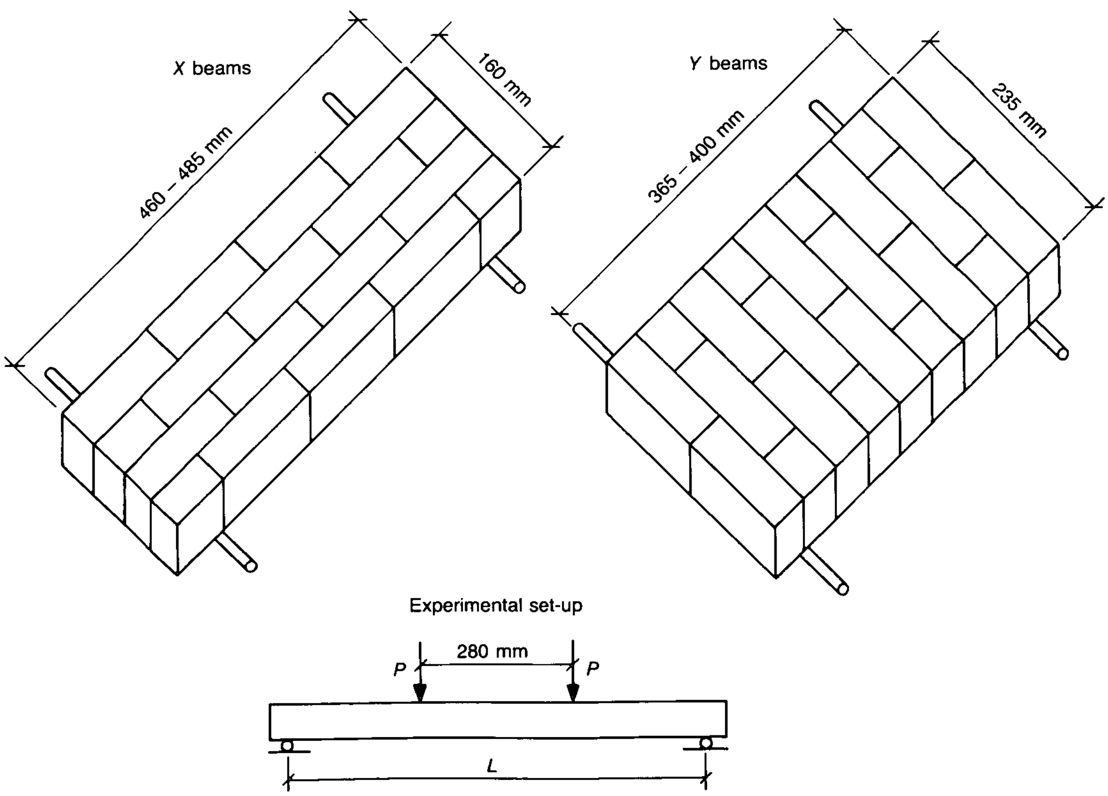


Fig. 2. Beam configuration and experimental set-up

ettes in compression. The compressive strain was measured by using the vibrating wire gauges. The values of moduli of elasticity were also obtained in bending (Fig. 2) and compared with those obtained in compression, and no significant difference was recorded. The average values of the tangent moduli of elasticity and the Poisson's ratio were

$E_x = 17\,750$ N/mm^2, $\upsilon_{yx} = 0{\cdot}11$

$E_y = 13\,500$ N/mm^2 and $\upsilon_{xy} = 0{\cdot}15$

These values have been used for the elastic analysis.

Theoretical analysis

6. A standard computer program was used for the elastic analysis. The yield-line analysis was carried out for each of the walls. The work method[5] has been used for all test cases dealt with in this Paper. The idealized yield-line pattern, giving the lowest failure pressure, is shown in Figs 3–5 for each case.

Walls with four edges simply supported containing a central opening

7. If a virtual deflection of unity is given to the four corners, cdef, while the panel in Fig. 3 is collapsing, the external work done by the uniformly distributed and line loads is given by

$$\frac{q\alpha L^2}{3}(3\alpha + 3\beta - 8\beta\lambda) + (1 - 2\beta - 2\lambda l + 4\beta\lambda)$$

$$= \frac{q\alpha L^2}{3}(3 - 3\beta - 3\lambda + 4\beta\lambda) \quad (1)$$

The internal dissipation of energy on yield-lines ac, bd, fh and ge is equal to $\Sigma(mL_x\theta_x + \mu mL_y\theta_y)$, where

$\theta_x = 1/\lambda\alpha L$ and $\theta_y = 1/\beta L$

Hence, the internal dissipation of energy is equal to

$$4m\alpha\left(\frac{\beta}{\alpha^2\lambda} + \frac{\mu\lambda}{\beta}\right) \quad (2)$$

Equating the external and internal work done gives

$$q = \frac{12m\left(\frac{\mu\lambda}{\beta} + \frac{\beta}{\alpha^2\lambda}\right)}{L^2(3 - 3\beta + 4\beta\alpha - 3\lambda)} \quad (3)$$

Walls with upper edge free and three other edges simply supported

8. The solution is obtained by giving a virtual deflection of unity to the vertical yield-line joining points b and d in Fig. 4. Equating the dissipation of internal energy and the external work done gives

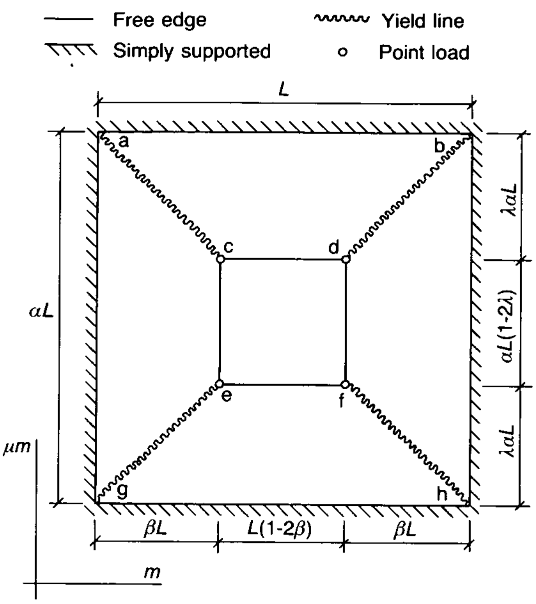


Fig. 3. Failure mechanism for walls with four edges simply supported

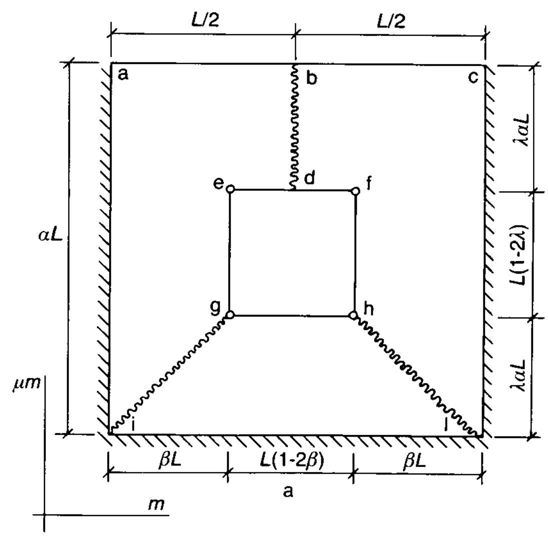


Fig. 4. Failure mechanism for walls with top edge free

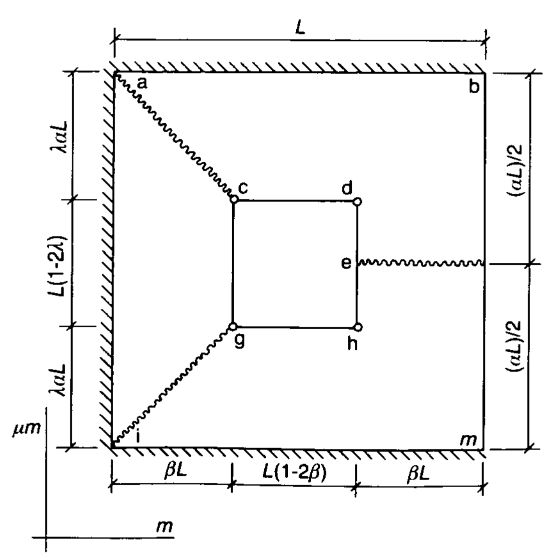


Fig. 5. Failure mechanism for walls with one lateral edge free

$$q = \frac{24m\left(\frac{\beta^2}{\lambda\alpha^2} + 2\mu\lambda^2\right)}{L^2(3\lambda + 20\beta^2\lambda - 18\beta\lambda + 12\beta - 12\beta^2)} \qquad (4)$$

Walls with one vertical edge free and three other edges simply supported

9. A similar solution is obtained by giving a virtual deflection of unity to the horizontal line between points e and f in Fig. 5. The predicted failure pressure is given by

$$q = \frac{24m\left(\frac{2\beta}{\alpha^2} + \frac{\mu\lambda^2}{\beta}\right)}{L^2(3\beta + 20\beta\lambda^2 - 18\beta\lambda + 12\lambda - 12\lambda^2)} \qquad (5)$$

The theoretical failure pressures obtained from equations (3)–(5) are shown in Table 1.

Discussion of results

10. The results are given in Tables 1, 2 and 3. From Tables 2 and 3, it is very clear that there is practically no difference between the flexural tensile strengths in two directions obtained from the wallettes extracted from the test walls after failure or built separately during its construction. This is also confirmed by statistical analysis of the results. There is a great deal of variation within the results, which has been reported in the literature[6] and which, as found in this test, is in no way unusual.

11. Some typical load–deflection relationships of panels 2 and 8 with the aspect ratios of 1 : 1 and 1 : 1·5 are shown in Fig. 6. The elastic analysis underestimates the deflections of the uncracked panels even at a very low pressure. At both low and failure pressures, the deflections of the panels at various points along the horizontal and vertical centre-lines (Fig. 6) are different compared with the predicted values

Table 1. Test results of walls

Test walls	Experimental pressure: kN/m²			Theoretical pressure: kN/m²		
	Cracking	Failure	Average failure	Elastic	Yield-line	BS 5628
1	5·0	7·9				
2	5·2	10·2	9·1	8·7	8·9	4·5
3	—	7·8				
4	—	7·2	7·5	5·2	6·1	3·1
5	6·8	7·4				
6	3·2	6·8	7·1	2·3	4·1	2·3
7	2·6	5·4				
8	4·0	6·4	5·9	2·6	5·7	2·9
9	—	3·1				
10	—	3·9	3·5	3·3	3·3	1·7
11	1·8	2·6				
12	2·2	2·6	2·6	2·1	3·3	1·7

Table 2. Wallettes built alongside test walls—flexural tensile strength

Wallettes	f_{tx}: N/mm²	f_{ty}: N/mm²
1	1·91	0·74
2	2·48	0·96
3	2·68	0·86
4	2·21	0·82
5	1·79	0·81
6	2·08	0·70
7	1·32	0·66
8	1·89	0·74
9	2·40	0·52
Mean	2·08	0·76
Standard deviation	0·41	0·13

Table 3. Wallettes extracted from the test walls—flexural tensile strength

Wallettes	f_{tx}: N/mm²	f_{ty}: N/mm²
1	2·10	0·83
2	2·41	0·61
3	2·35	0·64
4	1·59	0·60
5	2·24	0·54
6	2·07	0·50
7	1·83	0·81
8	1·23	0·47
9	1·77	0·48
10	1·83	0·89
11	1·95	0·59
12	1·76	0·89
13	2·02	0·89
14	1·90	0·86
15	2·30	0·57
16	2·20	0·55
17	1·89	0·67
18	2·15	0·65
19	2·61	0·89
20	2·51	0·78
21	2·12	0·77
22	2·55	0·82
23	2·21	1·07
24	2·33	1·10
25	1·85	1·01
26	1·48	1·62
27	2·18	1·53
28	3·11	0·62
29	1·61	1·07
30	2·24	1·42
31	2·01	—
32	2·08	—
33	2·48	—
34	2·15	—
35	2·97	—
36	2·46	—
37	2·33	—
38	2·19	—
39	2·00	—
40	2·47	—
41	2·27	—
42	1·81	—
43	2·26	—
Mean	2·14	0·82
Standard deviation	0·36	0·30

obtained by the elastic analysis. The analysis of the experimental deflections at 400 mm from the support in the symmetrical panel of the aspect ratio 1 : 1 suggests that the load distributes according to the flexural stiffness (i.e. stiffness orthotropy). In the conventional yield-line analysis as applied to reinforced concrete slabs, the question of the elastic orthotropy does not arise. Strictly speaking, ignoring this in obtaining the failure pressure of brickwork panels by the yield-line method may not be justified, as it violates the equilibrium condition and may explain the difference between the theoretical and experimental results. Fig. 7 shows the typical load–deflection relationship of point D (Fig. 1) which is non-linear for the test panels.

12. Before failure, initial cracks (Table 1) were noticed in the walls simply supported on four sides and in those simply supported on three sides with the vertical edge free. Walls 3, 4, 9 and 10 (Table 1), with three sides simply supported and the top edge free, did not show sign of cracking: they tended to behave like a strip spanning horizontally at the top, and the failure happened immediately after the development of vertical cracks at ultimate failure pressure.

13. The elastic analysis underestimates the failure pressure of the walls tested in this investigation. It also fails to predict the cracking pressure (Table 1). In the elastic method, it is assumed that the failure happens as soon as flexural strength in any one direction is reached. Thus no redistribution of moments can take place and, therefore, the strength orthotropy is neglected.

14. Although the typical crack patterns of the tested walls (Figs 8 and 9) were different and deviated from the idealized yield-lines, the correlation between theoretical predicted and experimental failure pressure (Table 1) was much better.

15. The experimental failure pressures for panels of aspect ratio 1 : 1 with three sides simply supported and with the vertical or top edge free were similar. This could be possible only if the strengths in the vertical and horizontal directions were the same, i.e. the panels exhibited strength isotropy. This is contrary to those failure pressures predicted theoretically by the yield-line method.

16. Figure 10 shows the relationship between the experimental failure pressure and those failure pressures predicted by both the yield-line method and the line of equality. In an ideal situation, all test results should lie on the line of equality. In this investigation, five test results of walls lie under this. However, in the case of the mean test results, all except one will be above the line of equality, which suggests that it is safer to use the yield-line method with all the shortcomings mentioned earlier for the

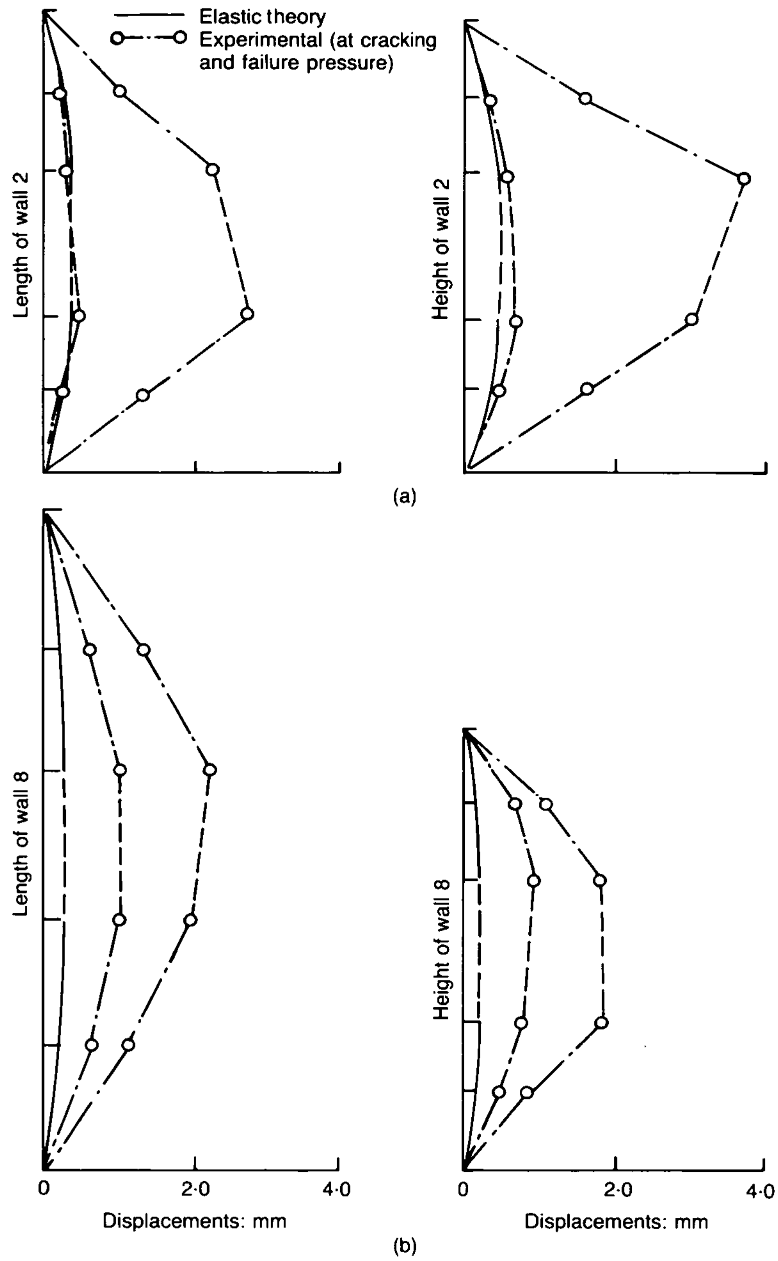


Fig. 6 (above). Load–deflection relationship of: (a) wall 2 (aspect ratio 1 : 1); (b) wall 8 (aspect ratio 1 : 5)

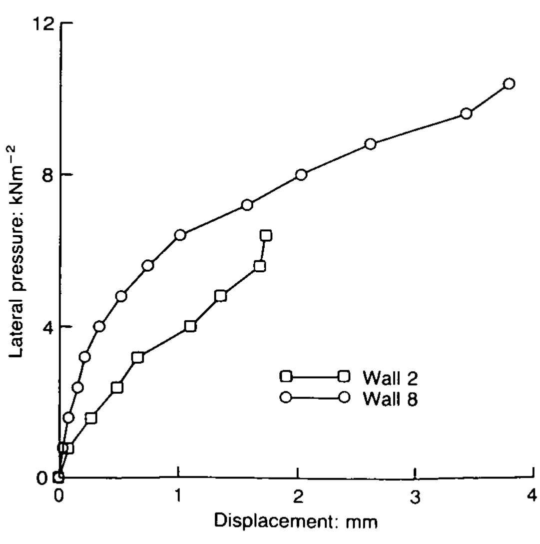
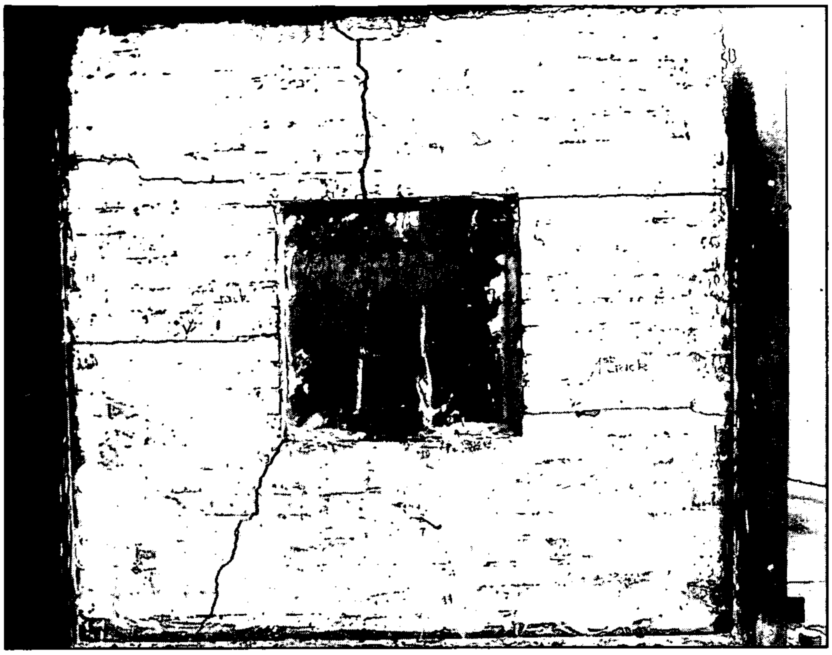


Fig. 7 (left). Load–displacement relationship of point D on walls 2 and 8 (see Fig. 1)



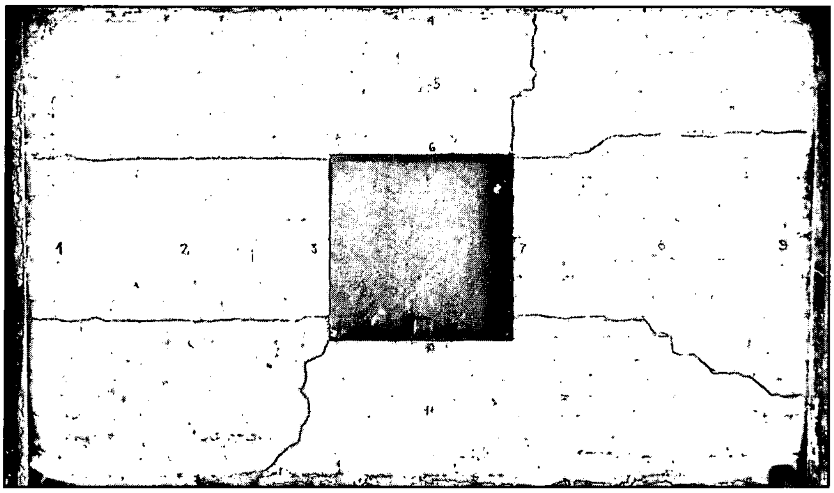


Fig. 8 (*top*). *Cracks after ultimate failure of wall 1 (simply supported on four sides and with aspect ratio of 1 : 1)*

Fig. 9 (*above*). *Cracks after ultimate failure of wall 8 (simply supported on four sides and with aspect ratio of 1 : 5)*

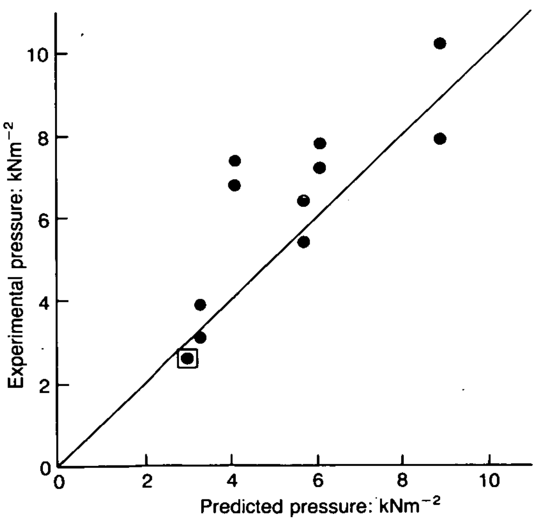


Fig. 10 (*right*). *Comparison between experimental and predicted (yield-line) failure pressure for laterally loaded walls*

design of brickwork panels, with openings subjected to lateral pressure.

17. Normally, the designer will use the published values of flexural strengths recommended by BS 5628[1] instead of test values; hence in Table 1, a comparison is made using the prescribed values of the characteristic flexural tensile strengths and the wall's test results. Because the ultimate failure pressures are being compared with the code,[1] the material partial safety factor has been assumed as one. The predicted failure pressures were many times lower than the experimental results. According to BS 5628,[1] the characteristic flexural strengths depend on water absorption; hence for these walls the allowable values are 0·4 N/mm^2 and 1·2 N/mm^2, which cause this underestimation of the pressure. The provision in the code of decreasing flexural strengths with increasing water absorption of bricks[7] seems obscure and may need revision in future as more data become available.

Conclusions

18. The flexural tensile strengths normal and perpendicular to the bed-joint obtained from the wallettes built independently or extracted from the undamaged portion of the tested walls are similar.

19. Compared with the elastic method, the yield-line method with all its limitations offers a better solution for predicting the lateral strength of brickwork panels with openings, and hence can be used with some confidence for the design of panels.

Acknowledgements

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A simplified ultimate load analysis of laterally loaded model orthotropic brickwork panels of low tensile strength

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Synopsis

Since many materials have low tensile strength and different strength and stiffness properties in two orthogonal directions, the analysis of structures made from them becomes more difficult. This paper presents a simplified approach for the ultimate load analysis of such orthotropic panels subjected to lateral loading. Initially, the simplified approach was developed for brickwork panels that not only have low tensile strength but exhibit different strength and stiffness orthotropies. At failure, these panels develop fracture lines very similar to the yieldlines in ductile plates, so that there has been a great temptation to apply yieldline analysis[1,3–6], which is really inappropriate and does not fully explain test results. This is understandable, since it is difficult to imagine fully plastic behaviour in a brittle material like brickwork. Invariably, yieldline analysis[7] overestimates the failure pressure[6,9]. A simplified method based on fracture lines is proposed, which could very well be applied to any brittle material having both strength and stiffness orthotropies.

Notation

/////////////	is the simple support
xxxxxxxxxxx	is the continuous support
———————	is the positive fracture line
– – – – – –	is the negative fracture line
m	is the ultimate moment/unit length along the bed joint
μm	is the ultimate moment/unit length normal to bed joint
$k=\frac{E_x}{E_y}$	is the ratio of modulus of elasticity in two directions
L	is the length
α	is the height/length ratio $\left(\frac{H}{L}\right)$
ω	is the failure pressure
β	is a factor
ν_x, ν_y	are Poisson's ratios

Fracture line analysis

Assumptions

All deformations take place along the fracture lines only, and the individual parts of the slab rotate as rigid bodies. The load distribution is in accordance with the stiffnesses in the respective directions. The fracture lines develop only when the relevant strengths are reached in the two directions.

Consider the idealised fracture lines for a four-sided panel with two simply supported and two continuous edges (see Fig 1).

Every portion of the panel into which it is divided by the fractures lines is in equilibrium under the action of external forces and reactions along the fracture lines and supports. Since it is symmetrical, only parts 1 and 2 need consideration:

The load on AFB$^{(1)}=\frac{1}{2}\omega\beta\alpha L^2$ and its moment along

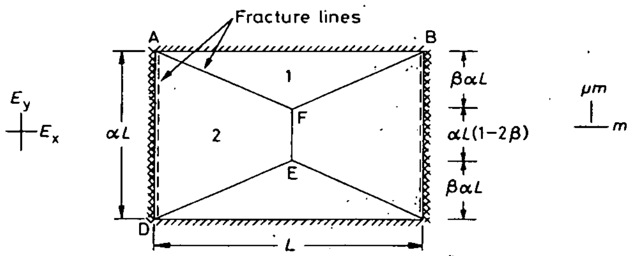


Fig 1. Idealised fracture lines

$$\text{AB} = \tfrac{1}{2}\omega\beta\alpha L^2 \times \frac{\beta\alpha L}{3}$$

$$= \frac{\omega\beta^2\alpha^2L^3}{6} \qquad \ldots(1)$$

For equilibrium

$$\frac{\omega\beta^2\alpha^2L^3}{6} = mL$$

$$\therefore \quad \frac{\omega\beta\alpha^2L^2}{6} = \frac{m}{\beta} \qquad \ldots(2)$$

Similarly, for AFED$^{(2)}$

$$\frac{\omega\beta L^2}{12} + \frac{\omega L^2}{8} - \frac{\omega\beta L^2}{4} = \frac{2\mu m}{k}$$

where $$k = \frac{E_x}{E_y}$$

or $$\frac{\omega L^2}{12}[\beta + 1{\cdot}5 - 3\beta] = \frac{2\,\mu m}{k}$$

or $$\frac{\omega\alpha^2L^2}{6}[1{\cdot}5 - 2\beta] = \frac{4\,\mu m\alpha^2}{k} \qquad \ldots(3)$$

From (2) and (3)

$$\frac{\omega\alpha^2L^2}{6}[1{\cdot}5 - 2\beta + \beta] = \frac{m}{\beta} + \frac{4\,\mu m\alpha^2}{k}$$

or

$$\frac{\omega\alpha^2L^2}{6}[1{\cdot}5 - \beta] = \frac{m}{\beta}\left[1 + \frac{4\mu\beta\alpha^2}{k}\right]$$

or

$$\frac{\omega\alpha^2L^2}{6}[1{\cdot}5\beta - \beta^2] = m\left[1 + \frac{4\mu\beta\alpha^2}{k}\right]$$

$$\therefore \quad m = \frac{\omega\alpha^2L^2}{6}\left[\frac{1{\cdot}5\beta - \beta^2}{1 + \frac{4\mu\beta\alpha^2}{k}}\right] \qquad \ldots(4)$$

For minimum collapse load or maximum value of moment m

$$\frac{d\left(\dfrac{m}{\omega}\right)}{d\beta}=0$$

From which,

$$\beta=\frac{k}{4\mu\alpha^2}\left[\sqrt{\frac{6\mu\alpha^2}{k}+1}-1\right] \qquad \ldots(5)$$

For a particular panel, the fracture pattern that gives the lowest collapse load should be taken as failure load. The values of m and β for panels of different boundary conditions are given in appendix A and were derived for comparison with the results of other work[8,9].

Experimental investigation

To check the validity of the analysis, experiments were carried out on 1/3 scale model brickwork panels and associated 1/3 control specimens as described below. The control specimens were used to determine the material properties and the elastic constants. The traditional method of brick laying was used to construct all model test panels and wallettes.

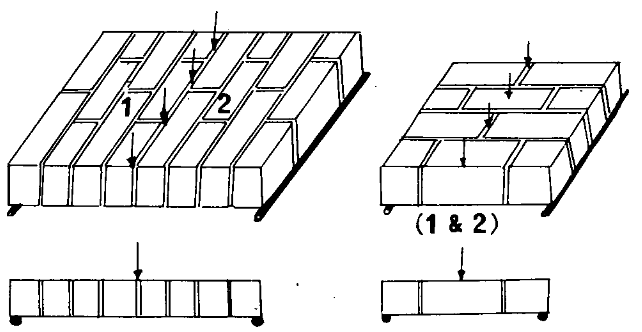


Fig 2. Test arrangement for determination of flexural strengths in two directions

Determination of strength orthotropy

Eight-course high, two-brick long wallettes were tested under central line loading as shown in Fig 2. Instead of the commonly used system of two-line loading, central-line loading was adopted for flexural testing to keep shear arm/depth ratios constant in both directions, and of such values (approx. 2) that its effect on the tensile strengths is negligible. Examination of published results[11] suggests that single- or double-point loading has no significant effect on the flexural strengths of brickwork. The wallettes were first tested for the determination of flexural strength normal to the bedding plane. Each of the resulting halves from the test specimens were further tested to obtain the tensile strength parallel to the bed joint. Thus the strength orthotropy was obtained from exactly identical specimens. This not only allowed a reduction in the number of test specimens, but also made it possible to obtain a realistic relationship between tensile strength normal and parallel to the bedding planes, as all the factors that affect the tensile strengths of brickwork remain the same. The result of the test is given in Table 2. The test on wallettes was carried out on the same day as that on the corresponding wall. Two wallettes with completely unfilled perpendicular joints were also made and tested as above. There was a 44% drop in the strengths, but the orthotropic ratio remained similar to the wallettes with properly filled vertical joints.

Determination of modulus of elasticity in two orthogonal directions

The tensile strength of brickwork normal to bedding plane is very low: it would therefore be difficult to obtain stress–strain curves and thus modulus of elasticity in bending. It was therefore decided to test wallettes similar to those described above under uniform axial compression in either direction to obtain the moduli of elasticity of the brickwork normal and parallel to the bedding plane. However, a few specimens were tested in flexure to determine modulus of elasticity parallel to the bed joint, and the value obtained was similar to the values obtained in axial compression. The strain was measured by means of 63·5 mm vibrating wire gauges for the test in both directions. The results of the tests are given in Table 1. The average orthotropic ratio was 1·4.

Test arrangement for brickwork panels

The brickwork panels were simply supported along their top and bottom to eliminate any unknown end restraint. Two other edges were bonded to two-brick-long short return walls, which were post-tensioned. Post-tensioning of the short return was necessary to avoid their premature failure. Under this arrangement, the maximum restraining moment at the return will be determined by the brickwork flexural strength in the plane parallel to the bedjoint, which was established by the test on wallettes made at the same time as corresponding wall panels.

The lateral load was applied to the test wall by an air-bag sandwiched between the wall panel and a rigid steel reacting frame. The pressure was applied at the rate of 0·5 kN/m^2/min approx. until failure by pumping air into the bag by means of an air compressor, the pressure being measured by water manometer.

Results and conclusions

The results of the tests on panels are summarised in Table 3. There is very good agreement between the theoretical and the experimental results. For interest, the failure pressures based on yieldline analysis[9] are also given in Table 3, and it is clear that the yieldline method, as such, should not be used for the design of brickwork panels.

The theory proposed in this paper was checked against other experimental results for which all the necessary data were available[8,9]. The result of the analysis is compared in Figs 3 and 4 with the average test results of 1/3 and 1/6 scale brickwork, and good agreement is found between them. Unfortunately, results of other work published in *The Structural Engineer*[6,10] could not be compared owing to lack of data, such as moduli of

TABLE 1—Relationship between the moduli of elasticity and Poisson's ratios

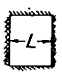
No. of specimens	Corresponding wall	E_x (N/mm^2) Parallel to bedjoint	E_y (N/mm^2) Perpendicular to bedjoint	ν_x	ν_y
1	A1	11 100	7 900	0·16	0·11
2	A2	13 500	9 000	0·13	0·09
3	B1	10 700	7 100	0·15	0·10
4	B2	10 100	8 400	0·14	0·12
5	C1	12 100	8 600	0·17	0·12
6	C2	12 900	9 900	0·17	0·13
Mean		11 733	8 483	0·15	0·11
SD		1323	954	0·016	0·0147
Coeff. of variation (in %)		11·3	11·2	10·9	13·0

TABLE 2—Relationship between flexural strengths in two orthogonal directions

Corresponding test wall	Flexural strength(f_{ty}) perpendicular to bedjoint (a) N/mm²	Flexural strength (f_{tx}) parallel to bedjoint (b) N/mm²	Ratio b/a (μ)	Remarks
A1	0·51*	1·62†	3·18	
A2	0·53*	1·62†	3·05	
B1	0·43*	1·13†	2·63	
B2	0·63*	1·34†	2·13	
C1	0·56*	1·90†	3·39	
C2	0·88*	2·02†	2·29	
	0·33*	0·89*	2·70	Completely unfilled perpend joints

* Average of two results
† Average of four results

TABLE 3—Failure pressure for wall panels four sides supported



$H = 796$ mm

Walls	Aspect ratio L/H	α	Ultimate pressure N/mm² x 10^{-3} Experimental	Yield-line	Proposed theory
A1	1·0	1·0	10·7	13·9	10·78
A2	1·1*	0·898	11·6	15·1	11·40
B1	1·5	0·674	4·7	5·8	4·60
B2	1·5	0·674	6·0	7·4	5·96
C1	2·0	0·505	5·2	6·2	5·16
C2	2·0	0·505	6·7	7·8	6·60

* $H = 708$ mm.

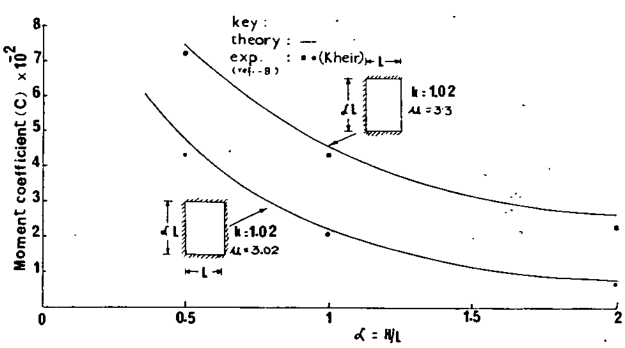


Fig 3. Comparison between experimental and theoretical moment coefficients

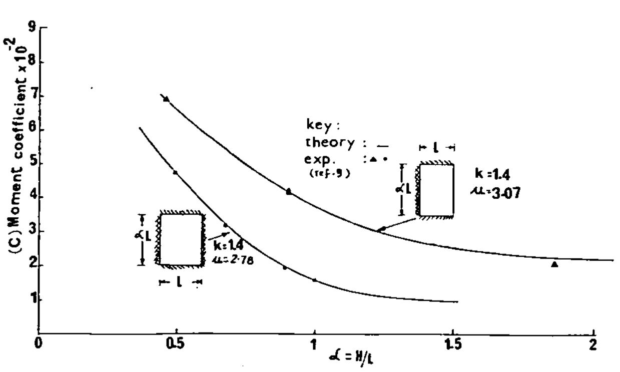


Fig 4. Comparison between experimental and theoretical moment coefficients

elasticity, corresponding strengths of wallettes for particular walls, and possible boundary restraint, and this precludes any valid comparisons.

Since there is very good agreement between the predicted lateral pressure and the experimental results of panels having different boundary conditions, the design of brickwork panels with similar stiffness orthotropy may be carried out on the basis of the bending moment coefficients given in Figs 2 and 3, with associated flexural strength normal to the bedding plane. If the stiffness orthotropy is quite different, the formulae given in appendix A and in the section on fracture line analysis may be used to calculate the coefficients by substituting the relevant values.

Acknowleldgements

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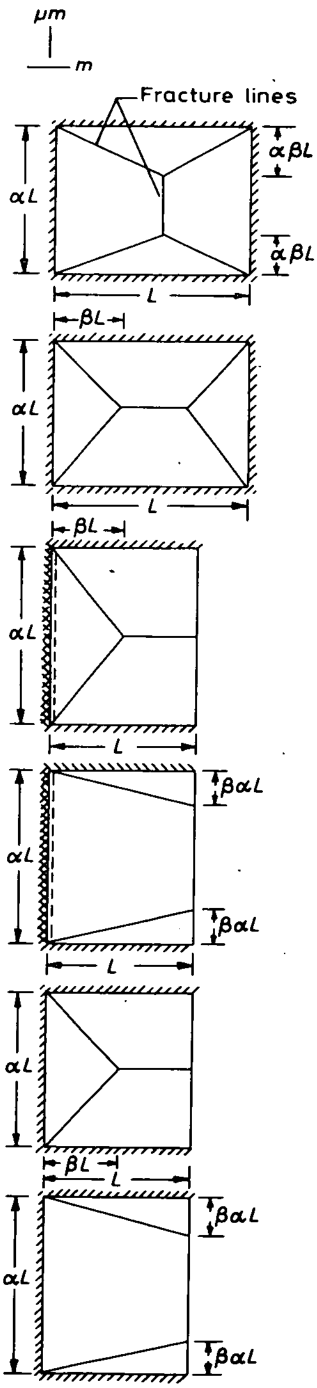
Appendix A



$$m = \frac{\omega\alpha^2 L^2}{6}\left[\frac{1\cdot5\beta - \beta^2}{1 + \frac{2\mu\alpha^2\beta}{k}}\right];$$

$$\beta = \frac{k}{2\mu\alpha^2}\left[\sqrt{\frac{3\mu\alpha^2}{k} + 1} - 1\right]$$

$$m = \frac{\omega\alpha^2 L^2}{6}\left[\frac{1\cdot5\beta - \beta^2}{2\beta + \frac{\mu\alpha^2}{k}}\right];$$

$$\beta = \frac{\mu\alpha^2}{2k}\left[\sqrt{\frac{3k}{\mu\alpha^2} + 1} - 1\right]$$

$$m = \frac{\omega\alpha^2 L^2}{12}\left[\frac{3\beta - \beta^2}{2\beta + \frac{\mu\alpha}{k}}\right];$$

$$\beta = \frac{\mu\alpha^2}{2k}\left[\sqrt{\frac{6k}{\mu\alpha^2} + 1} - 1\right]$$

$$m = \frac{\omega\alpha^2 L^2}{12}\left[\frac{3\beta - 2\beta^2}{1 + \frac{\mu\beta^2\alpha^2}{k} + \frac{0\cdot5\mu\alpha^2\beta}{k}}\right];$$

$$\beta = \frac{k}{2\mu\alpha^2}\left[\sqrt{\frac{3\mu\alpha^2}{k} + 1} - 1\right]$$

$$m = \frac{\omega\alpha^2 L^2}{6}\left[\frac{3\beta - \beta^2}{4\beta + \frac{\mu\alpha^2}{k}}\right];$$

$$\beta = \frac{\mu\alpha^2}{4k}\left[\sqrt{\frac{12k}{\mu\alpha^2} + 1} - 1\right];$$

$$m = \frac{\omega\alpha^2 L^2}{12}\left[\frac{3\beta - 2\beta^2}{1 + \frac{\mu\beta^2\alpha^2}{k}}\right];$$

$$\beta = \frac{k}{1\cdot5\mu\alpha^2}\left[\sqrt{\frac{2\cdot25\mu\alpha^2}{k} + 1} - 1\right]$$

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Collapse behaviour of rectangular steel box girders

continued from page 80

ing the validity of a 2D treatment for stiffened box-girder diaphragms[5]. Such a procedure has significant computational advantages, particularly when related to parametric studies.

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An ultimate load analysis of laterally loaded brickwork panels

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The paper presents a simplified approach for the ultimate load analysis of brickwork panels of various shapes subjected to lateral loading. The panels considered were octagonal, triangular and rectangular with hole. The results obtained from the theoretical analysis taking into account the strength and stiffness orthotropies compare favourably with the experimental results.

NOTATION

m	=	Ultimate moment/unit length along the bed joint
$\frac{\mu}{k}$ m	=	Modified ultimate moment/unit length normal to bed joint.
$k = \frac{E_x}{E_y}$	=	Ratio of modulus of elasticity in two orthogonal directions.
L	=	Length
$\lambda,\gamma,\alpha,\beta$	=	Factors
w	=	Failure pressure

Introduction

The new British limit state Code BS 5628 [1] recommends bending moment coefficients for the design of laterally loaded rectangular panels. The Code does not indicate the origin of these coefficients, but it is understood that some of them are mainly based on test results by West, Haseltine *et al* [5,6] who mentioned their analogy to coefficients derived by yield line analysis, as applied to reinforced concrete slabs. It is clear, however, that yield line analysis is not applicable to a brittle material and comparison between results derived from yield line formula and test results show that the yield line method consistently overestimates the failure pressure [4,5,6,7,9] when the orthogonal ratio for brickwork is interpreted as the strength ratio. The reinforced concrete

Code CP 110 [3], recommends the use of design bending moment coefficients based on elastic analysis and since brickwork panels exhibit different strength and stiffness properties in two orthogonal directions, the author has suggested a fracture line approach [8] taking into account both these orthotropies. This method has been applied successfully to predict the failure pressure of rectangular panels with the various boundary conditions. Before the method can be used universally for the design of laterally loaded panels, the validity of the method needs to be established for other shapes and types of panels. With a view to establishing this validity, the method was used to predict the failure pressures of octagonal, triangular and rectangular panels with an opening and, the theoretical results are compared with the experimental results obtained from testing them to failure.

Fracture line analysis

The assumptions made in the earlier paper [8] for the analysis hold good for the cases dealt with here.

Rectangular panel with opening

Consider the idealised fracture lines for a four sided panel with openings having two simply supported and two continuous edges as shown in Figure 1.

Every portion of the panel into which it is divided by the fracture lines is in equilibrium under the action of external forces and reactions along the fracture lines and supports.

Assume the deflection of line EF as unity at the time of failure. Consider the rigid regions 1, 3 and 2 and 4 as divided by the fracture lines.

External work done by the distributed load

$$= 4.\frac{1}{2}w\lambda H\frac{\alpha H}{2}\frac{1}{3} + 4w\frac{\alpha H}{2}\frac{H}{2}[1-2\lambda-\gamma]\frac{1}{2}$$
$$+ 2w\gamma H[\frac{\alpha H}{2} - \frac{\beta\alpha H}{2}][\frac{1-\beta}{2}] + \frac{1}{2}2w\alpha H\,\lambda H\frac{\lambda H}{3}$$

therefore external work

$$= \frac{wH^2}{6}[4\lambda\alpha + 3\alpha - 6\lambda\alpha - 6\alpha\gamma\beta + 3\gamma\alpha\beta^2]$$
$$= \frac{w\alpha H^2}{6}[3 - 2\lambda - 6\gamma\beta + 3\gamma\beta^2] \qquad (1)$$

internal work done

$$= 2\frac{\mu}{k}.m.\frac{2H}{\alpha H} + \frac{2\mu m}{k}(H-\gamma H)\frac{2}{\alpha H} + \frac{2.m\alpha H}{\lambda H}$$
$$= 2m[\frac{4\mu}{\alpha k} - \frac{2\mu\gamma}{\alpha k} + \frac{\alpha}{\lambda}] \qquad (2)$$

For equilibrium

$$2m[\frac{4\mu}{\alpha k} - \frac{2\mu\gamma}{\alpha k} + \frac{\alpha}{\lambda}] = \frac{wH^2\alpha}{6}[3 - 2\lambda - 6\gamma\beta + 3\gamma\beta^2] \qquad (3)$$

$$\text{or } \frac{m}{w} = \frac{H^2\alpha}{12}\left[\frac{3\lambda - 2\lambda^2 - 6\gamma\beta\lambda + 3\gamma\beta^2\lambda}{\frac{2\mu}{\alpha k}(2-\gamma)\lambda + \alpha}\right] \qquad (3)$$

Differentiating with respect to λ to get minimum value of $\frac{m}{w}$, we get $\frac{d\frac{m}{w}}{d\lambda} = 0$, therefore

$$[\frac{2\mu}{\alpha k}(2-\gamma)\lambda+\alpha][3 - 4\lambda - 6\gamma\beta + 3\gamma\beta^2]$$
$$-\frac{2\mu}{\alpha k}(2-\gamma).[3\lambda - 2\lambda^2 - 6\lambda\gamma\beta + 3\lambda\gamma\beta^2] = 0$$

$$\text{or } 3\alpha - 4\alpha\lambda - 6\alpha\gamma\beta + 3\alpha\beta^2\gamma - 2\lambda^2.\frac{2\mu}{\alpha k}(2-\gamma) = 0 \qquad (4)$$

$$\text{or } \frac{\lambda^2\mu}{k}[2-\gamma] + \alpha^2\lambda - \frac{3}{4}\alpha^2[1 - 2\gamma\beta + \gamma\beta^2] = 0$$

$$\text{therefore } \lambda = \frac{-\alpha^2 \pm \sqrt{\alpha^4 + 3\alpha^2\frac{\mu}{k}(1-2\gamma\beta+\gamma\beta^2)(2-\gamma)}}{\frac{2\mu}{k}(2-\gamma)} \qquad (5)$$

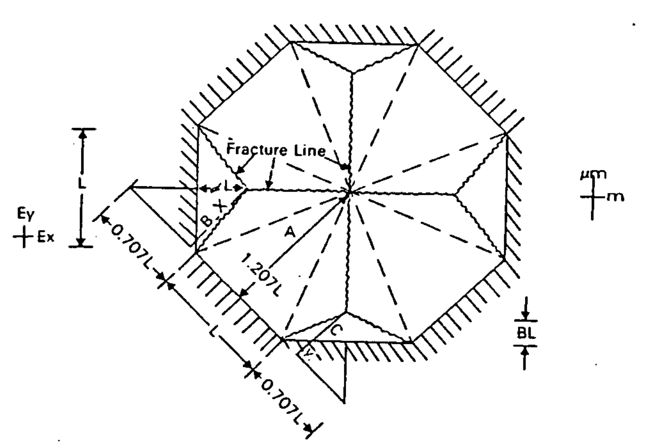


Figure 2. Generalised failure pattern.

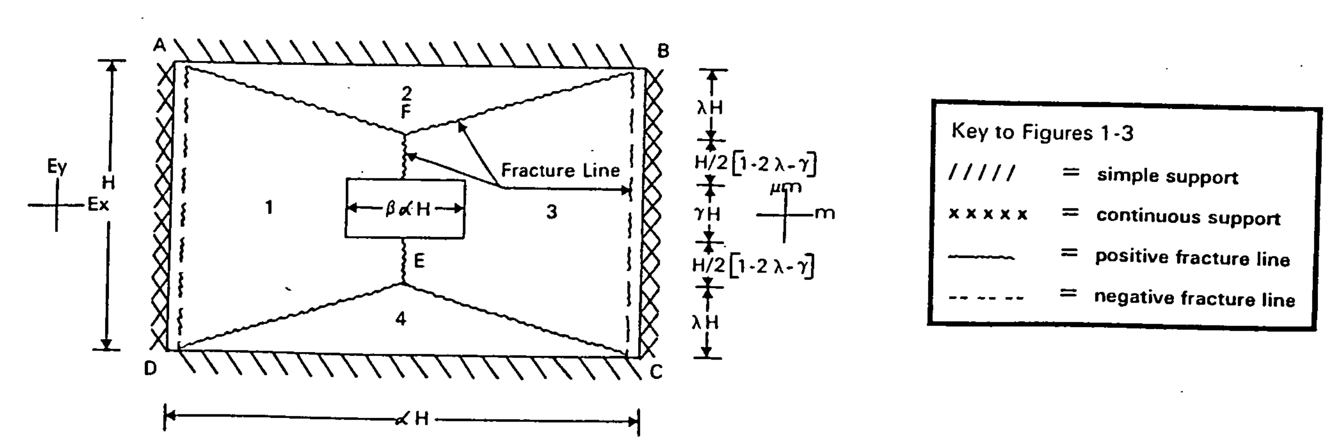


Figure 1. Idealised failure pattern.

considering + ve root of the equation:

$$\lambda = \frac{-\alpha^2 + \alpha\sqrt{\alpha^2 + 3\frac{\mu}{k}(1-2\gamma\beta+\gamma\beta^2)(2-\gamma)}}{2\frac{\mu}{k}(2-\gamma)} \qquad (6)$$

The value of λ from equation (6) can be substituted in equation (3) to obtain the failure pressure.

Octagonal panel
Due to the symmetry, only a quarter of the panel needs to be considered. For part A of the panel, equating the external and internal work gives:

$$\frac{m}{2}\left(1+\frac{\mu}{k}\right)L = \frac{1}{2}w\,2.414L\,1.207L\,\frac{1.207L}{3} - \frac{1}{2}\frac{w}{3}0.707L\,(x^2+y^2)$$

where $x = \frac{L(1+2\alpha)}{2\sqrt{2}}$ and $y = \frac{L(1+2\beta)}{2\sqrt{2}}$

therefore $m\left(1+\frac{\mu}{k}\right) = \frac{wL^2}{1+\frac{\mu}{k}}[1.1134 - 0.1178(\beta+\alpha) - 0.1178(\beta^2+\alpha^2)]$ (7)

similarly for part B and C of panel

$$\left(m+\frac{\mu}{k}m\right)\frac{L}{2} = \frac{1}{2}w\frac{L}{2}\frac{\beta^2L^2}{3} + \frac{1}{2}w\frac{L}{2}\frac{\alpha^2L^2}{3} \qquad (8)$$

From equations (7) and (8)

$$2m = \frac{wL^2}{\left(1+\frac{\mu}{k}\right)}[1.1134 - 0.1178(\beta+\alpha) + 0.0488(\beta^2+\alpha^2)] \qquad (9)$$

Further considering the equilibrium of B & C, we get

$$\frac{\mu}{k}\frac{mL}{mL} = \frac{w\alpha^2L^2}{6}\times\frac{6}{w\beta^2L^2} \qquad (10)$$

or $\alpha = \beta\sqrt{\frac{\mu}{k}}$

substituting the value of α in equation (9)

$$m = \frac{wL^2}{2\left(1+\frac{\mu}{k}\right)}\left[1.1134 - 0.1178\beta\left(1+\sqrt{\frac{\mu}{k}}\right) + 0.0488\beta^2\left(1+\frac{\mu}{k}\right)\right] \qquad (11)$$

$\frac{dm/w}{d\beta} = 0$ therefore $0.0976\beta\left(1+\frac{\mu}{k}\right) - 0.1178\left(1+\sqrt{\frac{\mu}{k}}\right) = 0$

therefore $\beta = 1.207\frac{\left[1+\sqrt{\frac{\mu}{k}}\right]}{1+\frac{\mu}{k}}$ (12)

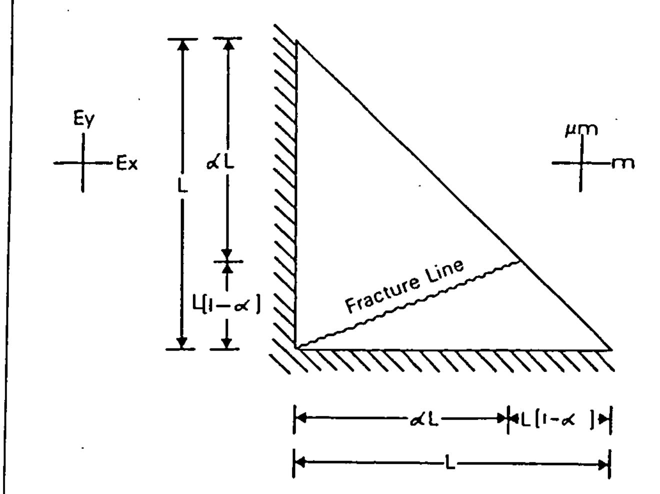


Figure 3. Idealised failure pattern.

From equation (10)

$$\alpha = 1.207\frac{\sqrt{\frac{\mu}{k}}\left[1+\sqrt{\frac{\mu}{k}}\right]}{1+\frac{\mu}{k}} \qquad (13)$$

This valid range of α is $0<\alpha<1.207$. However, examination of the value of α suggests the stationary maximum value of the equation occurs at a value which lies outside the upper limit due to the orthotropy. The value of α which is of more interest is its upper limit equal to 1.207.
From equation (6)

$$1.207 = 1.207\frac{\sqrt{\frac{\mu}{k}}\left[1+\sqrt{\frac{\mu}{k}}\right]}{1+\frac{\mu}{k}}$$; means $\frac{\mu}{k}$ must be equal to 1.

The failure pattern as shown dotted in Figure 2. Because $\frac{\mu}{k}>1$ the failure pattern does not conform to this pattern. The test failure pressure for 02 (Table 3) seems to lie between the theoretical values, which can be obtained from equation (11) for an isotropic or othotropic panel.

Triangular slab
Assuming unit deflection at D, the work equation for the panel is as follows:

$$\frac{m(\alpha L)}{L}\frac{1}{[1-\alpha]} + \frac{\frac{\mu}{k}mL\,[1-\alpha]}{\alpha L} = \frac{1}{2}wL\,L\frac{(1-\alpha)}{3} + \frac{1}{2}w\,L\frac{\alpha L}{3} \qquad (14)$$

$$\text{or } m\frac{\alpha^2+\frac{\mu}{k}(1-\alpha)^2}{\alpha-\alpha^2} = \frac{wL^2}{6}[1-\alpha+\alpha] = \frac{wL^2}{6}$$

$$\text{or } \frac{m}{w} = \frac{L^2}{6}\;\frac{\alpha-\alpha^2}{\alpha^2+\frac{\mu}{k}(1-\alpha)^2} \qquad (15)$$

$\frac{d\frac{m}{w}}{d\alpha} = 0$ therefore

$$(1-2\alpha)\left(\alpha^2+\frac{\mu}{k} - 2\frac{\mu}{k}\alpha + \frac{\mu}{k}\alpha^2\right) - (\alpha-\alpha^2)\left(2\alpha - 2\frac{\mu}{k} + 2\frac{\mu}{k}\alpha\right) = 0 \qquad (15)$$

or $\alpha^2\left(\frac{\mu}{k}-1\right) - 2\alpha\frac{\mu}{k} + \frac{\mu}{k} = 0$

$$\text{therefore } \alpha = \frac{+2\frac{\mu}{k} \pm \sqrt{\frac{4\mu^2}{k^2} - 4\left(\frac{\mu}{k}-1\right)\left(\frac{\mu}{k}\right)}}{2\left(\frac{\mu}{k}-1\right)} \qquad (16)$$

Considering the valid value of α

$$\alpha = \frac{+2\frac{\mu}{k} - 2\sqrt{\frac{\mu}{k}}}{2\left(\frac{\mu}{k}-1\right)} = \frac{\frac{\mu}{k}-\sqrt{\frac{\mu}{k}}}{\frac{\mu}{k}-1} \text{ where } \frac{\mu}{k}>1 \qquad (17)$$

The value of α is substituted in equation (15) to find the failure pressure.

Experimental details

All test panels and associated control specimens, eight course wallettes, were built in 1/3rd scale bricks. The traditional method of brick laying was used for the construction of the panels and the control specimens. The control specimens similar to that described earlier were used to find the strengths and moduli of elasticity in the two directions. The control specimens were tested the same day as the panels. The results of the tests on wallettes to determine the strength and stiffness orthotropies are given in Table 1 and Table 2.

Arrangement for testing the panels

The octagonal panels were built vertically in a rig. The triangular panels were built within the triangular testing frame. Due to the different shapes and size, special testing frames were built to suit their respective profile. The octagonal panels were simply supported on eight sides and the triangular panels were simply supported on two and free on the third. The rectangular panels with holes were built vertically on the strong floor of the laboratory with the lowermost course of the brick resting, but not bonded to, a polythene membrane, between the floor and the test walls. The rectangular panels were simply supported on the top and bottom and the reactions were sustained by an independent rigid steel frame, which was fixed to the strong floor. Two other edges were bonded to two brick long short returns which were post-tensioned to prevent premature failure of the returns.

Polythene air bags tailored to suit the shape and size of the panels were used to apply the lateral pressure. The lateral pressure was applied by the bag sandwiched between the test wall and a rigid steel frame. Lateral pressure was applied to the test walls until failure by pumping air into the bag by means of a compressor. The pressure in the bag was measured by mercury or water manometer.

Table 1. Relationship between flexural strengths in two orthogonal directions as determined from tests on control specimens (wallettes)

Corresponding test wall	*Tensile strength normal to bed joint f_{tn} N/mm^2	+Tensile strength parallel to bed joint f_{tp} N/mm^2	Orthotropy $\frac{f_{tp}}{f_{tn}}$
R_1	0.74	1.91	2.58
R_2	0.52	1.43	2.75
01	0.69	1.44	2.09
02	0.96	2.18	2.27
T1	0.68	1.74	2.56
T2	0.98	2.17	2.21

* Average of three tests.
+ Average of six tests.

Table 2. Relationship between the moduli of elasticity in two orthogonal directions.

Corresponding wall	E_y N/mm^2	E_x N/mm^2	Stiffness orthotropy $\frac{E_x}{E_y}$
R1	8900	12100	1.36
R2	9000	13100	1.46
01	8100	10000	1.23
02	9500	11800	1.24
T1	9300	12000	1.29
T2	10100	12000	1.19
			Mean: 1.30

Table 3. Comparison of theoretical and experimental failure pressure for the panels.

Panel	Failure pressure N/mm^2 x 10^{-3}	
	Fracture line	Experimental
Rectangular panel with openings R1	10.02) 8.76	10.34) 9.33
R2	7.50)	8.31)
Octagonal panel 01	5.28) 5.95	5.79) 5.83
02	5.36* 6.62)	5.86)
Triangular panel T1	4.93) 6.02	5.88) 6.76
T2	7.10)	7.65)

*Isotropic

Figure 4. Failure of panel with opening.

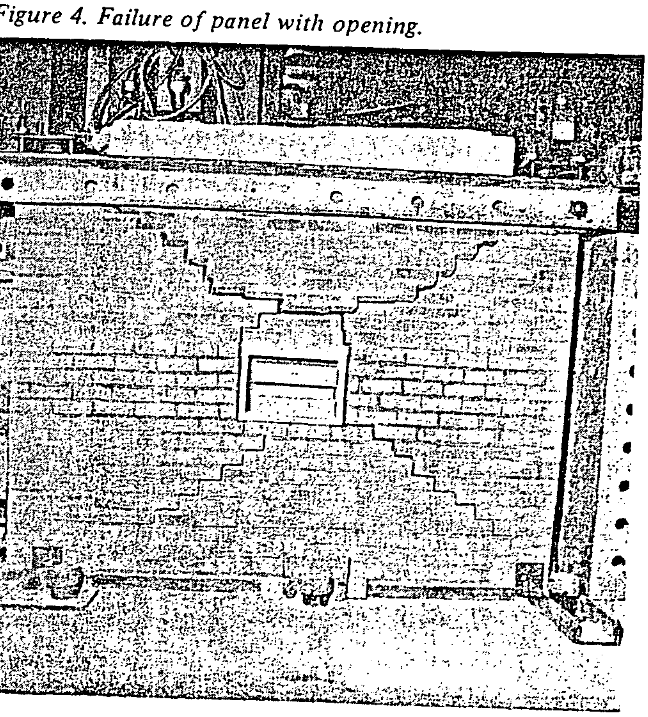
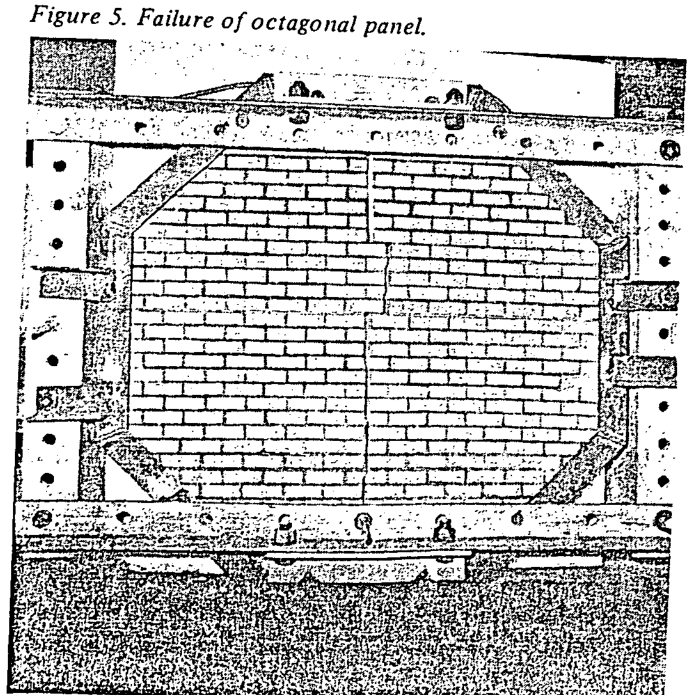


Figure 5. Failure of octagonal panel.



Results and discussion

All the walls failed suddenly without prior visible cracking. Figure 4, 5, 6 show the typical failure of the panels. The test results are summarised in Table 3.

It can be seen from Table 3 that there is very good agreement between the experimental failure pressure and those predicted by the fracture line method.

Conclusion

The method outlined has clearly demonstrated that it can be used to predict the failure pressure of laterally loaded panels with openings or panels of unusual shape or size. Previously, the method was found to predict the failure pressure of rectangular panels having different boundary conditions [8]. Hence, this method may be used with some confidence for the design of laterally loaded brickwork panels having various shapes and boundary conditions using the stiffness orthotropy and material strengths.

ACKNOWLEDGEMENT

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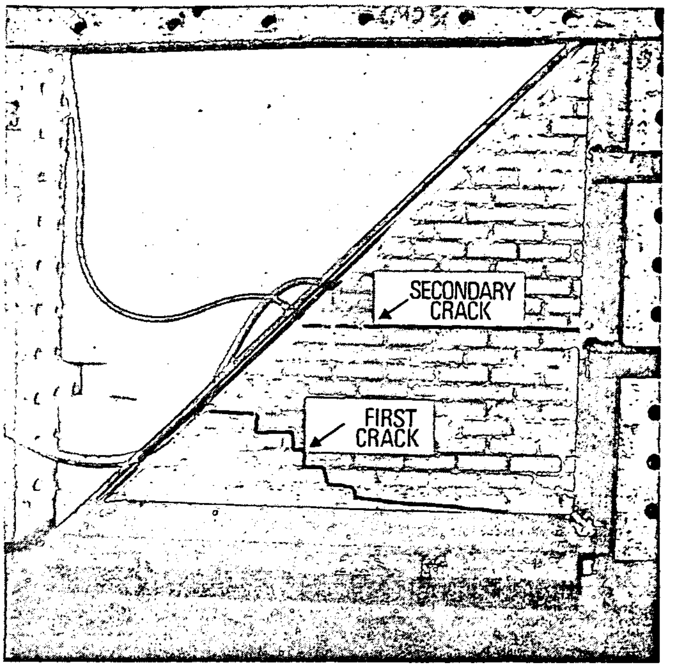

Figure 6. Failure of triangular panel.

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FAILURE CRITERION AND BEHAVIOR OF BRICKWORK IN BIAXIAL BENDING

By B. P. Sinha,[1] C. L. Ng,[2] and R. F. Pedreschi[3]

ABSTRACT: The paper describes a novel method for determining the failure criterion of masonry in biaxial bending. The conventional failure theories are not applicable to masonry that exhibits both strength and stiffness orthotropies. The failure criterion has been established from a novel test described in this paper. It appears that the weaker direction cracks in flexure first, in many cases, and the load is then shed to the stronger direction. Final failure happens only after the stronger direction fails in flexure. The cracking and failure in biaxial bending is predicted by the equations given in this paper.

INTRODUCTION

Masonry cladding panels supported on three or four sides bend like thin plates under lateral loading and are subjected to biaxial moments. The British code *BS5628* (BSI 1978) gives design coefficients for such panels. The coefficients are based on the yield-line method (Johanson 1972) originally developed for underreinforced concrete slabs. A brittle material such as unreinforced masonry does not behave in a rigid plastic manner that is assumed in yield-line theory, and thus cannot be applied universally to masonry panels. At present, no accurate mathematical solution is available that can predict cracking or failure of the laterally loaded panels. There are many failure theories such as that of Tresca, Von Mises, and Rankine, but they apply mainly to ductile or, to some extent, brittle isotropic materials (Fenner 1989). These theories are not applicable directly to masonry in bending, as it is brittle and exhibits both strength and stiffness orthotropies. An earlier attempt was made, with limited experimental results, to define the failure criterion for brickwork by subjecting a single joint to vertical and horizontal moments simultaneously (Baker 1979). On the basis of this test, Baker proposed an elliptical failure criterion for combined vertical and horizontal moments. This can be represented by the equation

$$\left[\frac{F_y}{F_{uy}}\right]^2 + \left[\frac{F_x}{F_{ux}}\right]^2 = 1 \tag{1}$$

where F_y and F_x = maximum flexural stresses in biaxial bending; and F_{uy} and F_{ux} = ultimate flexural strengths in weaker (y) and stronger (x) directions.

The method ignored the stiffness orthotropy and thus did not improve the understanding of the real behavior of laterally loaded panels. A rational approach can only evolve if the failure criterion in biaxial bending is known. Any attempt to define the failure criterion must be appropriate to anisotropic behavior of the material. Hence, a novel test method was developed by the first writer to understand the behavior of brickwork in biaxial bending (Mullholland 1980; Duarte 1993).

EXPERIMENTAL PROGRAM

The experimental program was carried in stages. The material properties were obtained from small specimen tests and the behavior in biaxial bending from the novel test specimen.

Materials

Half-scale solid bricks in 1:3 (rapid hardening portland cement:sand) mortar were used for the construction of all test specimens. All specimens were cured under a polythene sheet in the laboratory till the day of the test. The mortar cubes and cross beams with their associated wallettes were tested at the same age. All tests were carried out within 18–23 d of curing. Eighteen mortar cubes (100 mm) were tested and the average compressive strength of the mortar was 21.4 N/mm^2 with coefficient of variation 0.16.

Flexural Tests

Two types of wallettes, designated as A and B, were tested under a four-point loading system to obtain the strength in the two orthotropic directions (Fig. 1). These wallettes were built at the same time as the cross beams. A total of 36 wallettes were tested (half in each direction). The tests were done in a horizontal position as opposed to vertical, as was suggested

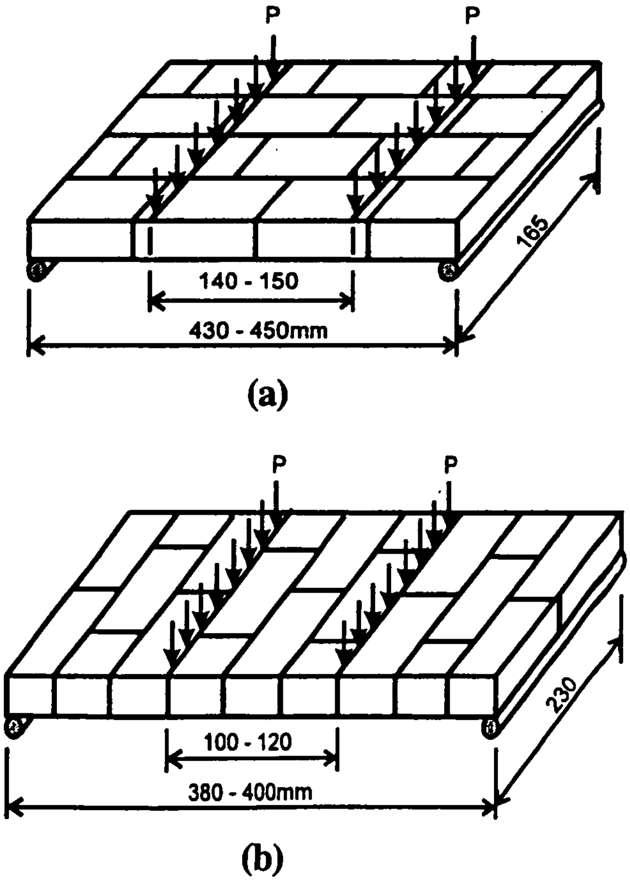

FIG. 1. Wallette Test Setup

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TABLE 1. Brickwork Properties

Material properties (1)	Normal to bed-joint (y-direction) (2)	Parallel to bed-joint (x-direction) (3)
Moduli of elasticity (MPa)	10,689	15,500
Poisson's ratios (ν_{xy} and ν_{yx})	0.16	0.11
Ultimate moment (Nmm/mm)	2.82	9.74
Coefficient of variation for moment	0.18	0.12

Note: Age 18–23 d

by the code (BSI 1978). The self-weight of each specimen was accounted for in calculating the bending moments. It was felt inappropriate to test them in a vertical attitude because the dead weight of the wallettes varies from the top to bottom and provides rotational restraint due to friction at the base. This effect of the dead weight is ignored in the code.

In specimen A, the tension develops parallel to the bed-joint whereas in B the tension develops normal to the bed-joint. Strain in the constant bending zone was measured by six electrical (10 mm) strain gauges, three on top and three on the bottom. The modulus of elasticity, needed for the theoretical analysis, was obtained from these measurements.

Some wallettes, similar to specimens A and B, were tested under a three-point loading system to obtain the flexural strength in two orthogonal directions. The average values of the ultimate moments normal and parallel to the bed-joint were 1.4 and 1.8% higher than the wallettes tested with four-point loading. The difference is insignificant and it is unlikely to cause any change to the failure envelope. Therefore, the ultimate moments obtained from wallettes with four-point loading were used to comply with the British code (BSI 1978).

Poisson's ratios were obtained by measuring the lateral and longitudinal strains from wallettes (225 × 225 × 55 mm) subjected to compression normal and parallel to the bed-joints. The lateral and longitudinal strains were measured by 63.5 mm vibrating wire gauges. The average ultimate moment, modulus of elasticity, and the Poisson's ratios in two orthogonal directions are given in Table 1. The ratio for both the modulus of elasticity and the Poisson's ratio parallel and normal to the bed-joint was 1.45. Tensile strength normal to the bed-joint is more variable than parallel to the bed-joint. This is usual for brickwork (Satti and Hendry 1973; Sinha and Hendry 1975; Lawrence 1975; West et al. 1977).

Cross-Beam Specimen

To simulate and apply both vertical and horizontal moments simultaneously, a novel specimen in the shape of a cross as shown in Fig. 2 was used. The central part of the cross, representative of a brickwork panel, was built with half-scale bricks and a 1:3 (rapid hardening cement:sand) mortar. The four arms attached to the central portion, comb-like in structure, were built with similar bricks using an epoxy sand mortar. This was done to prevent premature failure within the arm either in bending or shear. In addition, the comb-like structure allowed almost unhindered crack propagation within the central portion. The specimens were tested as simply supported beams subjected to a central point load. The load was applied through a disc (40 mm diameter) set in dental plaster to avoid any stress concentration at the mortar joint. The ratios of the arms, i.e., normal to the bed-joint in the direction of y and parallel to bed-joint in the direction of x, varied from 0.5 to 2.0. For each aspect ratio, three specimens were tested (a total of 33). The load was increased in steps until cracking and failure of the cross beam. Both the applied load and reactions were measured by the load-cells to check any discrepancies in the results. No difference was recorded between them. The advantage of this novel method is that not only the cracking load can be pinpointed, but also the redistribution of load in the two directions after one has failed. The measured pre- and

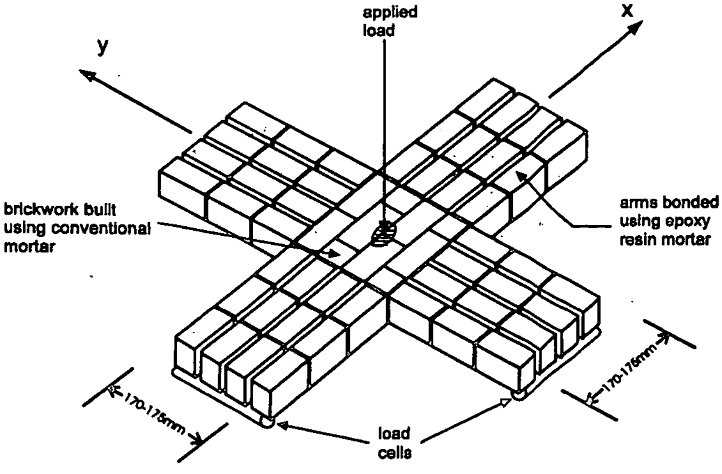

FIG. 2. Configuration of Cross Beam

TABLE 2. Comparison of Experimental and Theoretical Cracking Load for Cross Beams

		CRACKING LOAD (N)						
		Average Experimental Results[a]			Rankine Failure Theory		Finite Element	
L_x (mm) (1)	L_y (mm) (2)	Applied load (3)	P_x (4)	P_y (5)	P_x (6)	P_y (7)	P_x (8)	P_y (9)
300	585	3,384	3,124	260	3,836	276	2,686	195
445	585	2,284	1,811	473	1,943	590	1,408	424
585	585	1,690	936	754	855	590	818	544
690	585	1,284	613	671	521	590	569	595
860	585	987	318	669	269	590	357	649
1,140	585	841	165	676	116	590	207	682
585	300	1,816	489	1,327	224	1,150	249	1,164
585	445	1,391	392	999	494	775	525	755
585	690[b]	2,024	1,467	557	1,181	500	1,046	445
585	860[b]	2,066	1,595	471	1,848	401	1,424	342
585	1,140[b]	2,140	1,812	328	2,032	189	1,891	238

[a]Three specimens.
[b]Two specimens.

TABLE 3. Comparison of Experimental and Theoretical Failure Load for Cross Beams

		FAILURE LOAD (N)					
		Average Experimental Results[a]			Rankine Failure Theory	Finite Element	
L_x (mm) (1)	L_y (mm) (2)	Applied load (3)	P_x (4)	P_y (5)	P_x (6)	P_x (7)	P_y (8)
300	585	4,974	4,805	169	4,573	4,503	73
445	585	3,096	2,792	304	2,672	2,643	106
585	585	2,151	2,022	129	1,975	1,684	90
690	585	1,925	1,684	241	1,674	1,440	169
860	585	1,535	1,379	156	1,343	1,234	261
1,140	585	1,293	1,109	184	1,043	940	274
585	300	2,109	1,829	280	1,975	1,692	171
585	445	2,038	1,818	220	1,975	1,508	182
585	690[b]	2,050	1,814	236	1,975	1,663	63
585	860[b]	2,083	2,058	25	1,975	1,727	59
585	1,140[b]	2,266	1,968	298	1,975	1,920	208

[a]Three specimens.
[b]Two specimens.

postcracking reactions from the tests are given in Tables 2 and 3. The orthogonal moments at any stage of loading can be obtained accurately from the measured reactions multiplied by the lever arm.

THEORETICAL ANALYSIS

Rankine Maximum Stress Theory

Rankine maximum stress theory was used to obtain the failure load (Fenner 1989). It was assumed that the failure in the central part of the cross will take place if any one direction reaches its ultimate strength. Consider the simply supported cross beam subjected to an applied point load P at the center. Neglecting the effect of Poisson's ratio, the load will be shared as

$$P_x + P_y = P \quad (2)$$

where P_x and P_y = load carried by x- and y-directions.

From compatibility of deflection, we can write

$$\frac{P_x(L_x)^3}{48E_xI_x} = \frac{P_y(L_y)^3}{48E_yI_y} \quad (3)$$

$$\text{or } P_x = P_y\left(\frac{L_y}{L_x}\right)^3\left(\frac{E_x}{E_y}\right), \quad \text{since } I_x \approx I_y \quad (4)$$

The moduli of elasticity have been taken constant for the cross for simplification. The experimental value of modulus of elasticity of arm made in epoxy and sand was marginally different affecting the results up to a maximum of 2.5% only, hence it was neglected.

Substituting the value of P_x from (4) into (2), one can obtain the value of P_y in terms of applied load P. Hence, the applied moment in both directions can be obtained from

$$M_y = \frac{P_yL_y}{4} = \frac{PL_y}{4}\,\frac{1}{\left[1 + \left(\frac{L_y}{L_x}\right)^3\frac{E_x}{E_y}\right]} \quad (5)$$

similarly

$$M_x = \frac{P_xL_x}{4} = \frac{PL_x}{4}\,\frac{1}{\left[1 + \left(\frac{L_x}{L_y}\right)^3\frac{E_y}{E_x}\right]} \quad (6)$$

According to Rankine failure theory, the failure will take place in the direction that reaches its ultimate moment of resistance first, i.e.

$$M_y \geq M_{uy}; \quad \text{or } M_x \geq M_{ux} \quad (7, 8)$$

The theoretical cracking loads, P_y and P_x were calculated from (5) and (6) and equated to the ultimate moment given in (7) or (8). The minimum of the two values will define the cracking load. The ultimate moments in two orthotropic directions were obtained from the data in Table 1. In many cases during the tests, it was observed that once the y-direction (normal to bed-joint) cracked, it could not support any load and, hence, any subsequent resistance of the cross beam was solely due to the strength in the x-direction (parallel to bed-joint). By using this observation, the theoretical analysis at final failure becomes simplified by assuming the cross beam acts as single beam in uniaxial bending in the x-direction. Hence, the failure of the cross beam can only occur when the moment in this direction reaches its moment of resistance. The theoretical results are compared with the experimental values for the cross beam in Tables 2 and 3.

Finite-Element Method

The method described previously is simple, but it is not capable of predicting postcracking behavior such as load distribution in the cross beams. Hence, an in-house finite-element plate-bending program was used in conjunction with the Rankine failure theory to predict the pre- and postcracking load distribution in two orthogonal directions. Masonry was considered as a linear homogeneous orthotropic material in deriving the rigidity matrix using the properties obtained from the wallette tests.

Thus, before cracking, the rigidity matrix (D) for the plate element is represented by

$$D = \frac{t^3}{12}\begin{bmatrix} \frac{E_x}{(1-\nu_{xy}\nu_{yx})} & \frac{\nu_{xy}E_y}{(1-\nu_{xy}\nu_{yx})} & 0 \\ \frac{\nu_{yx}E_x}{(1-\nu_{xy}\nu_{yx})} & \frac{E_y}{(1-\nu_{xy}\nu_{yx})} & 0 \\ 0 & 0 & G_{xy} \end{bmatrix} \quad (9)$$

The preceding rigidity matrix is successively modified for each cracked element using the smeared crack modeling (Chen and Saleeb 1982). Thus, the modified rigidity matrix can be represented by

$$D_{cr} = \frac{t^3}{12}\begin{bmatrix} \frac{E_x}{(1-\nu_{xy}\nu_{yx})} & \alpha\frac{\nu_{xy}E_y}{(1-\nu_{xy}\nu_{yx})} & 0 \\ \alpha\frac{\nu_{yx}E_x}{(1-\nu_{xy}\nu_{yx})} & \alpha\frac{E_y}{(1-\nu_{xy}\nu_{yx})} & 0 \\ 0 & 0 & \beta G_{xy} \end{bmatrix} \quad (10)$$

The value of α is taken nearly as zero, which means that the cracked element is subjected to uniaxial stress parallel to the direction of crack. The value of β lies between 0 to 1. In this case the value of β appears to be 0.09.

Fracture Line Analysis

Fracture line analysis (Sinha 1978) has been used for laterally loaded brickwork panels; hence, the method was applied to the cross beams.

Assumptions

The material behaves in a linear elastic manner. All deformations take place along the fracture lines only, and the in-

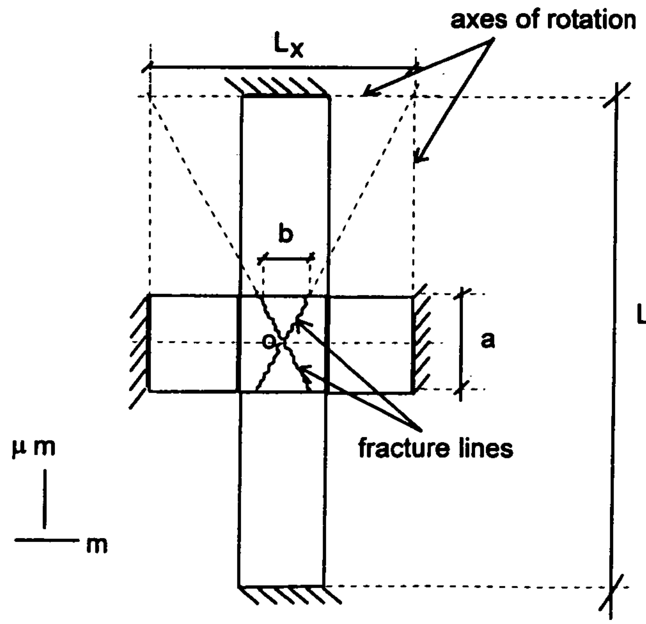

FIG. 3. Crack Pattern for Fracture-Line Analysis of Cross Beam

dividual parts of the cross rotate as rigid bodies. The load distributes according to the stiffness in the respective direction. The fracture lines develop when the relevant strengths are reached in the two orthotropic directions.

To deal with the stiffness orthotropy, the actual cross was converted to an affine isotropic cross having the same modulus of elasticity E_x in both directions. Consider the idealized fracture lines for the affine cross and give a virtual deflection of unity at the center o (Fig. 3). Hence, the following holds true:

$$\text{external work done} = \frac{1}{2} \cdot P \cdot 1 \tag{11}$$

The external work done must be absorbed by the internal work done on the fracture lines as follows:

$$\text{internal work done on the fracture line} = \frac{1}{2}\,(mb\theta_x + \mu ma\theta_y) \tag{12}$$

where a, b = projected lengths of fracture lines in two orthogonal directions; θ_x, θ_y = normal rotations along two orthogonal directions; and m, μm = normal moment/unit length.

Therefore

$$\text{internal work done} = \frac{1}{2}\left(\frac{4mb}{L_y} + \frac{4\mu ma}{L_x}\right) \tag{13a}$$

$$\text{internal work done} = 4m\,\frac{1}{2}\left(\frac{b}{L_y} + \frac{\mu a}{L_x}\right) \tag{13b}$$

External work done must be equal to internal work done, hence

$$P = 4m\left(\frac{b}{L_y} + \frac{\mu a}{L_x}\right) \tag{14}$$

The result obtained from (14) is shown in Table 4. Eq. (14) becomes the same as the yield-line equation if stiffness orthotropy is neglected, i.e., the deflection compatibility is not met.

ANALYSIS OF TEST RESULTS

Initially, the applied load was linearly distributed according to the stiffness in the respective directions (Figs. 4–6). Once the strength in any one direction reached its ultimate value, the specimen cracked. The value of the reaction dropped and the load was shed to the stronger direction. Three types of failure were detected in the experiments. In the first type, both directions failed simultaneously without cracking beforehand (Fig. 4). In the other two types, the cross beam first cracked normal to the bed joint (weaker y-direction) and the load was then shed to the stronger (x) direction immediately. If the shed load was large enough it would cause immediate failure in stronger direction (Figs. 5 and 6). If the shed load was insufficient to cause failure, the applied load can be increased till the ultimate strength in the stronger direction was reached (Fig. 6). From the results, it is very clear that after the section is cracked, it can no longer support any moment in that direction due to the brittle nature of the material. Some residual load in the y-direction was measured, but this is due to the dead weight and some frictional restraint at the support.

Comparison of Experimental and Theoretical Results

The results of the analysis are given in Figs. 4–6 and in Tables 2–4. From Figs. 4–6, it can be seen that the finite-element method predicts the trend obtained both in the pre- and postcracking phase of the experiments, but underestimates the cracking and failure load by 1–26% and 3–18%, respectively (Tables 2 and 3). The Rankine failure theory underestimates the cracking load by 8–24% and the ultimate load by 3–19%. The finite-element method provides little improvement. This underestimation in both cases is due to the fact that the uniaxial strength was used in the prediction, which is lower than that obtained in biaxial bending. At failure, the stronger (x) direction carries all the applied load.

Table 4 compares the experimental results of all the crosses with the yield-line method (Johanson 1972), the fracture-line theory (Sinha 1978), and the finite-element method using the failure criterion defined by (15). The yield-line method consistently overestimates the failure loads in all cases. In majority of cases, the fracture-line method provides slightly better prediction of the failure loads compared to the yield-line method. However, both methods give good agreement with the experimental results of the test panels where both directions failed simultaneously. The yield-line and the fracture-line methods can only be applied safely to these cases. These methods are not capable of predicting the failure load correctly if cracking in any direction precedes the failure. After the section cracks, it does not support any moment in that direction, which is contrary to the assumptions made either in the yield-line or fracture-line methods. The finite-element method, using the failure criterion proposed in this paper, slightly underestimates

TABLE 4. Comparison of Experimental and Theoretical Failure Loads

		Experiment	Theoretical Failure Load (N)					
L_x (mm) (1)	L_y (mm) (2)	Failure load (N) (3)	Yield-line (YL) analysis (4)	Ratio yield line (YL)/ experiment (5)	Fracture-line (FL) analysis (6)	Ratio fracture line (FL)/ experiment (7)	FEM with failure criterion (8)	Ratio FEM/ experiment (9)
300	585	4,974	5,104	1.03	5,062	1.02	4,740	0.95
445	585	3,096	3,143	1.02	3,091	0.99	2,877	0.93
585	585	2,151	2,639	1.23	2,570	1.19	1,958	0.91
690	585	1,925	2,163	1.12	2,366	1.23	1,717	0.90
860	585	1,535	1,658	1.08	1,793	1.16	1,424	0.93
1,140	585	1,293	1,326	1.03	1,410	1.09	1,301	1.01
585	300	2,109	2,290	1.09	2,435	1.15	1,976	0.94
585	445	2,038	2,335	1.15	2,541	1.24	1,816	0.90
585	690	2,050	2,473	1.21	2,422	1.18	1,850	0.93
585	860	2,083	2,363	1.13	2,331	1.12	1,986	0.95
585	1,140	2,266	2,327	1.03	2,309	1.00	2,291	1.01
585	690	2,875[a]	2,924	1.02	2,857	0.99	2,819	0.98
585	860	2,515[a]	2,595	1.03	2,556	1.01	2,460	0.98
585	1,140	2,482[a]	2,525	1.02	2,504	1.00	2,420	0.97

[a]Both directions failed simultaneously without prior cracking.

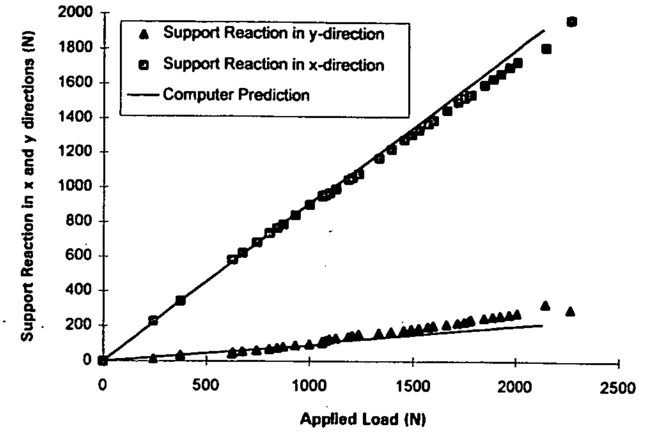

FIG. 4. Typical Load Distribution in Cross Beam—Failure Type 1 (Lx = 585 mm, Ly = 1,140 mm)

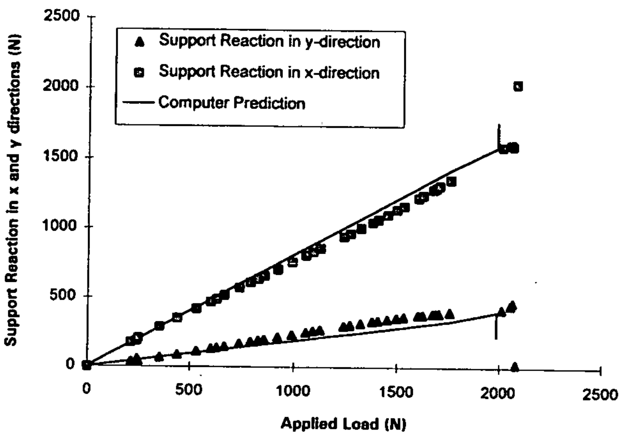

FIG. 5. Typical Load Distribution in Cross Beam—Failure Type 2 (Lx = 585 mm, Ly = 860 mm)

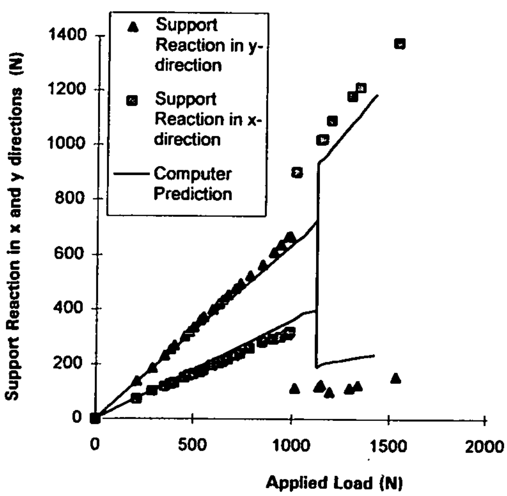

FIG. 6. Typical Load Distribution in Cross Beam—Failure Type 3 (Lx = 860 mm, Ly = 585 mm)

the failure load (Table 4) and thus can safely be used for predicting the strength of panels.

FAILURE CRITERION IN BIAXIAL BENDING

The results of the test have been plotted in nondimensional form in Fig. 7. The average values of uniaxial strength in two orthogonal directions were taken from testing of the wallettes.

The best envelope that fits all the points can be represented by

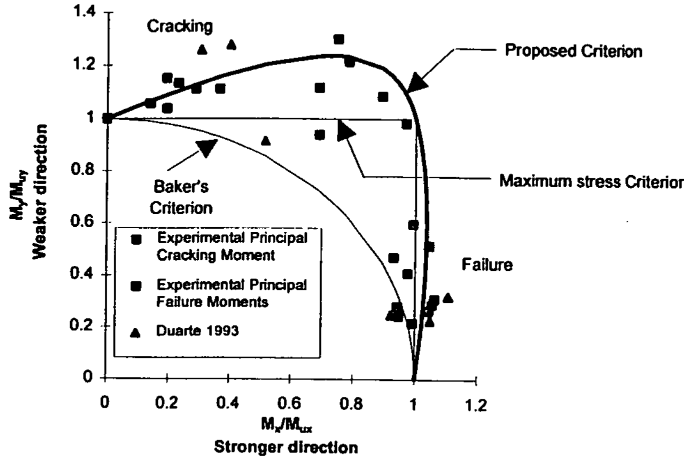

FIG. 7. Biaxial Failure Envelope

$$\left(\frac{M_y}{M_{uy}}\right)^2 - 0.75\cdot\frac{M_x}{M_{ux}}\cdot\left(\frac{M_y}{M_{uy}}\right)^2 - 0.25\cdot\frac{M_x}{M_{ux}}\cdot\frac{M_y}{M_{uy}} + \left(\frac{M_x}{M_{ux}}\right)^2 = 1.0 \qquad (15)$$

Cracking will precede failure if any point on the failure envelope lies to the left of the intersection point (1, 1). The final failure will result due to the load shedding from the weaker to the stronger direction. When the moment reaches the value of the moment of resistance in the stronger direction, failure takes place. Below the intersection point (1, 1), the failure will happen simultaneously in two directions or due to failure of the stronger direction, thus the weaker direction may or may not reach its ultimate strength. To evaluate the results against the Rankine maximum stress theory and Baker's failure criterion (Baker 1979), both failure envelopes are also shown on Fig. 7. The poor agreement between the experimental results and existing failure theories suggests very strongly that both theories are not applicable to masonry in biaxial bending. Baker's failure criterion does not predict cracking nor the ultimate failure of the cross beams in this paper.

Reduced-scale bricks have been used successfully to reproduce the behavior and the strength of the full-scale brickwork (Murthy 1964; Sinha 1967). In this investigation, both the behavior and the strength in biaxial bending have been obtained using half-scale bricks; hence, it can be concluded that the results are valid for full-scale brickwork.

SUMMARY AND CONCLUSIONS

The applied lateral load distributes according to the stiffness orthotropy of the brickwork; hence, the modulus of elasticity in the two orthogonal directions exert a great influence in the behavior and failure of brick panels. After the weaker direction cracks, it no longer supports any load. The load is then taken by the stronger direction until it fails.

Neither the conventional failure theories nor Baker's failure criterion are applicable to brickwork subjected to biaxial bending. The strength in the weaker direction is enhanced in biaxial bending. The criterion proposed in this paper defines the failure of brickwork in biaxial bending and may be used with the finite-element method for the design of laterally loaded panels.

The yield-line and the fracture-line theories predict very closely the failure load of the test panels when both directions failed simultaneously. These methods are not capable of predicting the failure load in majority of cases where cracking precedes the failure.

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APPENDIX II. NOTATION

The following symbols are used in this paper:

a, b = projected length of fracture lines in two orthogonal directions;
E_x, E_y = modulus of elasticity in orthogonal directions;
F_y, F_x = maximum flexural stresses in biaxial bending;
F_{uy}, F_{ux} = ultimate flexural strengths in weaker (y) and stronger (x) directions;
I_x, I_y = second moment of area;
L_x, L_y = span of affine cross in x- or y-direction;
$m, \mu m$ = normal moment/unit length;
M_x, M_y = applied moment in two orthogonal directions;
M_{ux}, M_{uy} = ultimate flexural moment;
P = total applied load;
P_x, P_y = load in x- and y-directions;
t = thickness of brickwork;
α, β = reduction factors;
θ_x, θ_y = normal rotations along two orthogonal directions; and
ν_{xy}, ν_{yx} = Poisson's ratios in two orthogonal directions.

The Quarry Project

(Group 2, Section 1.2.5 - Papers 20 to 29)

Proc. Instn Civ. Engrs, Part 1, 1976, 60, Feb., 153–162

TN 130 Structural testing of brickwork in a disused quarry

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The Note describes the development of a disused quarry as a full-scale structural testing station. The testing station was developed in 1967 by the Structural Ceramic Research Unit, Department of Civil Engineering and Building Science, University of Edinburgh, to examine the strength and rigidity of complete large panel multi-storey structures. The main advantage of this development is that the lateral loads are applied by jacking against the rock face, thus saving the cost of building a large frame for lateral loading. Several series of full-scale tests were carried out to investigate the behaviour of brickwork subjected to combined compression and shear, the behaviour of brick shear wall structure subjected to wind loading, progressive collapse in brick multi-storey structures, lateral strength of wall panels under different boundary conditions and interaction between walls and floor slabs. A brief account of the up-to-date results is reported.

Introduction

Until quite recently the structural use of brickwork had declined because of some reluctance on the part of engineers and architects to exploit its potential in situations where it might be the appropriate form of construction. To some extent this resulted from lack of research, which in turn led to the adoption of very high factors of safety in the design of the brick structures rendering them uneconomical. The purpose of the development of a disused quarry in 1967 at Torphin in Edinburgh was, therefore, to study the behaviour of brick structures at full scale, so as to assist in the development of rational design methods and put forward suggestions for modifications to the relevant code of practice.

2. The purpose of the Note is to give an outline of the work done so far in the quarry and the practical implications of the results of various experiments. The reason for the selection of a disused quarry for a full-scale testing station was to permit the application of lateral loads to structures up to five storeys high by jacking against the rock face, thus saving the cost of an expensive frame. In tests not requiring lateral load the quarry face provides a rigid support for stabilizing structures and a fixed plane of reference for measurements. The site was prepared by removing loose rocks from the floor and face of the quarry, lining the face with concrete and laying concrete on the floor to give a sufficient working area.

3. Several series of experiments were carried out.

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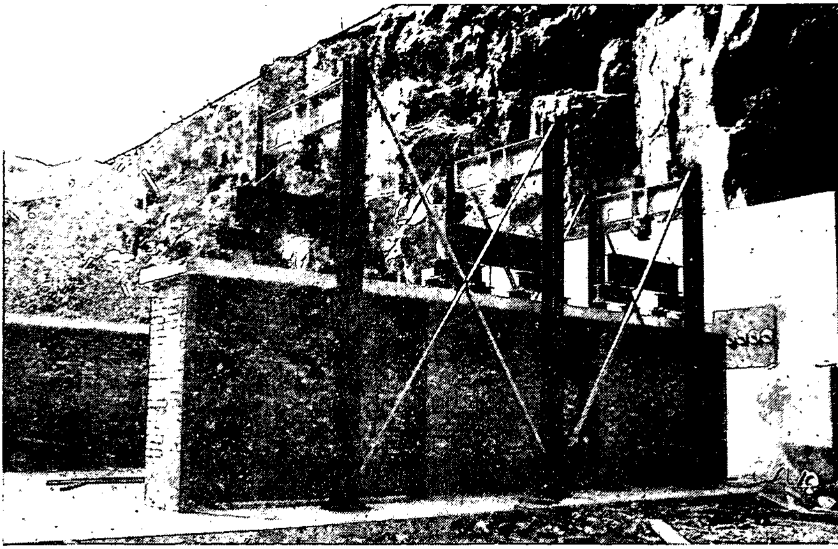

Fig. 1. Test arrangement for shear panel structure with precompression

Shear test in single-storey structures

4. Prior to the start of these tests a large number of tests were done o $\frac{1}{6}$ scale model brickwork structures to obtain the strength of brickwork unde combined compression and shear. The work was advanced to such a stag that it became necessary to carry out a limited number of full-scale tests to chec the validity before suggesting design rules for general acceptance. Six singl storey structures, three with openings and three without, were tested t destruction at various levels of precompression. Fig. 1 shows one of th structures. The test structure consisted of 102 mm thick continuous shea walls stabilized by four short returns with a 127 mm reinforced concrete sla on the top. The vertical load was applied by jacking against three 80 capacity portals, each of which was fixed to the 190 mm thick concrete base b means of eight expanding bolts. The shear load was applied by jacking agains the quarry face. As a result of this work[1] and earlier model work[2] a simpl formula to obtain the shear strength of brickwork subjected to precompressio was suggested which led to the upward revision of BS Code of practice CP 111 1964 by an amendment slip issued in 1970.[3] It appears that the recent Germa Code has also been modified in the light of the results of this test.

Behaviour of multi-storey brick structure subjected to win loading

5. In recent years a great deal of attention has been given to the analysis o shear wall structures. Design methods have varied from a simple approach in which lateral moments are apportioned between the shear walls in propor tion to their flexural rigidities, to highly analytical approaches taking int

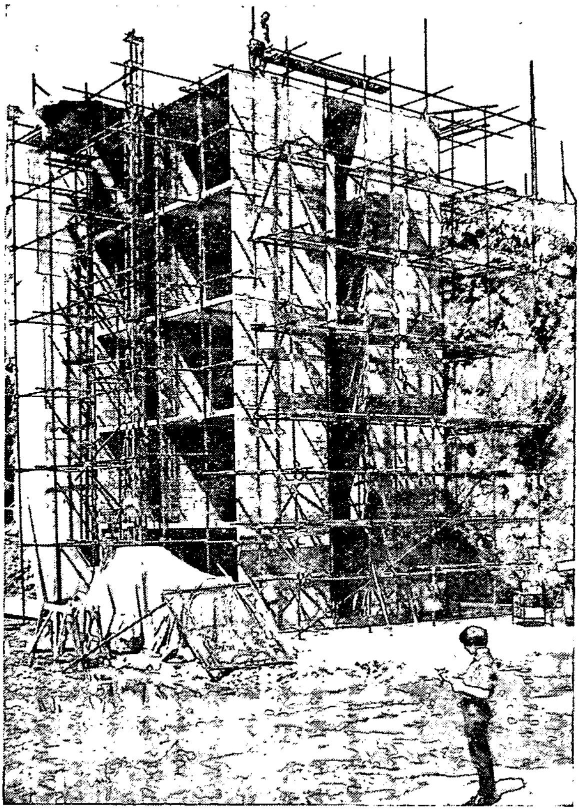

Fig. 2. General view of five-storey cross wall structure for lateral loading test

account interaction between the shear walls and interconnecting floor slabs or beams on the assumption of fully rigid connexion between the various elements. These methods of design have been used for the analysis of shear wall structures with limited confirmation of their validity from the testing of three-dimensional structures. Also, as a result of the upward revision of the design wind pressure in the UK,[4] the problem of a suitable method of design for calculating deflexion and stresses in such structures under lateral loading became urgent.

6. A five-storey structure was therefore built in Torphin Quarry (Fig. 2). The structure consisted of three pairs of 102 mm walls, the outside walls stabilized by two pairs of shear walls of similar thickness. The floor slabs were composed of 50 mm precast Omnia wide slab with 76 mm in situ concrete topping. The overall dimension of the building was 6·64 m × 6·23 m. The lateral load was applied by fifteen 10 t hydraulic jacks, three to a floor. The load from each jack was measured by a load cell. The overall lateral deflexion, lateral deflexion in ground floor shear walls and strains in the bottom shear walls were measured during the test. The test results have been reported in detail elsewhere.[5]

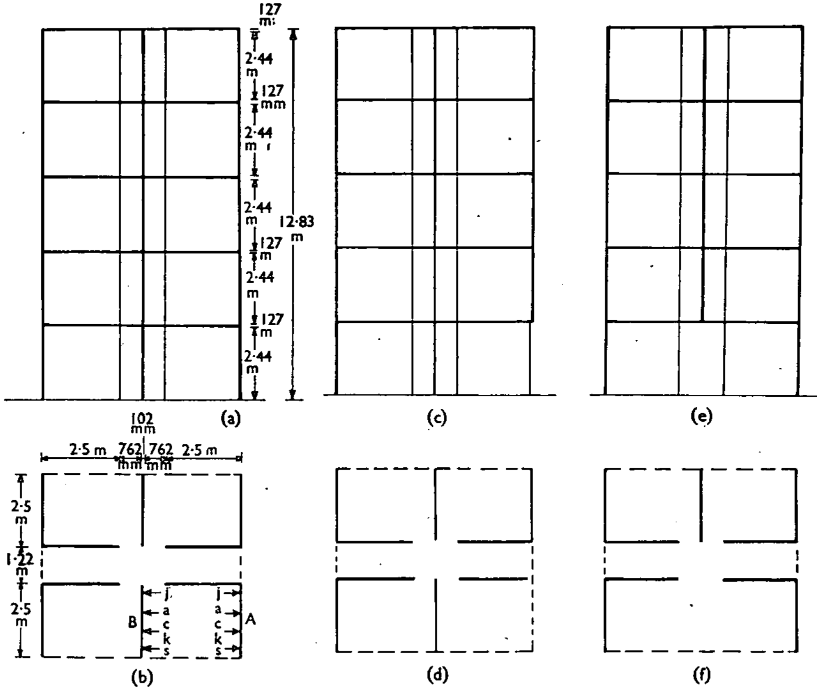

Fig. 3. Test arrangement for pull out test : (a) elevation ; (b) typical floor plan ; (c) elevation after first test ; (d) ground floor plan after first test ; (e) elevation after second test ; (f) ground floor plan after second test

7. The result of this experiment strongly suggested that the best approximation to the actual behaviour of a brick shear-wall structure is obtained by replacing the actual structure by an equivalent frame[6] in which the columns have the same sectional properties as the walls and the interconnecting slabs span between the centroidal axes of the columns. Any standard frame programme (which most design offices will have) can be used for the analysis of simple brickwork structures of this type, thus saving time in laborious arithmetical work.

Progressive collapse

8. Since the Ronan Point disaster in 1968 a great deal of attention has been focused on the problem of progressive collapse of a multi-storey structure. At the request of the Director of the Building Research Station three tests were carried out on the building referred to in §§ 5–7 to study whether it was susceptible to collapse following major local damage. Prior to these tests the stability of the building following the removal of a section of cross wall was assessed by methods described elsewhere.[7] These calculations suggested that the building would not collapse after removal of one of the major load-bearing walls. In each series, one of the main load-bearing walls (Fig. 3) was pushed out at the ground level by jacks with a view to testing the stability of the structure

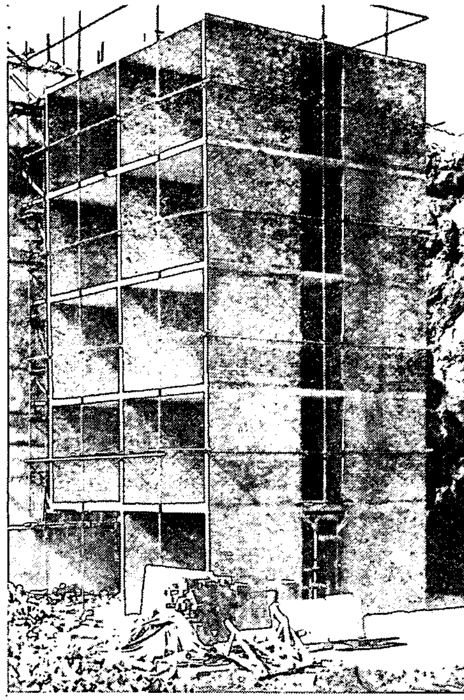

Fig. 4. Building after removal of main cross wall at ground floor level

in a damaged condition, as might occur following a gas explosion. The structure after one of the tests is shown in Fig. 4. The theoretical conclusion that the structure would remain stable was confirmed. It is worth mentioning that the structure was constructed for lateral loading tests prior to the Ronan Point collapse and was not especially designed to withstand this hazard.

Lateral strength of wall panels subjected to precompression

9. The strength of brick panels under lateral loading became an urgent problem following the upward revision of wind loading in the United Kingdom and the introduction of an amendment to the Building Regulations aimed at preventing progressive collapse. A series of transverse tests was planned on 280 mm cavity wall panels with and without returns subjected to precompression.

10. The walls were built within the experimental building used in the tests described in §§ 5–8 to provide realistic boundary conditions. This led to considerable modification of the existing building. Two cross walls and shear walls on the first and ground floors were removed from the side of the building nearest to the quarry face. The cross walls on the first floor were replaced by a 230 mm reinforced brickwork wall beam which supported the load above it and was in turn supported on two jacks and load cells resting on piers built in

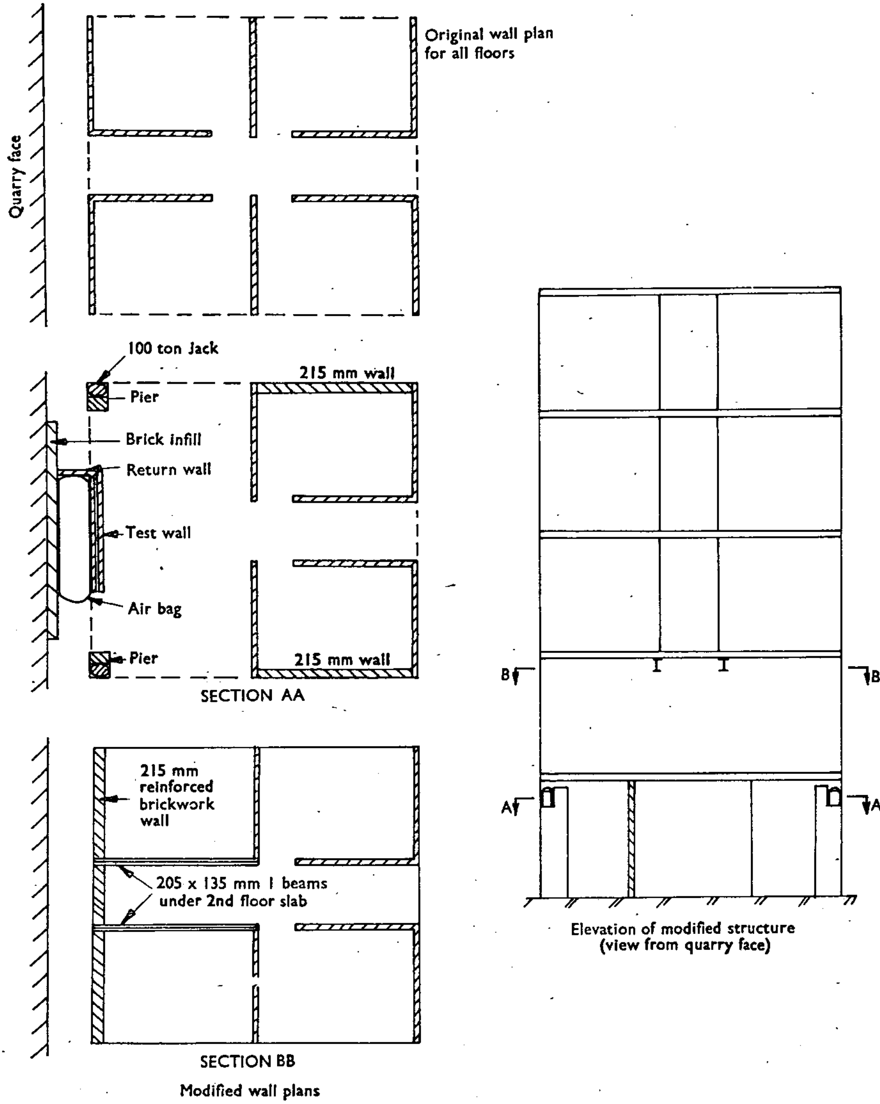

Fig. 5. Test arrangement for lateral loading of wall panels

two corners of the building in the ground floor (Fig. 5). This arrangement allowed the building to be lifted on the jacks while the test wall was under construction. Once the test wall attained sufficient strength the building was lowered gently to transfer the load on to the inner leaf of the wall.

11. It was important to know the exact height of the test wall, small chan-

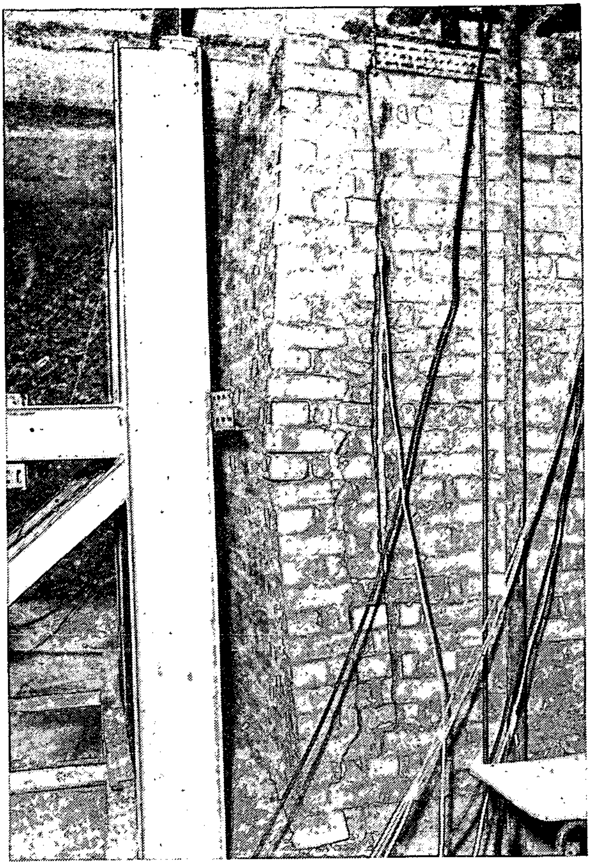

Fig. 6. Typical failure of wall with return

ges in height resulting in different precompressions due to the stiffness of the building. The lateral load was applied by an air bag sandwiched between the test wall and the quarry face. The deflexion of the panel and uplift of the building were measured during the test. An account of the work has been given elsewhere;[8] Fig. 6 shows the failure of a wall with return.

12. As a result of these tests a simple theory has been proposed for calculating the lateral strength of wall with and without returns subjected to precompression. This work was complimentary to a laboratory investigation of the lateral strength of brickwork with precompression carried out by West and others[9] at the British Ceramic Research Association and it is expected that the results will form the basis of relevant provisions in the new Code of Practice, at present under preparation.

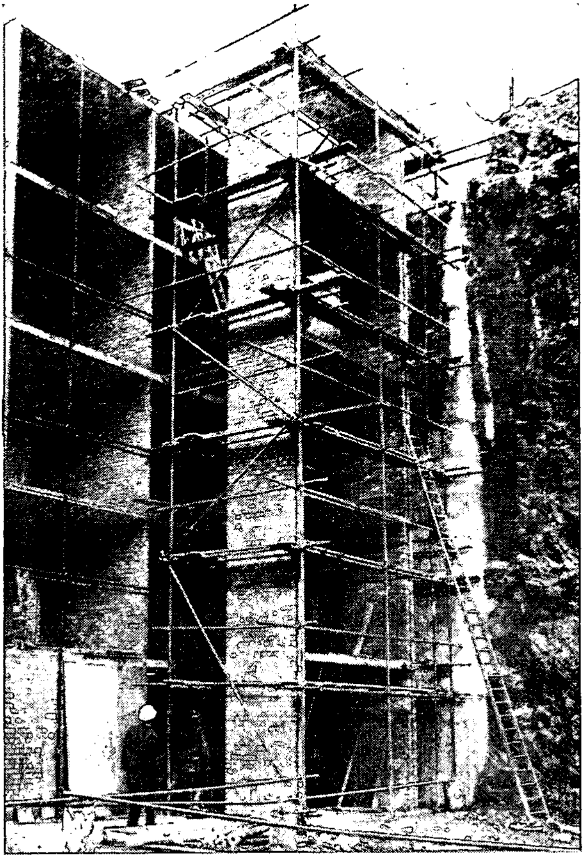

Fig. 7. Multi-storey cavity wall structure

Multi-storey cavity wall experiment

13. Cavity walls are commonly used as external walls in multi-storey buildings. The estimation of effective eccentricity, effective height and the distribution of loads from floor slabs between the leaves of a cavity wall is at present a matter of judgement, largely unaided by experimental results.

14. To improve this situation, a study of the structural behaviour[10] of multi-storey cavity walls was undertaken. Fig. 7 shows the cavity wall structure built at Torphin. The structure incorporated all the details used in practice; reinforced concrete floor slabs, damp proof course, etc. Near the quarry face the floor rested on both the leaves in every storey. On the other side the floor was supported on the inner leaf only at the first, second and fourth floor levels. Thus the outer leaf in this wall remained unsupported for three and two storeys.

15. Loading was applied to the floor slabs by pumping water into plastic

water tanks resting on the slabs. Strains and deflexions were measured at suitable points. From the test it appeared that the inner leaves carried about 86% of the total imposed load and only 14% was shared by the outer leaves. For the ground floor walls, the effective height equal to 0·75 actual height as stipulated in CP 111 appeared reasonable for this type of loading. The effective eccentricities resulting from the floor loading varied throughout the height of the test structure and it would not be reasonable to assume constant arbitrary eccentricity for the design as is done at present.

Test on three storey structure

16. The work described in §§ 13–15 is being extended to study the interaction between wall floor combinations which will afford some guidance to effective height and eccentricity to be assumed in design, and a three-storey two-bay structure consisting of three parallel 102 mm walls has been built at Torphin to examine these factors in an internal wall. The structure is under test at present, the system of loading and measurements being similar to those described above.

Conclusion

17. Full-scale tests are expensive and time-consuming compared with model tests. However, it is extremely desirable to carry out a limited number of such tests to verify models of theoretical approaches before general acceptance of conclusions based on the latter. The omission of such an approach has led to several spectacular structural failures in recent years with loss of life compared with which the cost of a large-scale test would be minimal. Further knowledge of the behaviour of real structures should lead to economy in the costs of construction through more refined structural design.

Acknowledgements

18. The work described in this Note has been supported by the British Ceramic Research Association, the Brick Development Association, Structural Clay Products Ltd. and the Science Research Council.

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shear tests on full-scale single-storey brickwork structures subjected to precompression

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This article presents a report of the first series of full-scale tests carried out at Torphin Quarry, Edinburgh. The object of these experiments was to obtain further information on the shear strength of brickwork panels built with wire-cut bricks, under varying degrees of precompression. Six structures were tested with precompressions up to 230lbf/in²—three with openings between the panels and three without. The results obtained were substantially in agreement with those obtained from corresponding model experiments and with other experimental evidence. A possible formula for permissible shear stress at a given level of precompression is put forward as a result of this work.

The lateral stability of multi-storey brickwork structures is provided by systems of walls stressed in compression and shear. Until recently there has been comparatively little information available on the strength of brickwork under this form of combined loading but a number of experiments were carried out by the Structural Ceramics Research Unit at Edinburgh University on one-sixth-scale model brickwork to examine this aspect of the design problem (1–4)*.

As a result of this work a possible revision of the relevant clause in CP 111:1964 has been proposed which would result in substantially higher permissible stresses in shear at higher values of precompression but it is clearly desirable to have some full-scale tests to give confirmation of the model work. A series of tests on six full-scale, single-storey shear-wall structures has been undertaken in a quarry where it was possible to apply horizontal loads by jacking against the rock face thus saving the cost of an expensive reaction frame for transmitting these loads to the ground.

This article embodies a description of the test facility and the results of this series of tests which forms Phase one of a more extensive programme now under way.

The test site

Torphin quarry

A suitable quarry for the tests was found at Torphin on the outskirts of Edinburgh. At this site there is a disused whinstone quarry having a sound rock face about 150ft long and almost vertical to a height of approximately 50ft. This face is accessible to all kinds of vehicles from both top and bottom. The floor is level and covered by a thin layer of broken rock and clay. Drainage by electric pump was existing and services were close at hand. The site was therefore such that it could be used for structural testing with very limited expenditure on preparation. Figure 1 shows a general view of the quarry at the beginning of operations.

Preparation of site for structural testing

Preliminary work on the site required the removal of about 1,000yd³ of crushed rock from the quarry floor, the removal of loose rock from the face and the cleaning of compacted clay and stones from the floor over the working area. In addition, a site hut was erected and an electricity supply brought to it.

Concrete was poured in three sections to give a level working area 36ft × 42ft with an average thickness of 7½in. This slab has nominal ⅜in mesh reinforcement. The concrete cube strength at seven days was 1898lbf/in² and tests on 1in dia cores taken at two months give a crushing strength of 5376lbf/in². This floor was of sufficient area to allow work to proceed on two test structures simultaneously.

The rock face, although sound, was irregular and to provide a suitable working surface, it was lined with concrete to a height of 15ft 5in for the first series of tests. A number of Rawl-bolts were set in the rock to provide anchorage for the concrete. The average crushing strength of 6in cubes at seven days were, 2354lbf/in² and 2287lbf/in².

**Numbers in parentheses indicate References at the end of the article.*

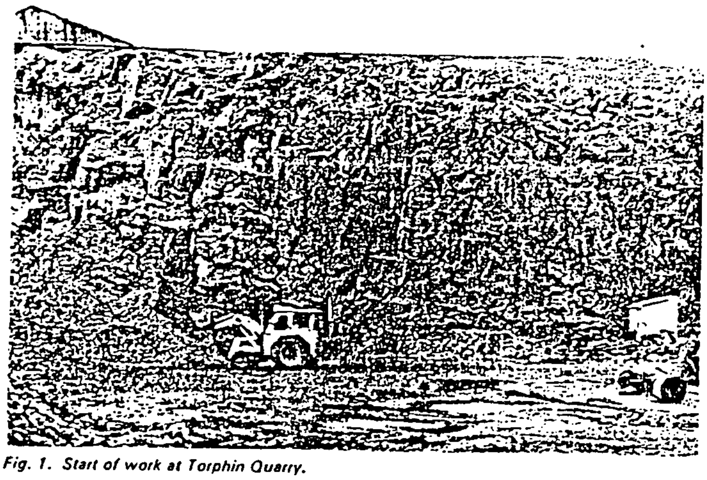


Fig. 1. Start of work at Torphin Quarry.

Concreting of the face was carried up to the top of the cliff to make it suitable for tests on structures up to five storeys in height.

Materials

Bricks

Solid, wire-cut bricks, 2⅝in thick, with an average crushing strength of 4960lbf/in² were used for all the tests. The physical properties are given in Table 1.

Table 1—Physical properties of bricks used in tests

	Range	Average	Standard deviation	Co-efficient of variation (%)
Compressive strength (lbf/in²)	4263–5968	4963	569	11·46
Suction rate (g/dm²/min)	15·20–21·59	16·47	2·33	14·0
Water absorption, 24-hr soak	13·97–16·0	15·05	0·7	4·7

Cement and Sand

A rapid-hardening cement, Ferrocrete, was used for all the tests. Ordinary building sand was used. The sand and cement conformed to BS 1200:1955 and BS 12:1958.

Mortar

A 1:¼:3 cement/lime/sand mix (by volume) was used for the construction of all the test walls. The materials were gauged in wooden measuring boxes and thoroughly mixed in the dry state before any water was added.

Six 4in mortar cubes were prepared from each set of test structures. The moulds were filled in two layers and compacted by tamping with a 1in square bar in a uniform manner. Out of six cubes, three were left in the quarry near the test structure and three brought to the laboratory and kept under water till tested. Generally, the cubes stored at the quarry in atmospheric conditions developed higher strength than those in the laboratory, except in two instances when the cubes were cast during warm weather: drying out of these cubes may have resulted in incomplete hydration of the cement.

Construction details

The test structures were built according to the B Ceram RA Model

Specification (4). There were four courses of brickwork to 1ft and all cross joints and collar joints were filled and flushed with mortar. During rain the work was protected with polythene sheeting.

In the course of building the test structures, three piers and three shear specimens, as shown in Fig. 13, were made for each structure. The piers were for assessing the diagonal tensile strength of the brickwork according to a method proposed by the Structural Clay Products Research Foundation (5).

Method of applying vertical load

The vertical load was applied by jacking against three 80 ton portals, each of which was fixed to the concrete base by eight projecting-type Rawl-bolts going to a depth of 7½in. This type of Rawl-bolt was selected for ease in moving the frames (together with the bolts) with a mobile crane from one test structure to another. Details of the loading rig are shown in Fig. 2a and b.

The loading equipment consisted of three 100 ton Tangye ship-type jacks with a maximum ram travel of 6in. The load from each ram was measured by means of a column-type 200 ton load cell. Each load cell was calibrated in the laboratory in an Avery universal testing machine prior to use. A load from each cell was transmitted to the floor slab of the test structure through spreader beams, as shown in Fig. 2. Rubber sheet, ¼in thick, was put between the rollers of the spreader beam and the slab to distribute the load as evenly as possible.

Method of applying racking load

The racking load was applied by jacking against the quarry face. The horizontal load from five 20 ton jacks was distributed to the quarry face through a ½in thick steel plate to avoid local crushing of the concrete. The load from each jack was measured with a 20 ton ball-seating load cell, also calibrated in the laboratory. The horizontal load was transmitted to the slab through a stiffened I-beam (8in × 5¼in) and rubber packing was inserted between the slab and I-beam to distribute the load evenly.

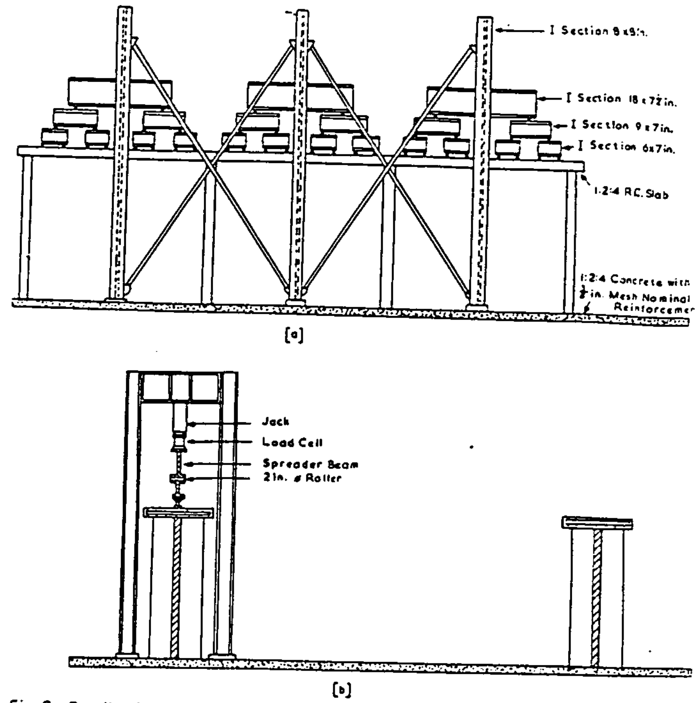


Fig. 2. Details of test structure and vertical loading rig.

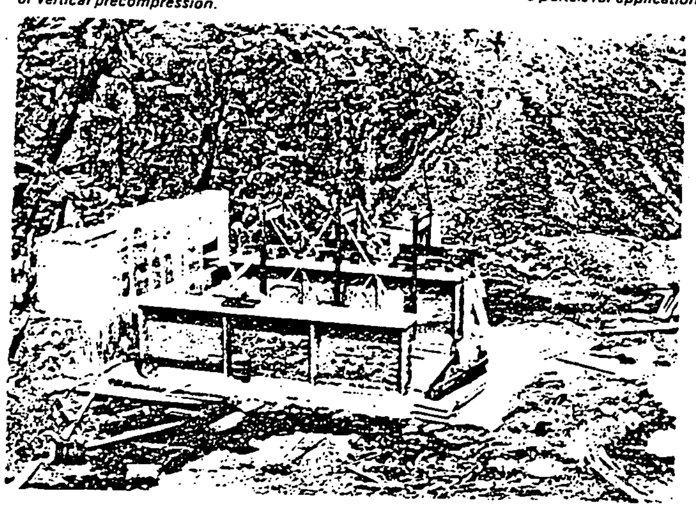
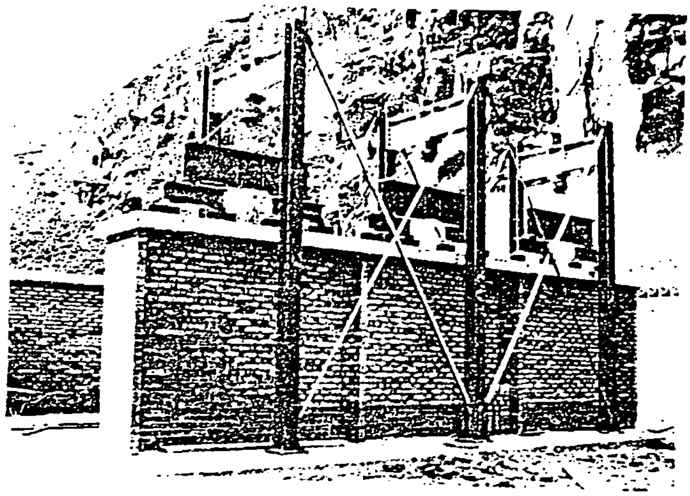


Fig. 3. General view of test site.



Below: Fig. 4. View of test structure showing jacks, spreader beams and portals for application of vertical precompression.

Pumping equipment

All eight jacks were operated by a single, mobile electro-hydraulic pump unit having four outlets with twenty tapping points. The vertical rams were operated by means of three separate controls, whereas the five jacks applying the racking load were connected to one outlet controlled by a single valve. Figure 3 shows a general view of the test arrangements and Figure 4 shows details of the loading jacks.

Experimental results

Tests on structures

Six full-scale structures, three without openings and three with openings, were tested to destruction at various levels of precompression. Table 2 gives a summary of the test results; Fig. 5 shows shear strength plotted against precompression for full-scale and model structures; and Table 3 gives the shear strength of model structures.

Horizontal deflections were measured by means of dial guages at slab level of each structure. In structures three and six deflections were measured at both ends and in structure six measurements were also made in the openings. Typical results are plotted in Fig. 6, 7 and 8.

Diagonal strains were measured by means of 12in and 24in Demec gauges in the central panel of structure two and in panels A and B of structure five. Principal stresses calculated from these measurements are shown in Fig. 9 and 10.

With one exception (structure one) failure took place by the development of cracks passing step-wise through the vertical and horizontal mortar joints and sometimes through the bricks. In the structures with openings, first failure was always in the double panel (B) followed by a crack in the single panel (A) leading to complete failure of the structure. Fig. 11 and 12 show the typical failure of structures.

Tests on shear specimens and piers

The three-brick shear specimens were tested in the laboratory as shown in Fig. 13. Precompression was applied across the bed joints of the specimen by a 6 ton hydraulic jack operated by a hand pump, the load being measured with a 5 ton load cell. The shearing load was applied by the testing machine in which the rig was placed.

The diagonal tensile strength (f_t) was obtained, from the relationship (5) $f_t = 2\sqrt{\sigma_c}$ where σ_c is the ultimate compressive strength of the brickwork piers.

Discussion of results

Strength of panel structures

It may be observed that the ultimate shear stresses plotted in Fig. 5 are based on the ultimate load divided by the total area of the shear panels, although the shear stress will not be uniformly distributed on

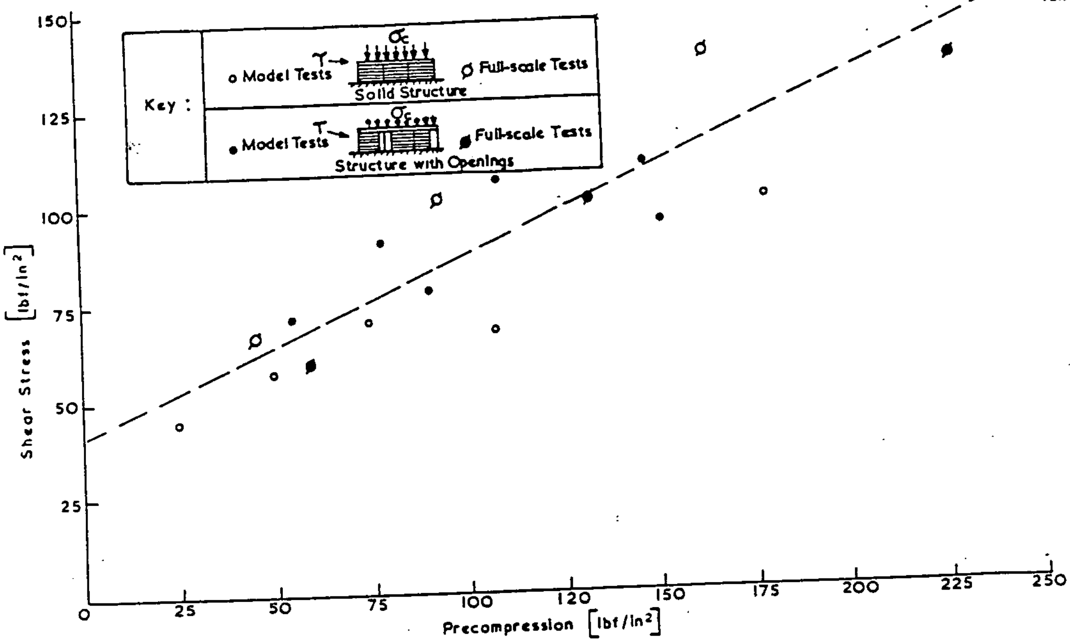


Fig. 5. Shear strength v. precompression for full-scale and model tests.

horizontal sections. A few strain readings were taken and from them the principal stresses shown in Fig. 9 and 10 were derived on assumed values of E and ν. These measurements indicate that panel (A) is the more highly stressed although in both model and full-scale structures initial cracking was usually noted first in panel (B). More detailed investigation would be required to explain this observation.

Examination of the shear strength:precompression results in Fig. 5 suggests that the three full-scale structures without openings were appreciably stronger than those with openings and also gave higher shear strengths than the corresponding models. The results as a whole however, indicate that for a given precompression both model and full-scale walls gave shear strength of a similar order.

Earlier work (1, 2, 3) suggested that there would be a break in the line relating shear strength to precompression with only a small increase in shear strength between precompression values in the range 100–200 lbf/in². The test results for model and full-scale structures are, however insufficiently detailed to confirm this effect—which was explained by the intervention of diagonal tension failure of the brickwork. The diagonal tensile strength of the full-scale brickwork as given by the pier test is practically the same as was found in the model test so that the results should be comparable where failure takes place in diagonal tension.

Strength of shear specimens

The results obtained from the tests on the three-brick shear specimens are plotted in Fig. 14 in relation to Sinha's couplet formulae (1). The

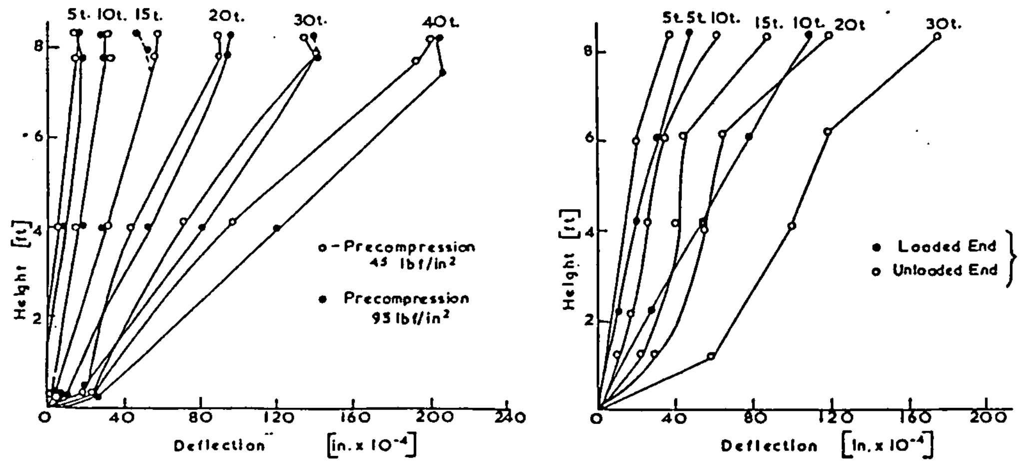
Table 3—Shear strength of model brickwork structures

Models without openings		Models with openings	
Precompression (lbf/in²)	Ultimate shear strength (lbf/in²)	Precompression (lbf/in²)	Ultimate shear strength (lbf/in²)
25	44·1	55	72·3
50	56·7	78	91·1
75	70·5	109	107·0
103	67·7	151	96·6
—	—	147·5	111·0

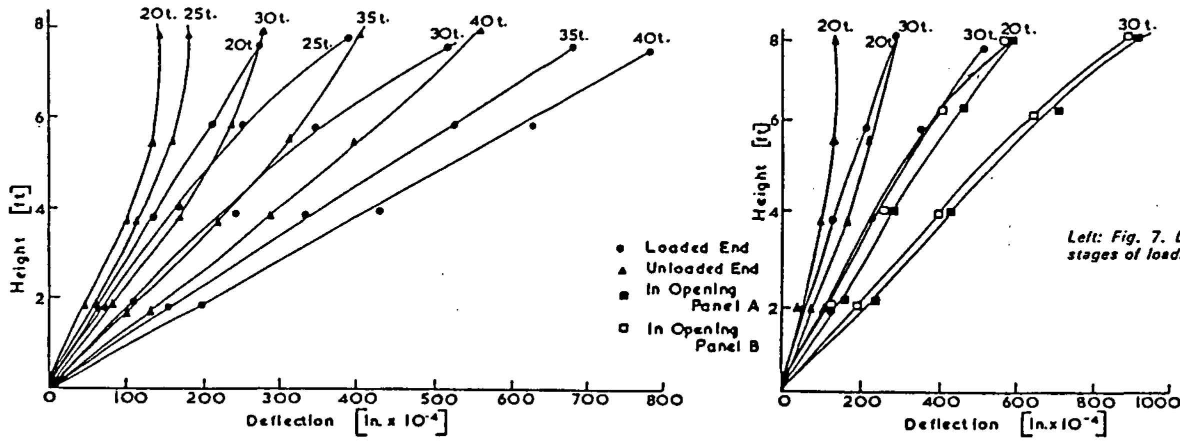
Table 2
Shear strength of full-scale storey-height shear-wall structures subjected to precompression

Test No.	Average compressive strength of mortar (lbf/in²)	Normal compressive stress (lbf/in²)	Ultimate racking load (tons)	Ultimate shear stress (lbf/in²)	Max. permissible stress according to CP:111:1964 (lbf/in²)	Factor of safety over CP:111:1964	Proposed modified shear stress according to tests (lbf/in²)	Load factor after modification
1*	1517 *L* / 1972 *Q*	45	41·5	66·7	20	3·34	22·5	2·97
2*	2415 *L* / 2554 *Q*	93	70·0	102·0	30	3·4	30·5	3·34
3*	2438 *L* / 1729 *Q*	163	95·0	138·6	30	4·6	42·2	3·29
4†	1153 *L* / 849 *Q*	60	28·5	57·4	20	2·9	25·0	2·3
5†	— / 1111 *Q*	133	50·0	101·0	30	3·4	37·2	2·71
6†	1610 *L* / 2088 *Q*	227	68·0	136·5	30	4·55	52·8	2·58

* = Structure without opening. † = Structure with opening. *L* = Laboratory. *Q* = Quarry.



Left: Fig. 6. Deflection of structures 1–3 (without openings) at different stages of loading.

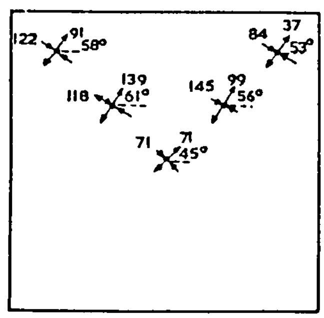


Left: Fig. 7. Deflection of panel at different stages of loading (precompression 227lbf/in²)

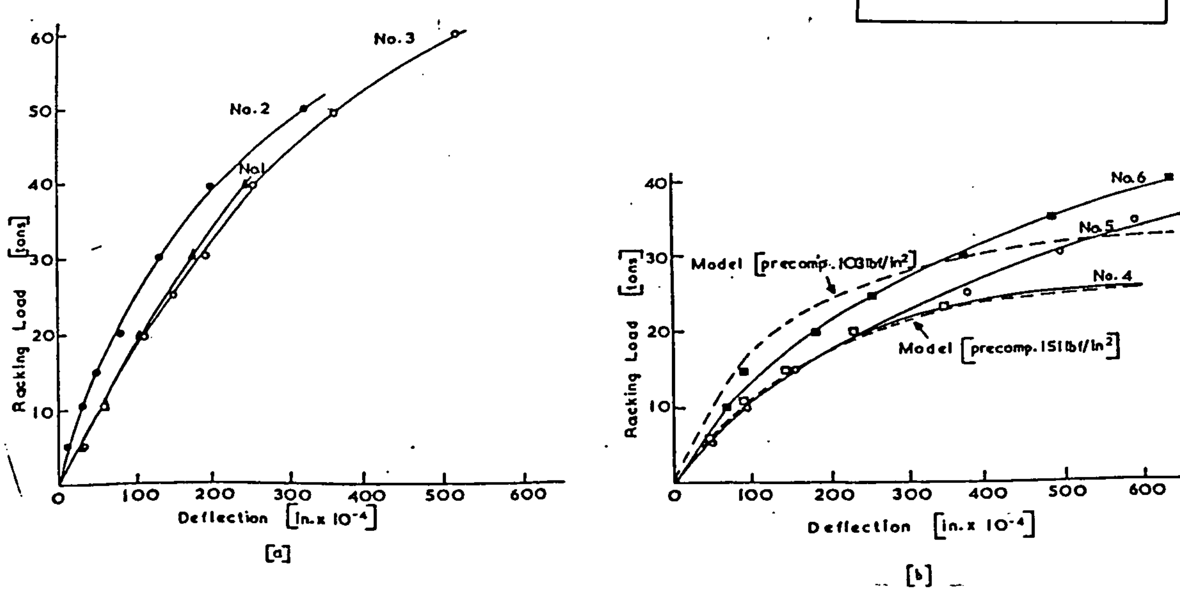
results might be considered as being consistent with the couplet formulae but the scatter at higher values of precompression is so great that it would be unwise to draw any definite conclusions from them. This type of test, however, does appear to give an indication of the shear strength of brickwork panels under precompression.

Deflections of full-scale and model structures

The readings obtained in the tests show that there is an appreciable difference in the deflections measured at the end of the structure at which the load was applied and at the unloaded end. As may be seen from Fig. 8 the load/deflection relation is distinctly non-linear, although

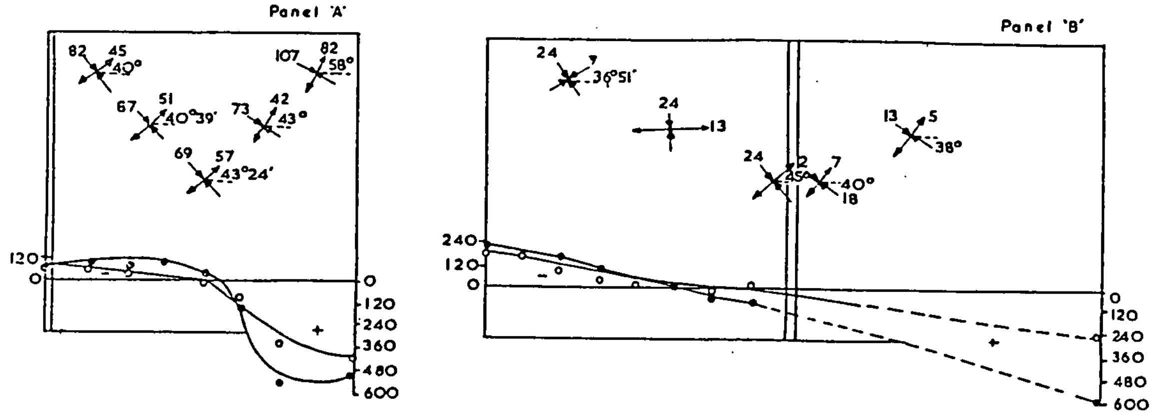


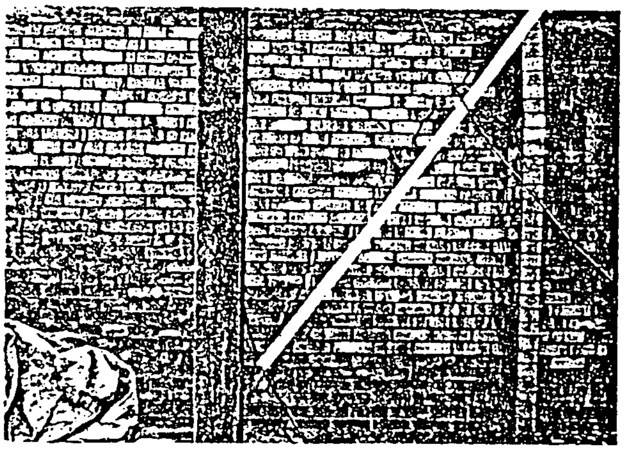
Left: Fig. 9. Showing the stress distribution in middle panel of structure 2 at 20 tons.

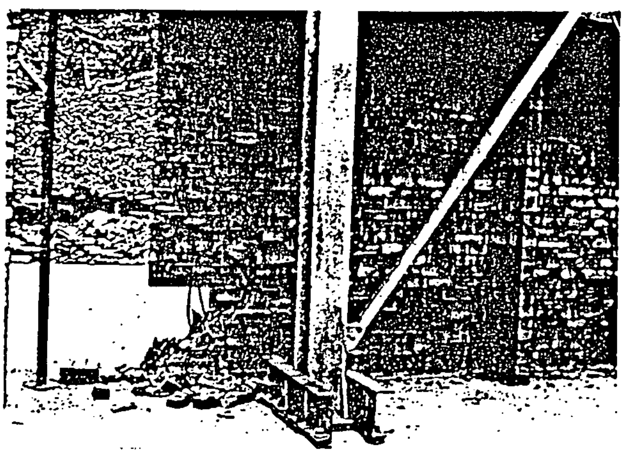


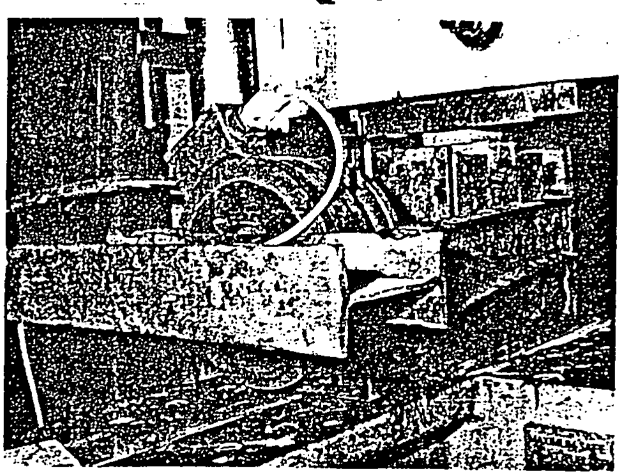
Left: Fig. 8. Horizontal deflections at slab level measured at unloaded end: (a) structures 1–3 (without openings); (b) structures 4–6 (with openings).

Right
Fig. 10. Showing principal stresses at 15 tons and vertical stresses at 30 and 45 tons in structure 6.
Below
Top: Fig. 11. Failure cracks in structure 3 (without openings).
Centre: Fig. 12. Failure of structure 6 (with openings).
Bottom: Fig. 13: Testing rig for couplet specimens set up in Avery testing machine.









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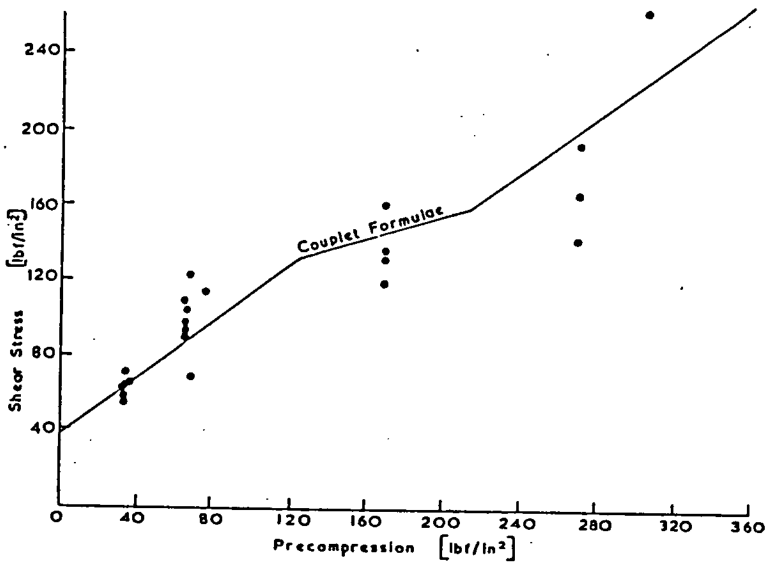
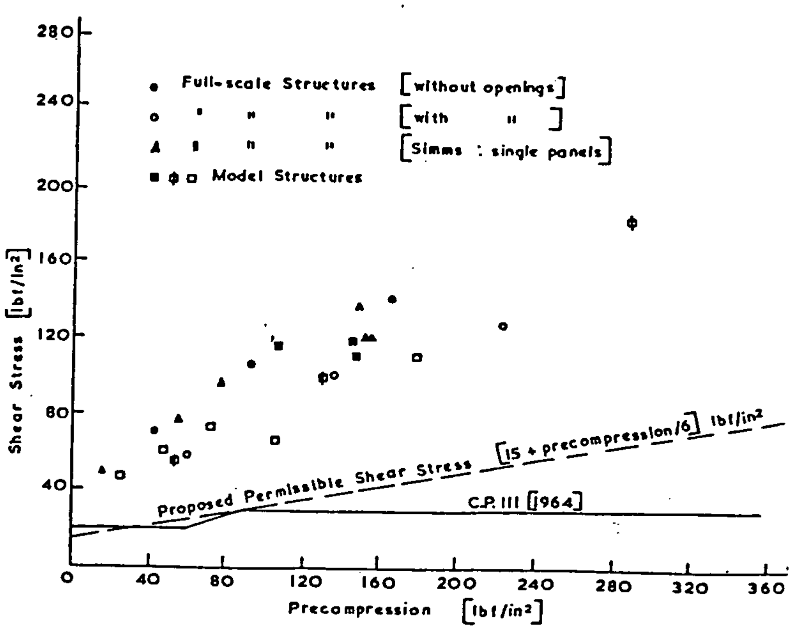
Top: Fig. 14. Results of shear test specimens in relation to Sinha's couplet formulae.

Bottom: Fig. 15. Test results for panel stuctures and proposed permissible shear stress.





elastic behaviour could be assumed as an approximation up to, say, one-third of the ultimate load. There does not seem to be any regular pattern in the relation between precompression and deflection at slab level and there is a wide range of effective values of the shear modulus. The form of the load/deflection curve obtained by scaling the model results to correspond with full-size is generally similar to those obtained in the full-scale tests but again there is no consistent correlation with precompression. The calculation of deflections of brickwork structures in the working load range is therefore unlikely to be very certain.

Permissible shear stresses in brickwork

In order to examine the question of permissible shear stresses for brickwork under precompression, relevant test results for panel structures have been collected and are plotted in Fig. 15. Also shown are the current shear stresses for brickwork in 1:¼:3 mortar according to CP.111:1964. There is reasonable consistency between all the recorded results which include model and full-scale tests and a variety of panel shapes and sizes. Examination of this data, with the adoption of a load factor of approximately 3 on the mean of the experimental results, gives a possible permissible shear stress equal to 15lbf/in^2 plus one-sixth of the vertical precompression, as indicated in Fig. 15. This has been fully validated up to precompression values of about 250lbf/in^2 and applies only to brickwork built, in wire-cut bricks.

Conclusions

1. The full-scale test results give substantial confirmation of the model tests.
2. The behaviour of the model brick structures is generally similar to that of the corresponding full-scale structures but the deformation of the models at higher shear stresses is proportionately greater.
3. Failure of structures, with and without openings, under racking loads was due to the breakdown of the bond at the interface of the brick and mortar leading to either diagonal cracking or to cracks passing through the bed joints.
4. A possible suggestion for permissible working stresses in shear for brickwork built in wire-cut bricks and set in 1:¼:3 mortar based on the present tests would appear to be 15lbf/in^2 plus one-sixth of the precompression.
5. Shear tests on three-brick specimens with precompression give an approximate estimate of the shear strength of full size brickwork panels.
6. Although additional work both on model and full-scale structures might be required to elucidate differences between model and full-scale tests, and solid walls and those with openings, nevertheless it is unlikely that further results of this nature could significantly change the recommendations of Fig. 15. However, the actual value to be taken for the load factor might have to be modified.

Acknowledgment

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SOME ASPECTS OF THE DESIGN OF BRICK STRUCTURES FOR WIND LOADING

by

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1. Introduction

In recent years, there has been increasing world-wide interest in load-bearing brick walls for non-framed cross-wall type of construction. A multi-storey building of this type has to be designed to resist lateral loads normally due to wind or due to seismic action in the earthquake zone. The resistance to the lateral forces caused by wind and earthquake is provided by the shear or cross-wall due to their shearing resistance and resistance to overturning as shown in Fig. 1. The paper deals only with some aspects of the design of brick structure subjected to wind loading. In a typical brick building the resistance to wind loading is provided mainly due to bending by the facade panel (Fig.2), which in turn transfers the load through the floor slabs to the cross or shear walls. Thus in the design for wind loading the following problems need to be considered:

i) Overall stability of the building

ii) The bending resistance of individual wall panels

The design of the panels subjected to wind loading will be dealt with in other sessions, hence only overall stability of the building is considered in this paper.

2. Overall Stability

If the cross-wall structure in Fig. 3 is examined, it can be seen that the structure is stable in the direction Y, but not in x-direction. The lateral load in x-direction will cause this structure to collapse like a "house of cards." To provide stability, and to stop such failure, adequate length of walls (cross or shear walls) parallel to the direction of the loading must be provided in both major axes. In addition, the floor and roof must be strong and stiff to transfer the load by diaphragm action. The designer must also ensure that the joint between the horizontal diaphragm and the walls are of adequate strength to resist the shearing stresses set up due to the loading.

In a brick building, walls can be arranged in a number of ways to provide for the overall stability. However, three basic patterns [1,2] (Fig. 4) can be identified according to the arrangement of the walls in the buildings:

i) Simple or double cross-wall system: The main cross-walls are load-bearing elements which also provide adequate resistance to wind forces parallel to them. Lateral stability in the other direction is provided by the corridor and flank walls.

ii) Cellular: Internal and external walls are both load-bearing and they also provide adequate lateral stability in both directions

iii) Complex: A combination of both systems as illustrated in Fig. 4. The laterals stability in two directions is provided by the wall systems or by the service core.

At the planning stage, the designer should try to avoid as far as possible very unsymmetrical wall arrangements. The building (Fig. 5) with unsymmetrical wall arrangements is subjected to torsion due to wind loading, which may produce undesirable stress condition in some walls.

3. Theoretical Methods for Wind Load Analysis

The calculation of the lateral stiffness and stresses in a system of symmetrically placed shear walls without openings subjected to wind loading is straightforward and involves simple bending calculations only. Shear walls containing openings present a much more complex problem. The complexity and the difficulty increases even more with a three dimensional multi-storey structure. The three dimensional structure is replaced and idealised by a hypothetical similar two dimensional structure for analysis. Generally, five methods are used for the analysis and design of shear walls with openings:

i) Cantilever Approach

ii) Equivalent frame

iii) Wide column frame

iv) Continuum and v/finite Element

Fig. 6 shows the idealisation of shear walls with openings for these methods.

3.1 Cantilever Approach

The structure is assumed to consist of a series of vertical cantilever walls. The floor slabs are assumed to act as a strut pin connected to the walls and thus only capable of transmitting the direct forces. The wind moment is apportioned to the walls according to their flexural rigidities. This is the most commonly used method for the design of brickwork structures. However, this over-simplification neglects bending of interconnecting beams or slabs which may require consideration in the design. The deflection of the individual wall is given by:

$$\Delta = \frac{w_1}{EI_1}\left[\frac{x^4}{24} + \frac{h^3 x}{6} + \frac{h^4}{8}\right] \qquad \text{(i)}$$

Also, $$\Delta = \frac{w_2}{EI_2}\left[\frac{x^4}{24} - h^3x/_6 + h^4/_8\right] \qquad \text{(ii)}$$

where $w_1 = \frac{w}{I_1 + I_2} . I_1$ and $w_2 = \frac{w}{I_1 + I_2} . I_2$

w - total uniformly distributed wind load/unit height h

h - height of building

I_1 and I_2 -Second moment of areas

3.2 Equivalent Frame

In this approach shear walls with openings are idealised and replaced by an equivalent frame (3,4). The columns and beams are assumed to have the same flexural rigidities as respective walls and floor slabs. The centroids of the walls become the centre line of the column and the span of the beams is taken to be the distance between the centroidal axes of adjacent columns. Any classical approach can be used for the analysis which may[5] or may not take shear and axial deformations of beams and columns into account. Standard computer programmes can be used for the analysis of the equivalent frame.

3.3 Wide Column Frame

This method[6] is a further refinement of the equivalent frame. The structure is idealised in a similar manner as an equivalent frame with the modification that between the centroid and the face of the shear walls the connecting members have infinite stiffness. Similar methods of analysis as equivalent frame can be used.

3.4 Continuum Approach

In this method, the beams connecting the walls are replaced by an equivalent continuous medium. Further assumptions are made that the medium has a point of contraflexure in the centre; axial and shear deformations are negligible. The moments and shear are assumed to distribute in the walls in proportion to their rigidities. On this basis a second order differential equation for the redundant shear force in the lamina can be expressed according to Coull and Chowdhary[7]

$$\frac{d^2T}{dx^2} - \alpha^2 T = -\beta x^2 \qquad \text{(iii)}$$

where $T = \int_0^x q\,dx$ (iv)

$$\alpha^2 = \frac{12I_p}{hb^3}\left[\frac{l^2}{I} + \frac{A}{A_1 \cdot A_2}\right] \qquad \text{(v)}$$

$$\beta = \tfrac{1}{2}\, w l \left(\frac{12I_p}{hb^3}\right)\frac{1}{I} \qquad \text{(vi)}$$

$$I = I_1 + I_2 \qquad \text{(vii)}$$

$$A = A_1 + A_2 \qquad \text{(viii)}$$

By putting appropriate boundary condition, ie for walls on rigid foundation, the value of T becomes:

$$T = \frac{2\beta}{\alpha^4}\left[1 + \frac{\text{Sinh}\,\alpha h - \alpha h}{\text{Cos h}\,\alpha h} \cdot \text{Sinh}\,\alpha x - \text{Cos h}\,\alpha x + \frac{\alpha^2 x^2}{2}\right] \qquad \text{(ix)}$$

Once the redundancy of system is found out the bending moments in wall 1 and 2 are given by:

$$M_1 = \left(\tfrac{1}{2} wx^2 - Tl\right)\frac{I_1}{I} \qquad \text{(x)}$$

$$M_2 = \left(\tfrac{1}{2} wx^2 - Tl\right) \cdot \frac{I_2}{I} \qquad \text{(xi)}$$

and the deflection will be

$$EI\,\frac{d^2y}{dx^2} = \tfrac{1}{2}\, wx^2 - Tl \qquad \text{(xii)}$$

For uniformly distributed load, the deflection at any section x from the top will be given by

$$y = \frac{1}{2}\frac{wh^4}{EI}\left\{\left[1/4 - 1/3\ {}^{x}/_{h} + 1/12\left(\frac{x}{h}\right)^4\right]\left[1 - \frac{1}{\mu}\right] - \frac{2}{\mu}\left[\frac{\left(\frac{x}{h}\right)^2 - 1}{2\,(\alpha h)^2} + \frac{\alpha h(\sinh \alpha h - \sinh \alpha x)}{} - \frac{\cosh \alpha\,(h-x) + 1}{(\alpha h)^4 . \cosh \alpha h}\right]\right.$$

where $\mu = 1 + \frac{A}{A_1 + A_2} \cdot {}^{I}/_{\ell^2}$

The continuum approach has developed from the pioneering work of Chitty[8] and various authors[9,10,11,12 & 13] have applied and extended it to shear wall with openings. Basically, the approach of all these authors are the same except for the choice of redundant function. Design charts[7,14] are available for rapid evaluation of the maximum deflection and stresses in the interconnecting shear walls.

3.5 Finite Element

All the above methods of analysis of shear walls assume linear elastic behaviour. The finite element method which is a powerful tool for the analysis of such structures enables non-linear effects to be included. In the finite element approach the walls are divided into a large number of rectangular or triangular elements which are assumed to be connected at nodes. The equilibrium equations at each node are expressed in matrix form and solved with the aid of a computer. Suitable computer programs are available which can deal with any complex structure. However, this may be costly and unnecessary in practical design situations.

4. Experimental Investigation

A good deal of analytical work has been done to determine the stresses and deflection of structures subjected to lateral loading, but very few tests have been carried out to establish the validity of these methods for practical structure. A five-storey structure was therefore tested in 1/6th scale bricks prior to expensive and time consuming full scale tests to study[15] the behaviour under wind loading. This was preliminary to more extensive tests[16] which were carried out in similar full-scale structure in a 'disused' quarry at Torphin in Edinburgh. The five-storey structure(Fig7)consisted of three pairs of 112.5 mm (4½") crosswalls and two pairs of corridor wall of similar thickness, bonded to crosswalls. The walls were built with 34.6 N/mm^2 brick in 1:1/4:3 (cement:lime:sand) mortar. The floor slabs were composed of 50mm thick precast panels with 75mm in-situ concrete topping. The loads were applied at each floor by three jacks reacting against the quarry face which was lined with concrete to provide a plane surface. The load from each jack was measured by load cells connected to a digital voltmeter. The deflection was measured by the dial gauges and strain at various positions on the brickwork at ground level was measured by Demec gauges. The deflection of the structure is reproduced in Fig. 8. The stress distribution obtained from the measurement of strain at the base of the test structure is also reproduced in Fig. 9. The structure [16,17] was replaced by a hypothetical two dimensional structure with appropriate areas and second moment of areas and analysed by all the methods mentioned earlier. The theoretical result of the analysis is given in Fig 8 and 9. The experimental strain and thus the stress distribution was non-linear along the shear walls which cannot be reproduced by most of the theoretical methods based on linear variation of strain with the exception of the finite element method. As a result, these methods do not give accurate distribution of stress in shear walls. The comparison between experimental and theoretical results suggests that the best approximation of behaviour of such brick structure can be obtained by replacing the actual structure by an equivalent frame. The vertical members of such a frame should have the same second moment of area as the walls and the span of the inter-connecting medium is taken between the centroidal axes of the adjacent walls. More rigorous and elaborate methods like finite element give comparable results, but for normal design may be expensive. The wide column frame and continuum methods are not recommended for the analysis and design of such structures as both overestimate the rigidity of the structure several times.

NOTATION

h Total height
h^1 storey height
ℓ distance between centroid axes of two walls
b clear distance between two walls
A_1, A_2 Cross-ssectional area
$A = A_1 + A_2$
$I = I_1 + I_2$ where I_1 & I_2 second moment of area of the walls
y horizontal deflection
x distance measured from top
q shear force intensity
T Total shear force in connecting medium
w intensity of loading /m
M_1, M_2 Bending moments in two walls

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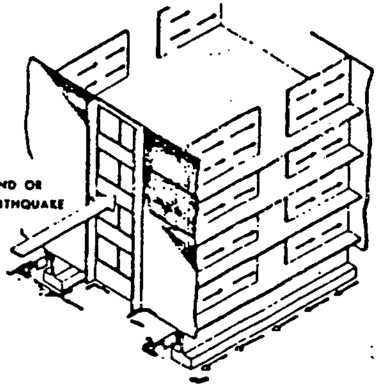


FIG. 1 Shear Wall Action

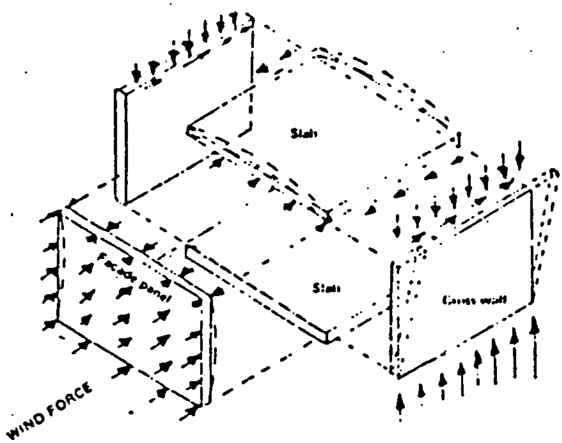
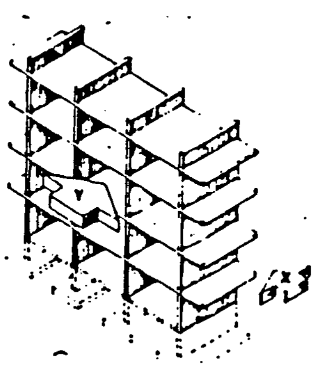


Fig. 2. Showing the action of wind forces on a building. Wind force is resisted by the facade panel owing to bending, and transferred via floor slabs to the cross or shear wall and finally to the ground. (Structural Clay Products Ltd.)



This crosswall structure is, therefore stable in direction Y but not in X

FIG. 3

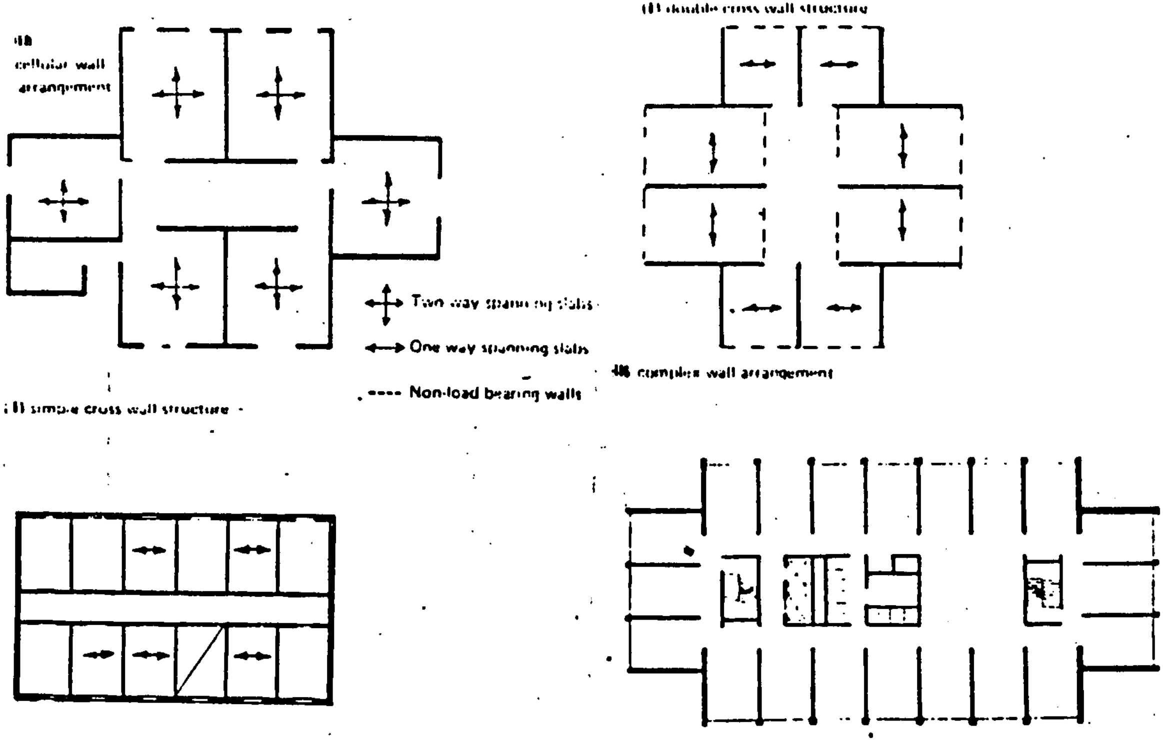


Fig. 4 – Typical wall arrangements in brickwork buildings.

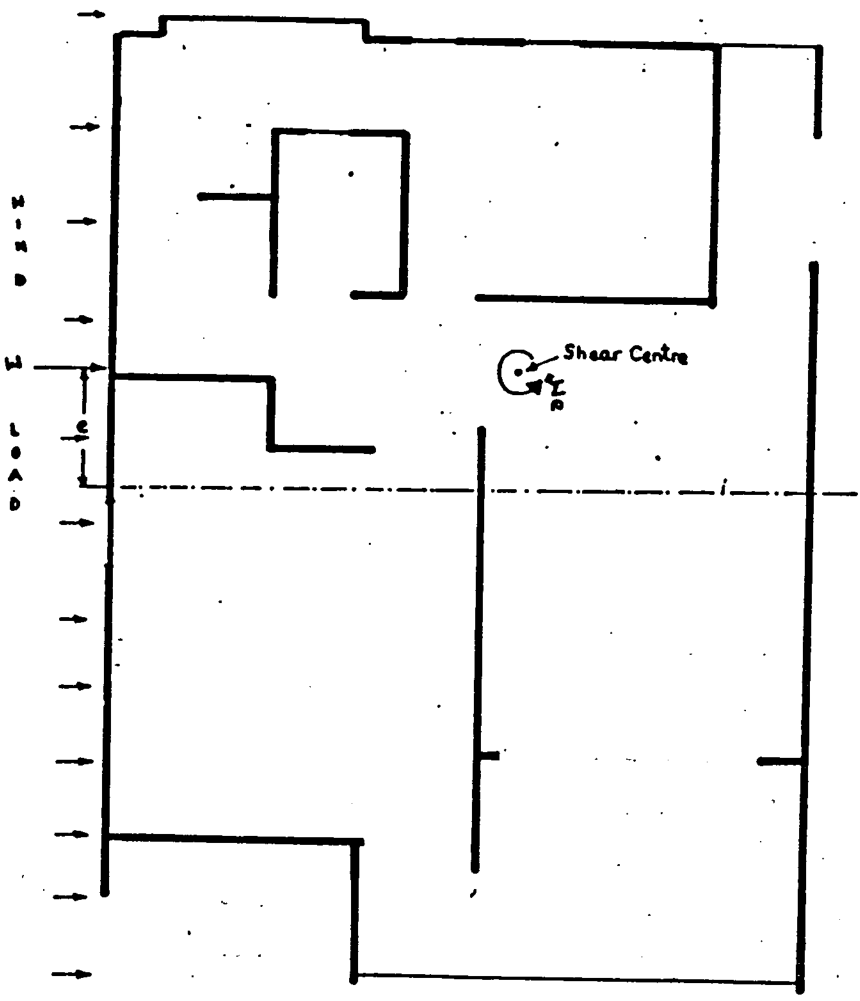


Fig.5 Torsion in unsymmetrical structure.

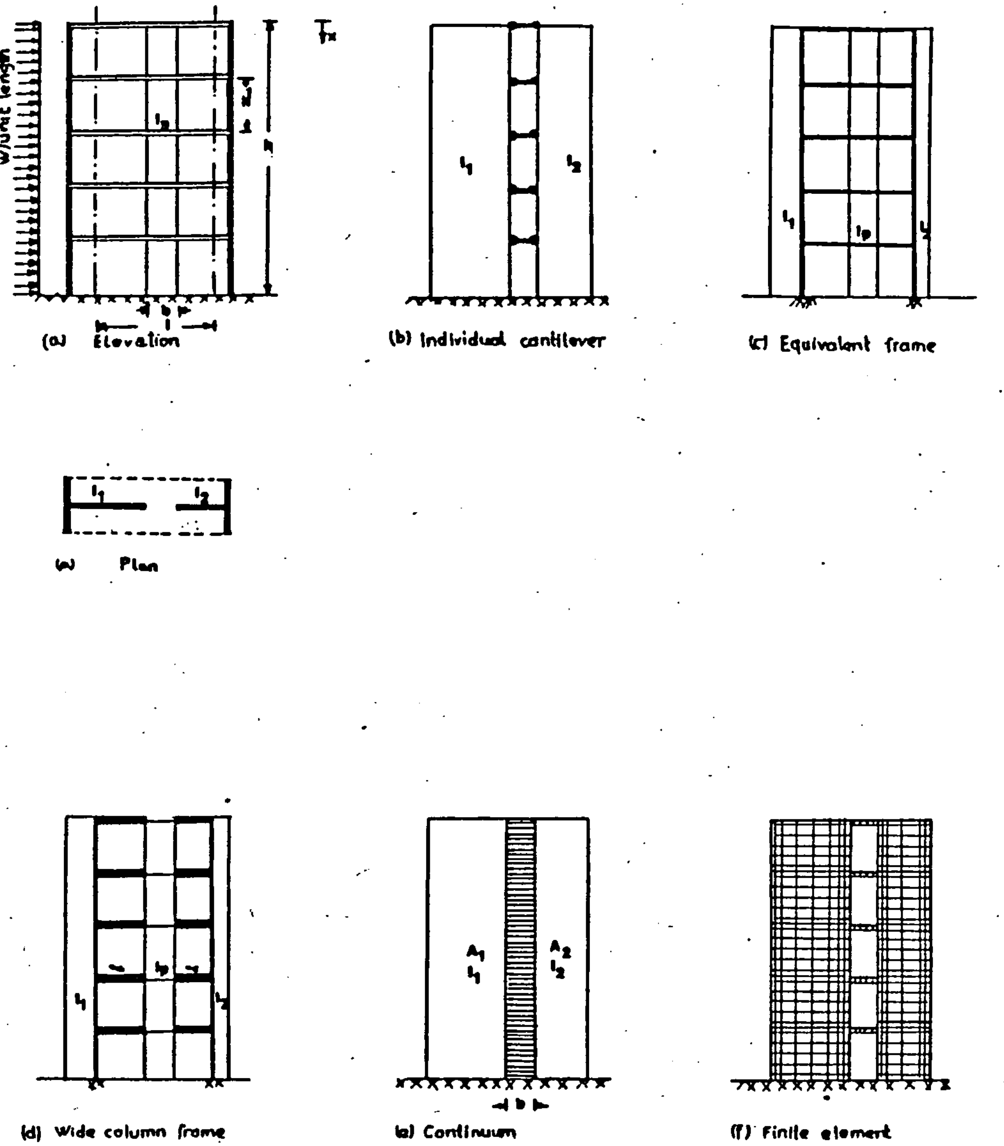
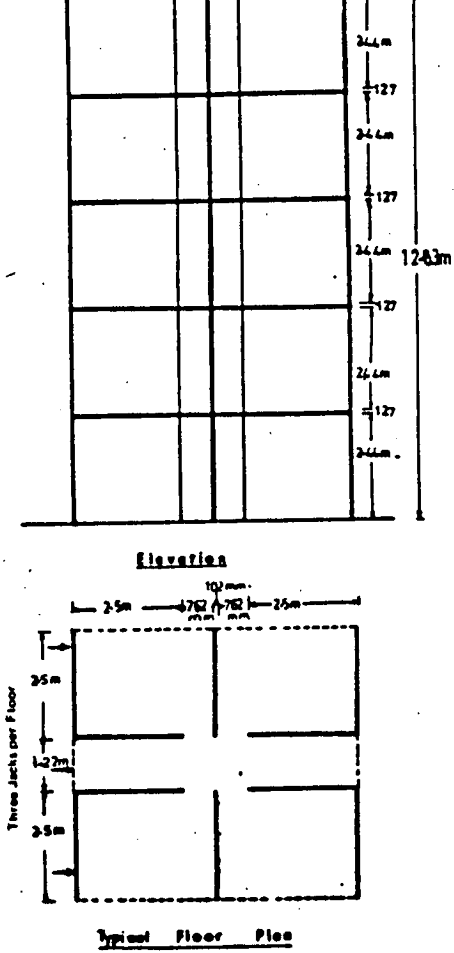


Fig. 6 : Idealisation of shear walls with opening for theoretical analysis



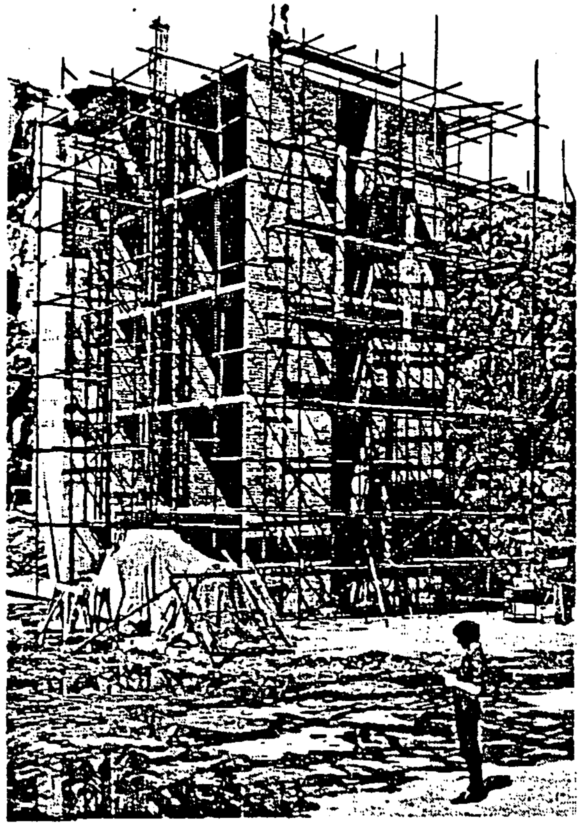


Fig.7 - Test Structure

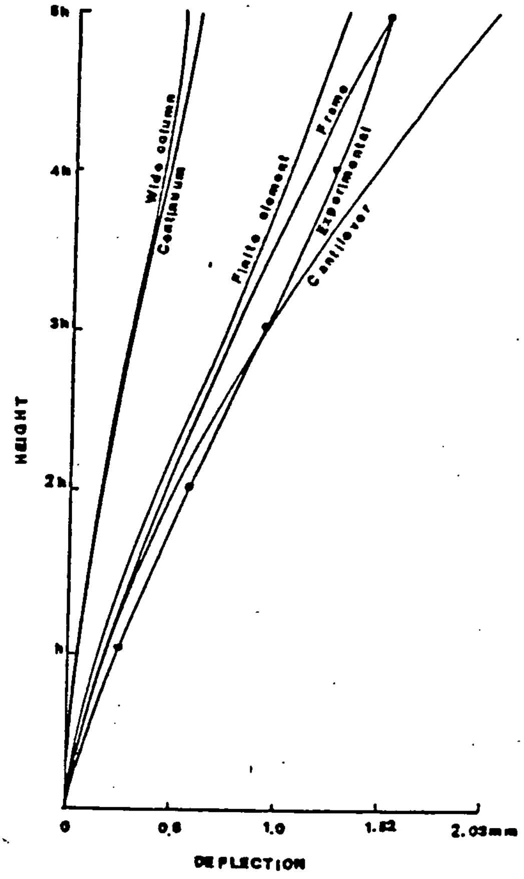
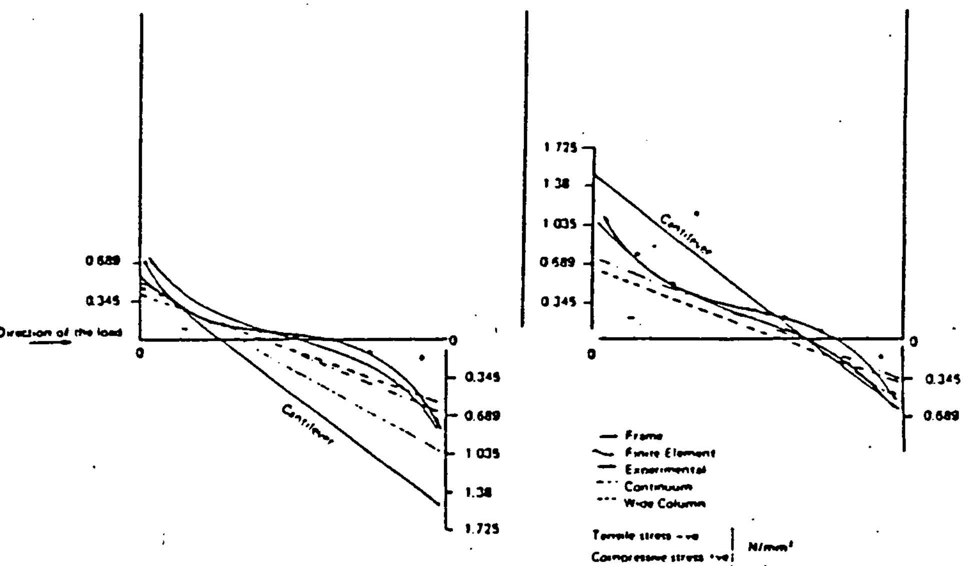


Fig. 8. Comparison of experimental and theoretical deflection results for an equivalent uniform load of 894 N/m^2 over the loaded face of the building.



ig. 9 - Stress distribution across the shear walls at the base for an equivalent uniform load of 894 N/m² over the loaded face of the building (only one-half of the structure is shown).

33.—Model and Full-scale Tests on a Five-Storey Cross-wall Structure Under Lateral Loading

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ABSTRACT

The work described is concerned with the rigidity of, and stress distribution in, a section of multi-storey cross-wall structure. Tests were first carried out on one-sixth-scale model clay brickwork and deflections measured under lateral loading were compared with those obtained by existing analytical solutions. The latter did not yield accurate results, but an approximate method based on the calculation of deflections, storey by storey, appeared more promising. The experiments were repeated on a full-scale structure. The test site and the results of lateral loading tests are described. Comparisons are made between the one-sixth-scale test and the full-scale tests and also with the results of deflection calculations by the approximate method, by the continuum method and by a finite element analysis. On the whole the full-scale test results give substantial confirmation of the model test.

Essais sur Modèle et en Vraie Grandeur sur une Construction en Mur de Refend de Cinq Étages Soumise à une Charge Latérale

Le travail décrit dans cet article est relatif à la rigidité et à la répartition des contraintes dans une section de construction de mur de refend de plusieurs étages. Les essais ont d'abord été effectués sur un modèle de maçonnerie en brique à échelle un sixième et les flèches sous charge latérale mesurées dans ce modèle sont comparées à celles obtenues d'après les solutions analytiques existantes. Ces dernières ne donnent pas de résultats précis, mais une méthode approximative basée sur le calcul des flèches, étage par étage, apparait plus prometteuse. Les expériences ont été répétées sur une construction grandeur nature. Le chantier d'essai et les résultats des essais de charge latérale sont décrits. Des comparaisons sont faites entre les essais à échelle un sixième et grandeur nature et également avec les résultats des calculs des flèches par la méthode approximative, par la méthode continue et par l'analyse d'élément fini. Dans l'ensemble les résultats de l'essai grandeur nature confirment l'essai sur modèle.

Versuche an einem Modell und an einem fünfstöckigen Gebäude mit Querwandkonstruktion unter seitlicher Belastung

Die Arbeit befaßt sich mit der Steifigkeit eines Abschnittes aus einer mehrgeschossigen Querwandkonstruktion und mit der darin auftretenden Spannungsverteilung. Zunächst wurden an einem Model aus Ziegelmauerwerk im Maßstab 1:6 Versuche durchgeführt. Dann erfolgte ein Vergleich der unter Seitenlast gemessenen Biegungen mit theoretisch ermittelten Werten. Letztere liefern keine korrekten Ergebnisse. Aber eine Näherungsmethode, nach der die Biegungen von Stockwerk zu Stockwerk berechnet werden, erschien aussichtsreicher zu sein. Die Versuche wurden an einer Konstruktion natürlicher Größe wiederholt. Bedingungen und Ergebnisse der Versuche mit Seitenlast sind beschrieben. Die Resultate der Versuche am 1:6–Modell und an der Konstruktion in natürlicher Grösse sowie die Ergebnisse der Biegungsberechnung nach der Näherungsmethode, der Kontinuumsmethode und der Analyse endlich großer Elementmengen sind miteinander verglichen. Im ganzen gesehen bestätigen die Resultate des Großversuchs die Brauchbarkeit des Modell–Tests.

1. INTRODUCTION

Recently it has been recognized that masonry can participate in resisting lateral load due to wind and other effects. In the design of tall buildings using current British Codes of Practice wind loading becomes a major criterion so that the advantage of shear-wall construction and the inherent strength of masonry cannot be overlooked. Thus a rapid increase has occurred in the use of high-rise load-bearing brickwork structures in which lateral stability is provided by shear panels. This demands a greater knowledge of the rigidity of brickwork under combined lateral and vertical forces and of the stress distribution in the various elements comprising the structure. In the design of such structures a simple (cantilever) method has normally been adopted in which the lateral moments are apportioned between the shear walls present in proportion to their flexural rigidity. A more refined method, the shear-connection method, takes into account the interaction between the shear walls and the floor slabs; the floors acting as shear connectors with full fixity assumed at the junction with the wall.

The first object of this work was to study the behaviour of a typical multi-storey cross-wall structure at model scale and then to compare the results with those obtained from existing theories. A one-sixth-scale model of a brick cross-wall structure was constructed and loaded horizontally at each floor level to simulate wind loading (Figure 1). The second objective was to compare and verify the results with a similar experiment at full scale. A similar full-scale structure was built in a disused quarry and lateral loading was applied by jacking against the quarry face (Figure 2).

2. MATERIALS

2.1 Bricks

One-sixth-scale model bricks with an average compressive strength of 4221, 3885 and 3435 lbf/in^2 were used in the

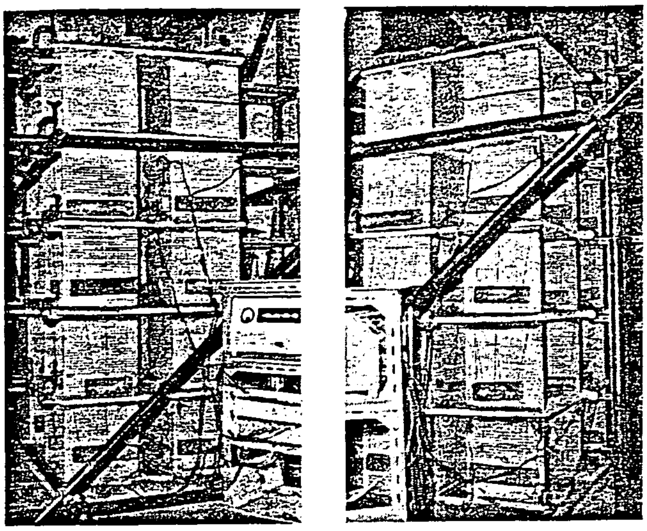


FIGURE 1—Model test arrangement.

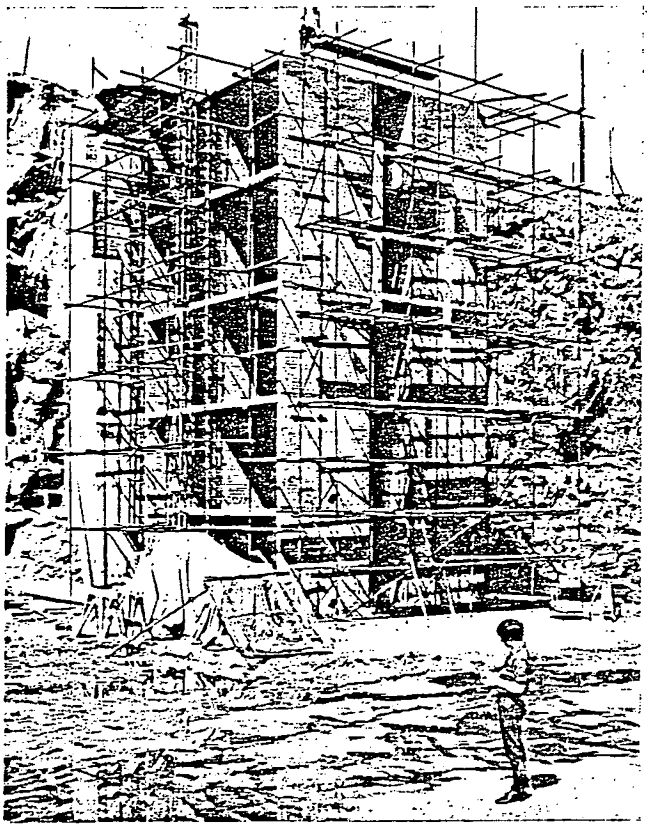


FIGURE 2—General view of the full-scale test structure.

construction of the model walls. Perforated bricks with an average strength of 5020 lbf/in^2 were used for the full-scale building.

2.2 Sand and Cement

Leighton Buzzard No. 19 and ordinary building sand, both conforming to B.S. 1200:1955, were used for the model and full-scale walls respectively. Rapid hardening Portland cement conforming to B.S. 12:1958 was used for both constructions.

2.3 Mortar

1:4 cement:sand mortar by weight (1:3 by volume) was used for the construction of all the model walls. The

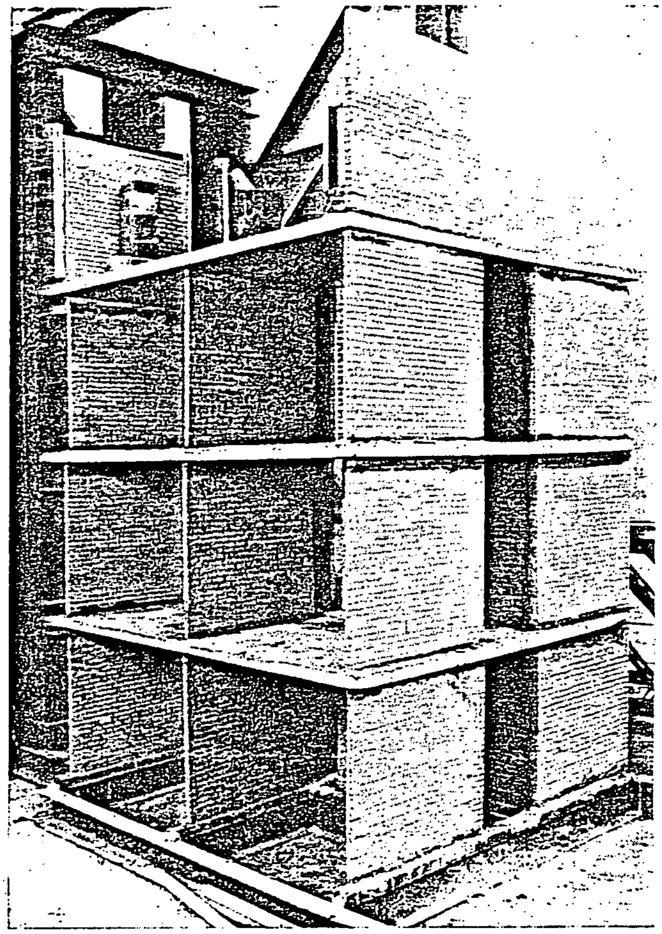


FIGURE 3—Method of assembling the model walls in position.

average compressive strength of the 1-in. mortar cubes was 1619 lbf/in^2 (range 1000–2307 lbf/in^2). A mortar mix of 1:1 was used for the joints between the wall panels and between the walls and floor slabs. 1:$\frac{1}{4}$:3 cement:lime:sand mortar by volume with an average compressive strength of 2100 lbf/in^2 at 28 days (4-in. cubes) was used for the construction of the full-scale walls.

3. CONSTRUCTIONAL DETAILS

3.1 Walls

3.1.1 *Model Wall Panels*

These were built in jigs and assembled as shown in Figures 3 and 4. Whilst assembling, care was taken to see that the walls remained plumb and level. The joints between the walls and between the walls and slabs were filled with mortar. The bottom of each ground-floor wall lay in a 1$\frac{1}{4}$-in. channel attached to the loading frame, the gap between the wall and the sides of the channel being filled with mortar.

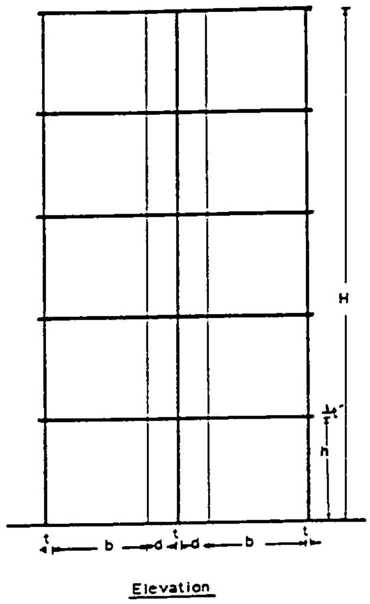
3.1.2 *Full-scale Walls*

The full-scale walls were built according to the B.Ceram.R.A. Model Specification.[1]

3.2 Reinforced Concrete Slab

3.2.1 *Model Slab*

Each slab was made of 1:1:2 concrete by weight. The maximum size of aggregate was $\frac{3}{16}$ in. About 1% reinforcement was provided at the top and bottom of



Dimensions

	Model	Full scale
W	41·5 in.	20 ft $5\frac{1}{4}$ in.
L	44·5 in.	21 ft $9\frac{3}{8}$ in.
C	8 in.	4 ft
b	16 in.	7 ft 10·5 in.
d	5 in.	2 ft 6 in.
t	0·688 in.	4·1 in.
h	17 in.	8 ft
H	7 ft $6\frac{1}{4}$ in.	42 ft 1 in.
t'	1 in.	5 in.

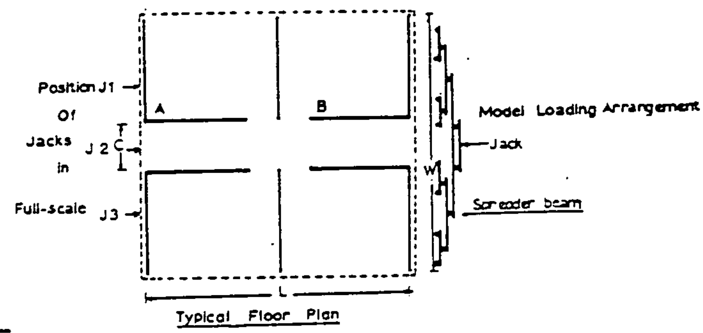


FIGURE 4—Cross-wall test structure.

each slab. The average compressive strength of 6-in. dia. × 12-in. cylinders was 5644 lbf/in² at 7 days and for 4-in. cubes was 7123 lbf/in². The modulus of elasticity derived from cylinder tests was $3{\cdot}2 \times 10^6$ lbf/in².

3.2.2 *Full-scale Slab*

The floor slabs were composed of 2-in. precast 'Omnia Wide Slabs' with a 3-in. *in situ* concrete topping (Figure 5). The result is similar to an *in situ* slab and there is also a saving of shuttering and time. The 2-in. precast sections do not bear onto the walls, therefore a good bond is obtained with the *in situ* concrete. A 1:2:4 ready-mixed concrete having a minimum strength of 3000 lbf/in² at 28 days was used. Mesh reinforcement ($\frac{1}{4}$-in. square twisted bars at 8-in. c/c) was provided in the bottom of the precast slab while similar reinforcement was provided in the top of the slab above supports.

4. ARRANGEMENT FOR APPLYING HORIZONTAL AND VERTICAL LOADS

4.1 Loading Frame for Model

A frame with a total capacity of 8 tons in horizontal loading was specially designed to test the one-sixth-scale, five-storey structure. The horizontal load could be applied simultaneously at all storeys. The details of the frame are given in an earlier report.[2]

4.2 Full-scale Structure

Since it would have been very expensive to build a special frame for applying horizontal load, a disused quarry was developed as a full-size testing station with very limited expenditure, where horizontal load could be applied by jacking against the quarry face.[3]

4.3 Application of Vertical and Horizontal Load to the Model

The estimated dead-weight stress (53 lbf/in²) in the lower storey of the prototype was simulated by lead billets hanging on the walls and lying on top of the slabs. The total weight of the lead billets was 2·02 tons. The slab weight was assumed to be evenly distributed on all the walls.

The lateral load was applied at each floor level with a 6-ton jack through a $7 \times \frac{3}{4} \times \frac{1}{2}$ in. beam of high-tensile steel supported on rollers 6-in. apart.[4] The load from each load-measuring beam was transmitted to each floor slab through a spreader beam (Figure 4). A plywood sheet was put in between the spreader beam and the slab to distribute the load evenly.

4.4 Application of Horizontal Load to the Full-scale Structure

No live load was considered in the full-scale structure. The horizontal load was applied by fifteen 10-ton jacks, three to each floor (Figure 4). The load from each jack was measured by a load cell (3, 5 and 10 ton capacity) calibrated in the laboratory. All the jacks were operated by a single, mobile electric-hydraulic pump unit having four separate outlets, each outlet having five tapping points. Two control valves were used: one for the jacks at roof level which were at half load, and one for the other twelve jacks.

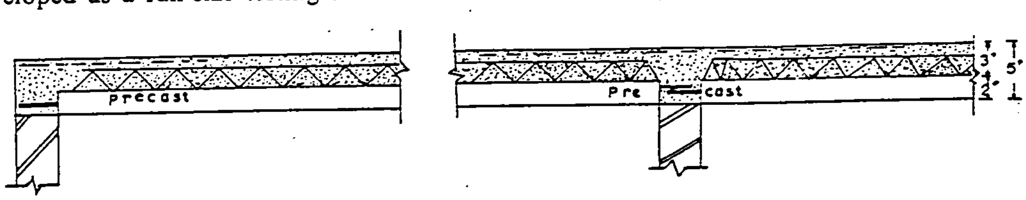


FIGURE 5—Typical section of full-scale slab.

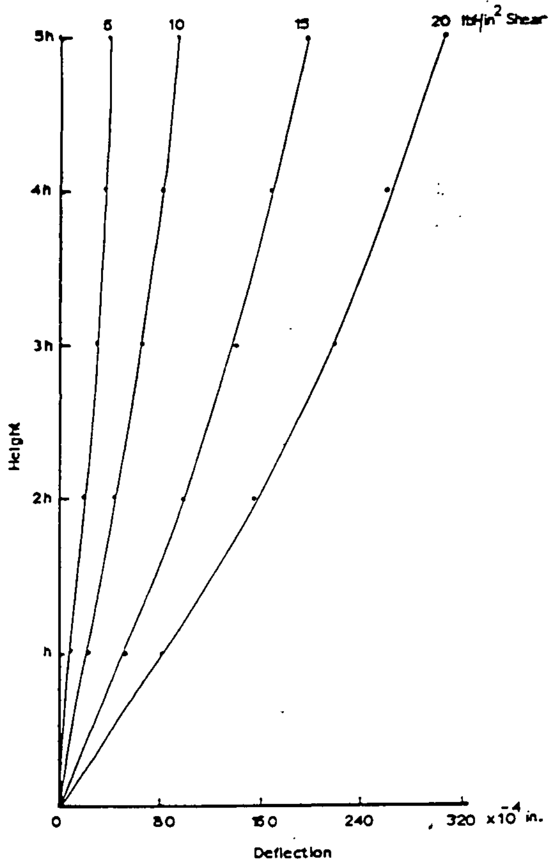


FIGURE 6—Deflection of model structure at various stages of loading.

5. DETAILS OF TEST ARRANGEMENTS

The details of test arrangements for both buildings are shown in Figures 1 and 2. The deflections were measured by dial gauges (0·0001 in. and 0·0005 in). In the full-scale structure, strain was measured with 12-in. and 24-in. Demec gauges.

6. EXPERIMENTAL INVESTIGATION

Lateral load was applied to both structures to investigate the overall deflections and in addition, for the full-scale structure, the distribution of the vertical strains in the ground-floor walls. The lateral load in terms of average shear stress in the bottom shear walls ranged from 5 to 28·4 lbf/in^2. The latter value is 1·5 times the shear produced by a designed wind speed[5] of 106 miles/h and greater than that allowed by C.P. 111 and its proposed modification.[6]

The deflection of both buildings at different stages of loading is given in Figures 6 and 7. The strains for the full-scale building are shown in Figure 8.

7. ANALYSIS OF THE STRUCTURE

The model was analysed by various methods[7–10]—individual cantilever, continuum and wide column—as indicated in Figure 9. The modulus of elasticity was taken as $0{\cdot}98 \times 10^6$ lbf/in^2 as obtained from a test on a small brick beam. It appeared from consideration of Figure 9 that these methods could not be used for the design of brick cross-wall structures without some modification.

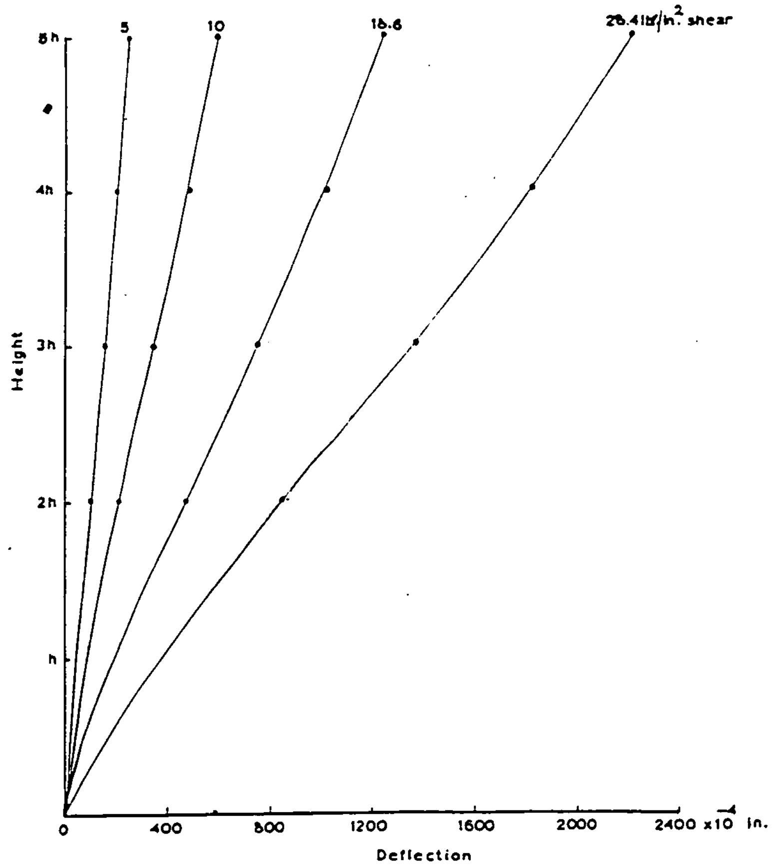


FIGURE 7—Deflection of full-scale structure at various stages of loading

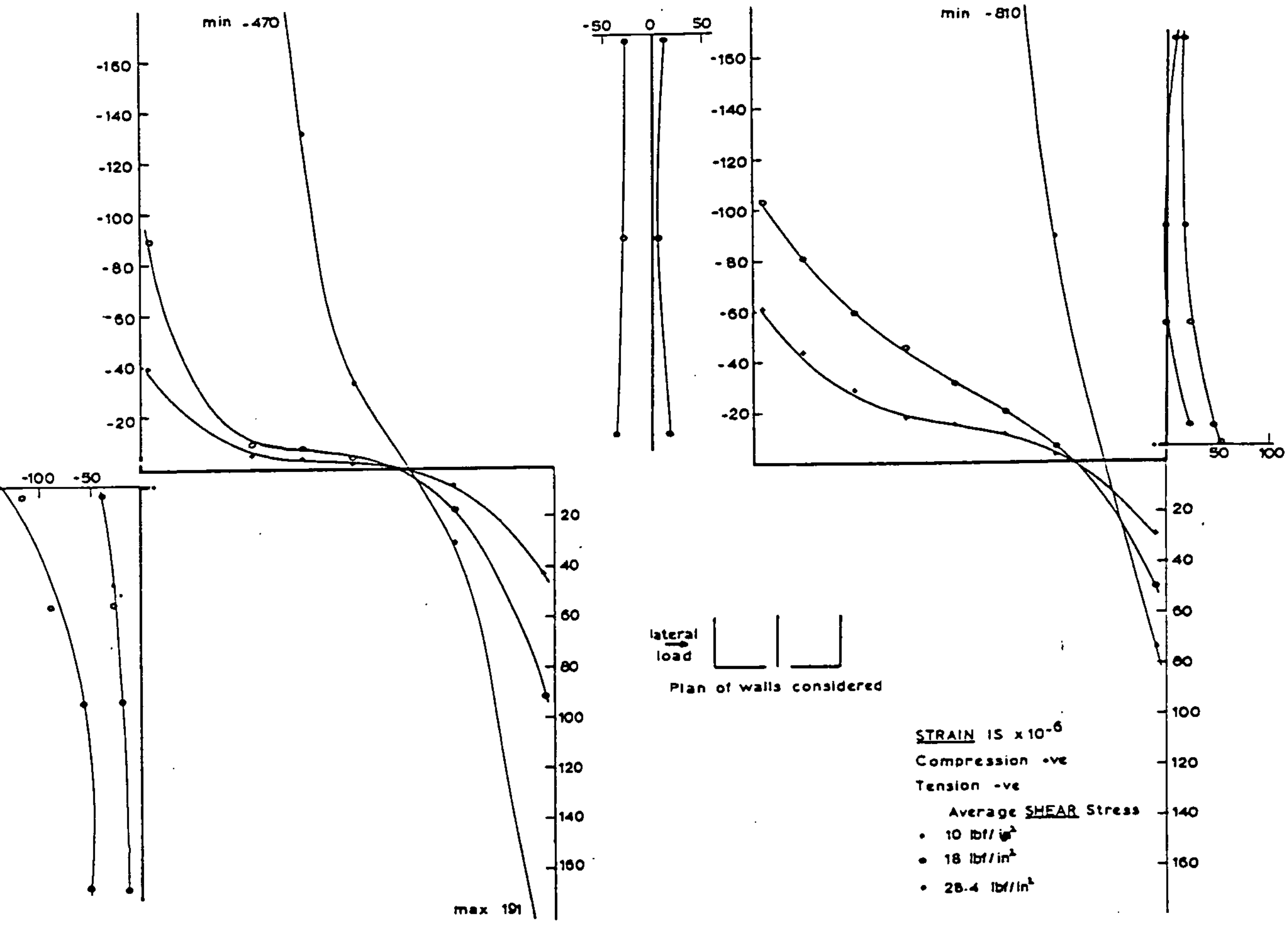
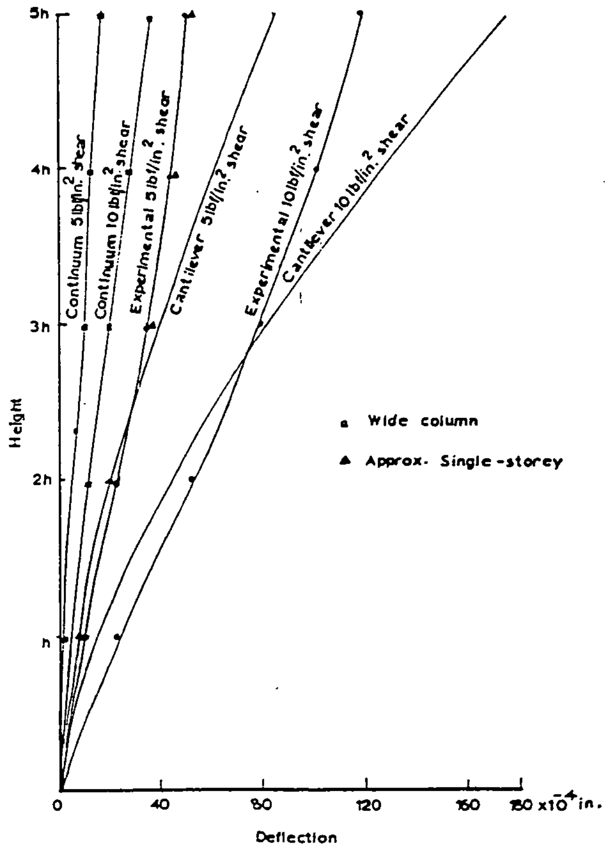


FIGURE 8—Vertical strains in the shear walls, flanges and centre walls.



GURE 9—Comparison of the analytical and experimental deflection results.

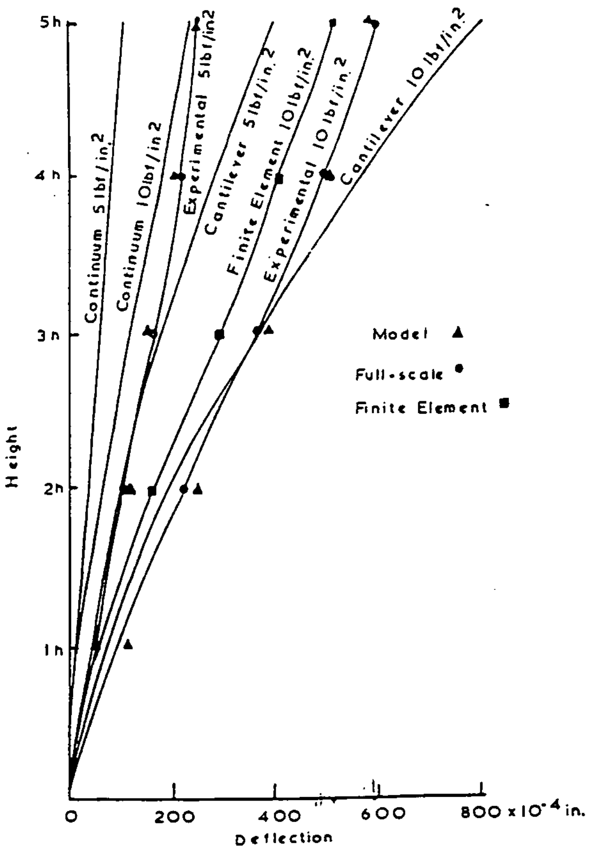


FIGURE 10—Comparison of the analytical and experimental full-scale and adjusted model deflection results.

The single-storey tests[4] show clearly that precompression increases the rigidity of the structure. Thus in the multi-storey structure the modulus of rigidity varied from one storey to another according to the precompression. The value for the modulus of rigidity was calculated from the single-storey test results for various precompressions. The deflection for the multi-storey structure was then calculated by an approximate method[2,7] which takes each storey as a separate unit and the results for each floor are added cumulatively—(shear and bending deflection) per storey. One-sixth of the flange was assumed to act integrally with the shear wall— as assumed in normal design of reinforced concrete L beams. The results, as shown in Figure 9, are promising.

The deflection of the full-scale structure was calculated similarly[8,9] and shown in Figure 10. The modulus of elasticity was assumed to be $1{\cdot}2 \times 10^6$ lbf/in². An analysis by the finite element method using the different values of modulus of rigidity obtained from model tests[11] for various precompressions is shown in Figure 10. Figure 11 compares the full-scale deflections with those obtained by the continuum method assuming various values for the effective width of slab, door opening and flange. Similarly the stress distribution is compared in Figure 12.

8. DISCUSSION

A comparison is made between full-scale and model deflections in Figure 10, the latter being increased by a scale factor of 5 taking into account the layout dimensions. Both curves have similar shapes and for low stresses up to 10 lbf/in² agree well, but with increasing stress the model results greatly exceed the full-scale results. A possible explanation is the difference in precompression between the two structures—the model having 53 lbf/in² while the full scale has 67 lbf/in². In the full-scale structure tensile cracking started in the range 10–18 lbf/in² shear, therefore this cracking would start earlier with a lower precompression and cause the increase in deflection.

Figure 13, showing deflection versus shear stress, shows that the load/deflection curve is non-linear, this effect increasing at higher stresses. Again this may be partly due to tensile cracks.

The strain distribution shown in Figure 8 is typical of a deep beam at low shear stresses. Tensile cracking occurs at high shear stresses, the effect becoming very marked above 18 lbf/in². In full-scale tests at 28·4 lbf/in² the cracks in the top of the bottom course in panel B were visible along the horizontal mortar joints in the tension zone. In practice these cracks will appear later

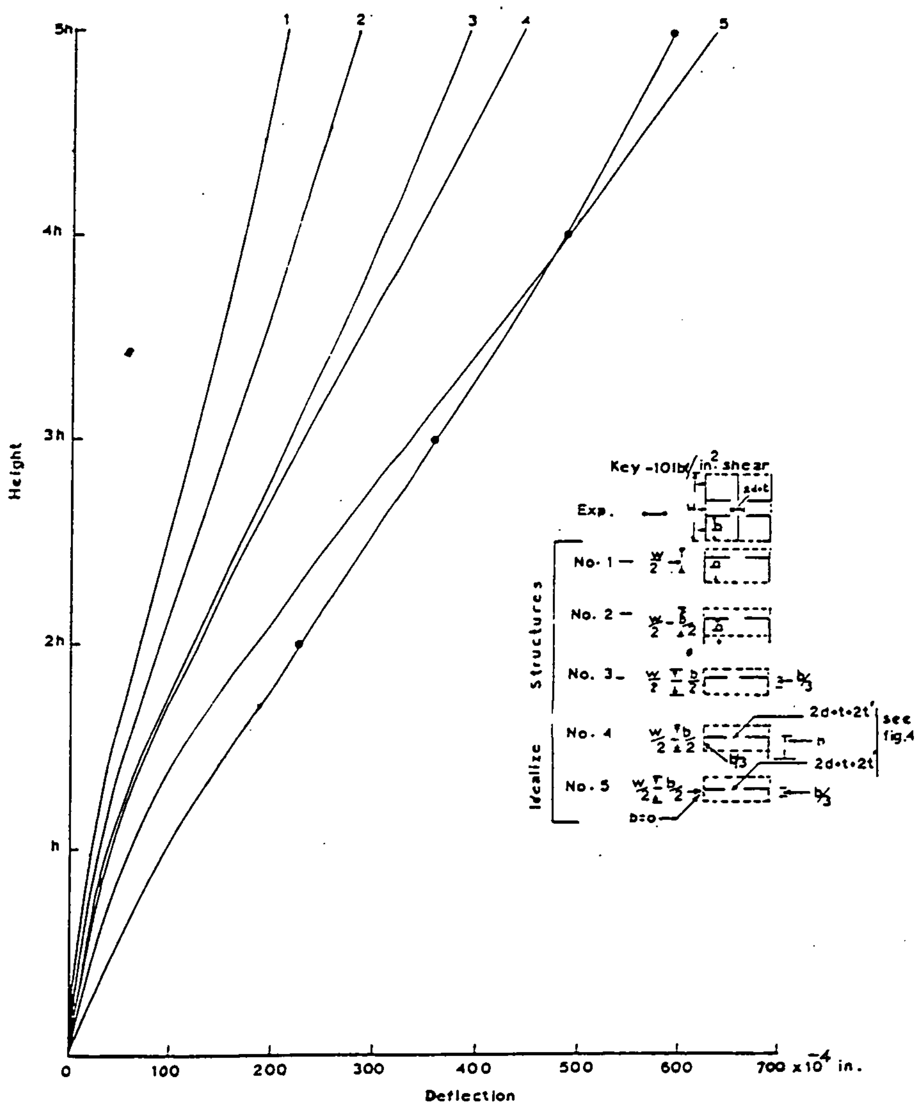


FIGURE 11—Comparison of the experimental deflection results with those obtained by the continuum method, assuming various effective slab and flange widths.

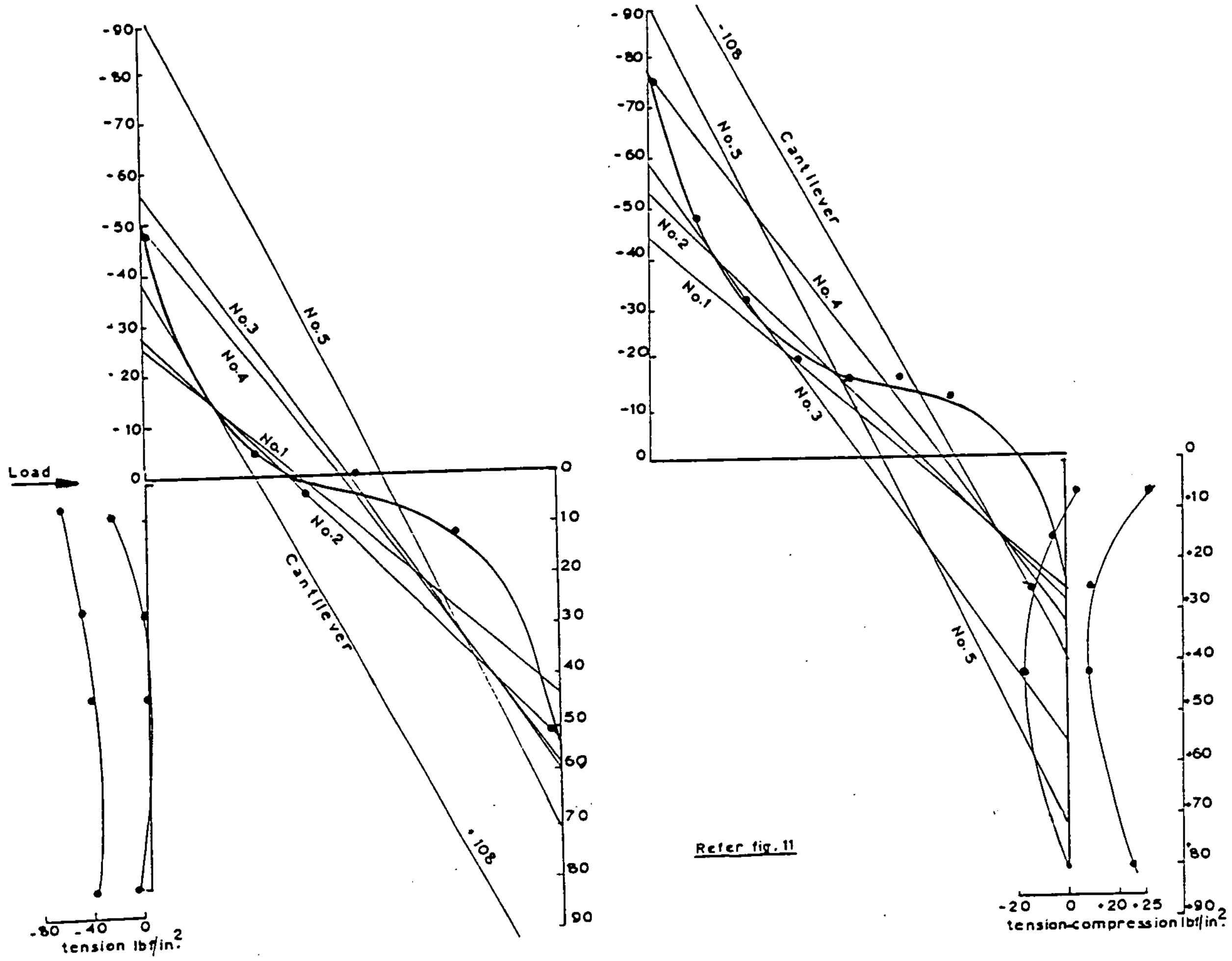


FIGURE 12—Experimental and analytical stress distribution in the shear walls and flanges.

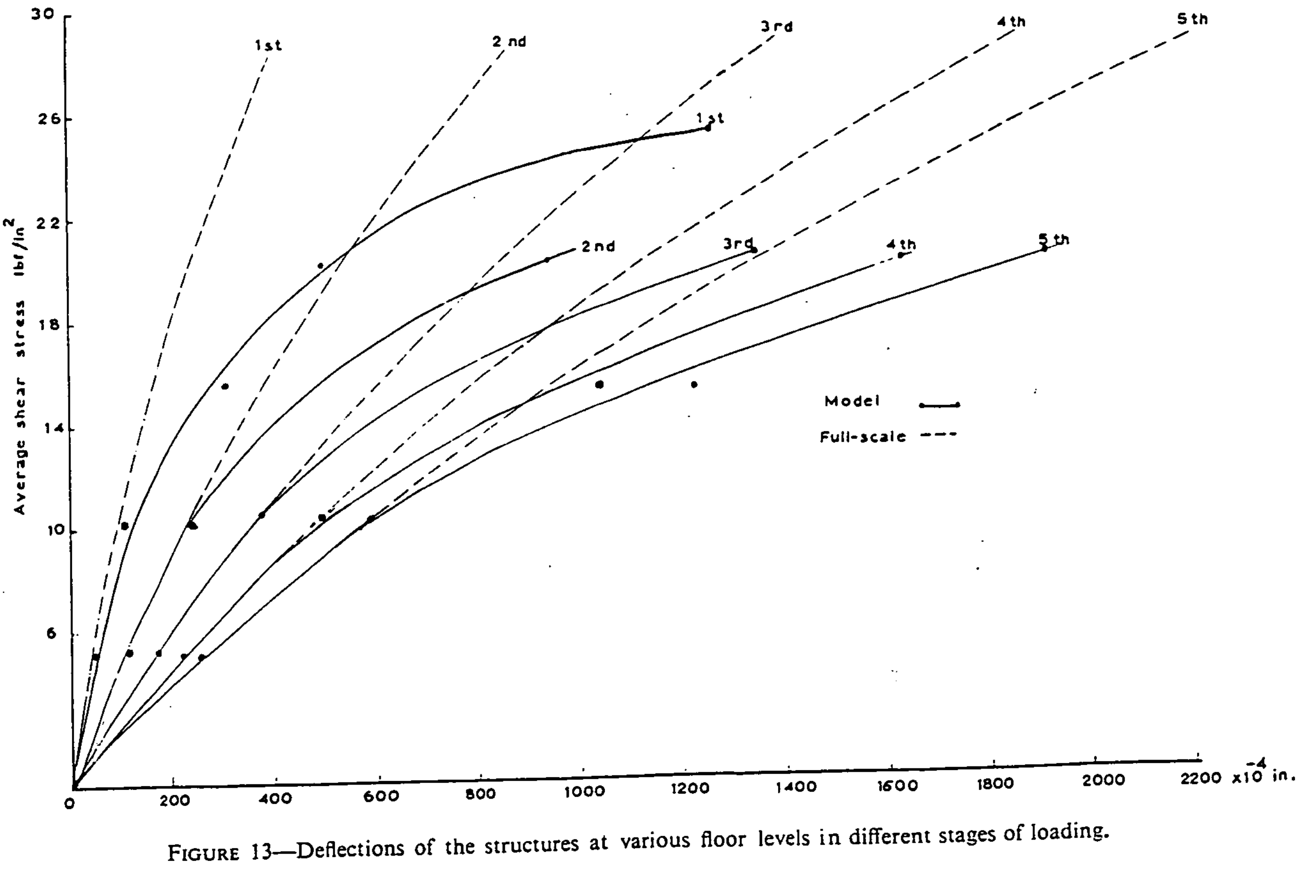


FIGURE 13—Deflections of the structures at various floor levels in different stages of loading.

since live load has been neglected here. The strain in the flanges and centre wall are shown for both sides of the wall, the strains in the centre wall mainly due to the horizontal movement of the slab causing bending stresses (tension and compression).

Most theoretical methods, except finite elements, assume a linear strain variation along the shear wall and thus will not give accurate results. The theoretical deflection curve for the individual cantilever method overestimates, while the shear connection method (continuum approach), finite element and the approximate method underestimate the experimental result. The calculations assumed flange and full width of slab acting integrally with the shear wall as well as a constant modulus of elasticity of $1{\cdot}2 \times 10^6$ lbf/in^2.

Figure 11 illustrates how the deflection curve according to the continuum method alters assuming various width of slab and flange—this reduces the difference between calculated and experimental results.

9. CONCLUSIONS

1. The model and full-scale deflections agree well at low shear stresses (Figure 10).

2. Existing analytical solutions do not give reliable results for stress or deflections in brick structures. The cantilever method overestimates the deflection (Figures 9 and 10) and the extreme fibre stresses (Figure 12). The continuum method (Figures 10, 11 and 12) underestimates the deflection while it gives approximate values for maximum compressive stress, but underestimates the maximum tensile stress. Finite element techniques (Figure 10) appear promising but further work is necessary before a reliable solution can be suggested.

3. The relationship between deflection, rigidity and shear load is non-linear. The rigidity decreases with ncrease of horizontal loading (Figures 6, 7 and 13).

4. The strain distribution along any cross-section of the shear walls is non-linear (Figure 8).

ACKNOWLEDGEMENT

This work was sponsored by the British Ceramic Research Association and they also produced and supplied the model bricks.

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Berechnung der Horizontalbelastung von Schubwänden aus Ziegeln, die durch Deckenplatten verbunden sind

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Kurzfassung

Für die Berechnung von mehrgeschossigen Mauerwerkskonstruktionen ist es üblich, entweder die Querwände idealisiert als einen Rahmen mit breiten Stielen bzw. als einzelne Kragträger, wobei die Decken als starre, gelenkig verbundene Aussteifungen wirken, zu behandeln oder die Decken durch ein Kontinuum (Schubverbindungsverfahren) zu ersetzen. Versuche an Modellen und Prüfkörpern in natürlicher Größe zeigten jedoch, daß das Verhalten von Querwänden aus Ziegelmauerwerk bei horizontalem Lastangriff nicht dem mit solchen Methoden vorherbestimmten Verhalten entspricht. Der Unterschied zwischen den experimentellen und theoretischen Ergebnissen ist vermutlich auf die Tatsache zurückzuführen, daß für das Zusammenwirken von verschiedenen Elementen Annahmen angesetzt werden, die nicht für alle in der Praxis verwendeten Ausführungsmethoden und Fugenmaterialien zutreffen. In diesem Beitrag werden kurz einige Ursachen dieser fehlenden Übereinstimmung zwischen den theoretischen und den experimentellen Ergebnissen besprochen und eine Bemessungsmethode solcher Bauwerke bei Windbelastung angedeutet. Ferner wird auch auf die Notwendigkeit einer genauen Ermittlung der Steifigkeit der Verbindungselemente eingegangen. Diese ist eine der Veränderlichen, welche das Verhalten von Mauerwerkskonstruktionen mit Querwänden weitgehend beeinflussen.

The Lateral Load Analysis of Brick Multi-Storey shear-wall structures

In structural design brick multi-storey, shear-wall structures are commonly idealised and replaced by a wide-column frame or the slabs are replaced by a continuum (shear connection method) or the walls are treated as individual cantilevers (the slabs merely acting as rigid pin-connected struts). The behaviour of actual brick shear wall structures under lateral loading differs from that predicted by these methods, as observed both in case of model and full-scale experiments. The difference between the experimental and theoretical results may be due to the assumptions regarding the interaction of the elements, which in a practical structure may not be valid due to the method of construction and jointing materials. The paper looks briefly into the cause of observed differences between theoretical and experimental results and suggests a possible method for the design of brick structures subjected to wind loading. The paper also discusses the importance of assessing correctly the stiffness of the interconnecting medium, one of the variables which affects the behaviour of brick shear wall structures to a great extent.

Analyse des effets dus aux charges latérales dans les immeubles-tour

Pour les calculs des immeubles-tour, et notamment pour l'étude des effets dus aux charges latérales, on part en général d'une structure idéalisée. On remplace la bâtiment soit par une ossature à colonnes rigides, soit par une ossature à poutres rigides (calcul des cisaillements dans les murs). Une autre méthode consiste à considérer les murs comme étant des poutres individuelles, encastrées dans les fondements (les planchers étant dans ce cas des poutres horizontales articulées).

Le comportement réel des bâtiments ne répond à aucune de ces hypothèses, comme le démontre tant l'essai sur maquette que celui sur bâtiment de grandeur nature.

On pourrait attribuer la différence entre les résultats d'essais et les calcules théoriques aux hypothèses concernant l'interaction des éléments qui, dans une structure réelle, peut très bien ne pas être valable, compte tenu de la méthode de construction et des matériaux de liaison.

L'article expose très brièvement la cause des divergences constatées entre les résultats expérimentaux et propose une méthode pratique pour le calcul des effets dus au vent.

Les auteurs soulignent l'importance d'une évaluation précise de la rigidité des nœuds, qui constitue l'une des variables ayant une grande influence sur le comportement des structures.

Schubwände sind ein wirksames Mittel zur Erzielung von Steifigkeit in modernen, vielgeschossigen Tragkonstruktionen. Derartige Wände bilden im Zusammenwirken mit den Geschoßdecken ein hochgradig unbestimmtes System. Eine genaue rechnerische Untersuchung ist schwierig, so daß für die Berechnung das Bauwerk meistens idealisiert und durch einen breitstieligen Rahmen [5] ersetzt wird; man ersetzt auch die Decken durch ein Kontinuum [3] (Schubverbund-Verfahren) oder die Wände werden wie einzelne Kragarme behandelt (wobei die Decken lediglich wie steife, Punktverformung verbundene Aussteifungen wirken). Während in der Praxis diese Verfahren für die rechnerische Untersuchung und den Entwurf von Reihen ebener Wände angewandt werden, die durch Decken oder Balken verbunden sind, stellt die rechnerische Untersuchung eines zusammengesetzten, dreidimensionalen, vielgeschossigen Bauwerks ein noch schwierigeres Problem dar. Außerdem wurde beobachtet, daß die Ergebnisse dieser Berechnungsverfahren sogar für den einfachen zweidimensionalen Fall nicht mit dem Verhalten wirklicher Schubwandkonstruktionen von Ziegeln übereinstimmten [1]. Sie können daher nicht allgemein als ein Normberechnungsverfahren angewandt werden. Der große Unterschied zwischen den experimentellen und theoretischen Ergebnissen kann auf die Annahmen hinsichtlich der Wechselwirkung zwischen den einzelnen Bauteilen zurückgeführt werden, die in einem wirklichen Bauwerk wegen der verwendeten Konstruktionsverfahren und Verbindungsmittel nicht immer gültig sind. Dieser Aufsatz soll kurz die Ursache des Widerspruchs zwischen den theoretischen und experimentellen Ergebnissen beleuchten. Unter Heranziehung experimenteller Ergebnisse werden mögliche Verfahren für die Berechnung von Ziegelbauten unter Windlast vorgeschlagen [1, 2, 4].

rechnung des Bauwerks

s in den Abb. 1 und 2 gezeigte Bauwerk wird durch ge- chte Wände und Balken ersetzt, welche die gleichen Flächen d Trägheitsmomente wie das Bauwerk aufweisen. Die cken sind durch Balken ersetzt, deren wirksame Breite gleich vollen Deckenbreite angenommen wird. Die Verformun- n infolge Axiallast in den Balken sowie die Axial- und Schub- rformungen der Stützen werden vernachlässigt. Durch diese iherungen können die Ersatzsysteme leicht berechnet wer- n, vorausgesetzt, daß die Werte der elastischen Kennwerte kannt sind. Der Einfachheit halber wurde für die nachste- nd aufgeführte Berechnung das Kraftgrößenverfahren ange- andt. Man beachte die Drehung Θ_1 der Wände am Decken- schluß (Abb. 3).

$$= {}_0\!\int^{l} \frac{M}{E_w I_w}\, dx - [R_1 + R_2 + R_3 \ldots + R_n] \cdot l \cdot h \text{ (s. Abb. 1)} \quad (1)$$

ierbei ist E_w gleich dem Elastizitätsmodul der Wand; I_w Träg- eitsmoment und Mdx gleich der Fläche unter der Biege- omentenlinie infolge der aufgebrachten Last.

$$\Delta = l\Theta_1 \quad (2)$$

ach Abb. 3 kann die Decke wie zwei an jedem Ende mit der nbekannten Kraft R_1 belastete Kragarme behandelt werden.

aher

$$\Delta = \frac{R_1 \cdot l^3}{3\, E_b\, I_b}:$$

ie wirksame Länge des Balkens wird innerhalb der Grenz- verte l_1 und l_2 angenommen. (3)

E_b, I_b sind Elastizitätsmodul beziehungsweise Trägheitsmoment ler Decke

Aus (1), (2) und (3) ergibt sich

$$R_1 \left(\frac{l^3}{3\, E_b\, I_b} + \frac{l^2 \cdot h}{E_w\, I_w}\right) + \left(\frac{R_2 + R_3 \ldots + R_n}{E_w\, I_w}\right) l^2\, h = \left[\frac{M\, dx}{E_w\, I_w}\right]_o^h \cdot l \quad (4)$$

In der gleichen Art erhält man n Gleichungen für n Unbekann- te. Wenn man annimmt, daß $l_2 = l$ ist, was der Rahmenbe- rechnung entspricht und die Gleichung in Matrixform aufstellt, erhält man:

Symmetrisch

$$\begin{bmatrix} R_1 \\ R_2 \\ R_3 \\ \vdots \\ R_n \end{bmatrix} \cdot \begin{bmatrix} \frac{l^3}{3E_bI_b} + \frac{l^2 \cdot h}{E_w I_w} & & & \\ \frac{l^2 \cdot h}{E_w I_w} & \left(\frac{l^3}{3 \cdot E_bI_b} + \frac{2\, l^2\, h}{E_w I_w}\right) & & \\ \frac{l^2 \cdot h}{E_w \cdot I_w} & \frac{2l^2 \cdot h}{E_w I_w} & \left(\frac{l^3}{3E_bI_b} + \frac{3\, l^2\, h}{E_w I_w}\right) & \\ \frac{l^2 \cdot h}{E_w I_w} & \frac{2l^2 \cdot h}{E_w I_w} & \frac{3\, l^2\, h}{E_w I_w} & \ldots \\ & & \ldots & \frac{l^3}{3E_bI_b} + \frac{nl^2\, h}{E_w I_w} \end{bmatrix} = \begin{bmatrix} U_1 \\ U_2 \\ U_3 \\ \vdots \\ U_n \end{bmatrix}$$

Hierbei ist $U_1 = l. \int_o^h \frac{M}{E_w I_w}\, dx$, $U_2 = l \cdot \int_o^{2h} \frac{M\, dx}{E_w I_w}$ usw. (5)

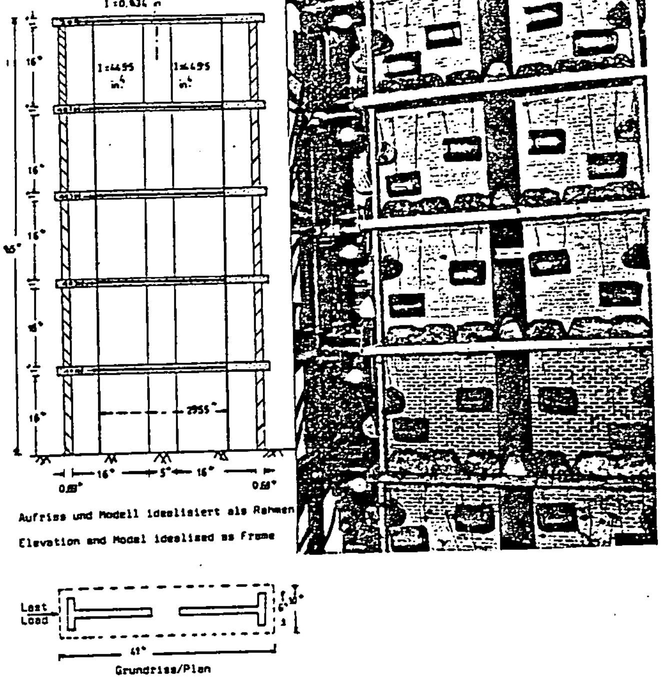


Abb. 1: Grund- und Aufriß von Modell 1
Fig. 1: Showing the plan and elevation of Model 1

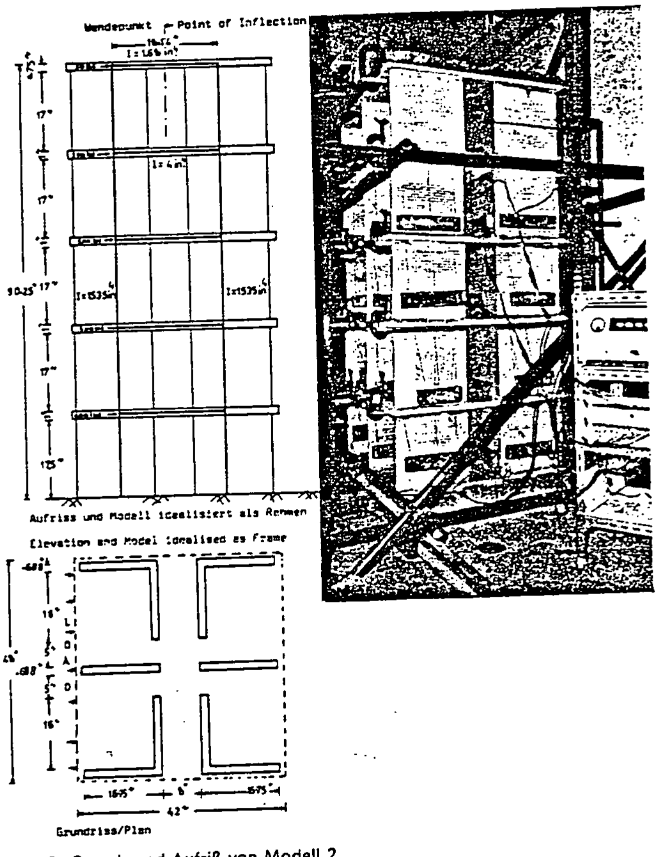


Abb. 2: Grund- und Aufriß von Modell 2
Fig. 2: Showing the plan and elevation of Model 2

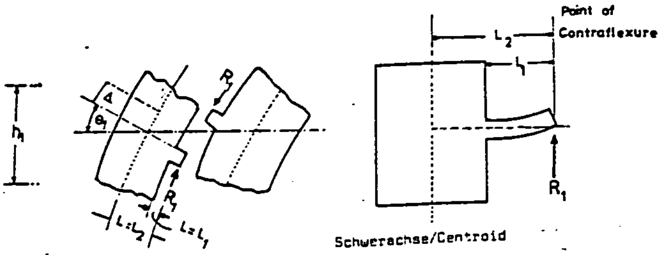


Abb. 3: Linke Hälfte der Konstruktion
Fig. 3: Left half of the structure

Wenn man davon ausgeht, daß der Balken zwischen der geometrischen Mittellinie und der Außenseite der Schubwand unendlich steif ist, d. h. einem breitstieligen Rahmen entspricht, ergibt sich folgende Gleichung:

$$\begin{bmatrix} R_1 \\ R_2 \\ R_3 \\ \vdots \\ R_n \end{bmatrix} \cdot \begin{bmatrix} \left(\frac{R_{-1}\, l_1^3}{3\, E_b I_b}+\frac{l^2 \cdot h}{E_w I_w}\right) & & & \\ \frac{l^2 \cdot h}{E_w I_w} & \left(\frac{R_{-1} l_1^3}{3 E_b I_b}+\frac{2\, l^2\, h}{E_w I_w}\right) & & \\ \frac{l^2 \cdot h}{E_w I_w} & \frac{2\, l^2 \cdot h}{E_w \cdot I_w} & \left(\frac{R_{-1} l_1^3}{3\, E_b\, I_b}+\frac{3\, l^2 \cdot h}{E_w I_w}\right) & \\ \frac{l^2 \cdot h}{E_w I_w} & \frac{2\, l^2 \cdot h}{E_w I_w} & \frac{3\, l^2\, h}{E_w I_w} \quad \cdots & \frac{R_{-1}\, l_1^3}{3\, E_b\, I_b}+\frac{n\, l^2\, h}{E_w I_w} \end{bmatrix} = \begin{bmatrix} U_1 \\ U_2 \\ U_3 \\ \vdots \\ U_n \end{bmatrix} \qquad (6)$$

Venn die Unbekannten einmal gefunden wurden, entweder lurch Lösung des Gleichungssystems oder durch Matrixinver-ion, so kann man die sich ergebende Biegemomentenlinie lurch Überlagerung erhalten. In der gleichen Art kann man lie Durchbiegung der Konstruktion nach dem Kraftgrößen-'erfahren erhalten.

)ie Spannungen in den rechten und linken Wänden in irgend-inem Querschnitt ergeben sich aus

$= \frac{R}{A} + \frac{MY}{I_w} - \frac{W}{A}$ beziehungsweise $\frac{R}{A} - \frac{MY}{I_w} - \frac{W}{A}$. Hierbei t

. = Axialkraft
1 = sich ergebendes Moment in der Wand
. = Wandfläche
w = Trägheitsmoment der Wand
' = Abstand der Randfaser von der Schwerachse der Wand
V = Eigengewicht des Bauwerks oberhalb des betrachteten chnittes

)ie Größtspannungen sollten die nach den Baubestimmungen ılässige Spannung nicht überschreiten. Außerdem sind keine ugspannungen in irgendeinem Wandquerschnitt erlaubt.

'ergleich mit den Versuchsergebnissen: Die aus der obigen erechnung nach Gleichung 5 erhaltenen Durchbiegungen für ie in den Abb. 1 und 2 gezeigten Modelle werden in den .bb. 4 und 5 mit den Versuchsergebnissen verglichen. Die Verte für die Elastizitätsmoduln des Ziegelmauerwerks und des etons im Modell wurden durch Biegeversuche an einfachen alken beziehungsweise Druckversuche an Zylindern ermittelt. ür Ziegelmauerwerk im natürlichen Maßstab wurde der lastizitätsmodul in einem Druckversuch ermittelt. Es besteht ne gute Übereinstimmung zwischen den experimentellen nd den rechnerischen Ergebnissen. In den Abb. 4 und 5 wer-n auch die Ergebnisse der rechnerischen Untersuchung am eitstieligen Rahmen (Gleichung 6), nach dem Schubver-und und dem Kragarmverfahren gezeigt. Die ersten beiden eser Verfahren schätzen die Durchbiegung des Bauwerks zu edrig, das Kragarmverfahren schätzt sie zu hoch ab (Abb. 4 d 5). Die Widersprüche sind hauptsächlich auf das Auftreten aterschiedlicher Scheibenkräfte in den gleichwertigen Ge-uden zurückzuführen (Abb. 6 und 8), die sich aus den unter-hiedlichen Annahmen ergeben, die den verschiedenen Ver-ren zugrunde liegen. Der einzige Unterschied zwischen den

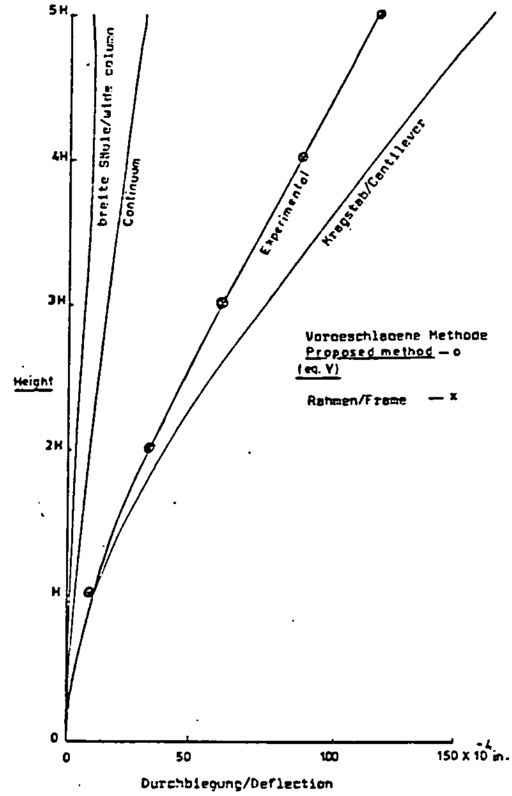


Abb. 4: Experimentelle und rechnerische Durchbiegung des Modells 1 (Abb. 1)
Fig. 4: Experimental and analytical deflection of Model 1 (Fig. 1)

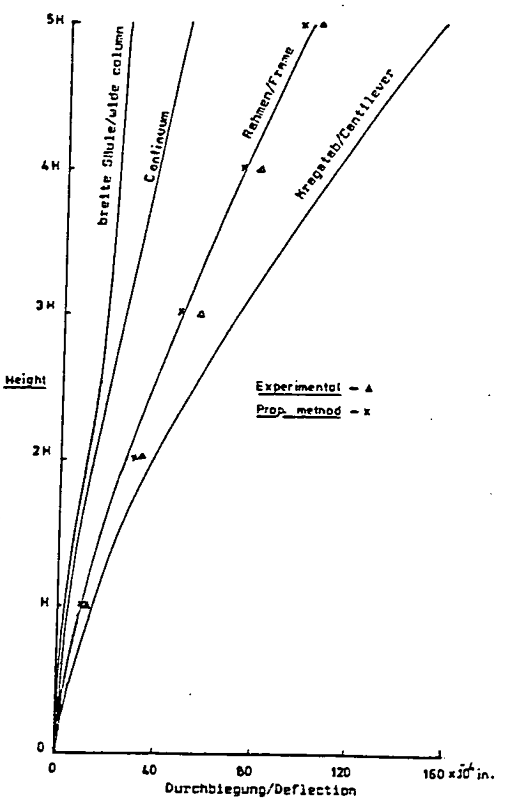


Abb. 5: Durchbiegung des Modells 2 (Abb. 2) nach verschiedenen Berechnungsverfahren und den Versuchsergebnissen
Fig. 5. Deflection of Model 2 (Fig. 2) by different analytical methods and the test results

Matrizen 5 und 6 liegt in der Hauptdiagonalen, und da die Glieder im zweiten Falle entsprechend dem breitstieligen Rahmen kleiner sind, wird der Scheibenschub vielfach höher abgeschätzt als vergleichsweise im ersteren Fall (Fall 5), der einer Rahmenberechnung entspricht. Dies führt zu einer zu niedrigen Abschätzung des Biegemomentes und der Durchbiegung beim Breitstiel-Verfahren. Dies trifft gleichfalls beim Schubverbund-Verfahren zu, bei dem ein Kontinuum zwischen den Wänden angenommen wird.

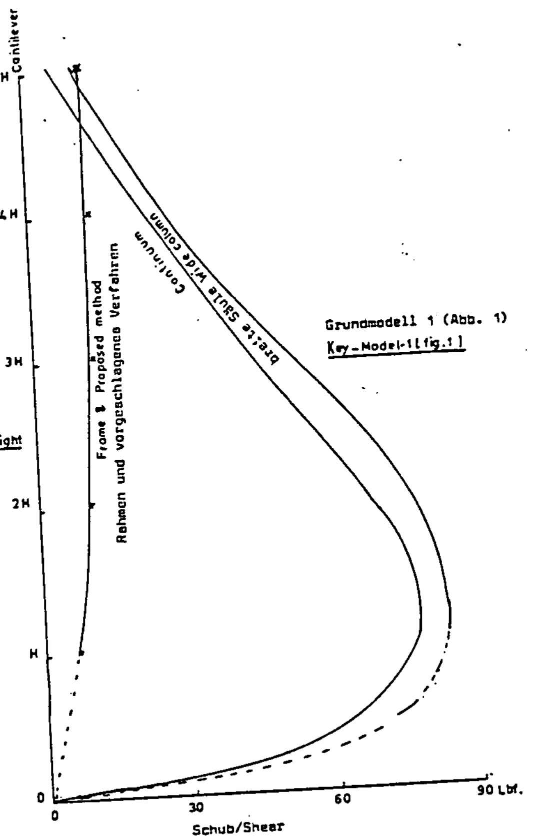


Abb. 6: Schubverteilung im Verbundbaukörper
Fig. 6: Shear distribution in connecting medium

Es entsteht kein Scheibenschub, wenn man davon ausgeht, daß die Konstruktion aus einzelnen Kragstäben zusammengesetzt ist. Somit werden die Biegemomente und die Durchbiegung zu hoch angesetzt. Obwohl dies eine sichere Annahme für die Berechnung der Schubwände sein kann, können die verbindenden Deckenplatten in einigen Fällen Schäden erleiden, da die Biegemomente in ihnen vernachlässigt werden. Der Verlauf des Biegemoments und des Scheibenschubs in beiden Modellkonstruktionen – berechnet nach den verschiedenen Verfahren – wird in den Abb. 6, 7 und 8 gezeigt. Eine gründliche Prüfung aller dieser Verfahren zeigt, daß bei einer bestimmten Kombination von Schubwänden und Öffnungen das veranschlagte Trägheitsmoment der verbindenden Balken und Deckenplatten das Bauwerksverhalten entscheidend beeinflußt. Der Einfluß einer Trägheitsmomentenzunahme des verbindenden Bauteils auf das größte Biegemoment sowie auf die Durchbiegungen werden für die Modellkonstruktionen 1 und 2 – ermittelt nach dem Kraftgrößenverfahren – in den Abb. 10a und 10b angegeben. Aus Abb. 10a und 10b ist zu ersehen, daß diese Wirkung in einem gewissen Bereich von Bedeutung ist und daß ein weiteres Anwachsen des Balkenträgheitsmoments darüber hinaus keine merkliche Wirkung auf die Steifigkeit oder das größte Biegemoment des Bauwerks hat. Nach den in den Abb. 10a und 10b gezeigten Beziehungen und den nach dem Schubverbundverfahren für die Modellkonstruktionen 1 und 2 ermittelten Durchbiegungen ist offensichtlich das wirkliche Trägheitsmoment der Deckenplatten beim Kontinuumsverfahren für diese Modelle um Faktoren von 113 bzw. 11 überschätzt wurden, d. h. im Verhältnis $1_2^3/1_1^3$. (Abb. 3). Zur weiteren Kontrolle wurden beide Konstruktionen nochmals mit vergrößertem Trägheitsmoment (im Verhältnis $1_2^3/1_1^3$) nach dem vorgeschlagenen Verfahren berechnet und die Ergebnisse mit

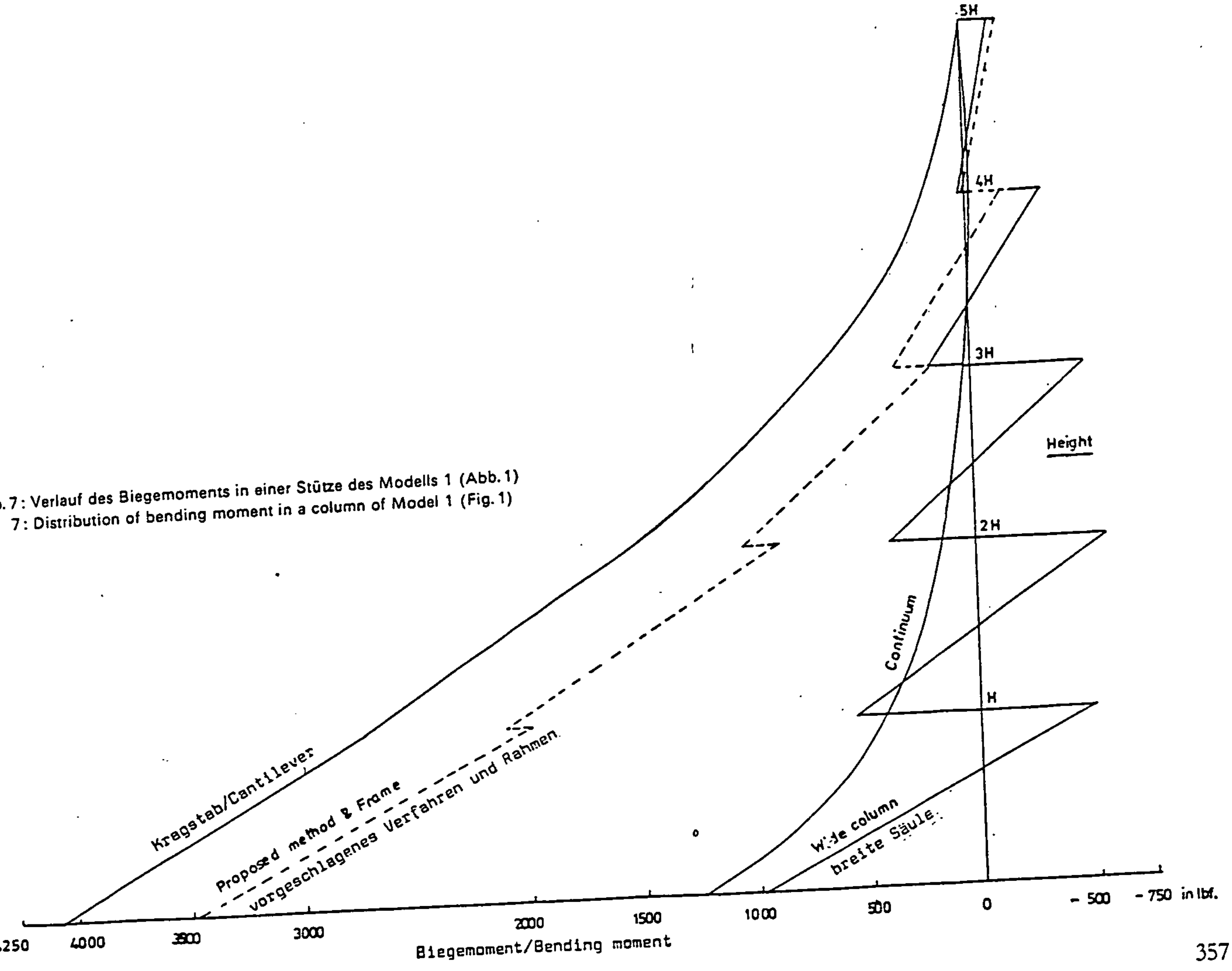


Abb. 7: Verlauf des Biegemoments in einer Stütze des Modells 1 (Abb. 1)
Fig. 7: Distribution of bending moment in a column of Model 1 (Fig. 1)

denen des Kontinuumverfahrens verglichen. Durchbiegung, größtes Biegemoment und Schub in der Scheibe waren die gleichen (Abb. 9). Die Werte der Biegemomente in halber Stockwerkshöhe waren ungefähr die gleichen, jedoch unterschiedlich an den Knoten. Dies liegt daran, daß das Kontinuumverfahren den Mittelwert für jedes Stockwerk gibt. Es scheint jedoch, daß das Schubverbundverfahren auch auf derart ähnliche Konstruktionen angewendet werden könnte, indem man die Spannweite des Bauteils bis zur Schwerachse der Schubwand vergrößert und somit den Wert der Schubkonstanten verändert. Die Anwendung des Schubverbundverfahrens ist jedoch begrenzt, da seine Anwendung sehr schwierig wird, wenn mehrere Wände durch Deckenplatten verbunden sind.

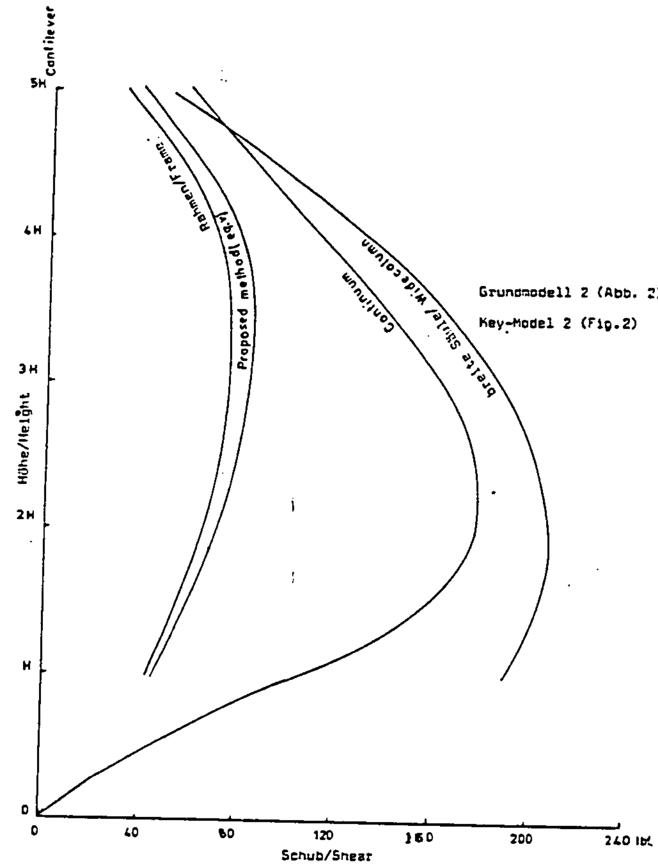


Abb. 8: Schubverlauf im Verbundbauteil
Fig. 8: Shear distribution in connecting medium

Da die meisten Entwurfsbearbeiter auf ein genormtes Rahmenprogramm zurückgreifen können, wurde es als zweckmäßig angesehen, die Möglichkeit der Verwendung eines solchen Programms für Schubwandkonstruktionen aus Ziegelmauerwerk ohne Änderung zu untersuchen, um die für die Rechenarbeit aufgewandte Zeit in wirtschaftlichem Rahmen zu halten. In vorliegendem Fall wurde IBM 1130 STRESS verwendet. Die Ergebnisse entsprechen etwa dem vorgeschlagenen Lösungsverfahren (Abb. 3); hieraus könnte man schließen, daß das genormte Rahmenprogramm für die Berechnung vielgeschossiger Gebäude mit Schubwänden aus dem gleichen Grunde zugelassen werden kann wie die Berechnung nach dem oben beschriebenen Kraftgrößenverfahren.
Nachdem verschiedene rechnerische Näherungen mit den Ergebnissen von Versuchen an Ziegelmauerwerksmodellen verglichen worden waren, hat man ein Bauwerk in natürlicher Größe, wie vorstehend beschrieben, mit dem in Abb. 11 gezeigten Ergebnis berechnet. Aus dieser grafischen Darstellung ist ersichtlich, daß die theoretischen Ergebnisse den experimentell erhaltenen sehr nahe kommen.

Schlußfolgerungen:

1. Die Vergleiche der verschiedenen betrachteten Berechnungsverfahren (nämlich: einfacher Kragstab, Rahmenberechnung, breitstieliges Analogiesystem und das Schubverbundverfahren) mit den experimentellen Ergebnissen weisen nachdrücklich darauf hin, daß die beste Annäherung an das wirkliche Verhalten eines Bauwerks mit Schubwänden aus Ziegeln zu erhalten ist, indem man die wirkliche Konstruktion durch einen Rahmen gleicher Steifigkeit ersetzt, in dem die Stützen die gleichen Querschnittseigenschaften wie die Wände

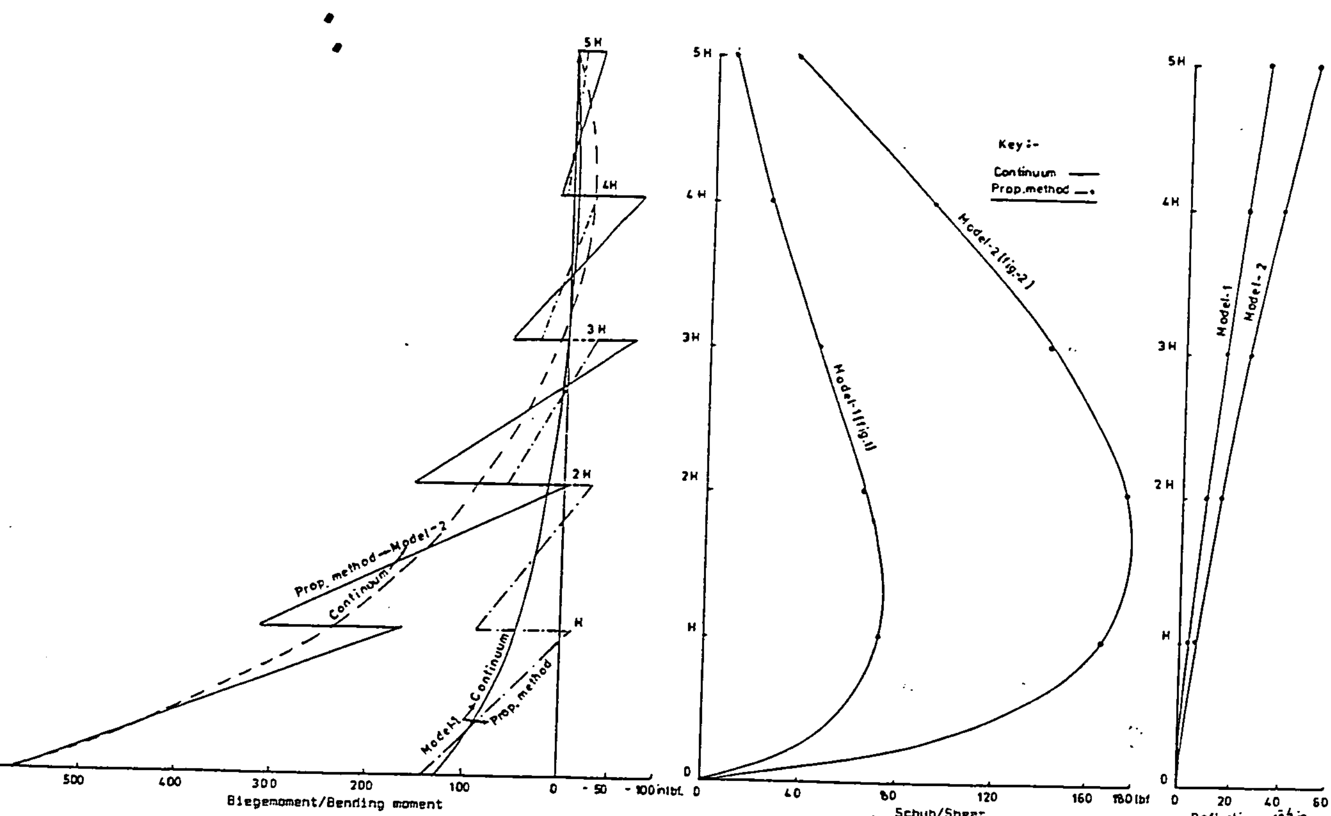
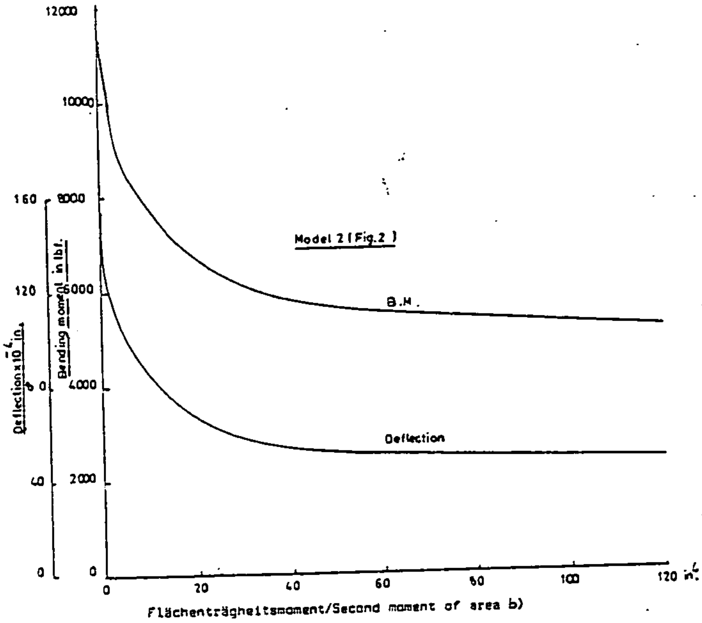


Abb. 9: Vergleich zwischen Kontinuum (Schubverbund) und vorgeschlagenem Verfahren bei Vergrößerung der Balkensteifigkeit mit einem Faktor L_2^3/L_1^3

Fig. 9: Comparison between continuum (shear connection) and proposed method by increasing the beam stiffness by a factor L_2^3/l_1^3



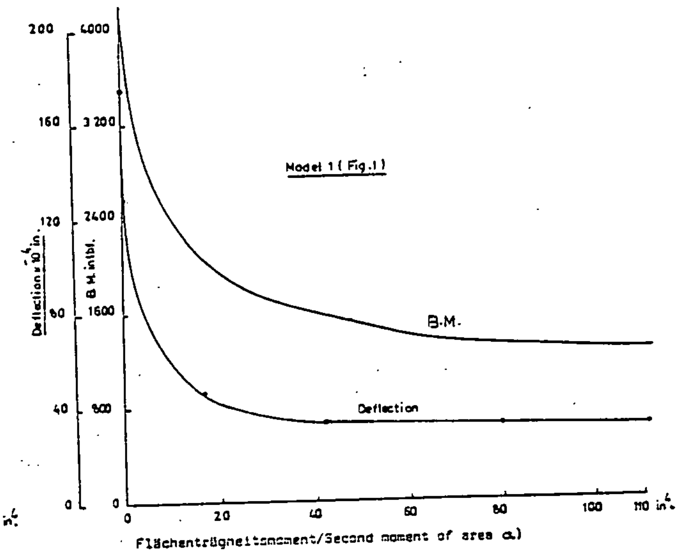


Abb. 10: Wirkung einer Vergrößerung des Flächenträgheitsmomentes der Deckenplatte auf das größte Biegemoment und die Durchbiegung der Modelle

Fig. 10: Effect of increasing the second moment of area of the slab on maximum bending moment and deflection of the models

haben und die verbindenden Decken zwischen den Achsen der Stützen gespannt sind. Die rechnerische Untersuchung kann mit dem in dem Aufsatz beschriebenen Verfahren durchgeführt werden oder durch ein genormtes Computerprogramm.

2. Die begrenzten experimentellen Ergebnisse haben gezeigt, daß es nicht ratsam ist, Schubwände aus Ziegeln, die durch Geschoßdecken verbunden sind, wie breitstielige Rahmen bei der Berechnung zu behandeln, da bei diesem Verfahren die Steifigkeit der verbindenden Deckenplatten vielfach zu hoch angesetzt wird und das wirkliche Verhalten der Konstruktion dadurch nicht genau wiedergegeben wird. Das Schubverbundverfahren kann angewandt werden, vorausgesetzt, daß die Spannweite des verbindenden Bauteiles von Mittellinie zu Mittellinie der Wände gerechnet wird. Mit diesem Verfahren werden jedoch die Biegemomente weniger gut erfaßt.

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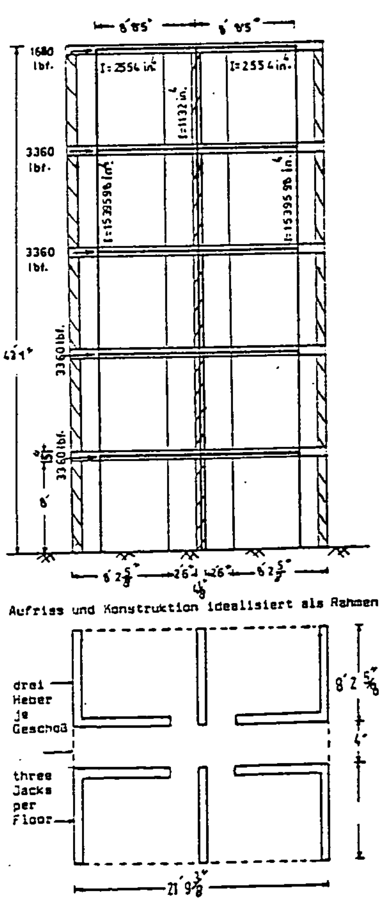

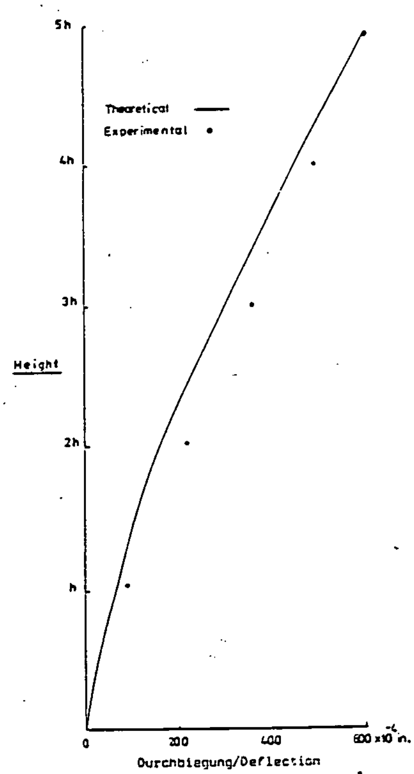

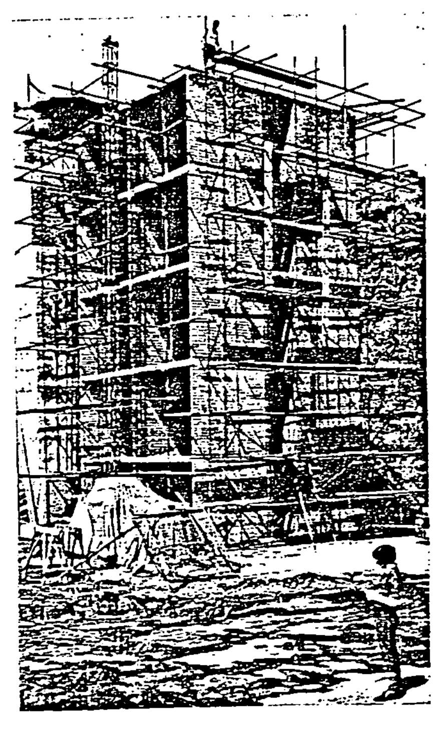

Abb. 11: Grund- und Aufriß eines Bauwerks im natürlichen Maßstab mit theoretischen und experimentellen Durchbiegungsergebnissen

Fig. 11: Showing the plan and elevation of full-scale building with theoretical and experimental deflection results

The Lateral Load Analysis of brick shear Walls Connected trough floor slabs

Shear walls are an effective means of providing rigidity in modern loadbearing multi-storey structures. Such walls acting together with the floor slabs form a highly indeterminate structure. Rigorous analysis is difficult, so for design the structure is commonly idealised and replaced by a wide column frame [5] or the slabs are replaced by a continuum [3] (shear-connection method) or the walls are treated as individual cantilevers (the slabs merely acting as rigid, pin-connected struts). While these methods are used in practice for the analysis and design of rows of plane walls connected by slabs or beams, the analysis of a complex three dimensional multi-storey structure presents an even more difficult problem. Furthermore, it has been observed that the results of these methods of analysis are not consistent [1] with the behaviour of actual brick shearwall structures even in the simple two dimensional case, and therefore cannot be universally applied as a standard design procedure. The wide gulf between the experimental and theoretical results may be due to the assumptions regarding interaction between the elements, which in a practical structure may not be valid due to the method of construction and the jointing materials. This paper looks briefly into the cause of the inconsistency between the theoretical and experimental results and in the light of experimental results suggests [1, 2, 4] possible methods for the design of brick structures subjected to wind-loading.

Approximate Analysis of the Structure

The structure shown in Figs. 1 and 2 are replaced with hypothetical walls and beams having the same areas and moments of inertia.

The slabs are replaced with beams whose effective width is taken as the full width of the slab. The deformations due to axial load in the beams and axial and shear deformations of the columns have been neglected. With these approximations, the substitute structures can easily be analysed provided the values of the elastic constants are known. For simplicity, the moment-area method has been used in the following analysis. Consider the rotation Θ_1 of the walls at the junction of slabs (Fig. 3).

Left half of the structure

$$\Theta_1 = {}_0\!\int^h \frac{M}{E_w I_w}\,dx - [R_1 + R_2 + R_3 \ldots + R_n]\cdot l\cdot h \text{ (see Fig. 1)}$$

where E_w – modulus of elasticity of wall; I_w – moment of inertia and Mdx is the area of the bending moment diagram due to the applied loading.

$$\Delta = l\Theta_1 \tag{2}$$

From fig. 3 the slab can be treated as two contilevers loaded at each end with unknown force R_1.

Hence

$$\Delta = \frac{R_1 \cdot l^3}{3\,E_b\,I_b};$$

the effective length of the beam is taken within the limits l_1 and l_2.

$E_b \cdot I_b$ are the modulus of elasticity and moment of inertia of the slab respectively.

From (1), (2) and (3) we will get

$$R_1\left(\frac{l^3}{3\,E_b\,I_b} + \frac{l^2\cdot h}{E_w\,I_w}\right) + \left(\frac{R_2 + R_3 \ldots + R_n}{E_w\,I_w}\right) l^2\,h = \left[\frac{M\,dx}{E_w\,I_w}\right]_0^h \cdot l \tag{4}$$

Similarly, we obtain n equations for n unknowns. If we assume $l_2 = l$, which corresponds to frame analysis and arrange the equations in matrix form, we get:

$$\begin{bmatrix} R_1 \\ R_2 \\ R_3 \\ \vdots \\ R_n \end{bmatrix} \cdot \begin{bmatrix} \frac{l^3}{3E_bI_b} + \frac{l^2\cdot h}{E_w I_w} & & & \text{Symmetrical} \\ \frac{l^2\cdot h}{E_w I_w} & \left(\frac{l^3}{3\cdot E_bI_b} + \frac{2\,l^2\,h}{E_w\,I_w}\right) & & \\ \frac{l^2\cdot h}{E_w\cdot I_w} & \frac{2l^2\cdot h}{E_w\,I_w} & \left(\frac{l^3}{3E_bI_b} + \frac{3\,l^2\,h}{E_w\,I_w}\right) & \\ \frac{l^2\cdot h}{E_w\,I_w} & \frac{2l^2\cdot h}{E_w\,I_w} & \frac{3\,l^2\,h}{E_w\,I_w} & \ldots \quad \ldots \quad \frac{l^3}{3E_bI_b} + \frac{nl^2\,h}{E_w\,I_w} \end{bmatrix} = \begin{bmatrix} U_1 \\ U_2 \\ U_3 \\ \vdots \\ U_n \end{bmatrix}$$

$$\text{where } U_1 = l.\int_0^h \frac{M}{E_w\,I_w}\,dx,\quad U_2 = l\cdot\int_0^{2h}\frac{M\,dx}{E_w\,I_w} \text{ etc.} \tag{5}$$

If we assume the beam is infinitely stiff between the geometric centre line and the face of shear wall, in other words corresponding to a wide column frame, the equations will be:

$$\begin{bmatrix} R_1 \\ R_2 \\ R_3 \\ \vdots \\ R_n \end{bmatrix} \cdot \begin{bmatrix} \left(\frac{R_{-1}l_1{}^3}{3\,E_b\,I_b} + \frac{l^2\cdot h}{E_w\,I_w}\right) & & & \\ \frac{l^2\cdot h}{E_w\,I_w} & \left(\frac{R_{-1}l_1{}^3}{3E_b\,I_b} + \frac{2\,l^2\,h}{E_w\,I_w}\right) & & \\ \frac{l^2\cdot h}{E_w\,I_w} & \frac{2\,l^2\cdot h}{E_w\cdot I_w} & \left(\frac{R_{-1}l_1{}^3}{3\,E_b\,I_b} + \frac{3\,l^2\cdot h}{E_w\,I_w}\right) & \\ \frac{l^2\cdot h}{E_w\,I_w} & \frac{2\,l^2\cdot h}{E_w\,I_w} & \frac{3\,l^2\,h}{E_w\,I_w} \quad \ldots & \frac{R_{-1}l_1{}^3}{3\,E_b\,I_b} + \frac{n\,l^2\,h}{E_w\,I_w} \end{bmatrix} = \begin{bmatrix} U_1 \\ U_2 \\ U_3 \\ \vdots \\ U_n \end{bmatrix} \tag{6}$$

Once the unknowns are found either by solving the simultaneous equations or by matrix inversion, the resultant bending moment diagram can be obtained by superimposition. Similarly, the deflection of the structure × can be obtained by the moment area method.

The stresses in the right and left hand walls at any cross-section will be given by

$\sigma = \frac{R}{A} + \frac{MY}{I_w} - \frac{W}{A}$ and $\frac{R}{A} - \frac{MY}{I_w} - \frac{W}{A}$, respectively. Where

R = the axial force on the wall at the section under consideration
M = the resultant moment on the wall
A = the area of the wall
I_w = the moment of inertia of the wall
Y = the distance from the centroid of the wall to its extreme fibre
W = Dead wt. of the structure above the section under consideration

These maximum stresses should not exceed the permissible stress from the code of practice. Furthermore, no tension is allowed at any cross-section of the wall.

Comparison with the test results : The deflections obtained from the above analysis using equation 5 for the model buildings shown in Figs. 1 and 2 are compared in Figs. 4 and 5 with the experimental results. The values of the elastic moduli of model brickwork and concrete were found by simple beam tests in pure bending and compressive tests on cylinders, respectively. For full-scale brickwork the modulus of elasticity was found by compression test. There is good agreement between the experimental and analytical results. Also in Figs. 4 and 5 are shown the results of analysis by the wide column frame (equation 6), the shear connection and cantilever methods. The first two of these methods underestimates, whilst the cantilever method overestimates the deflection of the structures (Figs. 4 and 5). The discrepancies are chiefly due to the generation of different laminar forces in the equivalent structures (Figs. 6 and 8) due to differing assumptions upon which the various methods are based. The only difference between the matrices 5 and 6 is in the leading diagonal, and as these terms are smaller in the second case corresponding to the wide column frame, it overestimates the laminar shear many times as compared with the former (case 5) corresponding to a frame analysis. This leads to the underestimation of the bending moment and the deflection by the wide column method. Similarly, this is true in the case of the shear connection method, in which a continuous medium extends between the walls. No laminar shear exists when assuming the structure is composed of individual cantilevers and thus the bending moment and deflection are overestimated. Although, this may be a safe assumption for the design of shear walls the interconnecting slabs may suffer damage in some cases since the bending moments in them is neglected. The distribution of the bending moment and laminar shear in both model structures is shown in Figs. 6, 7 and 8 as calculated by the various methods. A thorough examination of all the methods indicates that for a particular combination of shear wall and opening, the assessed moment of inertia of the connecting beams and slabs significantly affects the structural behaviour. The influence of increasing the moment of inertia of the connecting medium on the maximum bending moment and on the deflections for model structures 1 and 2 are shown in Figs. 10a and b as given by the moment area method. From Fig. 10a and b it can be seen that this effect is significant over a certain range, beyond which the increased moment of inertia of beam will have no appreciable effect on the rigidity or on the maximum bending moment induced in the structure. From the relationship in Fig. 10a and b and the deflections obtained by the shear connection method for model structures 1 and 2 it appears that the actual moment of inertia of the slabs in the continuum method have been overestimated in these models by approximately 113 and 11 times i.e. ratio of $1_2^3/1_1^3$ (Fig. 3) respectively. To make a further check, both the structures were analysed again with increased moment of inertia (in ratio of $1_2^3/1_1^3$) by the proposed method and the results compared with that given by the continuum method. The deflection, maximum bending moment and shear in the laminae were the same (Fig. 9). The values of the bending moments throughout the mid-height of the storeys were about the same, but different at the joints.

This is because the continuum method gives the average value for each storey. However, it would appear that the shear connection method could also be applied to such similar structures by extending the span of the medium to the C. G. of the shear wall and thus modifying the value of the shear constant. The use of shear connection method is however limited as it becomes very difficult to handle if several walls are interconnected by slabs.

As most design engineers will have access to a standard frame programme, it was thought useful to investigate the possibility of using such a programme without alteration for a brick shear wall structure to economise the time spent on arithmetical work. In this case IBM 1130 STRESS was used. The results were similar to the proposed method of solution (Fig. 3), hence it could be argued that the standard frame programme may be adopted to analyse multi-storey shear wall structures on the same basis as the analysis using the area-moment method set out above.

Having examined various analytical approaches by comparing with the results of brickwork model tests, the analysis of a full-scale structure was carried out as above with the result shown in Fig. 11. It may be seen from this diagram that the theoretical results are very similar to those obtained from experiment.

Conclusions :

1. The comparisons between the various analytical methods considered (namely, simple cantilever, frame analysis, wide-column analogy, and the shear connection method) with experimental results strongly suggest that the best approximation to the actual behaviour of a brick shear-wall structure is obtained by replacing the actual structure by an equivalent rigid frame in which the columns have the same sectional properties as the walls and the interconnecting slabs span between the axes of the columns. The analysis may be carried out by the method described in the paper or by a standard frame computer programme.

2. From the limited experimental results it is not advisable to treat brick shear walls connected by the floor slabs as a wide column frame for design as this method overestimates the stiffness of the connecting slabs many times and does not correctly represent the actual behaviour of the structure. The shear connection method could be used provided that the span of the medium is taken between the centre lines of the walls but will not give as good a presentation of bending moments.

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The stability of a five-storey brickwork cross-wall structure following the removal of a section of a main loadbearing wall

Synopsis
It is now a requirement under the Building Regulations that structures of five storeys and over should remain stable following the removal of a specified length of loadbearing wall, although at a substantially reduced safety factor. Three experiments are described in this paper which had the object of providing confirmation that this could be achieved in a simple five-storey brickwork cross-wall structure. In each test a section of loadbearing wall was removed and measurements were made of applied loads, deflexions and strains. The theoretical conclusion that the structure would remain stable under these conditions was confirmed and some information was obtained concerning the strength of 114 mm (4·5 in) thick wall panels subjected to lateral loading.

Introduction
This paper describes the results of three experiments on a section of a five-storey brickwork cross-wall structure in which sections of the main cross-walls were removed at ground-floor level with a view to testing the stability of the structure in a damaged condition, as might occur following an internal explosion. The structure (Fig 1) had been constructed previously for a series of lateral loading tests and was not specially designed to resist the stresses set up by partial collapse, but prior to the present tests the stability of the structure following the removal of a section of cross-wall was assessed by the methods described elsewhere.[1] These calculations indicated that the structure would not collapse following the removal of a major loadbearing element.

In each of the present tests, a section of loadbearing wall was removed under controlled conditions and measurements of applied loads, deflexions and strains were made in the course of the tests, thus affording information as to lateral rigidity and strength of a 114 mm (4·5 in) single leaf wall.

The test structure

Plan form and elevation
The structure consisted of three pairs of 114 mm (4·5 in) cross-walls stabilized by two pairs of shear walls, as indicated in Fig 2. The wall layout was the same on each of the five storeys of the structure.

Fig 1. Test structure

Bricks
Wire-cut bricks with an average crushing strength of 34·6 MN/m² (5020 lbf/in²) were used for the construction of the building. The average compressive strength of six-course brick prisms was 17·2 MN/m² (2495 lbf/in²) at 28 days.

Cement and sand
A rapid hardening cement 'Ferrocrete' to BS 12:1958 and ordinary building sand conforming to BS 1200:1955 were used for the construction of the walls.

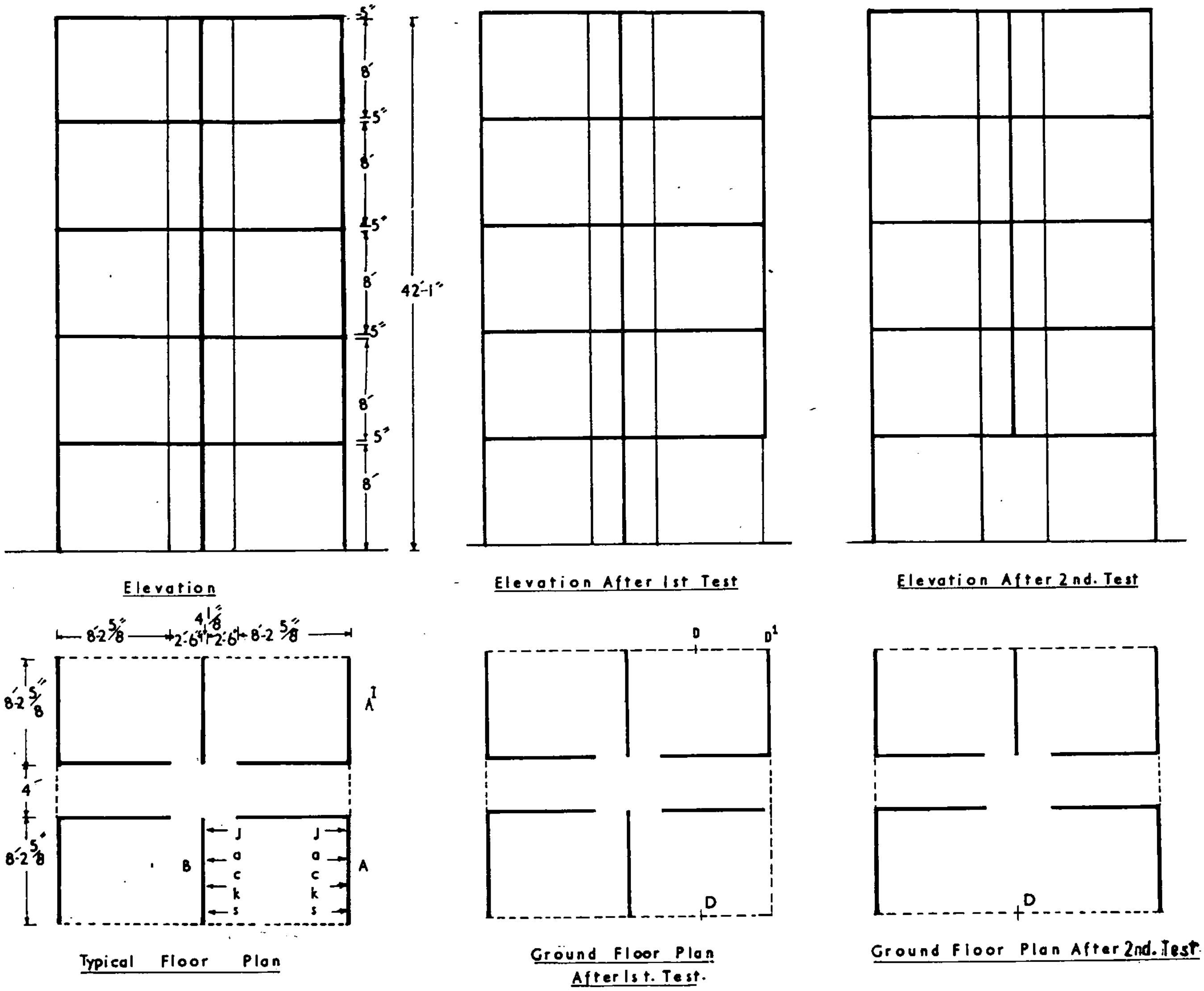

Fig 2. Test structure before and after use

Mortar
A 1:$\frac{1}{4}$:3 cement:lime:sand mix (by vol) was used for the construction of the building. The materials were gauged before dry mixing. The average crushing strength of 102 mm (4 in) mortar cubes was 14·5 MN/m^2 (2100 lbf/in^2) at 28 days.

Construction details

Walls
The walls were built according to the BCeramRA Model Specification.[2]

Reinforced concrete slabs
To save time and cost of shuttering, 50·8 mm (2 in) thick 'Omnia wide slabs' with 76·0 mm (3 in) *in situ* concrete topping were used for the floors (Fig 3). The floors were made of panels 3·16 m (10 ft 4·5 in) long and 1·2 m (3 ft 11·25 in) wide. The precast panels were lifted and kept in position by props with no bearing on the walls (Fig 4). A 1:2:4 ready-mix concrete having a minimum strength of 20·7 MN/m^2 (3000 lbf/in^2) at 28 days was poured on the top of the 50·8 mm (2 in) precast panels to give a 127 mm (5 in) thick slab throughout. By adopting this method of construction a good joint, similar to a cast *in situ* slab, was obtained between the finished slabs and the walls underneath. However, there is no through reinforcement (Fig 4) in the bottom of the slab over the brick wall. With this form of construction the minimum practical slab thickness is 127 mm (5 in) although it would be possible to use a thinner slab with normal *in situ* concrete. Mesh reinforcement [6 mm ($\frac{1}{4}$ in) square twisted bars at 203 mm (8 in) c/c] was provided in the bottom of the precast slab and in the top of the slab over supports. The details are given in Fig 3. The slab was designed to carry 1915 N/m^2 (40 lbf/ft^2) superimposed load and self weight of 2875 N/m^2 (60 lbf/ft^2); the design calculations are given in Appendix 1.

Arrangement for applying the tranverse load

The transverse load was applied by jacking against a steel frame, which was fixed to the concrete floor of the quarry by nine projecting-type Rawlbolts going to a depth of 184 mm (7·25 in) (Fig 5). The transverse load from four 10 tonne jacks was distributed to eight point loads on a line across the width of the wall through 305 mm (12 in) span steel spreader beams 50 × 50 mm (2 × 2 in). The load from each jack was measured by a 3 tonne load-cell calibrated in the laboratory. Rubber packing was inserted between the rollers of the spreader beams and the wall for the proper distribution of the load. All four jacks were controlled by a single valve and supplied by a mobile electro-hydraulic pump unit.

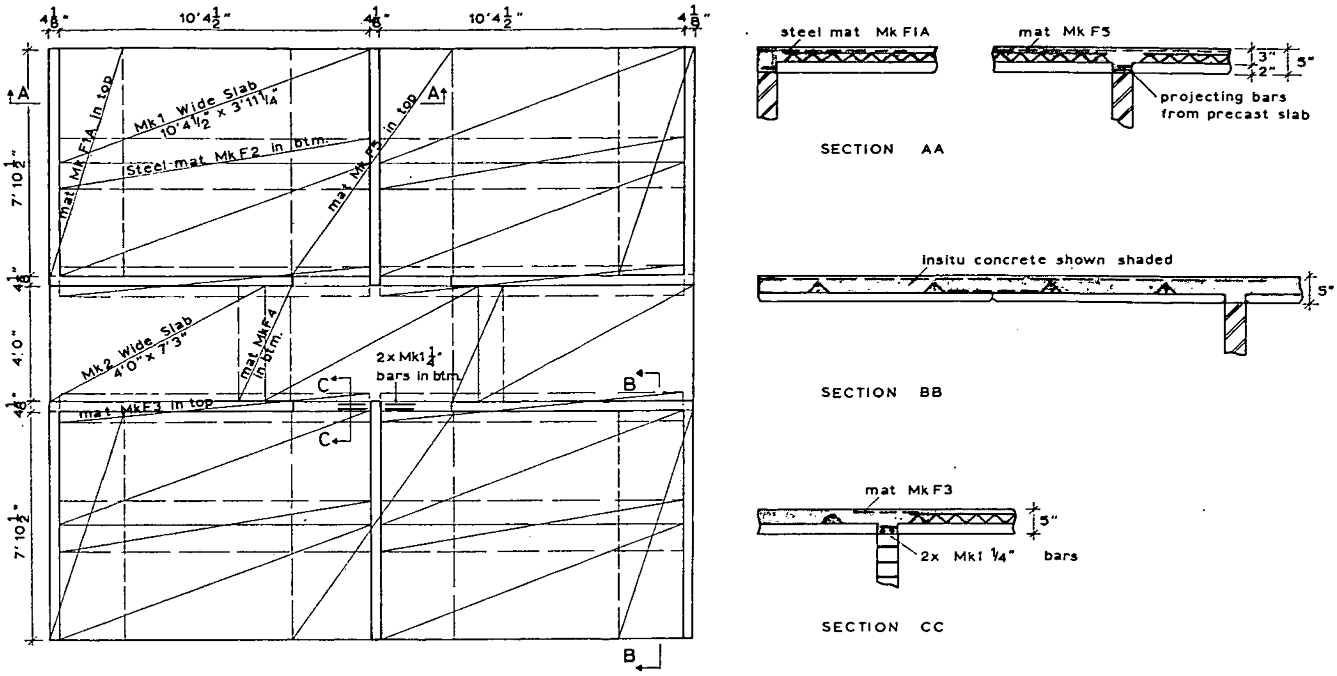

*Fig 3. Layout of precast slabs and reinforcement (*in situ *concrete)*

Tests on the structure

The tests were carried out in the following sequence.

(1) The outer loadbearing cross-walls of the ground floor (Fig 2, either panel A or A[1]) was loaded to destruction leaving no support in that region for the floor above it.

(2) The cross-wall in (1) was rebuilt.

(3) The central loadbearing wall (B, Fig 2) was similarly removed 15 days after the reconstruction of the outer wall.

The horizontal deflexions of the panel and the vertical deflexions of the floor slab were measured by a dial-gauge. Demec gauges of 610 mm (24 in) and 203 mm (8 in) gauge length were used to measure the strains in the panel. Typical deflexion results are given in Figs 6, 7 and 8.

The walls in both tests 1 (Panel A or A[1]) and 2 failed due to development of horizontal cracks at the centre of the panel at a brick/mortar interface. The ultimate loads were 43·1 kN (4·33 tonf—Panel A), 47·5 kN (4·77 tonf—Panel A[1]) and 62·5 kN (6·27 tonf) respectively. The calculated precompressions for the walls in test 1 and 2 were 365 kN/m^2 (53 lbf/in^2) and 510 kN/m^2 (74 lbf/in^2). In the first test (Panel A) the maximum deflexion after removal of the wall (Fig 2) could not be obtained as the falling wall knocked out the device for recording the vertical deflexion. The maximum deflexion of the floor slab was 4·04 mm (0·159 in) after removal of Panel A[1] (Fig 9). The deflexion of the floor slab at the centre (D, Fig 2—Panel A and A[1]) of the free edge was 1·45 mm (0·057 in) and 1·67 mm (0·066 in) respectively. In the second test the maximum slab deflexion (D—Fig 2) was 5·44 mm (0·214 in) after removal of the wall. No damage was noticed anywhere in the structure (Fig 10) except immediately above the wall removed where the joint between the first floor slab and wall was found to be broken and the wall appeared to be partly hanging from the second floor slab and partly supported by the first floor slab spanning between the ground-floor shear walls. From the recorded strains before and after the test this wall also appeared to have been relieved of load from floors above. In each case the structure as a whole was found safe after the tests and remained so for a week before the damaged element was replaced.

The uplift of the floor slab above the test wall was also recorded during the tests, with the results shown in Table 1.

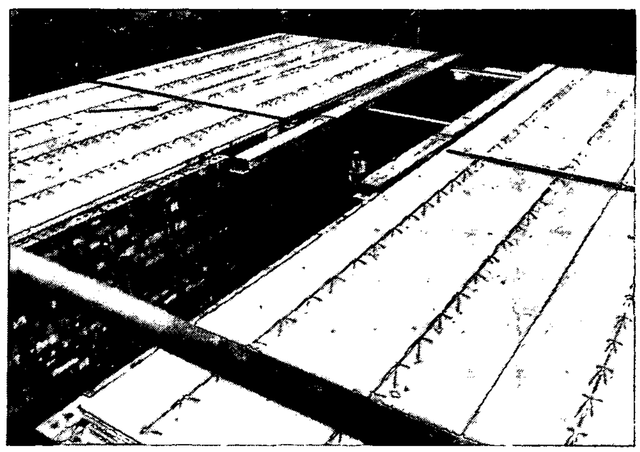

Fig 4. The precast Omnia slab in position

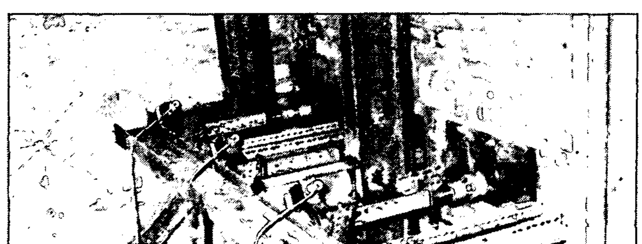

Fig 5. Test arrangement

TABLE 1

Showing the uplift of slab

Lateral load on panel	Test 1 (Panel A[1])	Test 2
	Slab Deflexion D[1]	Slab Deflexion D
28·21 kN (2·83 tonf)	0·102 mm (0·004 in)	—
35·9 kN (3·6 tonf)	—	0·229 mm (0·009 in)
40·3 kN (4·04 tonf)	0·508 mm (0·02 in)	—
47·5 kN (4·77 tonf)	4·87 mm (0·192 in)	—
49·8 kN (5 tonf)	—	0·762 mm (0·03 in)

Assessment of stability

Examination of the structure shows that if one of the central cross-walls is removed at ground-floor level the weight of each section of cross-wall on storeys above plus the self weight of the slab on which it rests and any imposed loading on it will have to be resisted by bending action in that slab. As we are concerned with ultimate load behaviour it is appropriate to apply a yield-line analysis. This permits the estimation of the load factor for the structure following the removal of the section of cross-wall. A detailed calculation, shown in Appendix 2, indicates that a load factor of 1·94 on dead load plus 1915 N/m^2 (40 lbf/ft^2) superimposed load will exist after removal of the centre cross-wall.

Where a section of the end cross-wall is removed, it is to be expected that each section of cross-wall above will be supported vertically by its connection to the corresponding shear wall and prevented from rotating by the floor slabs. The transfer of load to the shear walls will result in concentrated bearing stresses at the outer end of the shear wall on the ground floor. The actual distribution of loads is somewhat indeterminate but a safe assumption would be that the whole weight of the section of cross-wall above the one removed has to be supported by the floor slab below: in other words assuming that the bond between the cross-wall and the shear wall breaks. A calculation on this basis is shown in Appendix 2 and shows a load factor of about 2·44 after removal of the end cross-wall.

Discussion

The removal of one of the major loadbearing elements did not precipitate the collapse of the structure, hence it appears that the building is not susceptible to collapse following major local damage. Although the test was carried out without superimposed load on the floor slabs, this has been accounted for in the calculations for the worst condition. The slab appeared to be safe, even after taking superimposed load into consideration, and following the tests showed no sign of damage.

Precompression increased the load-carrying capacity of the wall in the transverse direction and from tests 1 (Panel A and A[1]) and 2 it can be inferred that a 114 mm (4·5 in) brickwork wall of the dimensions tested and under similar conditions will resist an equivalent uniform static pressure of 15·26 kN/m^2 (2·21 lbf/in^2) and 21 kN/m^2 (3·05 lbf/in^2) respectively, before failure. This is far below the design static pressure of 34 kN/m^2 (5 lbf/in^2) as required by the revision[3] to the Building Regulations and

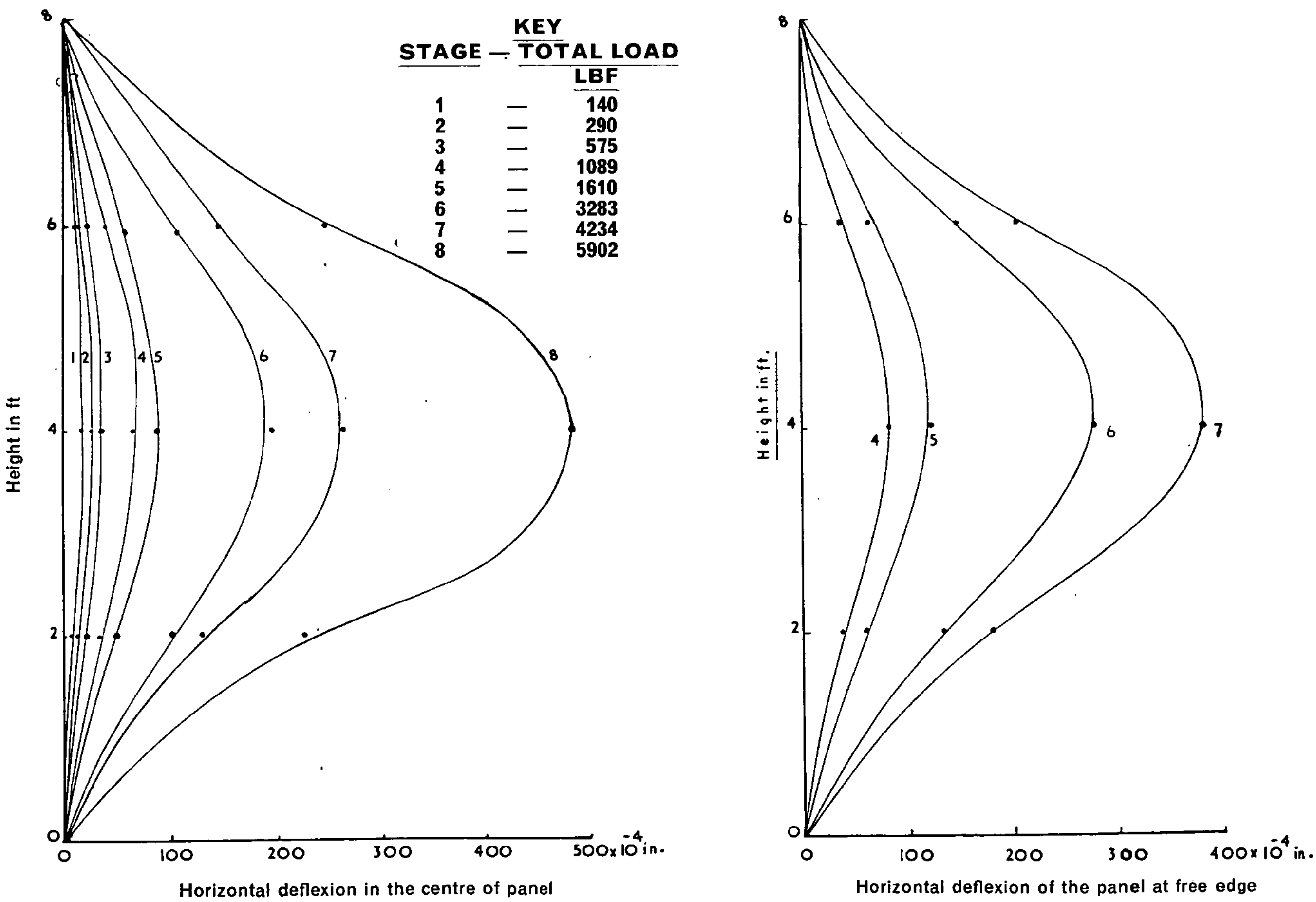

Fig 6. Panel deflexion at various stages of loading in the 1st test (Panel A)

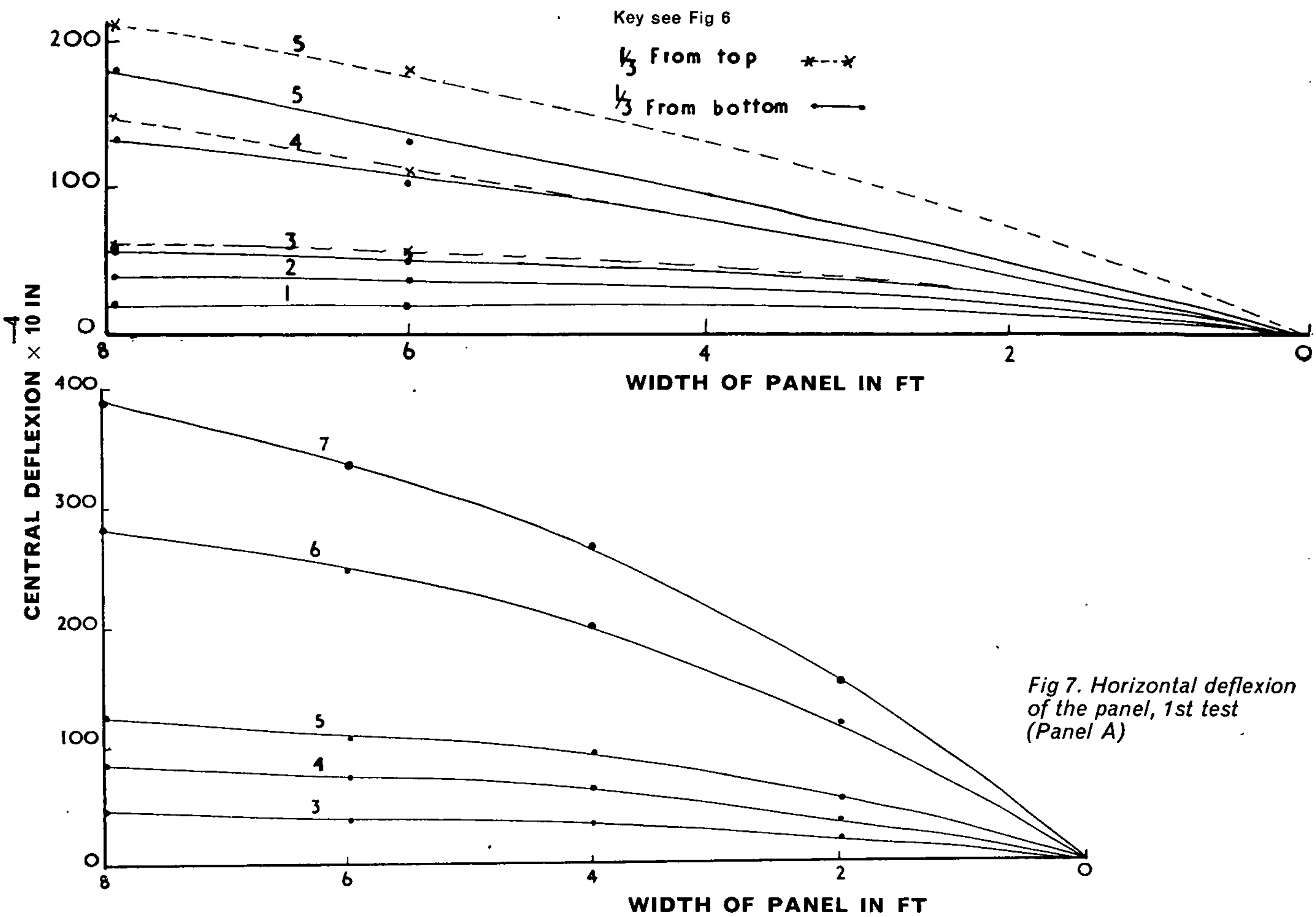

Fig 7. Horizontal deflexion of the panel, 1st test (Panel A)

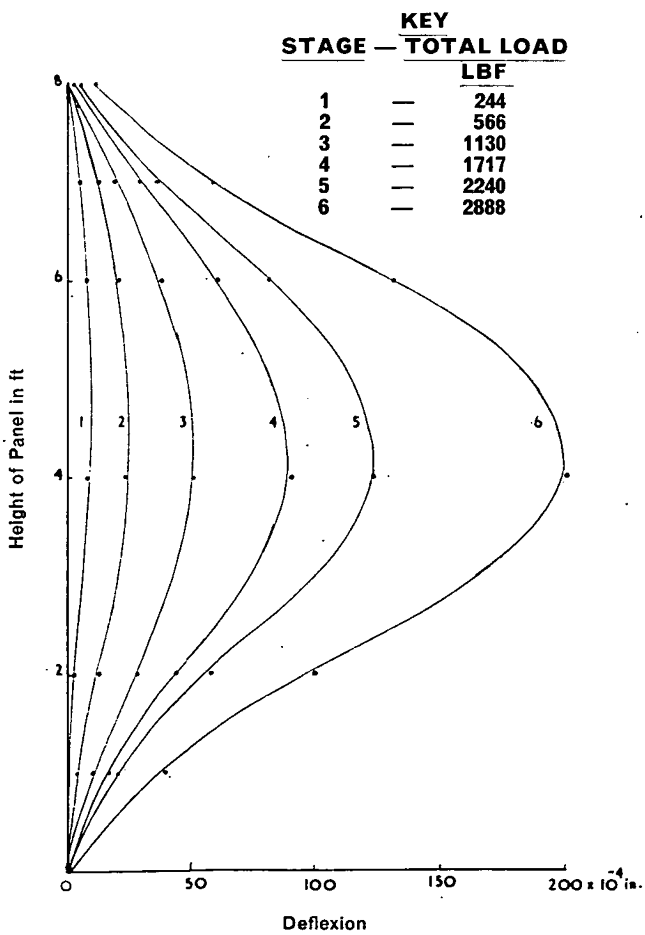

Fig 8. Horizontal deflexion of the panel in 2nd test at various loads

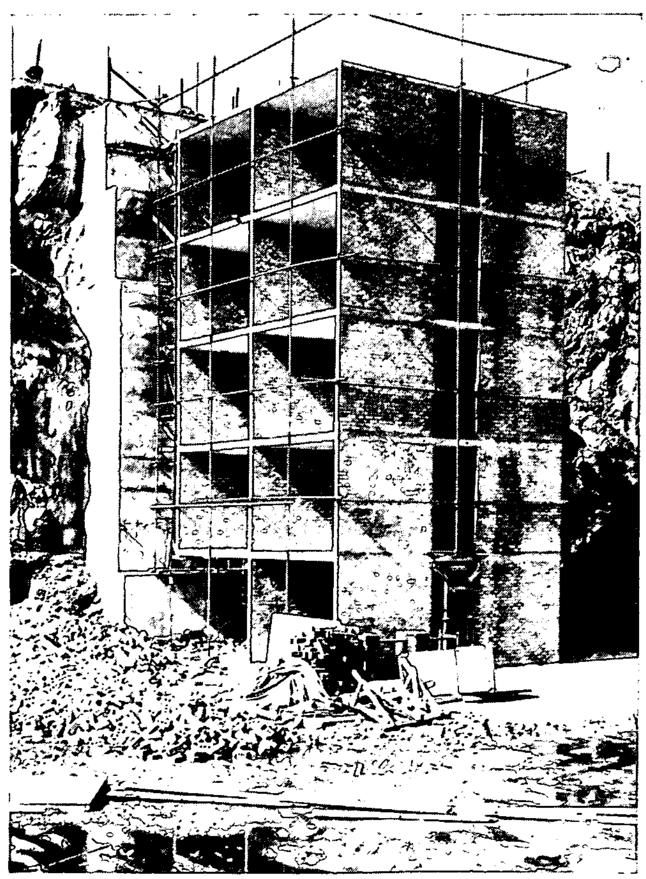

Fig 9. Structure after removal of the end loadbearing element

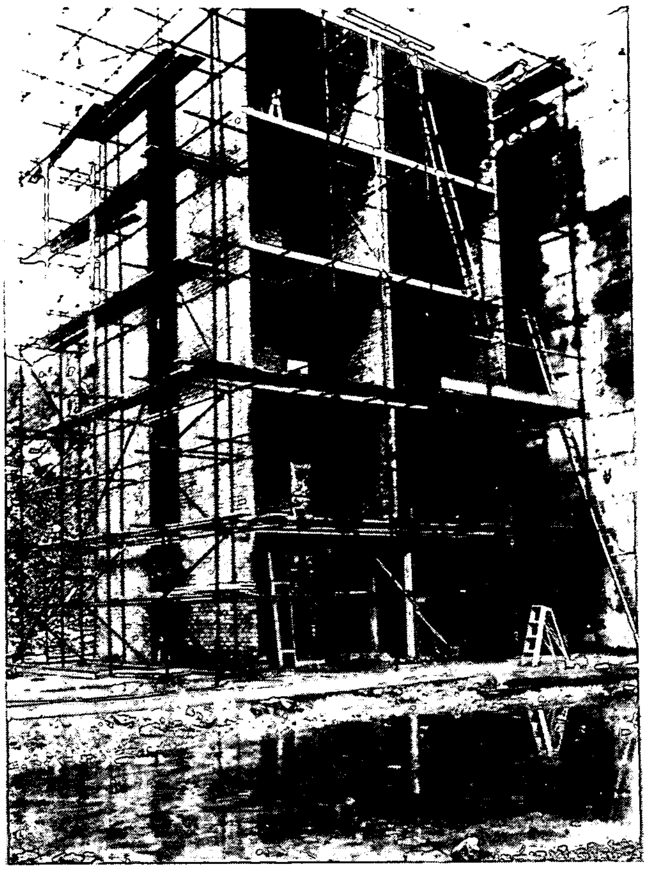

Fig 10. Structure after removal of central loadbearing element.

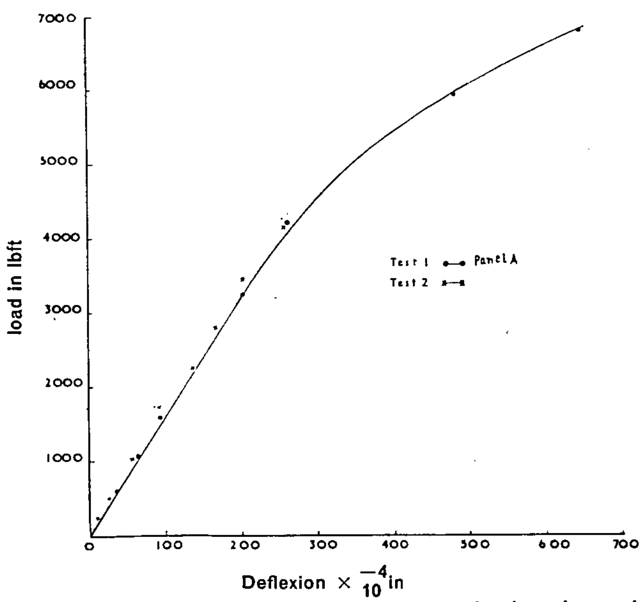

Fig 11. Relationship between transverse load and panel deflexion.

also in one case less than the 17 kN/m² (2·5 lbf/in²) recommended by the Institution of Structural Engineers.[4] Although the panel fails to achieve the above requirements, the stability of the structure as a whole was very adequate after removal of one section of cross-wall, although it was not designed with this in mind.

The deflexion of the panel at one-third height from top is slightly more than at the corresponding distance from the bottom (Fig 7), possibly due to different support conditions. The relationship between the central deflexion of the panel and transverse load is linear up to approximately half the ultimate load (Fig 11).

Conclusions

These tests demonstrated that a cross-wall loadbearing brickwork structure could remain stable in the event of one section of a main cross-wall being removed as a result of an explosion or other accident even though, as in the present structure, it did not have fully continuous *in situ* slabs. The test structure was not designed to withstand this treatment and it may therefore be concluded that the design of a brickwork structure of this type to meet the requirements of the *Building (Fifth Amendment) Regulations 1970* on the 'alternative path' basis would present no difficulty. In many cases, there would seem to be no necessity for additional elements to secure the safety of the structure.

The tests also indicate that a 114 mm (4·5 in) wall panel in a structure of the type tested would resist a lateral pressure of 14–21 kN/m² (2–3 lbf/in²).

Acknowledgement

The work described in this paper was carried out at the request of the Director, Building Research Station. Funds in support of the experiment were provided by the Ministry of Public Building & Works, now the Department of the Environment. The experimental structure was built in connection with research work sponsored by the Brick Development Association and the British Ceramic Research Association. A grant for staff salaries was provided by the Science Research Council.

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4. Institution of Structural Engineers, *Guidance on the Design of Domestic Accommodation in Loadbearing Brickwork and Blockwork to Avoid Progressive Collapse.* London, I.S.E., June, 1969.
5. Jones, L. L. and Wood, R. H. *Yield-line analysis of slabs.* London, Thames & Hudson; Chatto & Windus, 1967.

Appendix 1

*Design of floor slabs**

Wide slab units:

loading 40 lbf/ft² super $\quad P_{cb} = 1000$ lbf/in²

60 lbf/ft² self wt. $\quad P_{st} = 33\,000$ lbf/in²

$q = 100$ lbf/in²

Effective span 10·8 ft

Mid-span moment

Dead load $0{\cdot}071 \times 60 \times 10{\cdot}8^2 \times 12 =$ 5960 lbf/in

Live load $0{\cdot}096 \times 40 \times 10{\cdot}8^2 \times 12 =$ 5375 lbf/in

11 335 lbf/in

Support moment

Dead and live load $0{\cdot}125 \times 100 \times 10{\cdot}8^2 \times 12 =$

17 500 lbf/in

A_{st} for mid-span

$$\frac{11\,335}{33\,000 \times 4 \times 0{\cdot}95} = 0{\cdot}0905 \text{ in}^2/\text{ft}$$

Use 0·25 in square twisted bars at 8 in (0·095) centres both ways.

A_{st} for support moment

$$\frac{17\,500}{33\,000 \times 4 \times 0{\cdot}95} = 0{\cdot}134 \text{ in}^2/\text{ft}$$

*1 in = 25.4 mm; 1 lbf/in² = 6895 N/m²; 1 lbf/in = 0.113 Nm

Use $\frac{5}{16}$ in square twisted bars at 8 in centres with distributed steel 0·25 in (0·146 + 0.094) square twisted bars at 8 in centres.

$$\text{Shear stress } \frac{0{\cdot}62 \times 100 \times 10{\cdot}8}{12 \times 4 \times 0{\cdot}89} = 15{\cdot}7 \text{ lbf/in}^2$$

Lintol section 5 × 4 in

Bending moment $= 100 \times 6 \times 3^2 \times 1{\cdot}5 = 8100$ lbf/in

$$d_1 = \frac{8100}{4 \times 140} = 3{\cdot}8 \text{ in}$$

Depth provided = 4 in

$$A_{st} = \frac{8100}{33\,000 \times 4 \times 0{\cdot}896} = 0{\cdot}068 \text{ in}^2$$

Use 2 × $\frac{1}{4}$ in.

Appendix 2

*Stability calculations: yield-line analysis of slabs**

An approximate analysis of the resistance of the floor slabs to failure after removal of the walls has been carried out by yield-line theory according to normal procedures[5].

In this case (Fig 12) the slab is loaded by the weight of one storey-height panel of brickwork on the side fc.

Consider the yield-line pattern shown above. Let the slab be given a unit deflexion on line bc. To simplify calculations and since the load is distributed, the trapezoidal area B can be divided into a triangular and a rectangular area. Since bc deflects by one unit, the centre of gravity of each of the triangles abd and bde will deflect by one-third of a unit. Similarly, the centre of gravity of the rectangular area befc and the wall will each deflect by half of a unit.

Total external work $P = W \times \delta$

$= $ Total load × Deflexion of C.G. of loads

$= W' \times l \times \delta_w + w(A_A\delta_A + A_B\delta_B)$

where

δ = deflexion of C.G. of loads

w = distributed load (100 lbf/ft²)

A = area (ft²)

W' = wt. of wall/ft run (328 lbf/ft run)

l = length of wall (ft)

Suffix A, B, w represent sections abd, dbcf and the wall.

Hence,
$$P = W' \times 8 \times \tfrac{1}{2} + w[\tfrac{1}{2} \times X \times 8 \times \tfrac{1}{3} + \tfrac{1}{2} \times 8 \times X \times \tfrac{1}{3} + (10{\cdot}75 - X) \times 8 \times \tfrac{1}{2}]$$
$$= 5612 - 133{\cdot}33X \qquad \ldots(1)$$

Internal dissipation of energy in the yield lines (1, 2 and 3)

$D = \Sigma\, ml\theta_n$

where

m = normal moment/unit length on a yield line depending on the magnitude of the reinforcement.

θ_n = angle of rotation normal to yield line for the section under consideration.

l = length of yield line (ft)

$$D = \frac{0{\cdot}146}{0{\cdot}094} m \times 8 \times \frac{1}{X} + m \times 8 \times \frac{1}{X} + m \times X \times \frac{1}{8} + (2m \times 2{\cdot}5 + m \times 6 + 2m \times 2) \times \frac{1}{8}$$

(See design calculations for reinforcements)

$$D = m\left(\frac{20{\cdot}42}{X} + \frac{X}{8} + \frac{15}{8}\right) \qquad \ldots(2)$$

External work P = Dissipation of energy D, and by equating the expressions for P and D (equations (1) and (2)) we get:

$$m = \frac{5612 - 133{\cdot}33X}{\frac{X}{8} + \frac{15}{8} + \frac{20{\cdot}42}{X}}$$

$$\frac{dm}{dx} = 0$$

$$\text{or } 0 = -133{\cdot}33\left(\frac{X}{8} + 1.875 + \frac{20{\cdot}42}{X}\right) - (5612 - 133{\cdot}33X)\left(\frac{1}{8} - \frac{20{\cdot}42}{X^2}\right)$$

or $X^2 + 5{\cdot}72\,X - 120{\cdot}44 = 0$

$$X = \frac{-5{\cdot}72 \pm \sqrt{(5{\cdot}72)^2 + 4 \times 120{\cdot}44}}{2}$$

Acceptable solution $X = +8{\cdot}6$

$$m = \frac{5612 - 133{\cdot}33 \times 8{\cdot}6}{\frac{8{\cdot}6}{8} + \frac{15}{8} + \frac{20{\cdot}42}{8{\cdot}6}} = 838{\cdot}66 \text{ ft lbf/ft} = 10\,064 \text{ in lbf/ft}$$

This is less than the design moment at midspan and much less than the ultimate moment so that the slab will not fail when the wall under boundary fc is removed.

In this case (see Fig 13) because there was no through reinforcement over the wall in the bottom of the slab, it was assumed that a crack would develop along line cf as

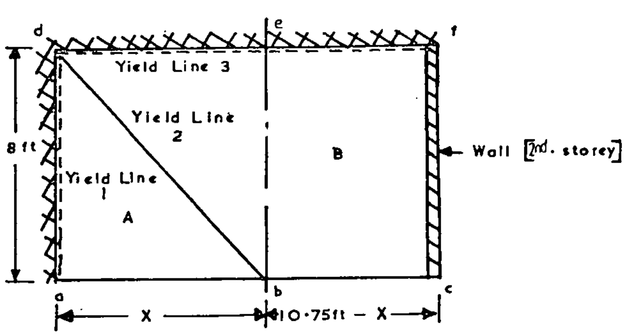

Fig. 12. Yield-line analysis of slabs: test 1.

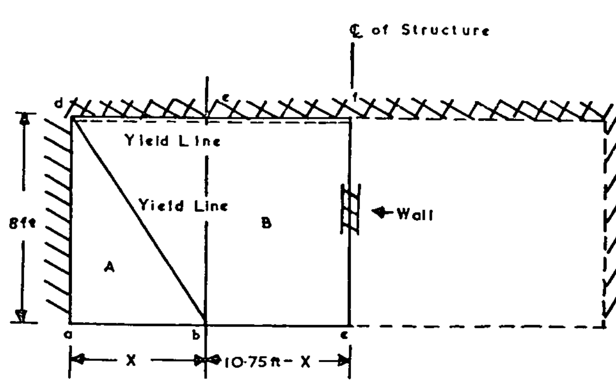

Fig. 13. Yield-line analysis of slabs: test 2.

* 1 ft = 0.305 m; 1 lbf/ft² = 47.88 N/m²; 1 lbf/ft = 1.356 Nm.

soon as wall support below is removed. The wall load will then be equally shared by the two sections of the slab.

External work $P = 2W' + w\,(8\,X/3 - 4X + 43)$ (Calculation similar to Test 1)

$$= 4956 - 133{\cdot}33X \qquad \ldots(3)$$

$$\text{Dissipation of energy } D = m\left(\frac{X}{8} + \frac{15}{8} + \frac{8}{X}\right) \qquad \ldots(4)$$

(See the design calculation, Appendix 1)

By equating P and D we get (equations (3) and (4)):

$$m = \frac{4956 - 133{\cdot}33X}{\frac{X}{8} + \frac{15}{8} + \frac{8}{X}}$$

$$\frac{dm}{dX} = 0 = -133{\cdot}33\left(\frac{X}{8} + \frac{15}{8} + \frac{8}{X}\right) - (4956 - 133{\cdot}33X)\left(\frac{1}{8} - \frac{8}{X^2}\right)$$

or $869{\cdot}4937\,X^2 + 2133{\cdot}28X - 39\,648 = 0$

or $X^2 + 2{\cdot}45\,X - 45{\cdot}60 = 0$

$$X = \frac{-\,2{\cdot}45 \pm \sqrt{(2{\cdot}45)^2 + 4 \times 45{\cdot}60}}{2}$$

Acceptable solution $X = +\,5{\cdot}64$

$$m = \frac{4956 - 133{\cdot}33 \times 5{\cdot}64}{\frac{5{\cdot}64}{8} + \frac{15}{8} + \frac{8}{5{\cdot}64}} = 1051 \text{ ft lbf/ft} =$$

$$= 12\,618 \text{ in lbf/ft}$$

This is greater than the design moment at midspan but still much less than the ultimate moment of the slab so that failure will not take place when the wall is removed.

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institution notes

Continued from page 440

should be expressed orally, in the drawing office. In particular, it is necessary to consider whether any colloquial terms could possibly be taken to relate to the inch instead of the millimetre, or vice versa, leading to errors in production.

2. *Units of Length*

   In the Engineering industries there is a general intention to use the millimetre for most applications. For the larger dimensions the metre will be used. For very small dimensions in precision engineering the micrometre or decimal fractions of the millimetre will be used.

3. *Synonym for Millimetre*

   There are three alternatives:
   (*a*) millimetre
   (*b*) milli
   (*c*) mil

   The full name 'millimetre' will undoubtedly be widely used and is recommended particularly in the early stages of the changeover. Of the two colloquial names, 'milli' is preferred as there is a possibility of 'mil' being confused with the Americanism for the one-thousandth of an inch.

4. *Synonym for Micrometre*

   The only feasible alternative to the word 'micrometre' in full is the 'micron'. Although this is deprecated by some authorities it is nevertheless extensively used and is preferable to the 'micrometre' which, apart from being unwieldy, bears a resemblance to the measuring instrument, the 'micrometer'.

5. *Name for 'O'*

   The alternatives here are:
   (*a*) Oh
   (*b*) nought
   (*c*) zero

   The third alternative, 'zero', is gaining popularity and may well become the accepted international standard but this is likely to take some time. Meanwhile it is recommended that the second alternative, 'nought', be used in preference to 'oh' which is difficult for some Europeans and could cause confusion in communication with metric countries. Thus, 0·001 mm would be expressed as point nought nought one.

6. *Decimal Fractions of the Millimetre*

   The main alternatives here are:

Dimension	Form 1	Form 2	Form 3
0·1 mm	point one	one tenth	one hundred microns
0·01 mm	point nought one	one hundredth	ten microns
0·001 mm	point nought nought one	one thousandth	one micron

In the drawing office and for formal expression of dimensions, Form 1 is recommended.

On the shop floor it is likely that, in due course, 'one tenth' and 'one hundredth' will be used respectively for 0·1 mm and 0·01 mm but where there is a possibility of confusion with imperial measurements, Form 1 is recommended for an interim period of time.

On the shop floor and in the tool room the word 'micron' for 0·001 mm will undoubtedly be widely used and is recommended.

As has been mentioned, the Royal Society/CEI Joint Committee on metrication would welcome constructive comments from members of the Institution upon the extent and intensity of the problem and upon the various recommendations set out above for colloquial usage. Comments should be addressed to the Royal Society/CEI Joint Committee on Metrication Secretariat, Institution of Production Engineers, 10 Chesterfield Street, London W1X 8DE.

Erratum

Attention is drawn to an error in the presentation of equation (2) in the paper 'Flexure-shear strength of reinforced concrete deep beams' by Dr. M. A. Sheikh, Professor H. A. R. de Paiva and Professor A. M. Neville, published in *The Structural Engineer*, Volume 49, No. 8, August 1971, pages 359–363.

Equation (2), page 360, describes an interaction curve between v/f'_c and f_x/f'_c. The left hand side of equation (2) should therefore read $\frac{v}{f'_c}$ and not f'_c.

Computers in Civil Engineering

Members of the Institution are invited to join members of the Institution of Civil Engineers at a meeting being held there on Monday 22 November at 5.30 pm when Mr. K. M. Vine-Lott, CEng, FICE will introduce an informal discussion 'Use of computers in civil engineering'. Introductory notes for the discussion will be available from ICE, Great George Street, London SW1P 3AA from 12 November and any member who wishes to attend should send for a copy of these notes.

Courses for Technicians

The C&CA has announced three training courses for structural engineering technicians currently undertaking HNC studies: 'Concrete construction for higher technicians' (part 1) will extend from 25 to 29 October and will deal with the production, handling and placing of concrete; part 2 of the course from 1 to 5 November will cover more advanced concrete mix design, testing, quality control and project work. 'Concrete for technicians—refresher course' from 6–10 December is intended for those technicians who have completed the HNC and require a refresher before undertaking the Institution's Technician Test.

Full details of each of these courses may be obtained from the Registrar, C&CA Training Centre, Fulmer Grange, Fulmer, Slough SL2 4QS.

Building Exhibition 1971

Members are reminded that the International Building Exhibition will be held at Olympia, London from 17 to 27 November 1971. As usual a series of conferences will be held in association with the Exhibition including on 24 November a one-day symposium 'Prospects for fibre reinforced construction materials' sponsored by the Building Research Station; and on Thursday 25 November, sponsored by the CITB, an afternoon conference 'Management in the construction industry'.

Full Scale Tests on the Lateral Strength of Brick Cavity Walls with Precompression

By

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ABSTRACT

This paper gives an account of tests on the lateral strength of 10½ in. (267 mm) brickwork cavity walls, with and without returns, under varying degrees of precompression. The walls were built within an experimental five storey cross-wall structure which provided the precompression with realistic boundary conditions. The walls were of storey height and varied in length from 4 ft (1.22m) to 15 ft 6 in. (4.72 m). Precompression was applied to the inner leaf of the wall under test. Lateral load was applied by hydraulic jacks or by an air bag. The results of the tests show the effect of one or two returns on the lateral strength of walls as compared with walls without returns having the same ratio of height to width and precompression. A simple theory for the strength of walls with returns is in development which shows reasonable agreement with the experimental results.

1. INTRODUCTION

Simply supported brickwork panels can resist only rather small transverse loads because of the low tensile strength of the material. However, both precompression in load bearing walls and boundary restraints in the case of infill panels can produce substantially increased lateral resistance as a result of "arching" effects within the thickness of the wall. This paper presents the results of a series of tests on 10½ in. (267 mm) cavity walls having different lengths, numbers of returns and precompressions. The test walls were built into an experimental multi-storey structure so as to obtain realistic end conditions. The test results are compared with those given by an approximate theory based on rigid body statics and assumed failure conditions.

2. EXPERIMENTAL

Deep frog Fletton common bricks were used for all tests except for two 9 in. (229 mm) walls for which a three-hole perforated wirecut brick was used. The average crushing strengths were

Table 1
Mortar and Brickwork Strengths

Test No.	Mortar Strength N/mm²	Average Age days	Brickwork Strength N/mm²	Average Age days	Notes
1	16.75	16	18.15	17	Brickwork strength
2	8.90	36	27.15	36	
3	12.55	10	5.85	10	1. Tests 1 & 2 use
4	11.70	13	7.30	12	215 mm cubes.
5	14.40	12	5.50	9	
6	8.50	9	7.20	16	Tests 3-18 use 6
7	8.55	12	7.05	10	brick prisms.
8	7.90	9	7.40	11	
9	9.05	19	8.15	8	2. All brickwork
10	13.15	9	6.55	8	specimens tested
11	9.75	11	7.45	14	with plywood top and
12	14.50	17	8.55	21	bottom.
13	17.90	23	6.55	23	
14	15.35	14	8.10	15	
15	14.70	21	5.50	21	
16	14.10	12	7.05	13	
17	17.10	26	6.85	20	
18	22.50	39	5.60	39	

26.13 N/mm² and 37.92 N/mm² respectively. A 1:¼:3 rapid hardening Portland cement:lime:sand mix by volume was used for all tests.

The walls were built to the B.C.R.A. Model Specification[1]. One bricklayer usually took two to three days to complete each wall. Galvanised steel ties in the cavity walls were spaced according to C.P.111:1964[2] (36 in. (0.91 m) horizontally and 18 in. (0.46 m) vertically). The outer leaf contained a damp proof course. Walls were cured for a minimum of 7 days before testing. 4 in. (102 mm) mortar cubes and 6-course single brick prisms were made as control specimens and tested in compression on the same day as the corresponding wall test; results for both cubes and prisms are shown in Table 1.

To apply precompression to the test walls and to simulate the actual end conditions in practice, the five storey experimental building built at the Torphin Quarry[3] was modified according to the following scheme. (Figure 1). Two cross walls and shear walls on the first floor and two cross walls on the ground floor adjacent to the quarry face were first removed and temporarily replaced by Acrow props. On the first floor, across the whole width of the building, the cross walls were replaced by a 9 in. reinforced brickwork beam consisting of two leaves built in stretcher bond with a 1 in. (25.4 mm) gap between them with inter-connecting ties. Horizontal reinforcement was provided in the bed joints and, through the gap, vertical reinforcement was

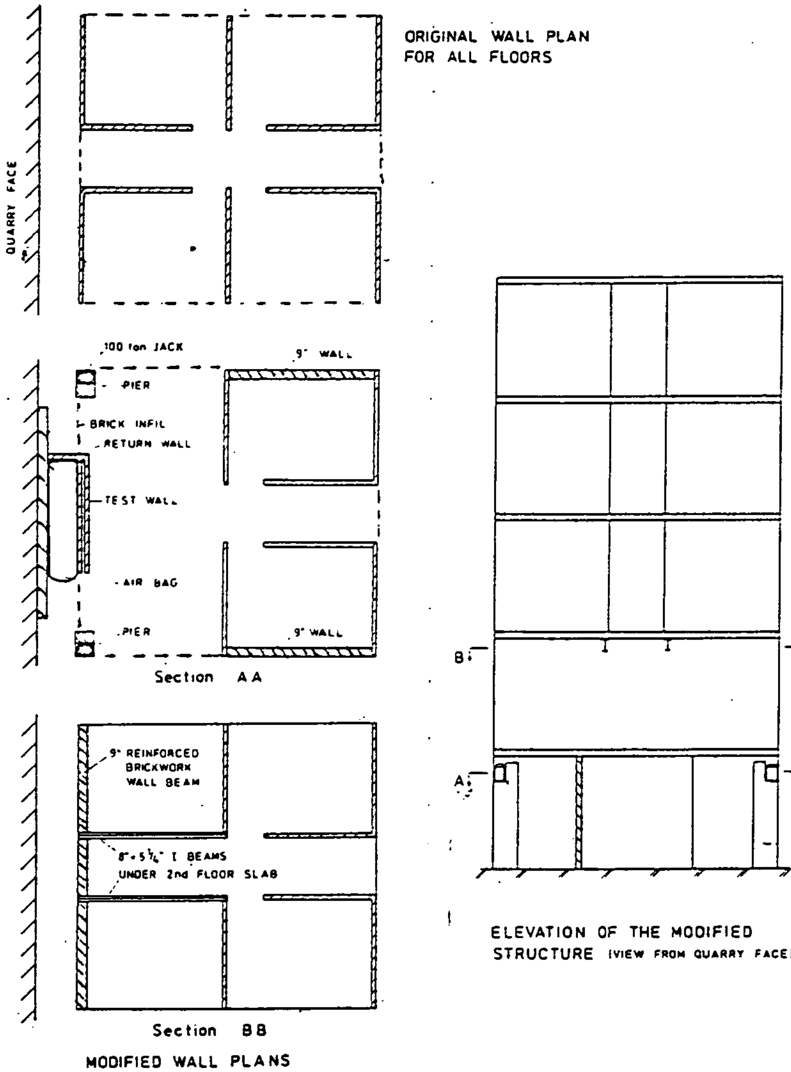


Figure 1 — Modified five storey brick structure.

tied to the first floor slab by Rawlplugs and passed through holes drilled in the second floor slab and bolted. The gap between the leaves was then grouted. Two 8 × 5¼ in. (203 × 133 mm) steel I sections replaced the shear walls, the sections spanning between the top of the reinforced beam and the centre cross walls. On the ground floor two brickwork piers were built under the corners of the slab. The tops of the piers were stepped, the lower step holding a 100 ton (996.4 kN) jack and a load cell, the higher step holding packing plates and another load cell. Two extra 9 in. (229 mm) shear walls, on the ground floor, were built in the opposite side of the building – they increased the shear resistance provided by the two 4½ in. (114 mm) shear walls which remained thus enabling the building to resist the transverse load applied to the test walls. After 14 days curing, the weight of the building was taken off the Acrow props by the jacks on the piers. After removal of the props the building was lowered onto the top step of the piers, the piers now taking the weight of the building. Two shear walls on the ground floor were then knocked out to complete the modifications.

2.1 Measurement and Application of Precompression to Test Walls

The proportion of the weight of the building bearing onto the test wall had to be found in each case. This depended on the height of the test wall – small changes in height resulting in different precompressions due to the stiffness of the building – increasing wall height causing an increase in precompression. To check this variation, a datum was fixed at the level where the building rested on top of the piers. A permanently fixed dial gauge then measured all changes in level from that datum. To obtain the relationship between load and level, the building was lifted and lowered over a small range covering the variation of wall height and wall lift expected. The level was obtained from the dial gauge and the load from the load cells situated on top of the jacks while the load cells on the top level of the piers gave an added check on the load at datum level. The results are shown in Figure 2.

The day after a test wall was built, the building was lowered onto the wall. Before doing this, the datum level was checked by the dial gauge and the readings from the load cells positioned on top of the piers were taken. The building was then raised by the 100 ton (996.4 kN) jacks to allow the load cells to be removed from the tops of the piers and to enable a mortar bed to be spread

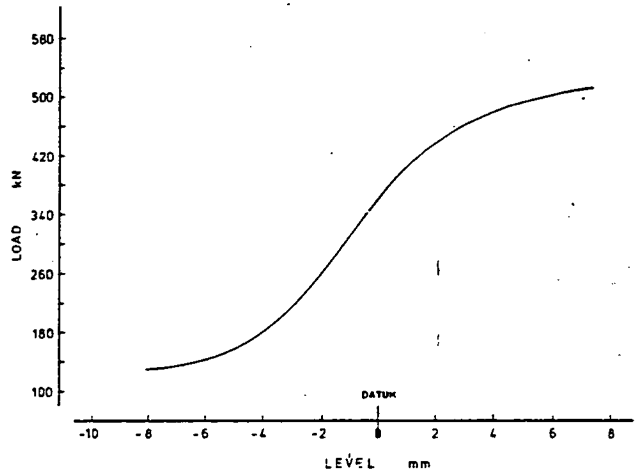


Figure 2—Relationship between load and level of the building.

on top of the test wall—in the case of cavity walls on the inner leaf only. The building was then slowly lowered onto the wall, the wall now taking all the weight. The dial gauge reading was again taken. Knowing the load at datum level (from the load cells) and the change in level between this level and the final level of the wall (from the dial gauge), the precompression on the wall was found with the aid of Figure 2.

2.2 Application of Transverse Load

Transverse load was applied by jacks or air bags. For the 4 ft. (1.22 m) wide walls, the load was applied by eight evenly spaced 5 ton (49.8 kN) jacks acting between the quarry face and the wall. The other tests used an air bag. A nylon net enclosed the bag and a 1 in. (25.4 mm) thick foam rubber sheet was inserted between the bag and the test wall to protect the bag from falling bricks. Air was supplied by a small mobile compressor while the pressure was measured by a manometer and an electrical pressure transducer.

2.3 Test Procedure

Dial gauges were fixed in position to measure the transverse deflection of the test wall. In some cases Demec discs were positioned at 12 in. (305 mm) gauge lengths to give an indication of the strain distribution over the cross section of the wall. Two displacement transducers were arranged to measure the uplift of the structure as the wall failed. The manometer and air compressor were connected to the air bag by plastic tubing. The pressure and displacement transducers were connected to a pen chart recorder. If there were any returns to the walls, the returns were prestressed to preclude premature failure by shearing of these elements.

Pressure was applied in suitable increments, the pressure being held constant at each stage while readings were taken. Dial gauges were removed at approximately three quarters of the maximum load. The 100 ton (996.4 kN) jacks were then allowed to follow the uplift of the building so that at failure there would only be a small drop in level. The jacks were installed with non-return valves as an added safety precaution. Care was taken to see that the jacks did not relieve the wall of any load. On failure, the jacks took the weight of the building which was then lifted to enable the load cells to be replaced on top of the piers. Load cell readings were noted together with the permanent dial gauge as a check on the load carried by the wall.

Transverse deflection results are shown in Figure 3 and examples of recorded uplifts close to the failure load are given in Figure 4. A summary of the test results is contained in Table 2.

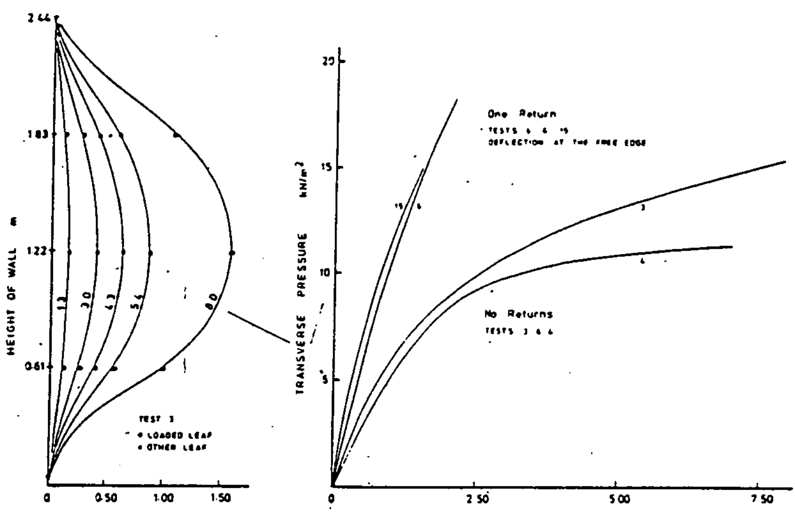


Figure 3 (a) — Overall transverse deflection.

(b) — Deflection at mid-height versus transverse pressure.

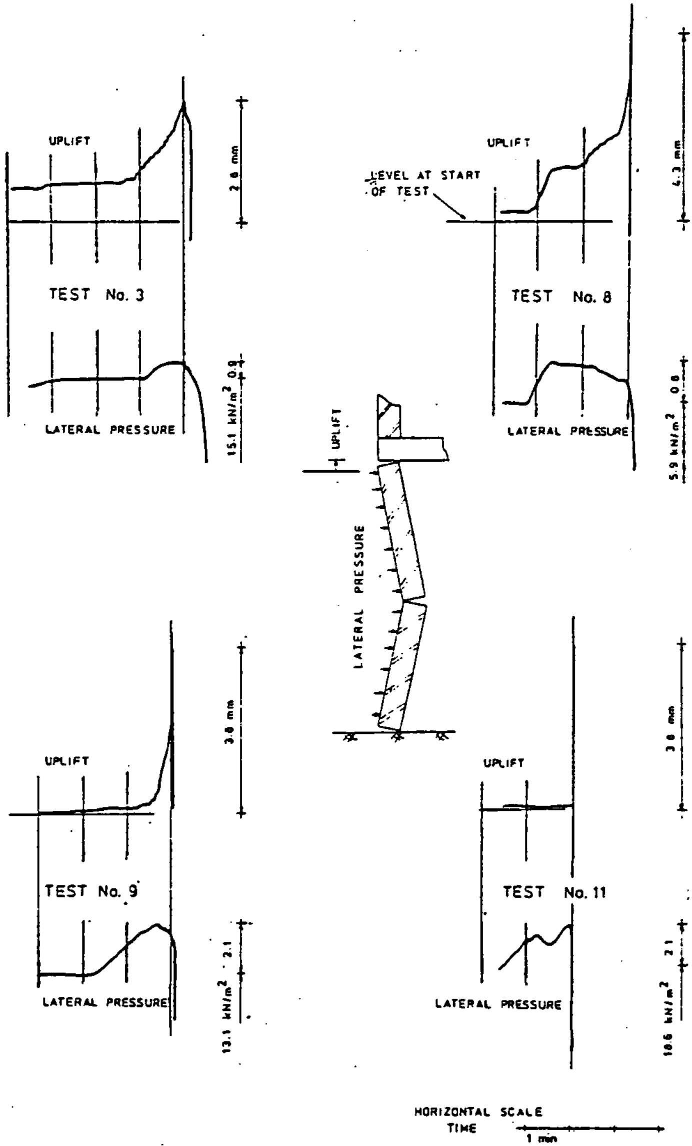


Figure 4 — Uplift of walls at failure.

Table 2
Test Results

Test No.	No. of Returns	Overall Length m	ǂ Length/ Height Ratio	Thickness mm	Cross-sectional Area of wall cm^2	Load on Wall kN	Wall Stress N/mm^2	Max. Uplift mm	Max. Lateral Load		Ratio Exp./ Simple Theory
									Exp. kN/m^2	Theory * kN/m^2	
1	0	1.22	0.5	213	2600	–	–	–	72.4	–	–
2	0	1.22	0.5	213	2600	–	–	18	84.1	–	–
3	0	1.24	0.5	102:76:102 Cavity	1265	143	1.14	2.8	16	16	1
4	0	1.24	0.5	,,	1265	120	0.944	2.8	14	13.1	1.05
5	1	1.37	0.52	,,	1395	159	1.14	2.3	26.2	15.9	1.7
6	1	1.37	0.52	,,	1395	134	0.96	3.0	25.5	13.3	1.9
7	0	2.49	1.0	102	2530	122	0.48	4.6	7.6	6.7	1.15
8	0	2.49	1.0	102	2530	131	0.52	4.3	6.7	7.2	0.95
9	1	2.59	1.0	102:76:102 Cavity	2630	159-184	0.61-0.70	3.8	15.2	8.4-9.7	1.8-1.6
10	1	2.59	1.0	,,	2630	136	0.52	4.3	16	7.2	2.2
11	2	2.72	1.0	,,	2760	133	0.48	3.8	20.7	6.7	3.1
12	2	2.72	1.0	,,	2760	149	0.54	2.8	23.9	7.6	3.1
13	1	4.67	1.9	,,	4750	229	0.48	3.6	5.5	6.7	0.85
14	1	4.73	1.9	,,	4800	264	0.55	3.8	6.2	7.6	0.8
15	1	1.32	0.5	,,	1350	135	1.00	2.8	24	13.9	1.75
16	1	1.88	0.75	,,	1910	140	0.73	2.8	17	10.1	1.7
17	2	3.89	1.5	,,	3950	149	0.38	3.0	12	5.2	2.2
18	2	4.57	1.8	,,	4645	169	0.37	2.0	9.7	5.1	1.9

Notes: Height of all walls 2.44 m; Theoretical max. uplift for 213 mm wall thickness = 37 mm
,, ,, ,, ,, 102 mm ,, ,, = 8.4 mm

ǂ length without return
* Theory without return

2.4 Behaviour of the Wall up to Failure

At low transverse loads, walls without returns behaved as homogeneous units, the precompression delaying the start of tensile cracking. Cracking started soon after the tensile bending

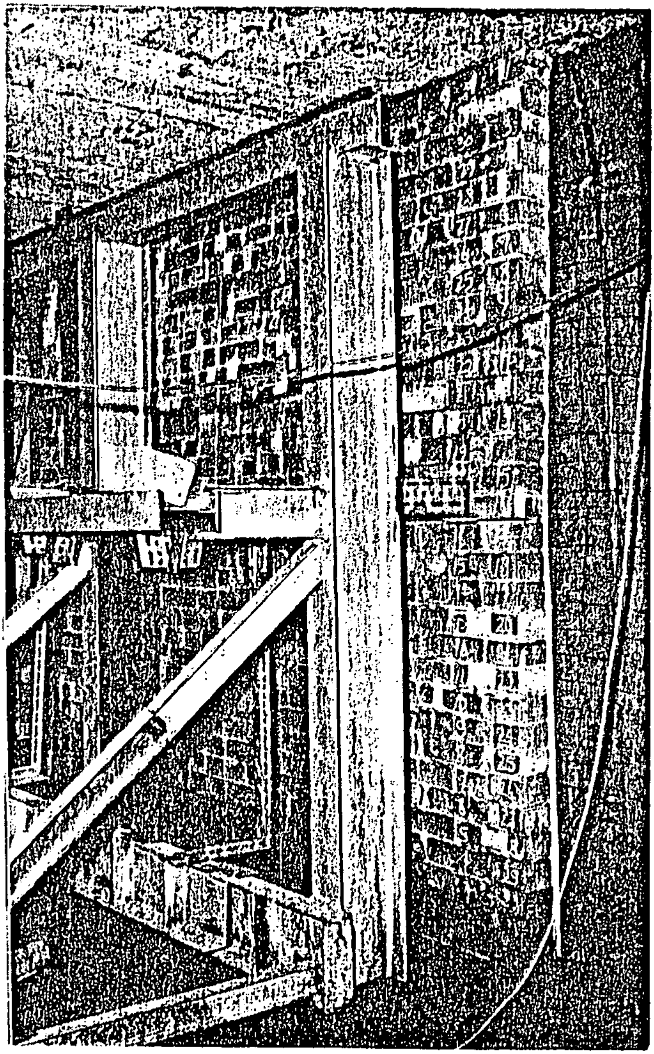
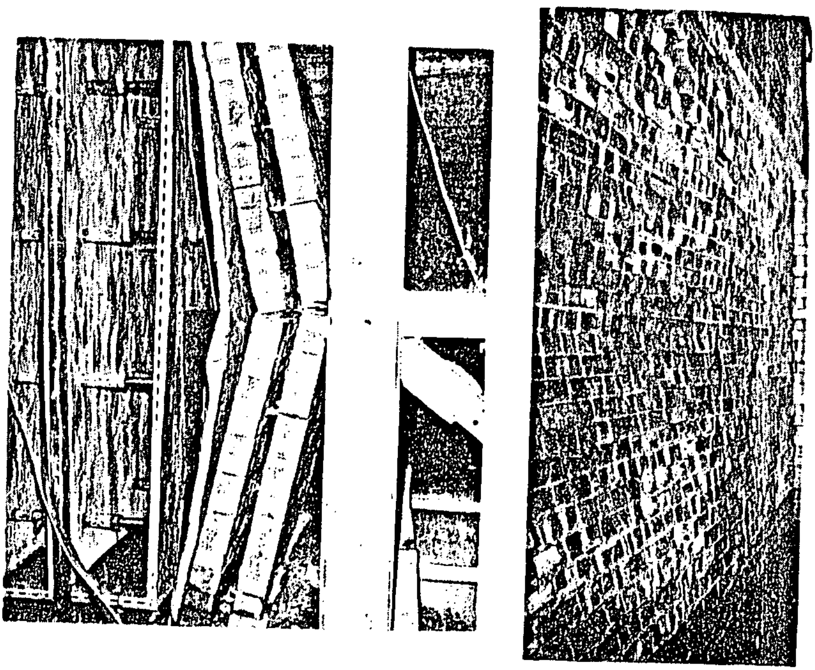


Figure 5 — Typical failure of wall without return.

strain exceeded the precompressive strain. With increasing transverse load cracking gradually extended through-out the cross section. When the crack extended across most of the cross section, greatly increased transverse deflections occurred together with lifting of the building, associated with only a small increase in transverse pressure. There was usually a small amount of crushing in courses adjacent to the points of rotation. Typical failures can be seen in Figs. 5, 6 and 7.

The effect of returns on the strength of walls of varying L/H ratios is shown in fig. 8. In the case of short walls, the vertical joints at the returns failed first, after which the walls buckled outwards as in the case of strip walls. The strengthening effect of returns fell away as the length of the wall increased and the failure mode changed to the development of a "roof" pattern of cracked lines.



(a) (b)

Figure 6 — Typical failures of wall with one return :—
(a) Test 5
(b) Test 14.

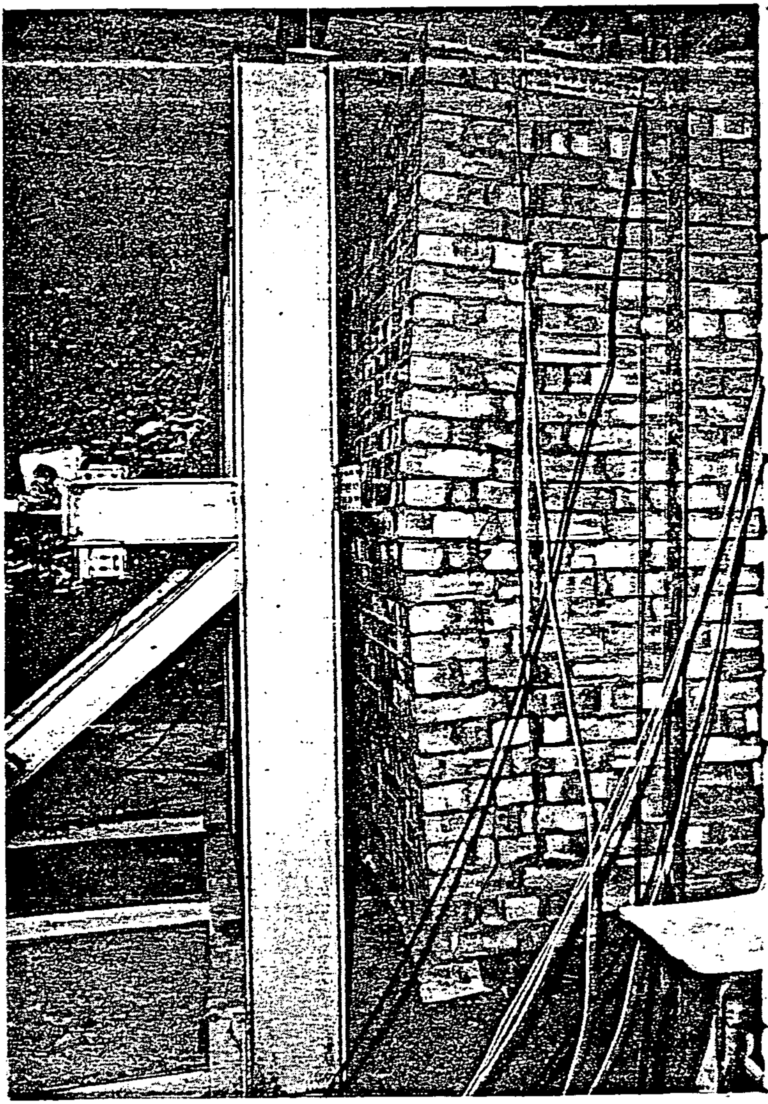


Figure 7 — Typical failure of return.

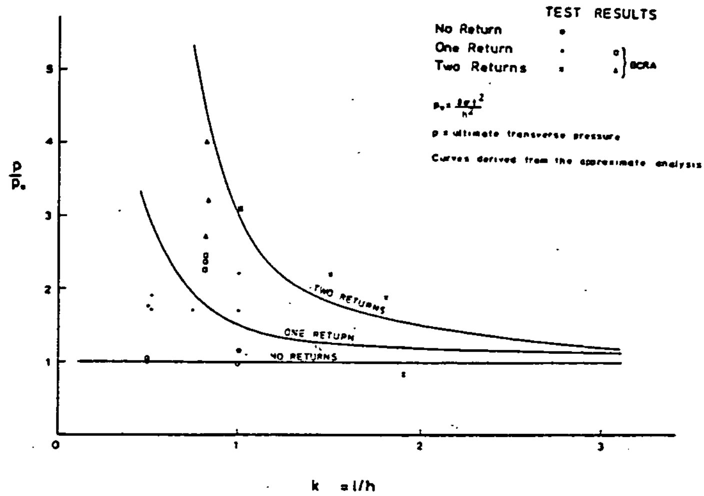


Figure 8 – Effect of returns on the lateral strength of walls with varying L/H ratios.

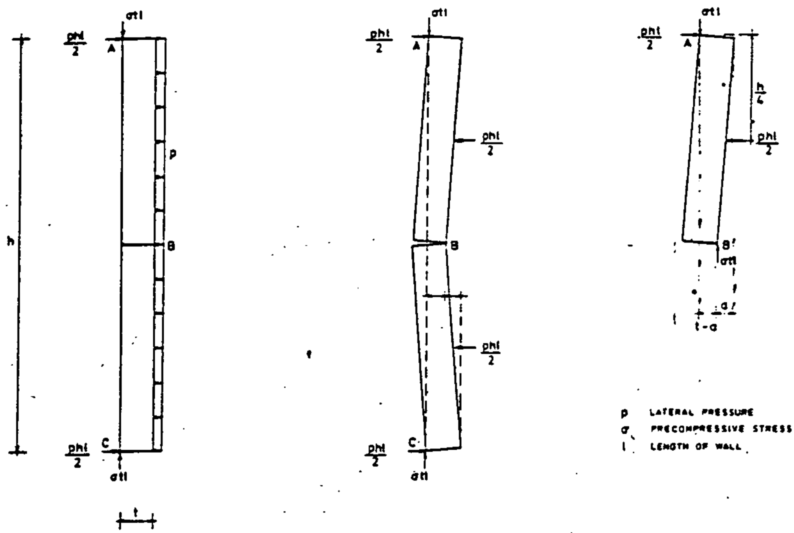


Figure 9 – Simplified failure mechanism of walls supported top and bottom.

3. AN APPROXIMATE THEORY FOR THE LATERAL RESISTANCE OF WALLS WITH RETURNS

An approximate analysis of the behaviour of transversely loaded walls without returns is possible on the following assumptions:

(1) Rigid materials (including supports)
(2) Failure occurs by horizontal cracking at the top, centre and bottom of the wall causing rotation about points A, B and C. (Figure 9).

The forces on the wall are a vertical precompressive load and a uniform transverse pressure. Consider the forces acting on the top half of the wall and take moments about A:

$$\sigma tl(t-a) = \frac{phl}{2} \cdot \frac{h}{4} \qquad (1)$$

$$p = \frac{8\sigma t(t-a)}{h^2}$$

σ = precompressive stress
t = thickness of wall
l = length of wall
h = height of wall
p = transverse pressure
a = horizontal distance through which centre of the wall has moved.

If the precompressive stress is constant throughout uplift of the wall at failure, then the maximum transverse pressure occurs when a = o.

$$p_o = \frac{8\sigma t^2}{h^2} \qquad (2)$$

If the precompression increases with uplift of the wall, it is possible for the moments of resistance, $\sigma tl(t-a)$, to increase even though the moment arm $(t-a)$ decreases.

Considering now walls with returns, part of the lateral pressure on the main wall is transmitted to the return, the interaction setting up tensile and bending stresses in the latter along the junction. In this analysis the bending moment is neglected and the tensile forces are assumed to be distributed in a triangular manner on the height of the wall, as indicated in Figure 10a.

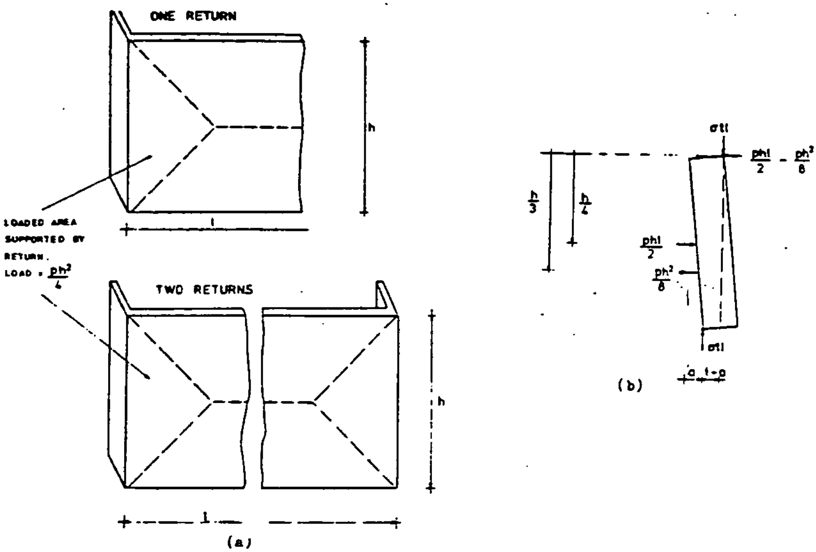


Figure 10 — Simplified failure mechanism for walls with returns.

In a wall with one return, considering the forces acting on the top half of the wall (Figure 10b) and taking moments about the top:

$$\sigma tl(t-a) = \frac{phl}{2} \cdot \frac{h}{4} - \frac{ph^2}{8} \cdot \frac{h}{3}$$

$$p = \frac{8\sigma t(t-a)}{h^2} \cdot \frac{1}{1-\frac{1}{3k}} \qquad (3)$$

where $k = {}^{l}/h \geqslant 0.5$

For constant precompression, the maximum transverse pressure occurs when

$$a = 0$$

thus $p_1 = \dfrac{8\sigma t^2}{h^2} \cdot \dfrac{1}{1-\frac{1}{3k}}$

or letting $p_o = \dfrac{8\sigma t^2}{h^2}$ (Eqn 2)

$$p_1 = \frac{p_o}{1 - \dfrac{1}{3k}} \qquad (4)$$

Similarly, in a wall with two returns:

$$p_2 = \frac{p_o}{1 - \dfrac{2}{3k}} \qquad (5)$$

where $k \geq 1.0$

Figure 8 shows p_1/p_o and p_2/p_o calculated from (3) and (5) above for a range of length/height ratios along with experimental values from the tests described in this report; a number of additional points are included from tests carried out by the British Ceramic Research Association[4]. In calculating the strength of the cavity walls, the effect of the unloaded leaf was neglected.

CONCLUSIONS

The results of these experiments confirm that the strength of strip walls with precompression can be calculated with good accuracy by the simple "arching" theory. The effect of returns has been investigated and it would appear that the strength of these walls can be predicted with reasonable accuracy by a very simple analysis, provided that the L/H ratio is greater than 0.75. For aspect ratios lower than this the actual strength of a wall is less than that indicated by the simple theory.

The results obtained from full scale experiments at Torphin Quarry are consistent with laboratory tests carried out by the British Ceramic Research Association.

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TEST ON A THREE-STOREY CAVITY WALL STRUCTURE

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TEST ON A THREE-STOREY CAVITY WALL STRUCTURE

The paper describes a test done on a three storey model cavity wall structure carried out as part of a programme of work intended to establish a basis for the calculation of the effective wall height, effective eccentricity due to floor loadings and the amount of load shared between the leaves in actual construction. It also compares the results obtained from a full scale test done simultaneously in Torphin Quarry.

VERSUCH AN EINER ZWEISCHALENKONSTRUKTION VON DREI STOCKWERKEN

Dieses Referat beschreibt einen Versuch der an einem Modell von Zweischalenkonstruktion von drei Stockwerken ausgeführt worden ist. Der Versuch war Teil eines Untersuchungsprogramms, aufgestellt zur Ermittlung einer Basis für die Berechnung der effektiven Mauerhöhe, der effektiven Exzentrizität und der Lastverteilung zwischen den beiden Wandscheiben.

Ein Vergleich des Resultats mit dem eines Versuchs am Modell auf wahrer Grösse, ausgeführt im Steinbruch von Torphin wird gemacht.

ESSAI SUR UNE STRUCTURE DE MUR CREUX A TROIS ETAGES

Cette communication décrit un essai effectué sur un modèle de mur creux à trois étages, essai entrepris en tant que partie d'un programme conçu comme base pour le calcul de la hauteur de mur effective, de l'excentricité effective et de la répartition de la charge entre les deux feuilles d'un mur.

Le résultat est également comparé avec celui de l'essai sur grandeur réelle effectué à la carrière de Torphin.

PROEF OP EEN SPOUWKONSTRUKTIE VAN DRIE VERDIEPINGEN

Deze mededeling beschrijft een test die op een model van een spouwkonstruktie van drie verdiepingen is uitgevoerd als deel van een programma opgesteld als basis voor de berekening van de effektieve muurhoogte, de effectieve excentriciteit en de verdeling van de last tussen de twee muurbladen.

Het resultaat wordt ook vergeleken met dat van de proef op ware grootte in de steengroeve van Torphin.

1. INTRODUCTION

In the majority of load-bearing brickwork structures in the U.K. the external walls are built in cavity construction. Although this type of construction is common, there is a complete lack of experimental data which could be used as a basis for the assessment of load distribution from floor slabs to the leaves of a cavity wall, or concerning the floor/wall interaction, which could offer guidance as to the effective height and eccentricity of loading for such walls. A major research programme[1] in the University of Edinburgh was therefore undertaken to study the structural behaviour of multi-storey cavity wall structures. The work described in this paper forms a part of this programme.

Figs. 1 and 2 show a three-storey cavity wall structure built with $\frac{1}{3}$rd scale bricks in 1:$\frac{1}{4}$:3 (cement: lime:sand) mortar for testing in the laboratory. Third scale galvanised steel twist ties conforming to B.S. 1243 were used. The ties connecting the two leaves of the cavity wall were spaced 300 mm horizontally and 150 mm vertically staggered.

2. TEST ARRANGEMENT

The cavity walls were built inside two steel channels which in turn rested on load cells. Similarly, the single leaf wall was built inside the channel. Initially, to provide lateral stability the structure was tied to a light steel frame by steel struts (75 mm long) and pinned to the slabs at each level on the side of the single leaf wall. This arrangement proved very unsatisfactory since the structure tended to sway with no definite pattern. This was cured by connecting the structure at each slab level through a pinned strut to a heavy steel box frame fixed at the base to the strong floor of the laboratory. To increase the rigidity, the frame was put under tension by two steel props reacting against the strong floor and the horizontal members of the box frame.

3. LOADING ARRANGEMENT AND INSTRUMENTATION

Precompression (59.6 kN/m^2) was applied to the walls by hydraulic jacks operated by a hand pump. Three proving rings recorded the load applied to the top of the walls.

The design floor loading (217 Kg/m^2) was applied by lead billets, uniformly distributed over the floor slabs.

The deflection was measured by means of dial gauges (Fig. 1). The rotation at the wall/floor junctions was measured by an "Electrolevel" instrument reading to 0.05 m rad.

4. METHOD OF TESTING

4.1 Modulus of Elasticity

Six-course prisms were tested in compression to obtain the modulus of elasticity. From the stress-strain curve the average value of E for $\frac{1}{3}$rd scale brickwork was found to be 5.6 kN/mm^2.

The modulus of elasticity for the concrete slab was obtained from the results of a load-deflection test in which the slabs were tested as simply supported and subjected to uniformly distributed load. The average value of the modulii of elasticity were 8.5 kN/mm^2 for the 1st and 2nd and 9.9 kN/mm^2 for the third floor respectively.

4.2 Structure

Before loading the slabs, precompression was applied at the top of the walls by means of hydraulic jacks, as previously described. The floor slabs were loaded in turn by means of lead billets. After loading all the floors, measurements were taken of deflection of the walls and slabs, rotation at the wall/floor junctions and the load cells at the base of the cavity wall. Tests were repeated several times to obtain a realistic average for the deflection results. The test results are shown in Fig. 3.

5. ANALYSIS AND DISCUSSION OF THE TEST RESULTS

5.1 Rotation and Deflection of floor slabs

The measured rotations at the ends of floor slabs were less than the rotation of similar slabs simply supported. For simply supported slabs at 1st and 2nd floor levels the calculated end rotation was 3.6×10^{-3} radians and for the 3rd floor slab was 3.1×10^{-3} radians. Comparing these values with the experimental rotation (Fig. 3) it became evident that the floor slabs behaved as partially fixed at the ends. From the results of the rotation measurements, the fixed end moments for each slab were calculated and thus the points of inflection in the slab were fixed. The end moments were assumed proportionate to the end rotations. The theoretical deflection of the slab was calculated from the bending moment diagram of the partially fixed slabs. The result of this analysis is shown in Fig. 3. There is very good agreement between the calculated and experimental results.

Mid span deflections for the 1st, 2nd and 3rd floors are 1.1, 1.2 and 1.3 mm respectively. The deflection thus tends to increase with increasing floor level which is consistent with the findings of the full-scale test. It is difficult to compare directly the results of model and full-scale[2] tests because of wide differences in their constructional details.

5.2 Rotation and Deflection of Walls

The measured rotations at the base of the walls were zero.

Lateral deflection of the walls is shown in Fig. 3. Except in the top floor, the curvature is similar in both the leaves of the cavity wall as would be expected since they were connected by ties. However, the difference in curvature of the leaves at the third floor may be due to the ineffectiveness of the ties connecting the free end of the outer leaf to the inner wall. The deflection of the cavity wall is less than the single-leaf wall (Fig. 3); it appears therefore that the cavity wall is slightly stiffer than the latter, which is reasonable.

Points of inflexion deduced from the deflection curves are approximately:

Point of Inflexion	Storey Level					
	1		2		3	
	Exp.	Theory	Exp.	Theory	Exp.	Theory
Inner-Leaf (Cavity Wall)	0.2	0.19	0.5	0.47	0.46	0.43
Single Leaf	0.2	0.33	0.44	0.52	0.43	0.44

The theoretical results were obtained from a computer frame analysis which assumes rigid joints.

The effective height equal to 0.75 of the actual height as stipulated in the British Code of Practice[3] (CP.111:1970) for design appears reasonable for the ground floor for this type of loading. The effective height for the ground floor wall in the full-scale test[2], subjected to similar loading, was between 0.8 to 0.7 of actual height for the cavity walls which confirms the model test results.

5.3 Load Distribution and Eccentricity

The result of the approximate analysis (based on known location of the points of inflexion) is given in Fig. 2. The results from the computer programme are shown in brackets. The difference is due to the fact that the programme did not allow for partial fixity and hence the deflection of the cavity wall as shown in Fig. 4 is different in magnitude but similar in trend. From these analytical results it could be seen that the floor load transferred to the cavity wall is carried only by the inner leaf; the outer leaf does not carry any imposed floor loading. In a situation in design the outer leaf could practically be ignored, which is also confirmed from the full-scale[2] test. However, the moment appears to be equally shared between the leaves as may be seen from the deflection results.

The vertical reaction below the inner leaf as measured by the load cells was 2.94 kN compared to the approximate theoretical load of 2.88 kN which indicates very good agreement between the experimental and theoretical results.

Fig. 2 shows that load eccentricity varies throughout the height of the wall; hence it would not be reasonable to assume constant, arbitrary eccentricity, as at present, in the design of similar brick walls.

6. CONCLUSION

i) The imposed floor loading is mainly carried by the inner leaf of the cavity wall, hence for design for vertical loading it may be treated as single leaf wall.

ii) With increasing precompression of the walls (i.e. towards ground level) the floor deflections decreased. The effective fixity of the floor slabs varied between 33 - 50% for the test structure.

iii) The eccentricity varied throughout the height of the wall hence the structure should be analysed instead of assuming constant, arbitrary eccentricity of loading for the design.

v) In a structure similar to the one tested with all floors loaded, it would be reasonable to take the effective height for ground floor wall equal to 0.75 of the actual height as stipulated in the British CP.111 (1970).

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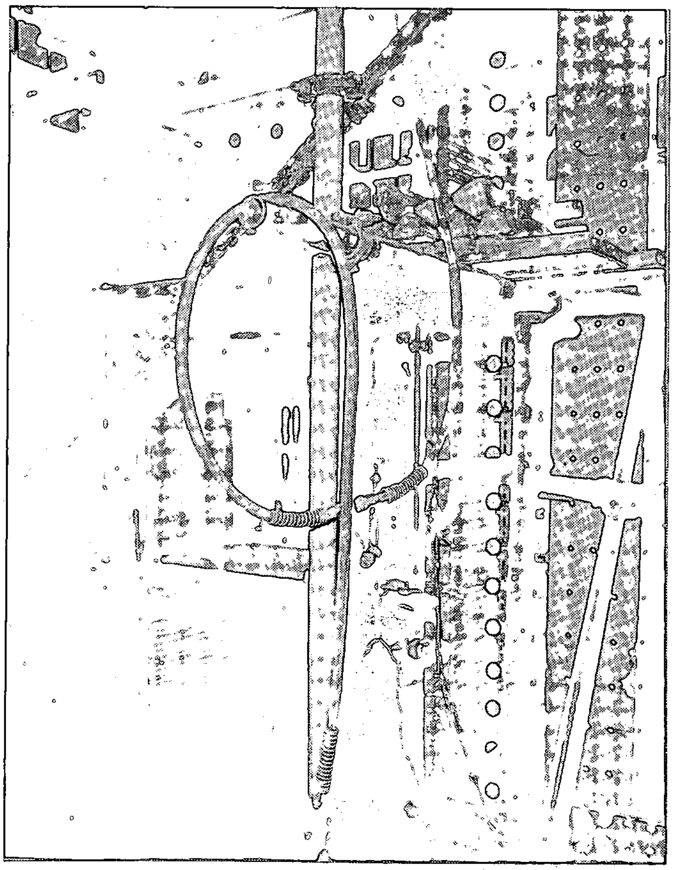


Fig. 1 Model Cavity Wall Test Structure

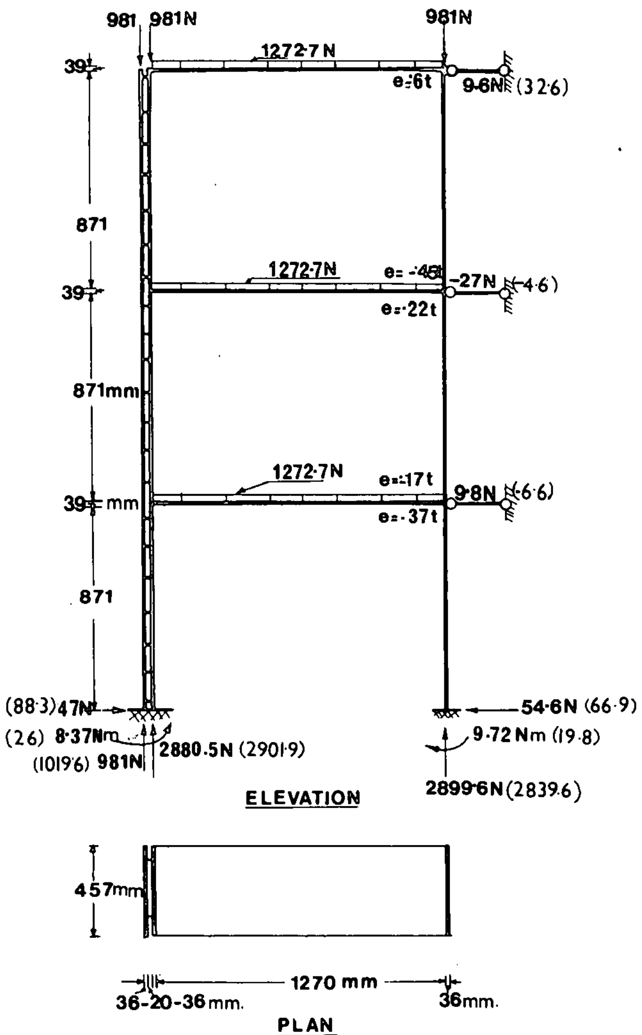


Fig. 2- Test arrangement and the overall load-distribution in the walls.

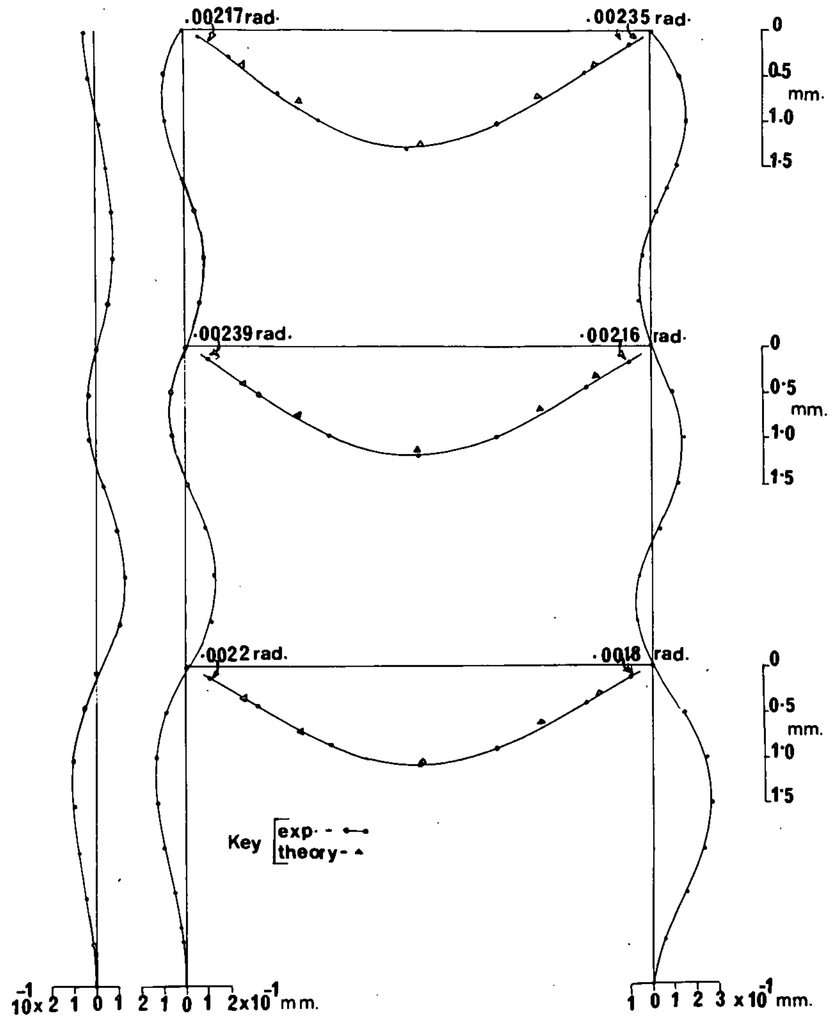
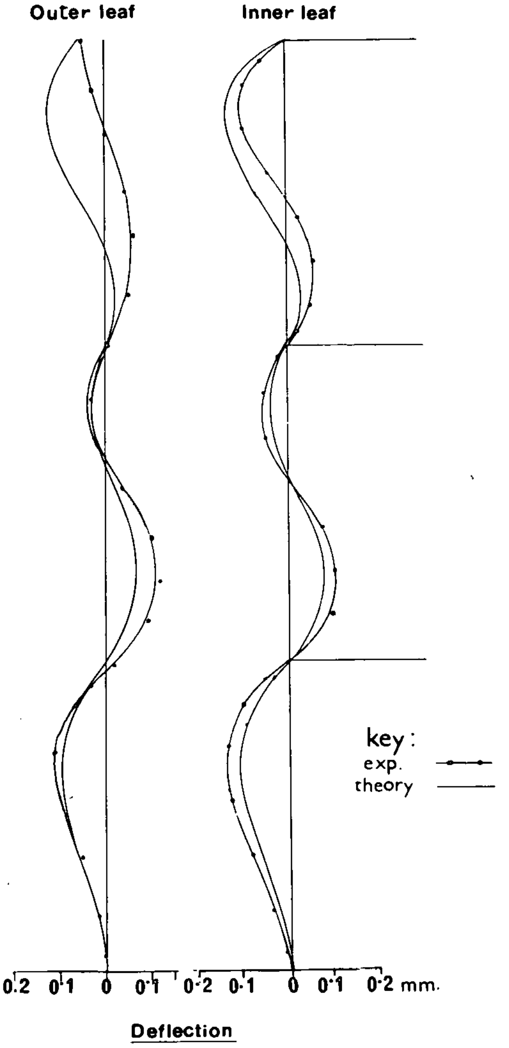


Fig. 3- Deflection of walls and floors when all floors are loaded.

Fig.-4 Experimental and theoretical deflection of cavity wall.



An Investigation into the Behaviour of a Five Storey Cavity Wall Structure

By

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ABSTRACT

The effect of floor loading was investigated in a full-scale, five storey brickwork structure. Reinforced concrete floors spanned between two cavity walls, the floors bearing on the inner leaf or on both leaves. The floors were uniformly loaded to 200 kg/m². The following was measured —deflection of the floors and walls, strain in the walls and rotation of the floors. Floor deflection increases towards the upper floors especially the top two. Tensile cracks occurred in the walls near their junction with the floors. The floor load was transferred at varying eccentricities to the walls. Most of the floor load was taken by the inner leaf of the cavity wall even where the floor bears on both leaves.

1. INTRODUCTION

The design of cavity walls in loadbearing brickwork is mainly empirical. In the Code of Practice[1] the ties connecting the two leaves are assumed to ensure the same lateral deflection in both leaves, implying that they share the applied moment in proportion to their rigidities. For slenderness ratio the effective thickness of the cavity wall is taken as two-thirds the sum of the actual thickness of the two leaves. Thus the stiffening effect of the outer leaf is taken into account to some extent even though it may not be under load.

This leaves many factors unanswered. For example, if the floor is carried through the cavity how is the load and moment actually shared between the two leaves? What is the magnitude of the moment? To help answer these questions a simple, full-scale, five-storey cavity wall structure was built (Figure 1), incorporating details used in practice—reinforced concrete floors, damp proof courses, outside leaf unsupported for a maximum of three storeys, standard spacing of ties, and the floor bearing onto one leaf or two leaves, in the latter case set back to allow brick slips to be fitted to the face.

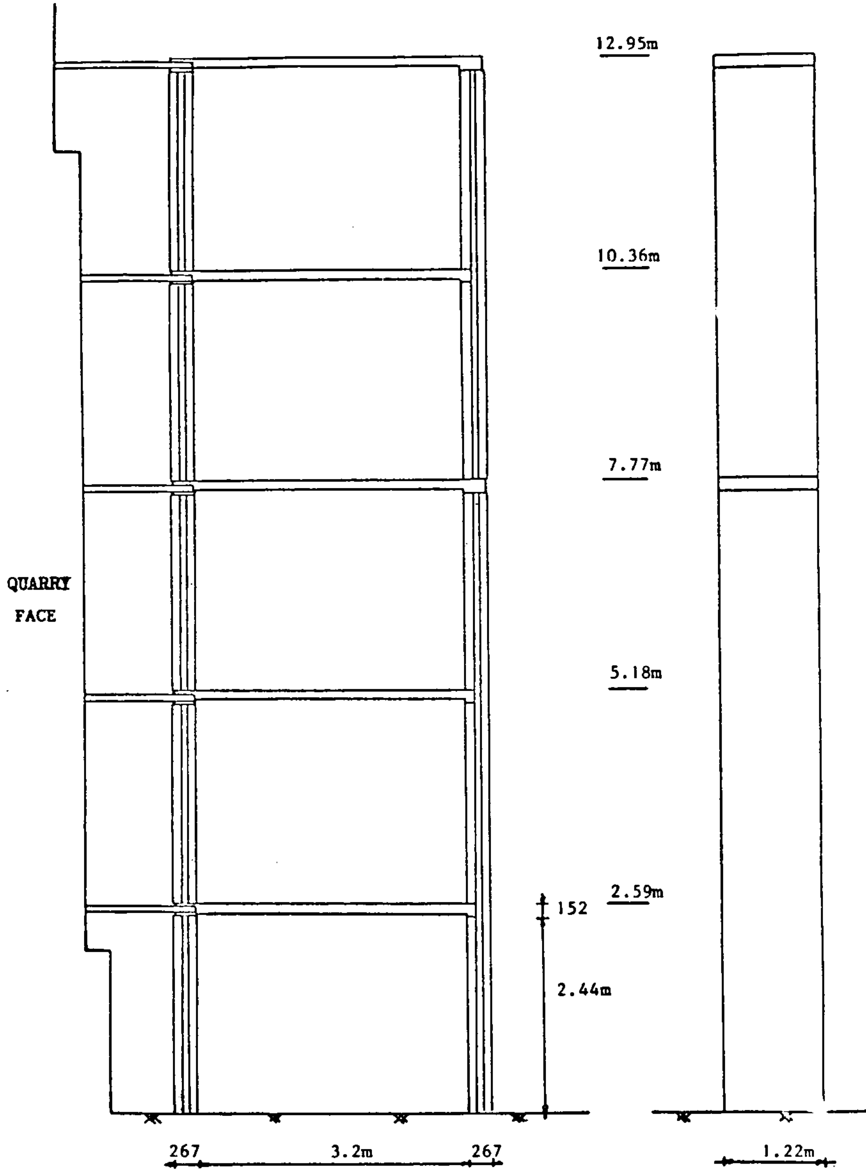


FIGURE 1. Elevation of the Cavity-Wall Structure.

2. MATERIALS AND CONSTRUCTION

Three hole perforated wire cut, common bricks were used. Their average compressive strength was 37·9N/mm^2 with a coefficient of variation of 19%. The mortar was 1:¼:3 rapid hardening Portland cement: hydrated lime : sand, using batching boxes to proportion by volume.

A 1 : 2 : 4 rapid hardening Portland cement : sand : aggregate mix by volume was used for the concrete slabs. The maximum size of aggregate was 20 mm and the water/cement ratio 0·5. Concrete cylinders (300 mm X 150 mm dia.) and a full scale slab were cast to obtain the modulus of elasticity. Four vibrating wire gauges were attached to the cylinder, which was tested in compression in an Avery 1MN test frame. A typical result was 26 kN/mm^2. The elastic modulus obtained from the deflection of the uniformly loaded test slab was 28 kN/mm^2.

Six brick high prisms were constructed and cured on site for quality control. The specimens were capped top and bottom with gypsum plaster before testing.

In addition prisms and small walls were built to be tested after the completion of the tests on the building to obtain the stress-strain relationship and the ultimate strength. They were tested under similar conditions to the concrete cylinders. The average modulus of elasticity was 14·3 kN/mm^2. The compressive strengths of mortar, concrete and brickwork are given in Table 1.

TABLE 1.

The Compressive Strength of Mortar Cubes, Concrete Cubes and Brickwork Prisms

Storey	Mortar Strength at 28 days N/mm^2	Concrete Strength 7 days N/mm^2	28 days N/mm^2	Brickwork Strength 28 days N/mm^2
1	14·0 (5)	34·0	42·2	—
2	18·2 (3)	36·9	37·7	24·1 (3)
3	18·1 (5)	37·4	28·5	23·5 (1)
4	17·1 (6)	27·3	37·8	21·2 (2)
5	21·9 (6)	—	40·2	24·4 (3)
Test Slab			41·5	

Notes: Numbers in brackets refer to the number of specimens tested.
Concrete strength based on an average of three specimens.
100 mm cubes used for mortar and concrete.

The Specification for the brickwork was based on the BCRA Model Specification[2]. Dimensions and spacing of the ties (galvanised strip fish-tail) are given in Figure 2. Two bricklayers and a labourer built a storey in one day. Coursing rods and levels were used. Until the floor was cast the walls were stabilised by two vertical steel rods threaded to Rawl bolts fixed in the base at the bottom of the cavity. The rods projected above the wall enabling plates to be bolted down onto the top of the wall thus post-tensioning the brickwork.

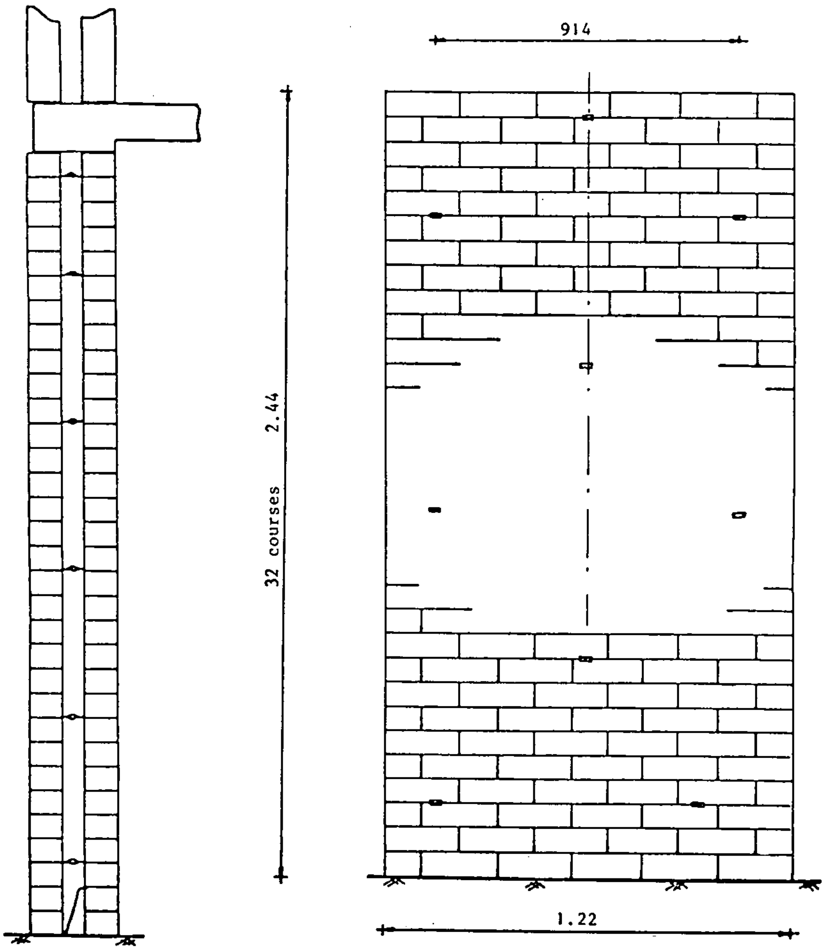


FIGURE 2. Layout of Brickwork and Ties.

The floor slabs were cast in-situ. The dimensions and reinforcement layout are shown in Figure 3. The design load was 300 kg/m^2. Concrete was compacted using a poker vibrator. On cold days the water was warmed before mixing the concrete. The top of the slab was levelled off with a wooden strike board and covered with polystyrene boards (25 mm thick) acting as insulation. Polythene in turn covered the polystyrene and scaffolding boards held it down. To ensure the stability of the structure one end of the slab was held in position by steel channels bolted to the quarry face with a hinge connection at the slab.

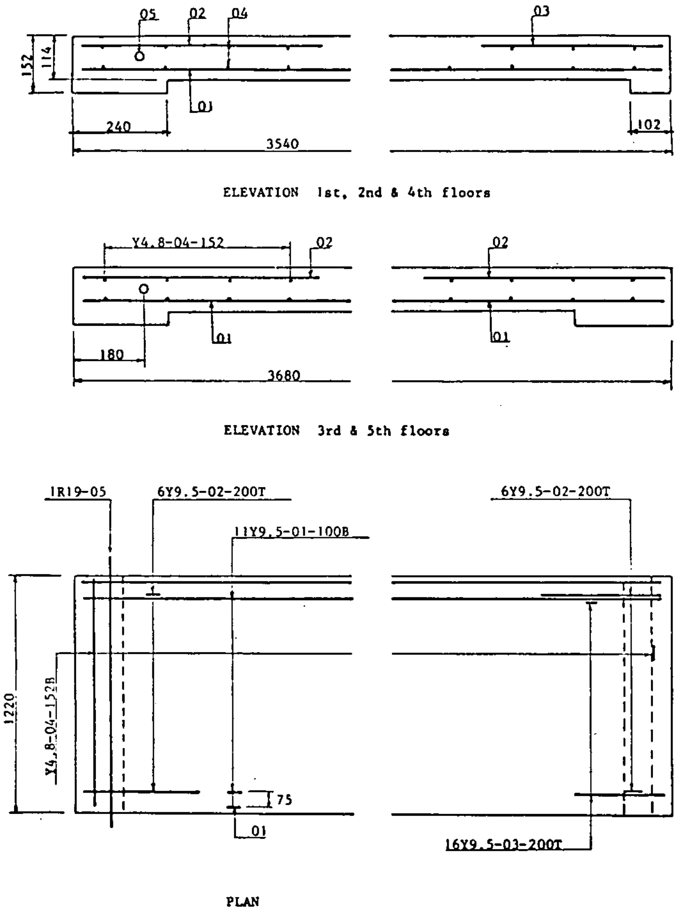


FIGURE 3. Reinforcement in the floor.

3. LOADING AND INSTRUMENTATION

Water in light flexible containers was used to ensure an easily applied uniform load—two to each floor (1·83X1·22 m and 1·22X1·22 m). The maximum level of water in the containers was 200 mm equivalent to a uniform load of 200 kg/m^2.

The deflection was measured using dial gauges reading to 0·002 mm. The guages were fixed to scaffolding tied back to the quarry face and independent of the structure. Theodolites were used to read the dial guages higher up the building. When not in use the gauges were enclosed in plastic bags to protect them from the weather.

Strains were measured in the cavity walls at ground level and at their junction with the floor slabs. 140 mm vibrating wire gauges were mounted both outside and inside the cavity wall. Where d.p.c. occurred the guages were mounted above the level of the d.p.c. All guages were connected to a data logger with print out.

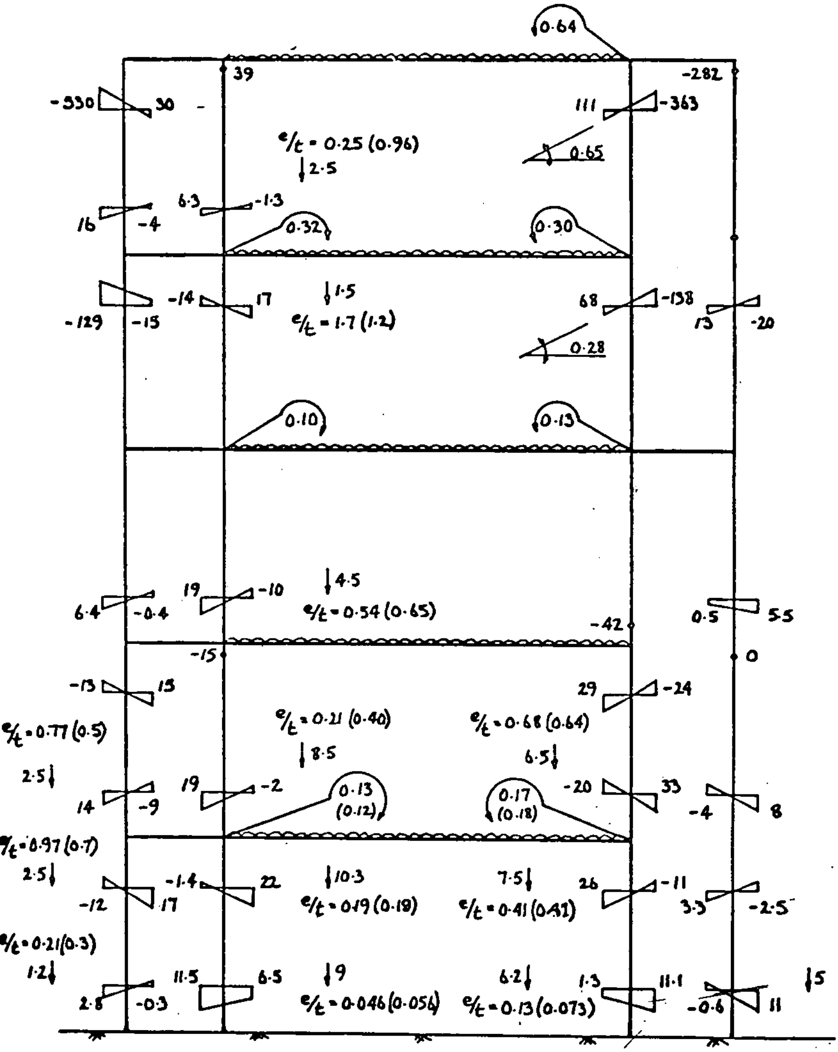
Rotation of the wall and floor at their junction was measured by an electronic level reading to 10 secs. or 0·05 mrad (estimate to 0·01 mrad).

To obtain good test results calm days with little or no change in temperature were necessary during the period of the test. Measurements were taken of strain and rotation at the wall-floor junctions and deflection of the walls and floors. Individual floors were loaded or all floors were loaded. It was necessary to repeat the tests several times since there was a variation in magnitude of the results although the trend was consistent.

4. RESULTS

4.1 Strains

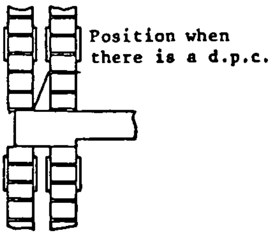
Strain results give an estimate of the load and moment distribution. The measured strains were very small—they rarely exceeded 22 micro-strain unless tensile cracking occurred. Thus great care was needed to obtain good results. Figures 4 and 5 show results as linear strain planes. For a linear stress-strain relationship, the slope of the strain plane is proportional to the bending moment and the change in position of the strain plane along the centre line of the wall is proportional to load. When tensile cracking occurs these assumptions are no longer valid.



For notation see figure 5.
Theoretical values are given in brackets.

FIGURE 4. Strain and Rotation—All floors loaded to 200 kg/m².

Position of strain gauges



Notation to figures 6 & 7

Strains on wall surface derived from gauge results

• Individual strain gauge results

Axial strain in wall due to floor loading

Floor rotation at its junction with the wall

e Eccentricity in walls due to floor loading (from strain results)

t Wall thickness = 102mm

Units strain x 10^{-6} rotation mrad

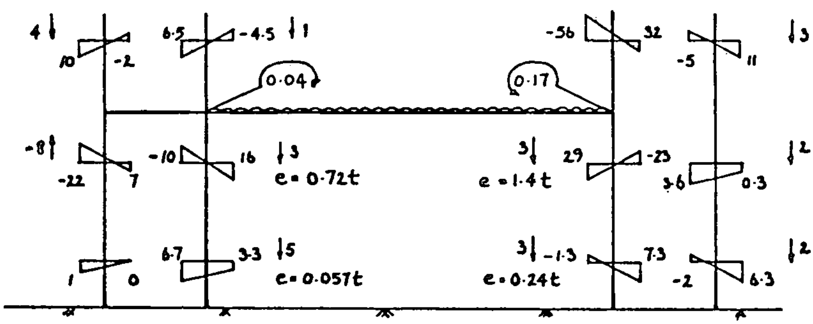


FIGURE 5. Strain and Rotation

First floor loaded to 200 kg/m^2.

The strains in the ground floor walls (Figure 4) just below the slab are higher than at the base except in case of the outer leaf where the floor bears only on to the inner leaf. The difference in the ground floor strain readings may be due to difference in the modulus of elasticity, local variation in workmanship, or the limitation in the resolution of the strain measuring equipment (1×10^{-6} under normal laboratory conditions). Within these limits, the total load worked out from the strains in the ground floor walls immediately below the slab appears to be 35·9 kN against 36·6 kN calculated from the weight of water (9·81 kN/m^3).

The floor load is mainly carried by the inner leaf. The ground floor inner leaf, where the floor goes through the cavity and rests onto both leaves, carries 50% of the total imposed load against 43% obtained from frame analysis. In the corresponding situation, where the floor bears only on to one leaf, the inner leaf carries 36% of the total imposed load instead of the theoretical 39%. The outer leaf which is unsupported up to the third floor carries theoretically 10% of the total load, about 2% higher than the outer leaf supported at each floor level. A similar trend is obvious from the strain results in the outer leaves at the base of the structure. As the strain in the outer leaf where the slab bears only onto one leaf is unexpectedly high, no definite conclusion could be drawn.

Some load eccentricities resulting from the floor loading have been calculated and are shown in Figure 4, these vary from 0·046t to 1·7t (t = wall thickness), clearly showing that a constant, arbitrary eccentricity should not be assumed. When the floor bears onto the inner leaf only, much larger eccentricities occur. In the immediate vicinity of the junction, most of the floor moment is applied to the inner leaf, only gradually distributing to the outer leaf. This is confirmed by wall deflections (Figure 6). This behaviour causes tensile cracking and hence the theoretical eccentricities are higher in most cases (Figure 4). In the ground floor walls immediately below the slab, the values of experimental and theoretical eccentricities are the same. It may be concluded that, in the present case, a precompression of approximately 0·46 N/mm^2 is required above the joint to make it behave as a stiff joint.

4.2 Rotation

Rotation of the floor at its junction with the wall is shown in Figures 4 and 5. The rotations for floors 3–5 are the results of only one or two tests.

Considering the rotations at the first floor junctions, carrying the floor through both leaves reduces the rotation. The rotation is 0·04 mrad when the first floor only is loaded and increases to 0·13 mrad when all floors are loaded which may be compared with the unchanging 0·17 mrad where the floor bears onto the inner leaf only.

Next consider the fifth floor rotation of 0·64 mrad. The slope of the strain plane at this junction is 0·65 mrad, clear evidence that the section covered by the gauges is cracked, allowing the floor to rotate about a point near the edge of the wall. A similar result is obtained from the fourth floor rotation of 0·30 mrad which compares with 0·28 mrad obtained from the strain results.

On the assumption that the top floor is not restrained at its junction with the wall, the fixity at the first floor junction when all floors are loaded is 80% of full fixity where the floor goes through the cavity and bears on two leaves. On the above assumption and under similar loading conditions the fixity at the first floor junction where the slab rests onto one leaf only appears to be 73% of full fixity.

4.3 Deflections

Lateral deflection of both leaves of the walls is shown in Figure 6. The two leaves of the cavity have similar curvatures and deflection which would be expected since they are tied together. Nevertheless there are differences where the floor bears onto the inner leaf only—here the curvature of the inner leaf is much greater than the outer leaf. This is also reflected in the greater slope of the strain planes for the inner leaf.

Points of inflexion deduced from large scale plots of the deflection curves are approximately:—

Storey level	1	2	3	4	5
Point of inflexion	0·2–0·3	0·45	0·45	0·45	0·55
Theory	0·2–0·3	0·5	0·5	0·5	0·47

The theoretical results (Figure 7) come from a computer frame analysis which does not allow for tensile cracks. The effective height for design equal to three-quarters of the actual height (CP 111)[1] appears quite reasonable for the ground floor inner leaves.

Factors affecting floor fixity at its junction with the wall are the rigidity of the wall and floor and the average compressive stress in the wall which affects the point at which tensile cracking occurs in the wall.

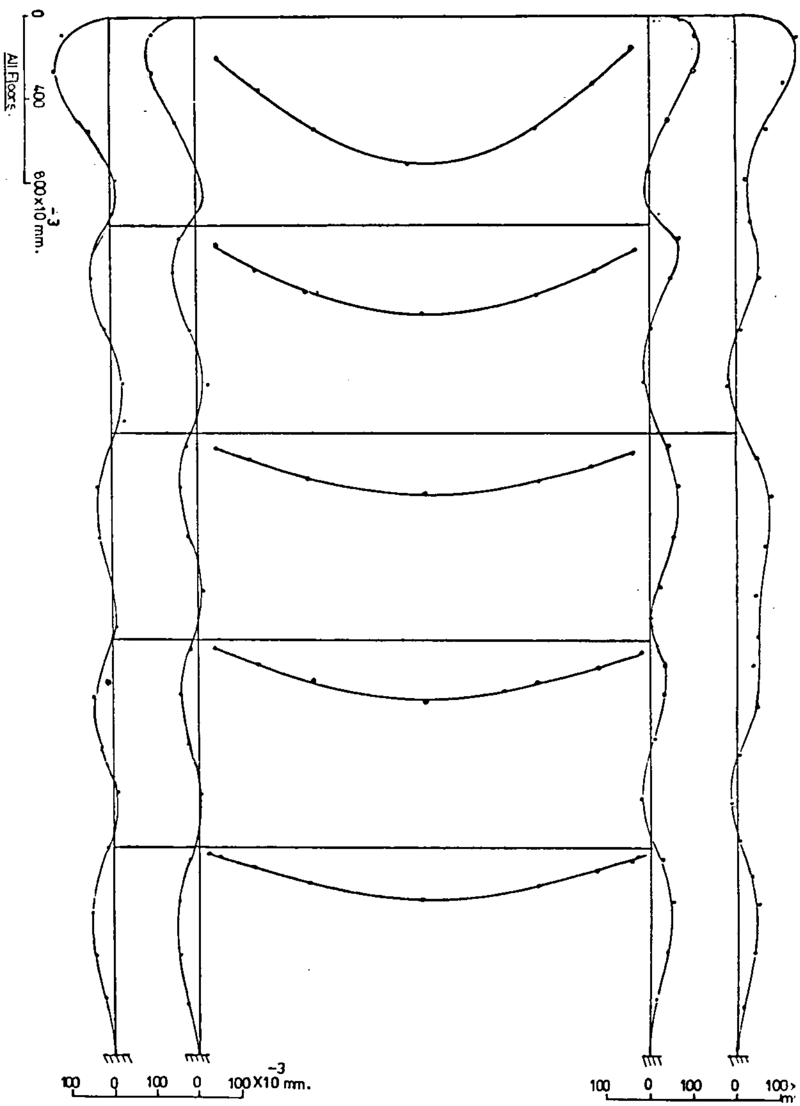


FIGURE 6. Deflection of the walls and floors when all floors are loaded.

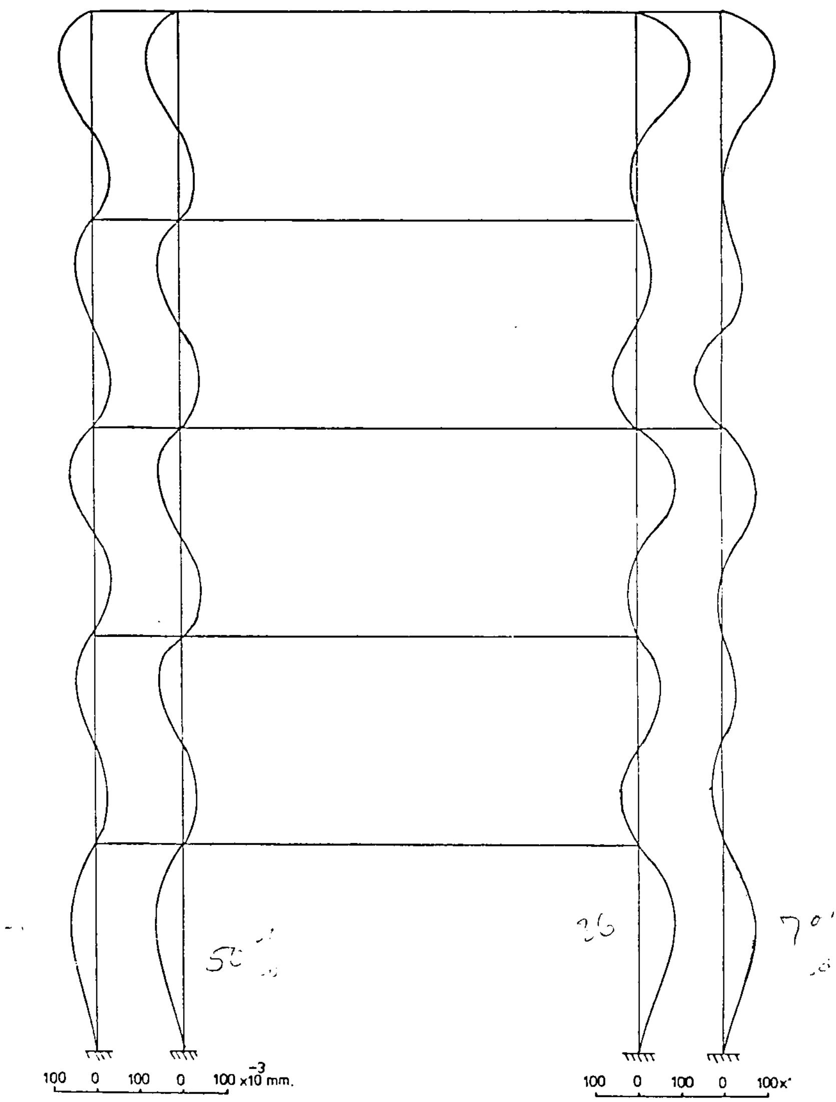


FIGURE 7. Theoretical deflection of the walls when all floors are loaded.

The floor deflections are also shown in Figure 6. Midspan deflections for 1st to 5th floors are 0·20, 0·23, 0·21, 0·33 and 0·59 mm respectively. The deflection thus tends to increase with increasing floor level. Initially the increase in deflection is small. The third floor deflection decreases because the slab is carried through both cavities. The fourth and fifth floor show large increases due to tensile cracking at the floor-wall junction (see strain results in Figure 4). At fifth floor level large tensile cracks occur and for practical purposes it could be considered as a simply supported floor. This would be a safe assumption although the theoretical floor deflection based on an elastic modulus of 28 kN/mm^2 gives a midspan deflection of 0·78 mm.

5. CONCLUSIONS

1. Most of the imposed load is taken by the inner leaf. About 86% of the total imposed load on the test structure was carried by the inner leaves of the cavity walls. Where the floors bear on two leaves at every storey, the inner leaf supports 50% of the load. On the other hand the inner leaf, where the floor bears only on one leaf, carries 36% of the imposed load. The remaining 14% of the imposed load is shared between the two outer leaves.

2. Where the floor bears onto one leaf, most of the moment is taken by the inner leaf at the wall floor junction, tending to cause tensile cracks in the wall near the junction. Away from the junction the moment is equally shared between the two leaves, as a result of the action of the wall ties.

3. Moving up the building (i.e. decreasing precompression) the floor deflections and rotations increase, due mainly to the tensile cracking in the walls at the wall-floor junction.

4. The fixity of floors passing through both leaves is greater than that of the floor bearing onto one leaf. For the first floor, the difference is large when only that floor is loaded and small when all floors are loaded.

5. Considering the ground floor walls, the effective height equal to 0·75 x actual height, as stipulated in CP 111, appears quite reasonable.

6. The effective eccentricities resulting from the floor loading vary throughout the height of the test structure and hence it may not be reasonable to assume a constant arbitrary eccentricity for the design.

ACKNOWLEDGEMENTS

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AN INVESTIGATION INTO THE BEHAVIOUR OF A BRICK CROSS-WALL STRUCTURE

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ABSTRACT

Strains, deflection and rotations resulting from floor loading was investigated in a full-scale, two bay, three-storey brick structure. The deflection of the floors and walls, strains in the walls and rotations of the wall/floor junctions were measured on internal as well as on end walls with all floors loaded or with alternate floors loaded. With all floors loaded the central wall should not deflect at all but in the test structure small deflections occurred due to accidental eccentricities. With alternate floor loading the deflection patterns suggested that the effective height of the central wall in the upper two storeys may be taken as equal to the actual height and in the ground floor approximately equal to 0.7 times the actual height. Floor deflection increases towards the upper floors. The floor load was transferred at varying equivalent eccentricities but in some cases the measured strains were so small that it would be difficult to draw any precise conclusions.

1. INTRODUCTION

The design of brickwork compression members of a multi-storey structure is at present based on empirical rules. The current code of practice[1] treats brickwork walls and columns as isolated elements loaded axially or eccentrically and gives allowable stresses for them accordingly. However, it offers no guidance to the designer for the calculation of eccentricity of loading and inadequate guidance relating to effective height in a practical situation. The effective height of a wall and the eccentricity imposed on it will depend on the type of floor loading, the relative stiffness of wall and floor and the degree of fixity at the joint which in turn may depend on the amount of precompression. The assessment of these factors in design is difficult due to lack of experimental data. This investigation

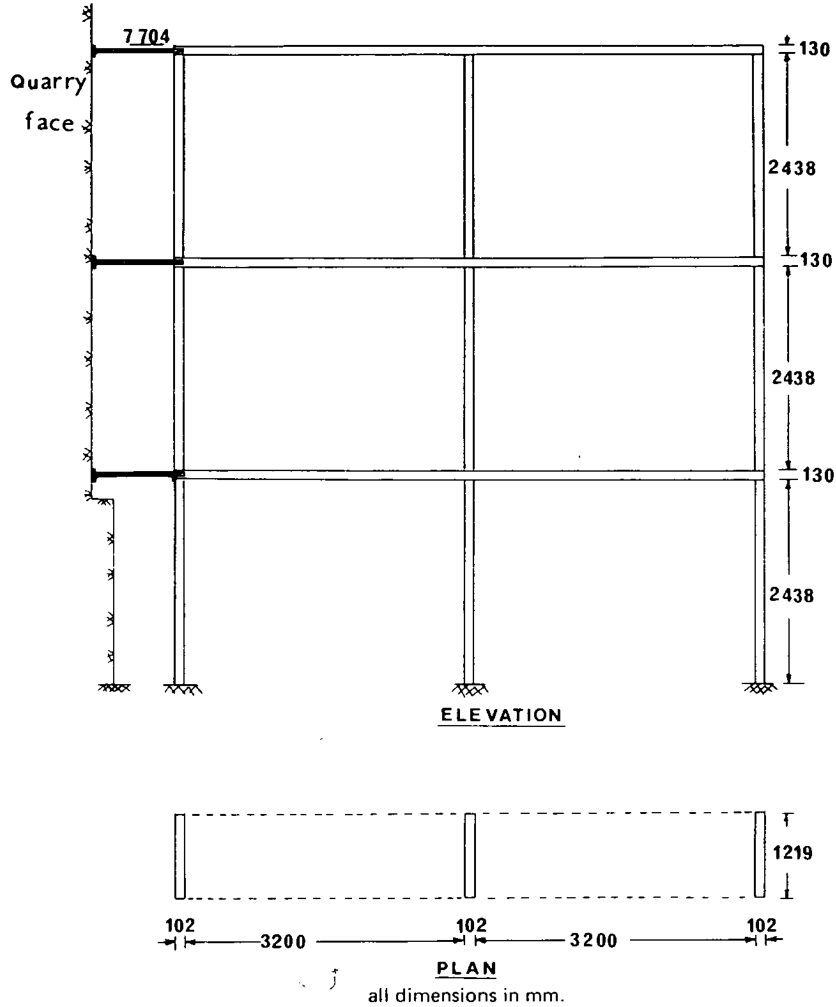


FIGURE 1—Test Structure.

is a part of a continuing research programme[2] to obtain data for the refinement of the design of brickwork structures subjected to dead or live loading.

A two bay, three-storey structure as shown in Figure 1 was built at Torphin Quarry, Edinburgh - so that the behaviour of an end and a central wall subjected to floor loading could be monitored at the same time.

2. EXPERIMENTAL

2.1. Materials and construction

Three hole perforated, wire-cut common bricks of compressive strength 37.9 N/mm^2 with a co-efficient of variation of 19% were used for the construction. 1:$\frac{1}{4}$:3 (rapid hardening cement: lime:sand) mortar was used. The average compressive strength of mortar for the different storeys is given in Table 1.

Table 1

Compressive strength of 100 mm mortar cubes, concrete cubes & brickwork prisms

Storey	Mortar		Concrete		Brickwork	
	7 days N/mm^2	28 days N/mm^2	7 days (cube) N/mm^2	1$\frac{1}{2}$ years (cylinder) N/mm^2	28 days N/mm^2	1$\frac{1}{2}$ years N/mm^2
1	8.0	12.7	18.0	25.8 *	17.0	
2	7.5	-	21.8	34.8 *	10.4*	20.0
3	-	11.8	13.5	24.6 *	9.7	

* One specimen only. Remainder means of 3.

Brick prisms six-courses high were built and cured on site for quality control. The specimens were capped top and bottom for testing. A few prisms were tested after the completion of the test on the building to obtain the modulus of elasticity of brickwork, the average value of which was 11.7 kN/mm^2 for the ground and first floor and 9.45 kN/mm^2 for the second floor walls.

50 mm thick 'Omnia' precast panels equal to the internal dimension of the building (3.2 m x 1.22 m) were lifted and kept in position by props with no bearing on the walls except for the reinforcement protruding at the ends of the panels (Figure 2). Ready-mix concrete (1:2:4) was poured on the top of the precast slabs to obtain a thickness of 130 mm throughout.

Negative reinforcement was provided as required for continuity at the end and at the central support, before pouring the concrete. By adopting this method of construction, not only considerable saving in time and cost of shuttering was achieved but a good joint similar to a cast-in-situ slab was obtained between the finished slab and wall underneath. The compressive strength of concrete cubes and cylinders is also given in Table 1. The average modulus of elasticity of concrete obtained from cylinder tests was 25.83 kN/mm^2.

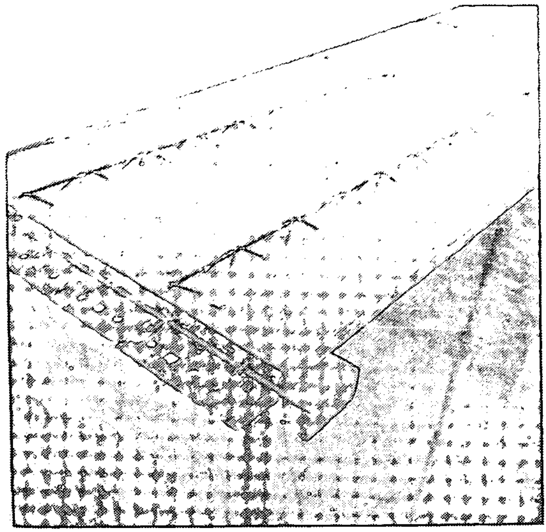


FIGURE 2—Omnia slab in position.

2.2 Loading arrangements and instrumentation

The floor slabs were loaded uniformly by water in flexible glass fibre tanks. A layer of soft felt was introduced between the tanks and each floor slab to distribute the load uniformly. The main pipe carrying the water supply from the storage tank on the quarry floor had two branches at each floor level to fill simultaneously the loading tanks on each side of the central wall. The quantity of water was metered accurately at the inlet of each tank (Figure 3). When alternate floors were loaded, one of the branches at each floor level was blocked. On completion of a test the water from the loading tanks was allowed to drain by gravity back into the storage tank.

The deflections of walls and slabs were measured by 75 dial gauges reading to 0.002 mm. Some were fixed to the quarry wall to measure the deflection of the end wall nearest to it. Others to measure the deflection of the central, far wall and slabs were fixed on independent frames which in turn were fixed to the brick walls and floors of a five storey structure built for earlier tests so as to be as stable as possible and also to minimise any effect due to thermal expansion of the gauge supporting frame.

The rotation at floor/wall junctions was measured by electrolevels measuring to 5×10^{-4} radians. Strains were measured by 140 mm vibrating wire gauges and a data logger was used to record the results.

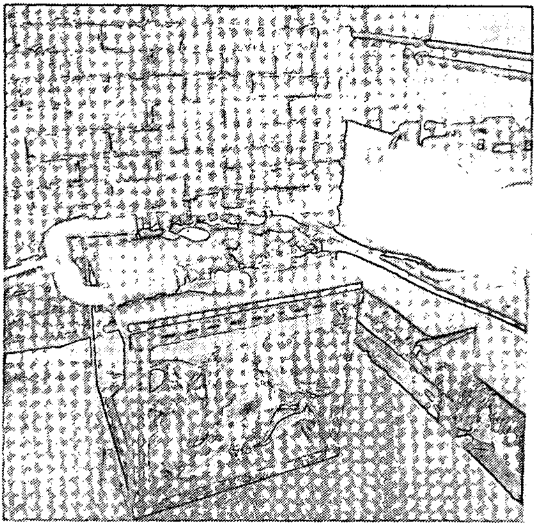


FIGURE 3—Loading tanks and water meter.

The structural behaviour under a load of 2.45 kN/mm^2 was investigated:

(i) with all floors loaded, (repeated 11 times).

(ii) with alternate floors loaded, (repeated 7 times).

The loading was higher than the design load in order to obtain strain and deflection readings which could be measured with reasonable accuracy. Because the tests were repeated several times there were very slight variations in the readings, but the trend was very consistent.

3. RESULTS AND DISCUSSION

The test results are shown in Figures 4 and 5. The theoretical results were obtained by analysing the structure as a rigid frame.

3.1. Strains

The measured strains were very small. Even the analytical values seldom exceeded 33 μs which would not cause tensile cracking. Except for the central wall at the ground floor level, the maximum axial strain due to floor loading was about 5 μs which is difficult to measure precisely with any degree of accuracy. The strains in all walls were mainly due to bending for both loading cases. Only the central wall was subjected to axial

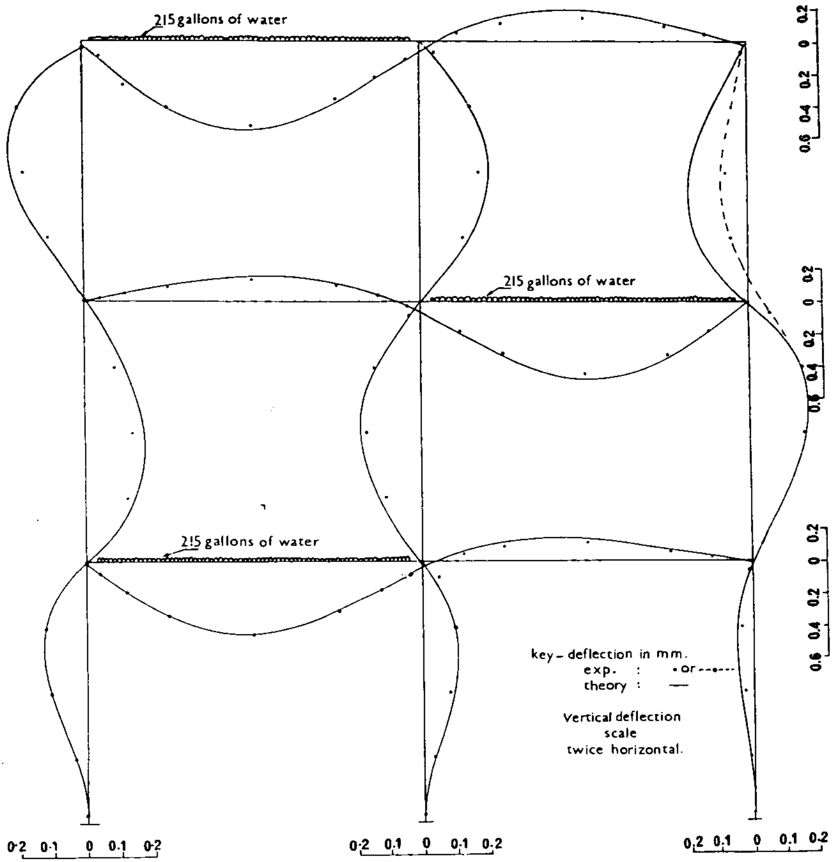


FIGURE 4—Deflection results for alternate floor loading.

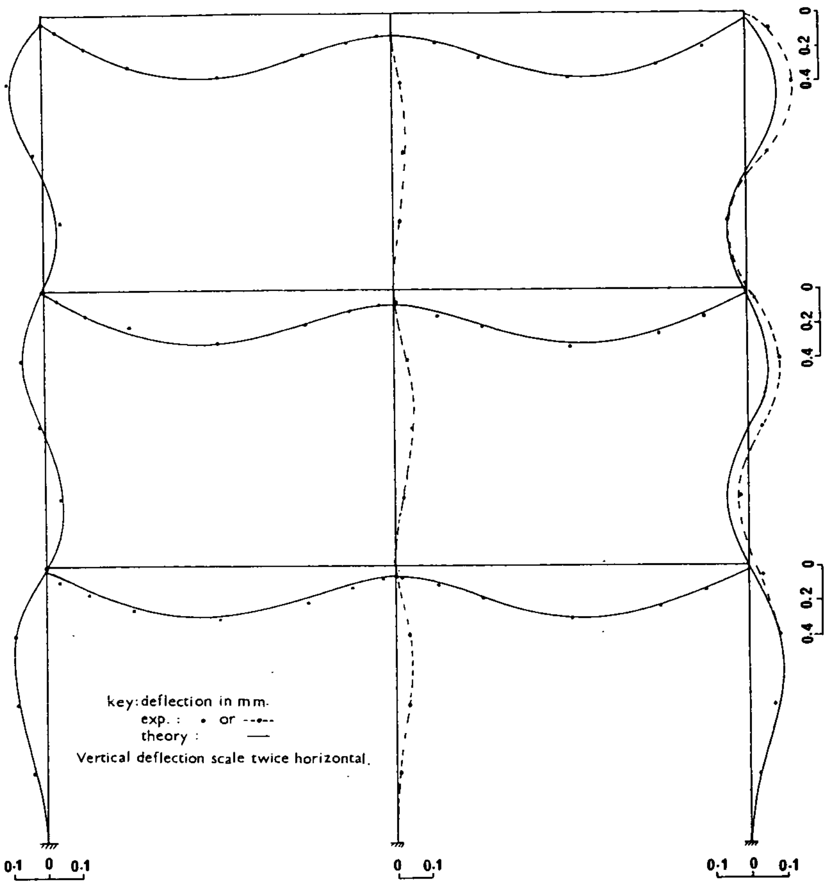


FIGURE 5—Deflection results for each floor loaded with 215 gallons of water.

loading in the case of all floors loaded uniformly. Figures 6a and b show the measured and calculated strains. With all floors loaded, the total load at slab and ground level in the ground floor central wall and the end wall situated farthest from the quarry face were 32.9 kN and 11.7 kN as calculated from the strains. This compares with the actual load of 33.9 kN and 11.7 kN respectively.

Some load eccentricities resulting from floor loading are shown in Figures 6a and b which vary from 0 to 1.89 t. From this it could be argued that the assumption of constant arbitrary eccentricity for the design is not correct. In most cases the equivalent eccentricities calculated from experimental results were lower than theoretical values. This could be due to several factors, partial fixity of joints or non-uniform strain distribution in the length of the wall. However, it would be safer in design to assume the theoretical eccentricities provided no tensile cracks develop at the joint due to loading.

3.2. Rotation

The rotations of wall and slab at its junction were similar for both loading cases. No differential rotation was recorded at any wall/slab junction.

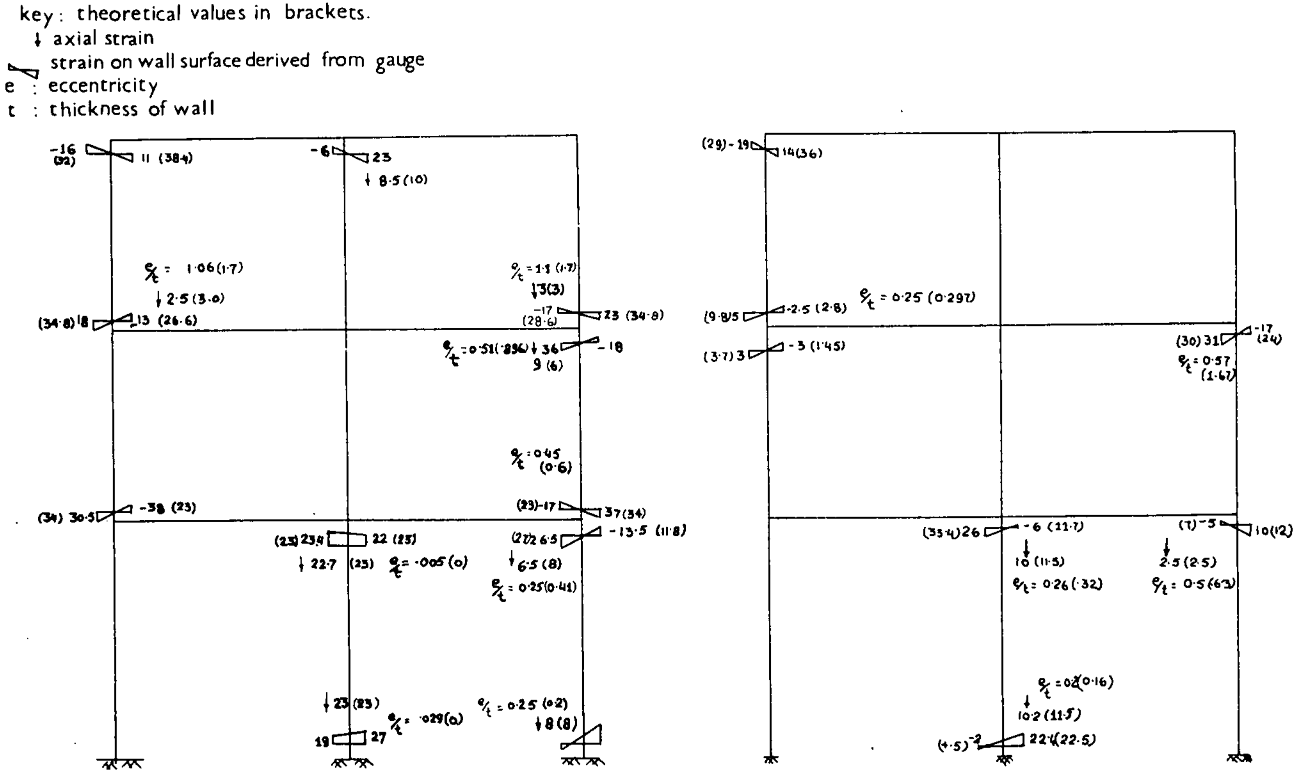
3.3. Deflections

In general there is good agreement between experimental and analytical (Figures 4 and 5) deflection results for both the floor slabs and the walls subjected to two different loading conditions. The theoretical result was obtained from a standard frame programme which takes into account the axial and shear deformations of columns and beams. In some cases the axial deformation of the wall was as much as 20% of the maximum slab deflection. To ignore this would lead to erroneous values of end fixity if these were based only on maximum slab deflections.

In the case of alternate floor loading, there were considerable differences between measured and calculated deflections for the top floor wall at the end farthest from the quarry face. This may have been due to the movement of the dial gauge frame which could not be made absolutely rigid, to shrinkage cracking or to a constructional defect which may only have allowed partial transfer of the bending moment.

With all floors loaded, the central wall of this structure, theoretically, should not deflect at all. In practice it would be very difficult to apply axial load. The maximum deflection of the central wall was of the order of 0.05 mm which could be the result of accidental eccentricity.

The effective height ratios deduced from the deflection curves are given approximately in Table 2.



(a) All floors loaded.

(b) Alternate floors loaded.

FIGURE 6—Strains

Table 2

Approximate effective height ratios

Floor	Alternate floors loaded			All floors loaded	
	End Wall	Central	End	End walls	Central
Second	0.84	1	0.95	0.53	0.95
First	0.95	1	0.84	0.50	0.95
Ground	0.7	0.7	0.7	0.7	0.82

Thus the effective height depends on the type of loading and particularly on the situation of the wall in the structure. The current CP 111 suggests an effective height ratio of 0.75 where a concrete slab spans onto a wall; such a blanket provision appears incorrect.

For the present, it would be reasonable to idealise the brick structure as a frame in order to calculate the effective eccentricities and effective height for particular loading conditions. However, for the future limit state code it will be necessary to test its validity in the ultimate state. It is planned to test this structure to destruction in future.

4. CONCLUSIONS

1. There is very good agreement between the experimental and analytical results, hence the brick structure can be idealised as a frame in design.

2. The effective height of a wall depends on its disposition and also on the type of floor loadings to which it is subjected. The effective height equal to 0.75 x actual height, as stipulated in CP 111, does not apply for all cases of loading.

3. The effective eccentricities resulting from floor loading vary throughout the height of the structure and hence it is incorrect to assume constant arbitrary eccentricity in design.

ACKNOWLEDGEMENTS

The work described in this paper is sponsored by the Brick Development Association and the British Ceramic Research Association.

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Reinforced Masonry

(Group 3 - Papers 30 to 33)

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An ultimate load-analysis of reinforced brickwork flexural members

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This paper outlines a simplified ultimate load analysis of reinforced brickwork flexural members, using the actual stress-strain relationship. Test specimens were built from low, medium and high strength bricks. The theoretical results are compared with full-scale test results and it is shown that the collapse moment of reinforced brickwork can closely be predicted by this method.

NOTATION

F_c	Compressive force	b	Breadth of section
F_t	Tensile force	$\lambda_1, \lambda_2, \lambda_3$	Coefficients
n	Depth of neutral axis	ϵ	Strain in brickwork
f_y	Steel stress	ϵ_{max}	Failure strain in brickwork
f_m	Masonry prism strength	σ	Stress in brickwork
M_u	Ultimate resistance moment	σ_{max}	Failure stress in brickwork
M_{um}	Ultimate resistance moment based on masonry	k	Constant
M_{ut}	Ultimate resistance moment based on steel	p	Constant
Z	Lever arm	E	Initial Tangent Modulus
d	Effective depth of section	E_{max}	Secant Modulus at failure

Introduction

At present, the design of reinforced brickwork in the UK is governed by the elastic design recommendation of the British Standard Code [1] of practice CP 111. The elastic design when compared with available test results [2,3,4] is very conservative and also does not fully reflect the behaviour of a flexural member. Various research workers [2,3,4,5] have suggested an ultimate load design approach for reinforced brickwork using stress block similar to the reinforced concrete, although the stress-strain relationship of concrete differs from that of brickwork [7]. Also, the simplified CP 110 approach for the calculation of ultimate moment is based on a concrete strain of 0.0035. This assumed failure strain is somewhat higher than found for brickwork specimens of various strengths from the test results of the author [3] and others [6,7,8]. The ultimate failure strain of various types of brickwork appears to be 0.0025 for high, 0.00275 for medium and 0.003 for low strength bricks. Earlier, the author [9] has suggested an approximate method for the ultimate load design of reinforced brickwork flexural member based on cubic parabolic stress-strain relationship and failure strain of 0.003 for all types of bricks. The present paper, however, uses the actual failure strain and curvilinear (curve with an initial linear branch) stress-strain relationship for the ultimate load analysis of reinforced brickwork and also compares the results of analysis with those obtained by assumed cubic parabolic stress-strain relationship. The results obtained from the theoretical analysis are compared with the test results.

Stress-strain relationship of brickwork

Six-course and three-course prisms were built and tested in axial compression, as shown in Table 2, the strains having been measured by a 'Demec' gauge. The ultimate strength of the prisms is given in Table 2. The strain measurements were very carefully monitored up to failure. Only a few typical stress-strain relationships for low-strength bricks are shown in Figure 1. The stress-strain relationship is almost linear initially, thereafter the strain increases rapidly compared to stress. The final failure of brickwork happened when the stress reached maximum at maximum strain.

The stress-strain relationship for various strengths of brickwork has been shown in dimensionless form in Figures 2 to 4 and it can be seen that the exact shape of the non-dimensional curve is dependent on brickwork strength.

The brickwork stress and strain relationship can mathematically be idealised and expressed in dimensionless form as:

$$\text{Linear part: } \frac{\sigma}{\sigma_{max}} = \frac{E}{E_{max}} \cdot \frac{\epsilon}{\epsilon_{max}}; \quad 0 > \frac{\epsilon}{\epsilon_{max}} \leqslant k \tag{1}$$

$$\text{Curve part: } \frac{\sigma}{\sigma_{max}} = \left[\frac{\epsilon}{\epsilon_{max}}\right]^p; \quad \frac{\epsilon}{\epsilon_{max}} \geqslant k \tag{2}$$

where k = 0.226 for low brick strength
= 0.2619 for medium strength
= 0.363 for high strength
p = 0.26 for low brick strength
= 0.245 for medium strength
= 0.2 for high strength

As can be seen from Figures 2 to 4 the mathematically idealised curves represent closely the experimental results. The area under all these non-dimensional curves for various strengths of brickwork can be related by λ_1 times the area of the enclosing rectangle and the position of their centres of gravity by λ_2.

$$\text{Thus, } \lambda_1 = \frac{E}{E_{max}} \int_o^k \frac{\epsilon}{\epsilon_{max}} \, d(\epsilon/\epsilon_{max}) + \int_k^1 \left(\frac{\epsilon}{\epsilon_{max}}\right)^p d(\epsilon/\epsilon_{max}) \tag{3}$$

$$\lambda_2 = \frac{\left[\frac{E}{E_{max}} \int_o^k \frac{\epsilon}{\epsilon_{max}} \, d(\epsilon/\epsilon_{max})\right]\left[k/3 + (1-k)\right] + \int_k^1 (\epsilon/\epsilon_{max})^p d(\epsilon/\epsilon_{max})(1 - \epsilon/\epsilon_{max})}{E/E_{max} \int_o^k \epsilon/\epsilon_{max} \, d(\epsilon/\epsilon_{max}) + \int_k^1 (\epsilon/\epsilon_{max})^p d(\epsilon/\epsilon_{max})} \tag{4}$$

The values of λ_1 and λ_2 for different types of brickwork calculated from the Equations (3) and (4) are given in Table 1.

The value of λ_1 and λ_2 are also given for cubic parabolic stress-strain relationship and it can be seen that these values are very similar to the actual values obtained from the stress-strain relationship.

Theoretical analysis

The theoretical analysis described in this paper uses the actual and also the cubic parabolic stress-strain relationship.

Assumptions for the analysis of collapse moment

i) Plane sections remain plane in bending
ii) The tensile strength of brickwork, being low is ignored
iii) The existence of good bond is assumed between brickwork, grout and steel
iv) The stress-strain relationship for brickwork in flexural compression and for reinforcement in tension is given by the appropriate uniaxial tests
v) The compressive strain in the brickwork at the extreme fibre at failure is equal to specified experimental value of ϵ_{max}.
vi) For a balanced section, the steel strain at the point of failure is equal to 0.0035, the stress being the appropriate test value

Table 1. Characteristics from curvilinear or cubic parabolic stress-strain relationship

Type of brickwork	λ_1		λ_2	
	Equation 3	Cubic parabola	Equation 4	Cubic parabola
High strength	0.734	0.75	0.397	0.4
Medium strength	0.746	0.75	0.41	0.4
Low strength	0.7486	0.75	0.414	0.4

The stress at any point in brickwork can be determined from the Equations (1), (2) or from the cubic parabolic relationship between stress and strain. Just before compressive

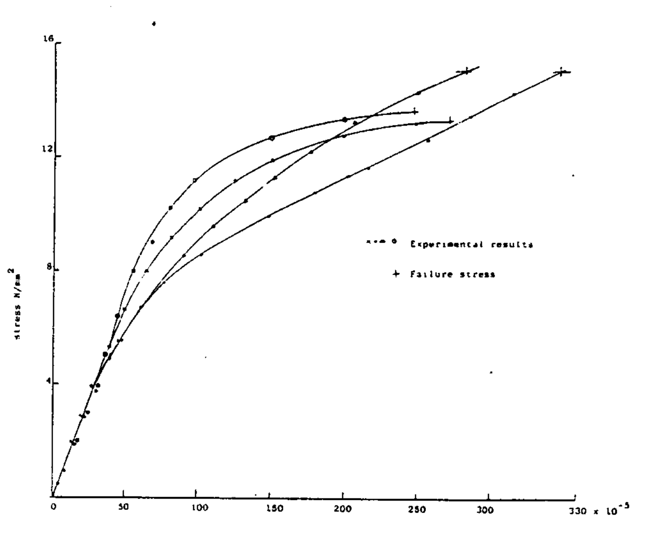


Figure 1. Typical stress strain curve for low strength brickwork.

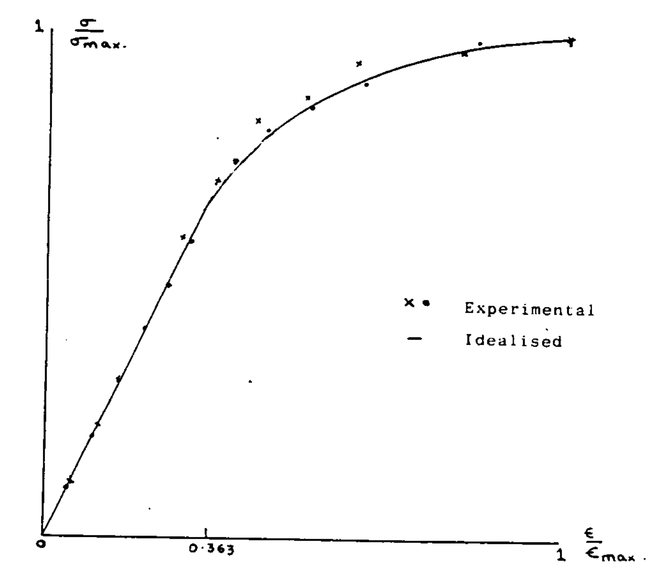


Figure 4. Non-dimensional stress strain curve for high strength brickwork.

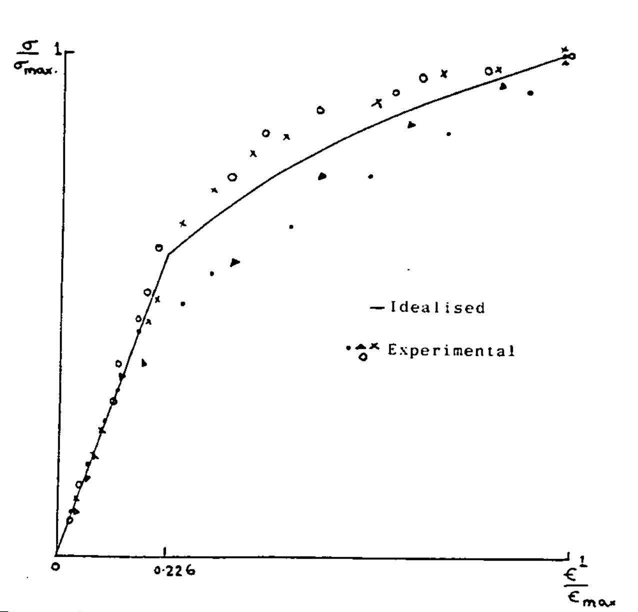


Figure 2. Non-dimensional stress strain relationship for low strength brickwork.

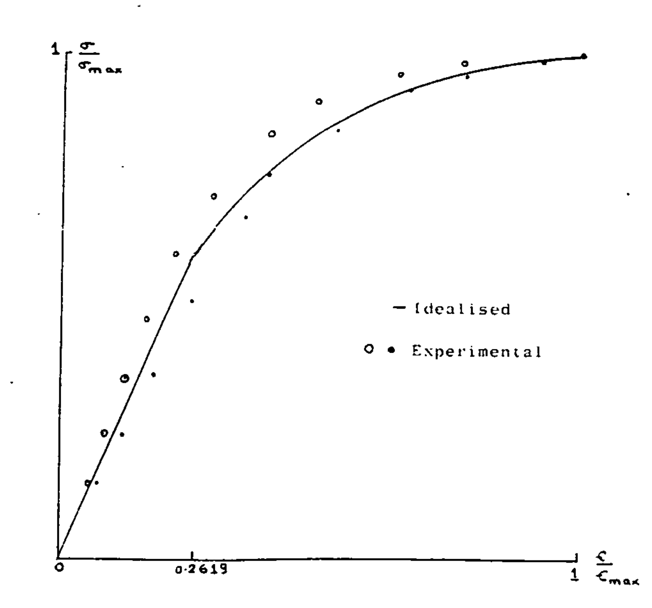


Figure 3. Non-dimensional stress strain curve for medium strength brickwork.

failure of the brickwork, the stress block becomes similar to the shape as shown in Figure 5 in (c) or (d) and characterised by λ_3 and λ_2. Also the area of the stress block can be related as λ_1 times the area of enclosing rectangle as shown in the previous section.

From Figure 5 (c and d) the total compressive and tensile force can be given by:

$$F_c = \lambda_1 . \lambda_3 . b . n . f_m \qquad (5)$$

$$F_t = A_s . f_y \qquad (6)$$

$$n = \frac{A_s . f_y}{\lambda_1 . \lambda_3 b . f_m} \qquad (7)$$

The ultimate moment M_u can be found by taking the moment of one of the forces about the line of action of either compressive or tensile forces.

$$M_{um} = \lambda_1 . \lambda_3 . b . n . (d - \lambda_2 n) f_m \qquad (8)$$

$$M_{ut} = A_s . f_y . (d - \lambda_2 n) \qquad (9)$$

The brickwork failure stress is assumed equal to the average compressive strength obtained from the test, hence $\lambda_3 = 1$.

For balanced section inserting numerical values from Table 1 in Equations (3) and (9) give the following:

(a) Curvilinear stress block (Figure 5d)

High strength brickwork

$$M_{um} = 0.734.1.b.0.42d(d - 0.397 \times 0.42d)\, f_m$$

$$M_{um} = 0.257\, f_m bd^2 \; ; \qquad (\epsilon_{max} = 0.0025) \qquad (10)$$

$$M_{ut} = A_s . f_y . d \left(1 - \frac{0.397 A_s f_y}{0.734 bd f_m}\right) \qquad (11)$$

$$= A_s . f_y . d\left(1 - \frac{0.54\, A_s f_y}{f_m\, bd}\right)$$

Medium strength brickwork

$$M_{um} = 0.746.1.b.0.44d^2(1 - 0.41 \times 0.44) f_m ; \; (\epsilon_{max} = 0.00275) \qquad (12)$$

$$= 0.269\, f_m bd^2$$

$$M_{ut} = A_s . f_y . d \left(1 - \frac{0.41 A_s f_y}{0.746 f_m bd}\right) = A_s . f_y . d\left(1 - \frac{0.55\, A_s f_y}{f_m . b . d}\right) \qquad (13)$$

Low strength brickwork

$$M_{um} = 0.7486.1.b.0.46d^2(1 - 0.414 \times 0.46) f_m ; (\epsilon_{max} = 0.003)$$

$$= 0.279\, f_m bd^2 \qquad (14)$$

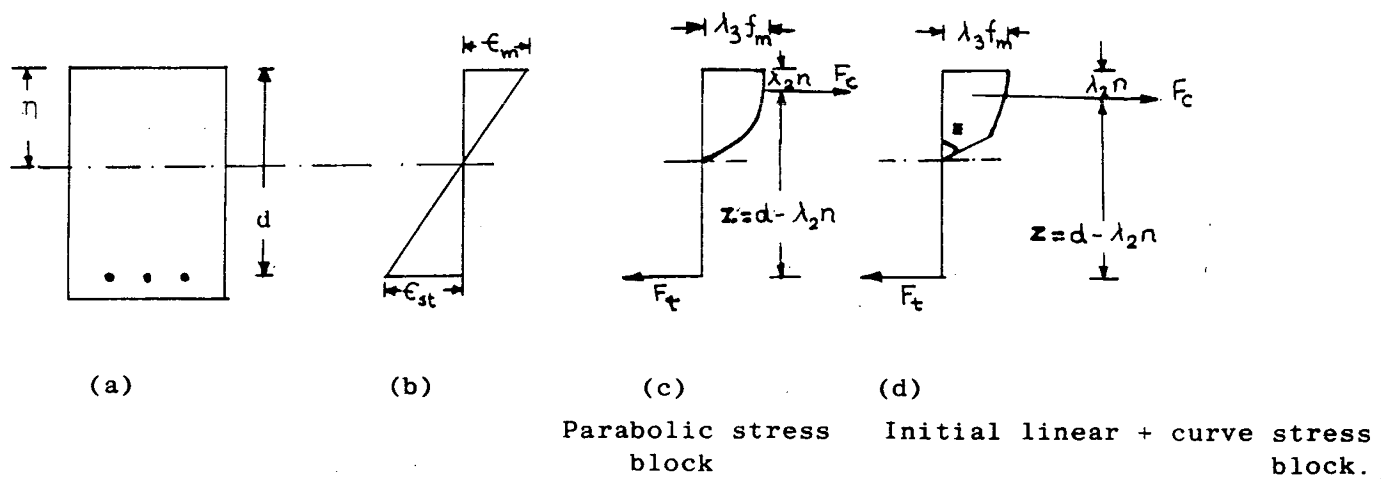


Figure 5. Strain distribution and stress block factors at collapse moment.

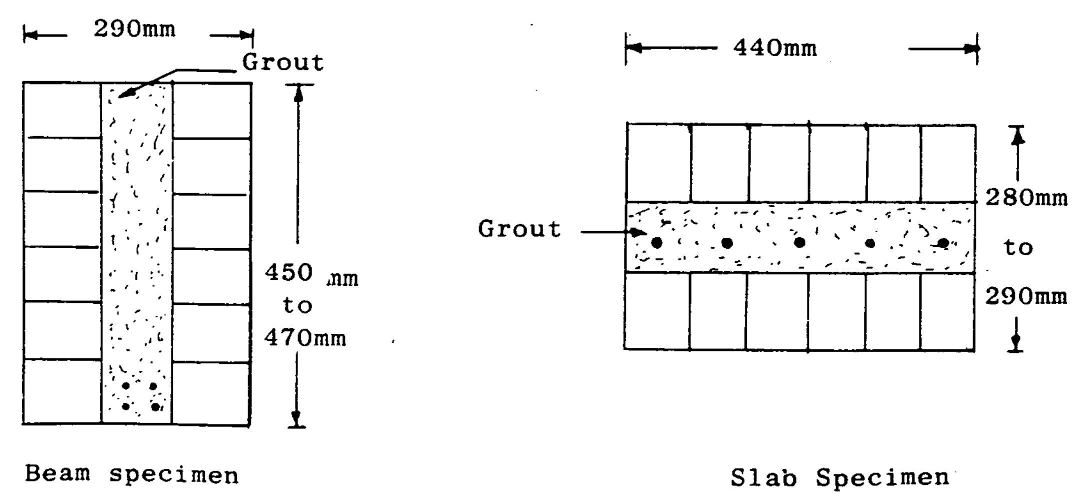


Figure 6. Cross-sectional dimensions of the test specimens.

$$M_{ut} = A_s.f_yd\left(1 - \frac{0.414\,A_sf_y}{0.7486f_mbd}\right) = A_s.f_y.d\left(1-\frac{0.553A_sf_y}{f_m.b.d}\right) \quad (15)$$

(b) Cubic parabolic stress block (Figure 5c)

High, medium and low brickwork strength (assuming constant brickwork failure strain $\epsilon_{max} = 0.003$)

$$M_{um} = 0.75.1.b.0.46d^2\ (1 - 0.4x0.46)f_m \quad (16)$$

$$= 0.2815\ f_mbd^2$$

$$M_{ut} = A_s.f_y.d\ \left(1 - \frac{0.4\ A_sf_y}{0.75f_mbd}\right) \quad (17)$$

$$= A_s.f_y.d\ \left(1 - \frac{0.533\ A_sf_y}{f_m.b.d}\right)$$

Brief experimental details

Slab and beam specimens and test arrangements

Two types of flexural specimens were tested; the cross-sectional dimensions are given in Figure 6. The % of high yield steel used was 0.9 for slab and 0.88 for beam specimens. The specimens were designed so that failure was expected in flexure and not due to shear which is the most common mode of failure in reinforced brickwork. Both types of specimens were tested under two point loading in a specially designed loading frame providing a pin and roller support as shown in Figure 7. The load was applied by means of two hydraulic jacks operated by a pump, and measured by load cells connected to a pen chart recorder and a digital voltmeter. The load was applied in stages till failure. The deflection and strain readings were both recorded during the experiment. The strain in the reinforcement was measured by electrical resistance gauges.

Results and discussions

Initially, under load both types of the specimens behaved as homogeneous beams with the neutral axes remaining approximately at the centre. With increased load, cracks appeared at the interface of brick and mortar below the level of reinforcement in the zone of maximum bending moment, thus causing the neutral axis to shift upwards. This continued till all the cracks were formed, after which the position of the neutral axis remained virtually constant until final failure. Figure 8 shows a typical relationship between the upward shift of the neutral axis in the zone of maximum moment and increasing load for an under-reinforced beam. The final failure of the specimens was either due to compressive failure of the brickwork, initiated by yielding of the steel or to the failure of both simultaneously (Figures 9 and 10).

In case of the slab specimen the compressive zone is located within the brickwork and the grout is only in the tensile zone, therefore, the grout does not contribute towards the ultimate strength of the slab. In the beam specimens the forces were not only resisted by the brickwork but also by the grout. As the modulus of elasticity obtained from the test of the grout cylinders was very similar to the modulus of elasticity of brickwork, hence the calculation based on a transformed section using the compatibility of strain at time of failure will not significantly increase the collapse moment.

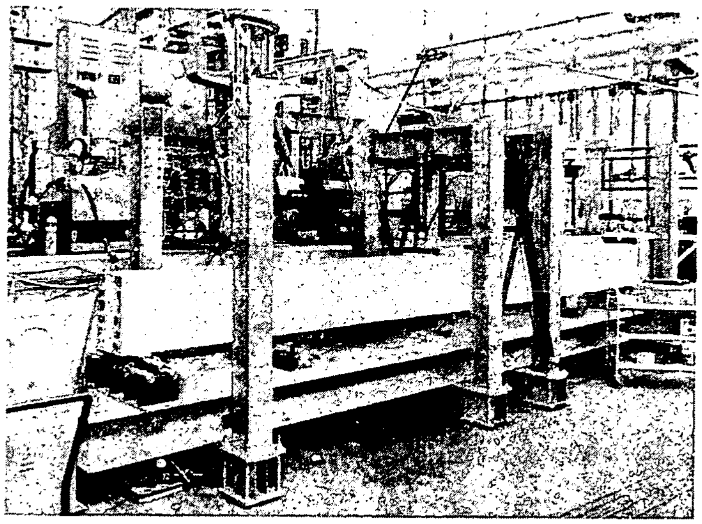
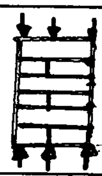
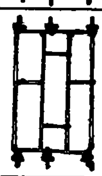


Figure 7. Test arrangements, showing roller and pin support.

Table 2. Comparison of experimental and theoretical ultimate moment for reinforced brickwork

Brick strength N/mm^2	Test Nos. *	Average compressive strength N/mm^2	Specimen type	Ultimate moment in kNm: Experimental		Theoretical: Cubic parabolic		Theoretical: Curvilinear (Initial linear + curve)	
Low: 21.55	1	11.17		26.78	28.24	27.89	28.25	27.65	28.04
	2			28.90		27.50		27.26	
	3			30.80		27.11		27.65	
	4			26.30		29.01		28.83	
	5			25.53		28.49		28.24	
	6			31.08		29.01		28.62	
Low: 21.55	7	12.92		183.0	185.7	172.5	171.67	171.23	170.43
	8			176.0		170.0		168.81	
	9			186.6		170.0		168.81	
	10			194.7		172.5		171.23	
	11			192.7		172.5		171.23	
	12			192.7		172.5		171.23	
Medium: 59.40	13	18.58	"	196.0	199.0	171.0	173.3	170.30	172.4
	14			197.2		173.10		172.24	
	15			204.0		175.78		174.66	
High: 88.33	16	22.04	"	200.0	196.75	176.9	176.9	176.3	176.3
	17			193.5		176.9		176.3	

** Note: 1 to 6 slab specimens*
7 to 17 beam specimens

The theoretical and experimental results are summarised in Table 2. As can be seen from the Table 2 there is a very good correlation between the experimental and the theoretical results.

Summary and conclusion

The collapse moment of a reinforced brickwork flexural member can be reliably predicted, utilising a curvilinear or a cubic parabolic stress block, average crushing strength and ultimate strains of brickwork on the basis of the formulae proposed in this paper. The ultimate strain has been derived experimentally and the crushing strength of brickwork may be found from a prism test, using a specimen of h/t ratio of 4.5 approximately.

ACKNOWLEDGEMENTS
The work described in this paper has been sponsored by the Building Research Station, Department of the Environment, U.K. and carried out in the Department of Civil Engineering and Building Science of the University of Edinburgh. The author gratefully acknowledges the valuable suggestions and encouragement given by Professor A.W. Hendry. The bricks used in these tests were donated by the London Brick Company and Messrs. G.H. Downing and Company.

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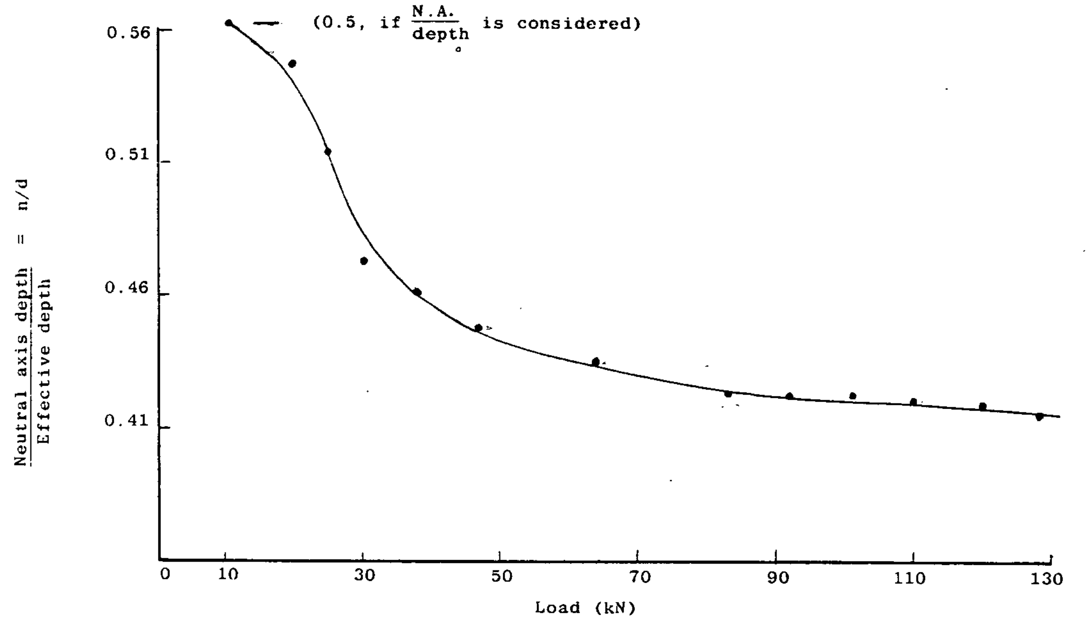

Figure 8. Relationship between load and depth of the neutral axis.

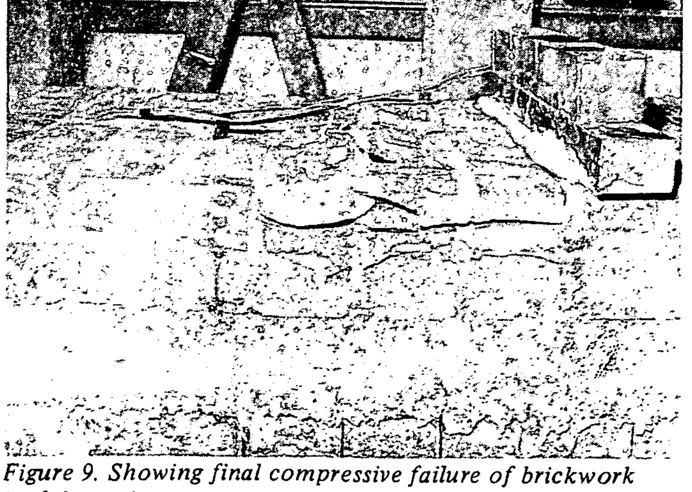

Figure 9. Showing final compressive failure of brickwork in slab specimen.

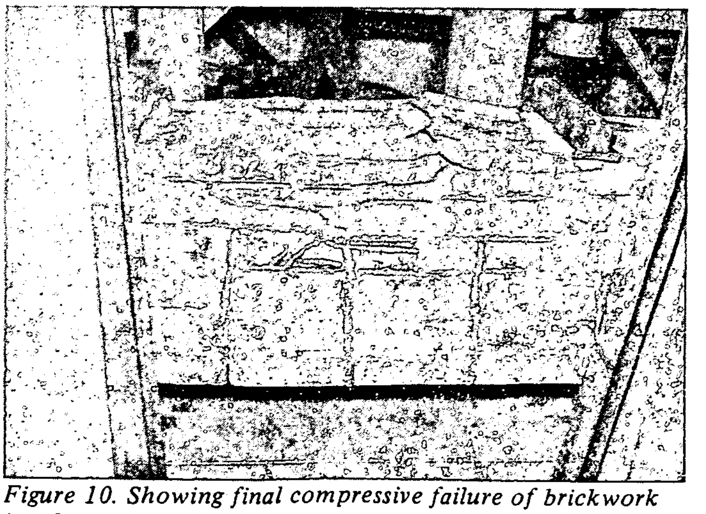

Figure 10. Showing final compressive failure of brickwork in a beam specimen.

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Reinforced grouted cavity brickwork

by B.P. Sinha

Reinforced and grouted cavity brickwork offers possibilies of a monolithic form of construction . [illegible] ; but there is little research-based data on this development. Within a research programme in the UK, the author has carried out extensive tests on such reinforced slabs and beams, with spans ranging from 1.3 to 6.3 m.

Results are reported on the ratio of shear span to effective depth, on proportions of tensile reinforcement, and on brick strengths, mortar grades and shear reinforcement. The characteristic shear strength for different percentages of tensile reinforcement is given, and a method is also outlined for calculating the ultimate moment.

Brickwork, being cheap and a low energy input material, has generated a lot of interest among engineers and architects during recent years. Although it is very strong in compression, its use is limited because of its low tensile strength. This disadvantage has been overcome in flexural members by incorporating reinforcement in the mortar joints. However, in such cases the thickness of the mortar joint (ref. 1) dictates the size of the reinforcing bars and further difficulty is envisaged in reinforcing it against shear, which is the most common mode of failure of reinforced brickwork elements.

These limitations can be overcome by the use of grouted cavity construction. Basically it consists of two skins of brickwork as used in an ordinary cavity wall, the cavity being used to accommodate the main and shear reinforcement. The cavity is finally grouted to form a monolithic construction and also to give additional protection to the reinforcement from the weather. This form of construction would have a wide field of application and may prove economical because of elimination of formwork. Although reinforced brickwork was the subject of investigations in the past (refs. 2, 3 and 4), no comprehensive tests were carried out on grouted cavity construction.

Because of this lack of experimental data, a comprehensive programme of research was undertaken. This paper describes the work done to investigate shear strength and the behaviour of grouted cavity reinforced brickwork beams covering the following variables:

(i) Shear span/effective depth a/d ratio. From very limited tests done in the past (refs. 5 and 6), it became evident that the a/d ratios have a profound effect on the shear strength of reinforced grouted beams. To establish this fact, the investigation was extended to cover a wide range of a/d ratios varying from 1.5 to 10.

(ii) % of tensile reinforcement. Five different percentages of tensile reinforcement varying from 0.88 to 2.54 were used in this investigation. The percentage of steel was kept constant while investigating the effects of other variables mentioned under section (i) and sections (iii) to (v). The percentage of high yield steel used as constant was 0.9 for slab and 0.88 for beam specimens.

(iii) Shear reinforcement. The variable considered was the spacing of the vertical stirrups and its effect on shear strength of beams built with various types and strenght of bricks. The typical detail is given in fig. 1. The steel hanger on the top was purposely omitted to avoid reinforcing the beam in the compression zone.

Notations:

a	=	shear span
b	=	breadth
d	=	effective depth
M_{um}	=	ultimate resistance moment based on masonry
M_{ut}	=	ultimate resistance moment based on steel
f_m	=	masonry prism strength
f_y	=	Steel yield stress

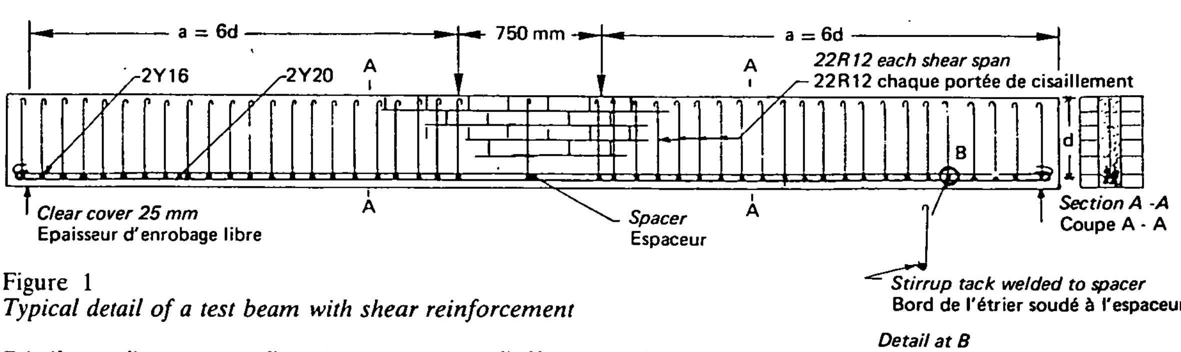


Figure 1
Typical detail of a test beam with shear reinforcement

Détail type d'une poutre d'essai avec armature d'efforts tranchants

Murs creux en briques armés et liaisonnés au coulis

par B.P.Sinha

Les murs creux en briques armés et liaisonnés au coulis permettent toutes sortes de formes de construction monolithique où la maçonnerie de briques constitue un coffrage permanent tout à fait acceptable, mais on ne dispose pas de nombreuses données de recherche sur ce développement. Dans le cadre d'un programme de recherche au RU, l'auteur a donc réalisé une série complète d'essais sur des dalles et des poutres armées de ce type, de 1,3 à 6,3 m de portée.

Nous donnons ici les résultats relatifs au rapport «portée en cisaillement — hauteur utile», aux proportions d'armatures tendues ainsi qu'aux résistances des briques, aux qualités de mortier et aux armatures d'efforts tranchants. Ce rapport indique également la résistance au cisaillement caractéristique pour différents pourcentages d'armatures tendues, ainsi qu'une méthode permettant de calculer le moment maximal.

La brique, matériau bon marché et dont la fabrication ne nécessite qu'une faible quantité d'énergie, suscite depuis quelques années, beaucoup d'intérêt chez les ingénieurs et les architectes. Malgré une très bonne résistance à la compression, son utilisation est limitée en raison de son peu de résistance à la traction. On a pu pallier ce défaut dans les éléments soumis à une flexion transversale en intégrant des aciers d'armature dans les joints en mortier. Mais, dans ce cas, c'est l'épaisseur du joint (réf. 1) qui dicte la dimension des barres et on se trouve confronté à une difficulté supplémentaire lorsqu'on veut armer ce type de maçonnerie contre le cisaillement qui est le mode de rupture le plus courant des éléments en maçonnerie de briques armée.

On peut surmonter ces inconvénients par l'emploi d'une maçonnerie creuse liaisonnée au coulis. Il s'agit essentiellement de deux parois de briques comme pour un mur creux ordinaire, mais dont la cavité sert à loger l'armature principale et l'armature d'efforts tranchants. Puis, on verse un coulis pour obtenir une construction monolithique et une meilleure protection des aciers d'armature contre les intempéries. Ce type de construction pourrait connaître un grand développement vu son caractère économique puisqu'il élimine les coffrages. La maçonnerie de briques armée a déjà fait le sujet d'études dans le passé (réf. 2, 3 et 4), mais aucun essai poussé n'avait été réalisé sur cette maçonnerie creuse liée au coulis.

C'est en raison de ce manque de données expérimentales qu'un vaste programme de recherche a été entrepris. Cet article décrit les travaux effectués pour mieux connaître la résistance au cisaillement et le comportement de ce type de poutres, travaux portant sur les variables suivantes:

a. Rapport portée en cisaillement/hauteur utile (a/d): d'après des essais très limités réalisés dans le passé (réf. 5 et 6), on s'était aperçu que les rapports a/d ont une influence certaine sur la résistance au cisaillement des poutres armées liaisonnées au coulis. Pour établir ce fait, les recherches ont été étendues pour couvrir une vaste gamme de rapports variant de 1,5 à 10.

b. Pourcentage d'armatures tendues: les chercheurs se sont basés sur cinq pourcentages différents d'armatures tendues, allant de 0,88 à 2,54. Le pourcentage d'acier a été maintenu constant pendant les recherches sur les autres variables mentionnées en *a.*, *c.*, et *e.* Le pourcentage d'acier à forte élasticité utilisé a été constamment de 0,9 pour les éprouvettes de dalles et 0,88 pour celles de poutres.

c. Armatures d'efforts tranchants: la variable étudiée a été l'espacement des étriers verticaux et son influence sur la résistance au cisaillement de poutres construites avec des briques de type et de résistance variés, comme on peut le voir sur la figure 1. Le crochet supérieur a été volontairement omis pour éviter d'armer la poutre dans la zone de compression.

d. Résistance des briques: trois types de briques, à résistance faible (21,55 N/mm^2), moyenne (59,38 N/mm^2) et élevée (88,33 N/mm^2) ont été utilisés pour la construction des éprouvettes.

e. Dosage du mortier: on pense que pour ce type de maçonnerie, le mortier de qualité III ne sera pas acceptable; aussi les recherches ont-elles porté essentiellement sur les qualités I (1: 1/4: 3) et II (1: 1/2: 4 1/2).

Eprouvettes et procédures d'essai

Deux types d'éprouvettes ont été utilisés pour les essais: une poutre et une dalle mince (cf. fig. 2). Les deux parois du mur creux étaient écartées dans les deux cas de 80-90 mm et liées par deux tirants en acier galvanisé à deux lames espacés de 450 × 300 mm et échelonnés. Pour la dalle, l'armature était au centre de la cavité; ainsi l'une des parois ne servait qu'à

Notations:
- a = portée en cisaillement
- b = largeur
- d = hauteur utile
- M_{um} = moment de résistance maximal basé sur la maçonnerie
- M_{ut} = moment de résistance maximal basé sur l'acier
- f_m = résistance d'un prisme de maçonnerie
- f_y = limite apparente d'élasticité de l'acier

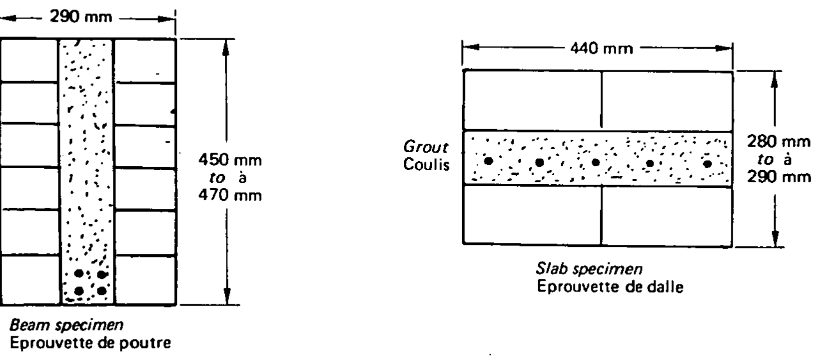
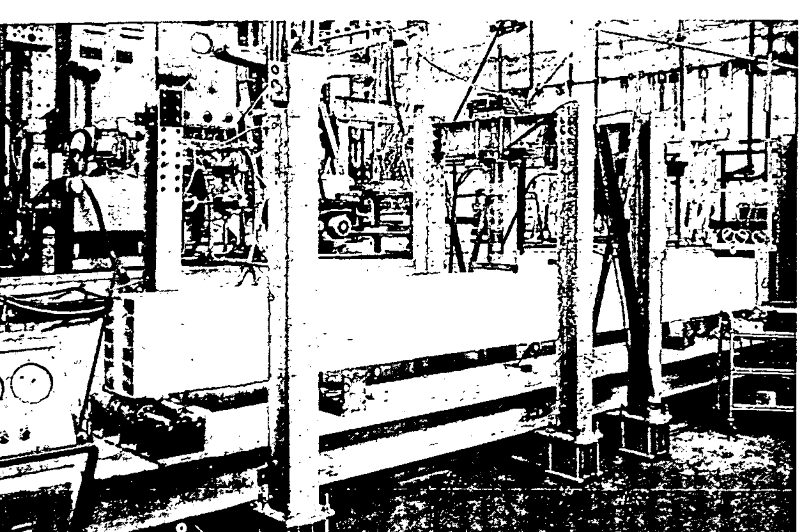


Figure 2
Cross section and dimensions of test specimens

Coupe et dimensions des éprouvettes d'essai

Figure 3
Test arrangement, with a 6.3 m span beam under test

Dispositif d'essai avec une poutre de 6,3 m de portée soumise à un essai.



(iv) Brick strength. Three types of bricks, low (21.55 N/mm^2), medium (59.38 N/mm^2) and high (88.33 N/mm^2) strength were used for the construction of the test specimens.

(v) Mortar grade. It is envisaged that for reinforced brickwork, the grade III motar will not be acceptable, hence the investigation focused on grades I (1 : 1/4 : 3) and II (1: 1/2: 4 1/2) mortars.

Test specimens and procedures

Two types of specimens, a beam and a thin slab, as shown in fig. 2, were used for the tests. The two leaves of the cavity wall forming both sets of specimens were 80-90 mm apart and were tied by galvanised fish-tail ties spaced 450 × 300 mm and staggered. For the slab the reinforcement was in the centre of the cavity; thus one of the leaves of the cavity wall was only providing cover to the reinforcement and used in this case merely as formwork. This type of construction is suitable as a retaining wall or when needed structurally to resist out-of-plane lateral pressure from either direction. For every specimen, full anchorage lengths for the reinforcement were provided following the concrete code (ref. 7). A 1: 0.1: 3: 2 (cement: lime: sand: peagravel) mix by volume was used for the grout with a constant water-cement ratio of 1: 2.

Both types of specimens were tested under two-point loading in a specially designed loading frame providing a pin and a roller support, as shown in fig. 3. The load was applied by means of two hydraulic jacks operated by a pump, the load being measured by the load cells connected to a pen chart recorder and a digital voltmeter. The load was applied at stages until failure and the deflection and strain readings were recorded. A 'demec' gauge measured the surface strain in the brickwork in the constant moment zone and electrical resistance gauges measured the steel strain. These gauges were fixed to the steel and wired prior to the grouting and construction of the beam.

Most of the test specimens for investigating the effect of shear span/effective depth ratios on the ultimate shear strength were made from low strength bricks. After the test it was found that this variable has no significant effect on the shear strength after a/d > 5. The effects of all other variables were investigated by keeping this constant at 6.

Results and discussion

Initially, under load both types of specimen behaved as a homogeneous cross-section with the neutral axis remaining approximately at the centre. With increasing load the cracks appeared at the interface of the brick and mortar below the level of reinforcement in the zone where the bending moment was maximum, thus causing the neutral axis to shift upwards. This continued until all the cracks had formed, after which the depth of the neutral axis remained virtually constant until final failure.

In the beam made from the high strength brick the cracks formed at the interface of brick and mortar at perpends and also along bed joints due to shear. The typical propagation of cracks and the shifting of the neutral axis under load is given in figs. 4 and 5 for an under-reinforced beam.

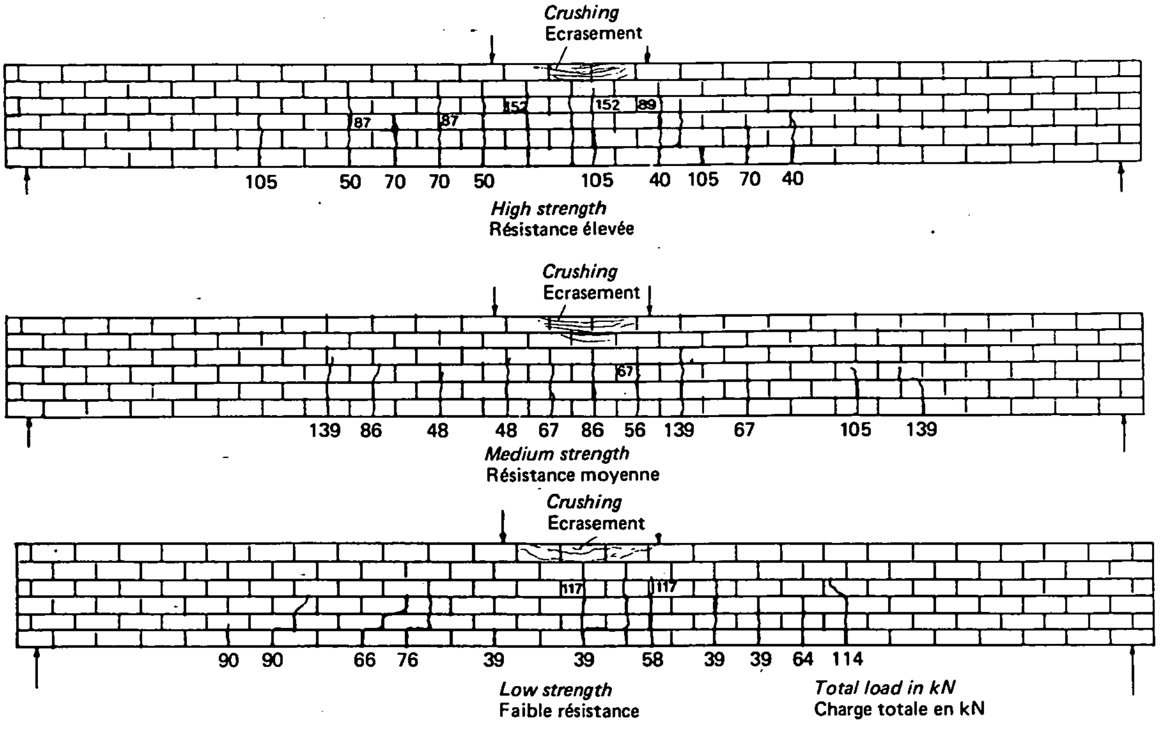


Figure 4
Typical crack pattern for beams with shear reinforcement shear span to effective depth ratio (a/d) = 6

Modèle type de fissuration pour des poutres avec armature d'efforts tranchants, rapport portée en cisaillement/hauteur utile (a/d) = 6

couvrir l'armature et n'était utilisée, dans ce cas, que comme coffrage. Ce type de construction convient comme mur de soutènement ou, si besoin est, pour résister à une pression latérale excentrée venant de l'une ou l'autre direction. Pour toutes les éprouvettes, des longueurs normales d'ancrage ont été assurées, conformément au code du béton (réf. 7). Un mélange 1: 0,1: 3: 2 (ciment: chaux: sable: gravillons) par volume a servi pour le coulis avec un rapport C/E constant de 2: 1.

Ces deux types d'éprouvettes ont été soumis à un essai de chargement en deux points dans un cadre de chargement spécialement conçu avec appuis à chevilles et à rouleaux. La charge a été appliquée au moyen de deux vérins hydrauliques actionnés par une pompe, la charge étant mesurée par des cellules reliées à un enregistreur et un voltmètre numérique. La charge a été appliquée par étapes jusqu'à rupture et les relevés de flèche et de déformation enregistrés. Un manomètre « demec » mesurait la déformation de surface dans la maçonnerie dans la zone de moment constant tandis que des jauges à résistance électriques mesuraient la déformation de l'acier. Ces jauges étaient fixées à l'acier et les fils branchés avant le coulage du coulis et la construction de la poutre.

La grande majorité des éprouvettes utilisées pour cette étude sur l'influence des rapports portée en cisaillement/hauteur utile sur la résistance maximale en cisaillement a été faite avec des briques à faible résistance. Cet essai a permis de se rendre compte que cette variable n'avait pas d'incidence significative sur la résistance au cisaillement après a/d > 5. Pour l'étude des effets des autres variables, on s'en est tenu à a/d = 6.

Résultats et discussion

Au départ, les deux types d'éprouvettes se sont comportées sous la charge de façon homogène, l'axe neutre restant approximativement au centre. Au fur et à mesure de l'augmentation de la charge, les fissures sont apparues à l'interface brique-mortier en-dessous du niveau des aciers d'armatures dans la zone de moment de flexion maximal, repoussant ainsi l'axe neutre vers le haut. Ce phénomène s'est poursuivi jusqu'à ce que toutes les fissures se soient formées, après quoi la hauteur de l'axe neutre est demeurée pratiquement constante jusqu'à la rupture finale.

Dans la poutre faite de briques à forte résistance, les fissures se sont formées à l'interface brique-mortier perpendiculairement aux joints entre lits et également le long de ceux-ci, en raison du cisaillement. Les figures 4 et 5 donnent la propagation type des fissures et le déplacement de l'axe neutre sous la charge pour

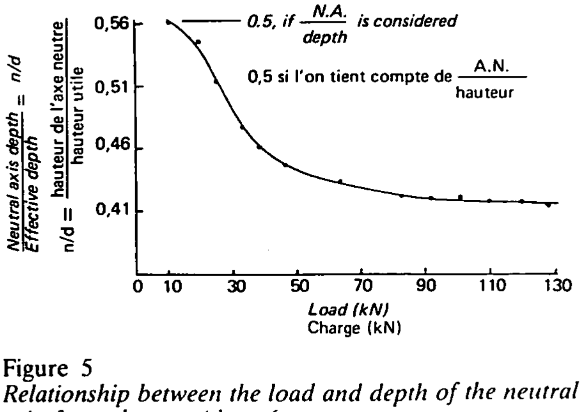


Figure 5
Relationship between the load and depth of the neutral axis for a beam a/d = 6

Rapport entre la charge et la profondeur de l'axe neutre pour une poutre de a/d = 6

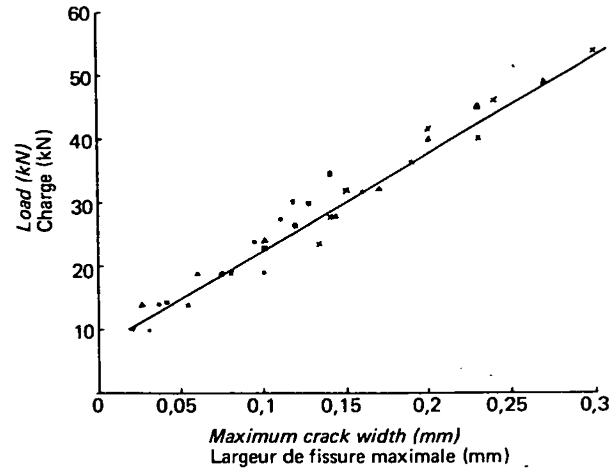


Figure 8
Relationship between load and maximum crack width for medium and high-strength brickwork, a/d = 6

Rapport charge/largeur maximale de fissuration pour des maçonneries de résistance moyenne et élevée, a/d = 6

Some typical load deflection curves for both types of specimens are given in figs. 6 and 7 and this relationship appears to be bilinear. The deflection increased once the external moment exceeded the cracking moment. The load at which this deviation in deflection takes place can be predicted very closely (ref. 5) by using the flexural strength of brickwork and uncracked section (full cross-section) of the beam or slab. The deflection of different beams under similar loading conditions is not the same because of variations in the modulus of elasticity of brickwork, the workmanship, and the presence of hair cracks at the brick/mortar interface, affecting the stiffness of the beams and slabs.

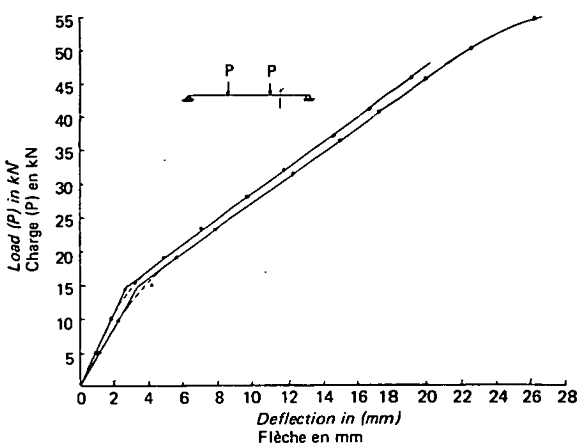


Figure 6
Load deflection relationship for low-strength brick beams, a/d = 6

Rapport charge/flèche pour des poutres en briques de faible résistance, a/d = 6

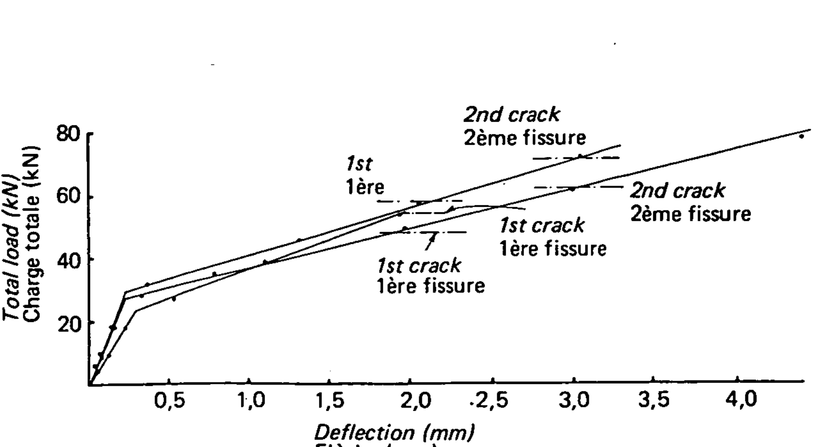


Figure 7
Load/deflection graphs for wall, a/d = 6

Courbes charge-flèche pour un mur de a/d = 6

Cracks width

In a few beams the crack widths at the bottom were measured. The cracks were largely confined to the brick and mortar interface and the mean crack spacing was usually at half-a- brick length. Some typical relationships are given in fig. 8 between crack width and the load for beam of a/d ratio 6. The maximum crack width of 0.3 mm is permitted by the concrete code. However, this crack width for reinforced grouted beams was exceeded only near the ultimate load.

Shear strength

Effect of shear span/effective depth ratio

As can be seen from fig. 9 and table 1, the shear strength of the grouted reinforced brickwork beams and slabs increases with the decreasing shear span/ depth ratio. The shear failure of beams and slabs with higher shear span/effective depth ratios is due to the development of a typical diagonal crack (fig. 10), whereas for beams and slabs with lower a/d ratio cracking is followed by the development of a 'tied arch effect'. With a lower a/d ratio this effect is greater, resulting in higher strength in shear.

Generally the ultimate shear strengths of the slabs were higher than those of the beams. This difference (fig. 9) is significant when the a/d ratio is lower than 6. From the results it may, however, be inferred that not only does the a/d ratio have an effect on the ultimate shear strength, but also the depth — the shallower the beam the higher is the shear strength. Hence care must be exercised while comparing the test results from the different sources (ref. 8). The characteristic shear strength of beams and slabs are given in fig. 9, which ignores the effect of the a/d ratio.

Because of flexural failures (fig. 12), the shear strength was slightly lower than expected for the wall of a/d = 8 built with low strength bricks.

une poutre insuffisamment armée. On trouvera, en figures 6 et 7, quelques courbes de flèches types pour les deux types d'éprouvettes et le rapport semble être bilinéaire. La flèche a augmenté une fois que le moment externe a dépassé le moment de fissuration. Il est possible de prévoir assez précisément la charge qui provoque cet écart de flèche (réf. 5) en utilisant la résistance à la flexion de la maçonnerie de briques et le profil non fissuré de la poutre ou de la dalle. La flèche des différentes poutres dans des conditions de charge similaires n'est pas la même en raison des variations du module d'élasticité de la maçonnerie, du travail des ouvriers et de la présence de « cheveux » à l'interface brique-mortier affectant la rigidité des poutres et des dalles.

Largeur des fissures

On a mesuré en bas d'un petit nombre de poutres, la largeur des fissures. Celles-ci affectaient essentiellement l'interface brique-mortier et l'espace moyen entre les fissures était généralement d'une demi-longueur de brique. La figure 8 donne quelques rapports types entre la largeur des fissures et la charge pour une poutre de a/d = 6. Le code du béton admet une largeur de fissure maximale de 0,3 mm. Cette largeur de fissure pour les poutres armées et liées au coulis n'a été dépassée que près de la charge maximale.

Résistance au cisaillement

Incidence du rapport portée de cisaillement/hauteur utile

Comme on peut le voir d'après la figure 9 et le tableau 1, la résistance au cisaillement des poutres et des dalles en maçonnerie de briques armée et liée augmente au fur et à mesure que décroît le rapport portée de cisaillement/hauteur utile. La rupture en cisaillement des poutres et des dalles avec des rapports plus élevés est due au développement d'une fissure diagonale typique (fig. 10), tandis que, pour des dalles et des poutres avec des rapports a/d inférieurs, la fissuration est suivie par le développement d'un « effet d'arc attaché ». Avec un a/d inférieur, cet effet est plus grand, entraînant une résistance plus forte au cisaillement.

En général, les résistances maximales au cisaillement des dalles ont été plus fortes que celles des poutres. Cette différence (fig. 9) est significative lorsque le rapport a/d est inférieur à 6. On peut conclure de ces résultats que non seulement le rapport a/d a bien une influence sur la résistance au cisaillement maximale, mais également la hauteur — moins la poutre est épaisse, plus la résistance au cisaillement est forte. Il faut donc faire extrêmement attention lorsque l'on compare des résultats d'essais de différentes sources (réf. 8). La figure 9 donne la résistance caractéristique au cisaillement des poutres et des dalles qui ne tient pas compte de l'incidence du rapport a/d.

En raison des ruptures en flexion (fig. 12), la résistance au cisaillement a été légèrement inférieure à celle qu'on escomptait pour le mur de a/d = 8 construit avec des briques de faible résistance.

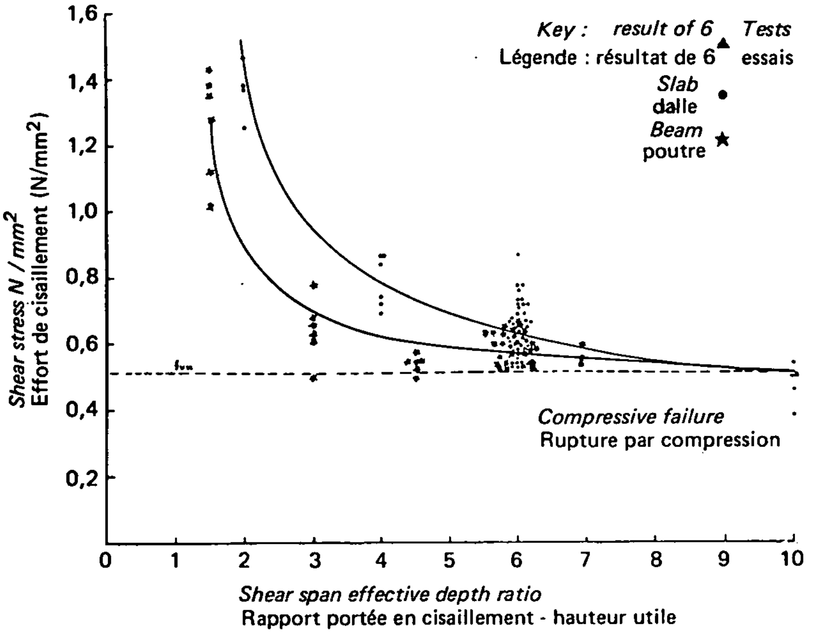


Figure 9
Shear strength in relation to the ratio of shear span to effective depth, a/d

Résistance au cisaillement en fonction du rapport portée en cisaillement/hauteur utile, a/d

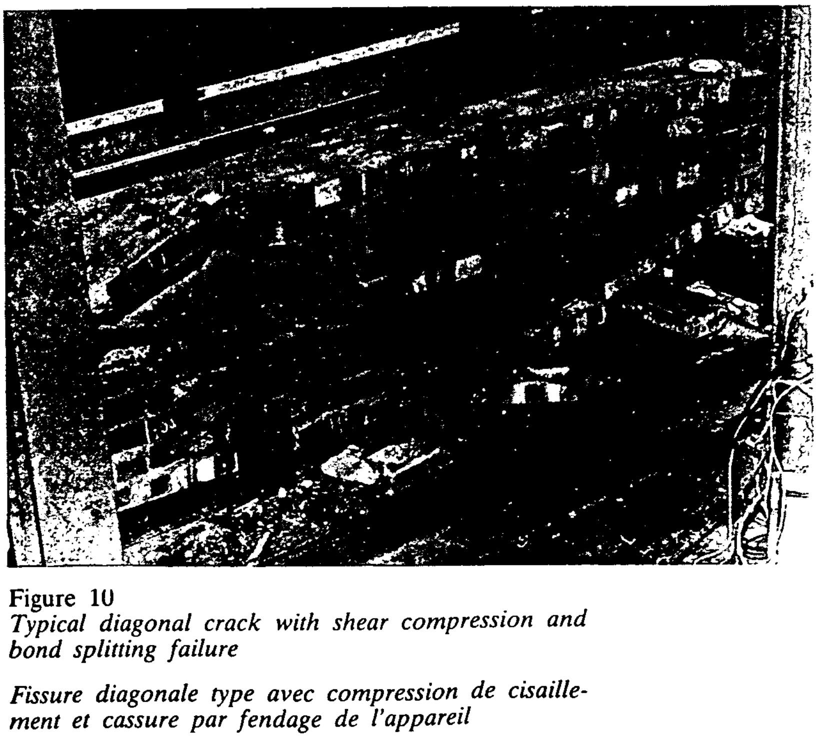


Figure 10
Typical diagonal crack with shear compression and bond splitting failure

Fissure diagonale type avec compression de cisaillement et cassure par fendage de l'appareil

Table 1 *Effect of shear span/effective depth ratios on the shear strength of reinforced grouted brickwork beams and slabs*

Type of specimen	Brick strength N/mm^2	No of speci-mens	Ratio shear span/ effective depth	1 : 1/4 : 3 mortar strength N/mm^2	1 : 0.1 : 3 : 2 grout strength N/mm^2	Nominal ultimate shear stress N/mm^2 V/bd	Average shear stress N/mm^2	Coeff. of variation in %	Remarks
Slabs		1	2	18.7	11.6	1.76			
(% of high		2	2	21.5	11.6	1.46			
yield steel =		3	2	24.9	10.0	1.38	1.51	16.0	
0.9 for all		4	2	24.0	10.0	1.25			
tests)		5	2	20.0	10.9	1.37			
		6	2	21.0	10.9	1.86			Shear failure at
		1	4	20.6	11.7	0.72			the interface of
		2	4	21.55	11.7	0.69			brick and grout
		3	4	21.67	16.5	0.86			
		4	4	21.22	16.5	0.86	0.785	9.8	
	Low strength	5	4	21.22	16.5	0.84			
	flat : 21.55	6	4	19.78	17.37	0.74			
		1	6	20.46	14.18	0.52			
		2	6	20.46	14.18	0.63	0.61	—	
		3	6	25.0	18.02	0.69			
		4	6	25.0	18.02	0.58			
		1	8	22.14	17.37	0.405			Yielding of steel
		2	8	18.86	13.04	0.44			and subsequent
		3	8	20.86	13.04	0.47	0.43	7.87	compressive
		4	8	19.66	13.56	0.43			failure of b.w.
		5	8	19.66	13.56	0.38			
		6	8	20.46	14.18	0.46			
Slabs	High strength	1	10	22.7	22.1	0.50			Shear failure
	flat : 88.33	2	10	22.7	22.1	0.50			
		3	10	22.7	22.1	0.54	0.48	12.6	
		4	10	19.2	19.2	0.46			
		5	10	19.2	19.2	0.38			
Beams		1	1.5	22.14	17.37	1.28			
(% of high		2	1.5	18.86	13.04	1.38			
yield steel =		3	1.5	20.80	13.04	1.43	1.26	12.99	Shear failure
0.88 for all	Low strength	4	1.5	19.66	13.08	1.35			
tests)	on bed : 21.55	5	1.5	19.25	13.08	1.12			
		6	1.5	21.96	14.18	1.01			
		1	3.0	20.6	11.7	0.68			
	on edge :	2	3.0	21.55	11.7	0.63			
	16.10	3	3.0	21.67	16.5	0.49	0.64	14.8	Shear failure
		4	3.0	21.22	16.5	0.61			
		5	3.0	19.78	17.37	0.66			
		6	3.0	22.14	17.37	0.78			
		1	4.5	18.7	11.6	0.49			
		2	4.5	21.5	11.6	0.51			
		3	4.5	24.3	10.0	0.53	0.53	4.4	Shear failure
		4	4.5	24.0	10.0	0.53			
		5	4.5	20.0	10.9	0.53			
		6	4.5	21.0	10.9	0.56			
		1	6.0	22.20	14.0	0.59			
		2	6.0	22.20	14.0	0.64			
		3	6.0	19.96	16.86	0.64	0.63	3.4	Shear failure
		4	6.0	19.96	16.86	0.63			
		5	6.0	19.96	17.24	0.65			
		6	6.0	22.86	17.24	0.64			
		1	6.8	19.3	17.5	0.53			Explosive failure
		2	6.8	22.5	14.0	0.55	0.56	—	Explosive compressive at top
		3	6.8	23.3	14.0	0.60			Slow compressive at top

Tableau 1 *Influence des rapports portée en cisaillement/hauteur utile sur la résistance au cisaillement des poutres et des dalles en maçonnerie armée liée au coulis* (% d'acier à forte élasticité = 0,9 pour tous les essais)

Type d'éprouvette	Résistance des briques (N/mm²)	N° des éprouvettes	Rapport portée en cisaillement/hauteur utile	Résistance du mortier 1 : 1/4 : 3 (N/mm²)	Résistance du coulis 1 : 0,1 : 3 : 2 (N/mm²)	Effort de cisaillement maximal nominal (N/mm²) (V/bd)	Effort de cisaillement moyen (N/mm²)	Coefficient de variation (%)	Remarques
Dalles (% d'acier à forte élasticité = 0.9 pour tous les essais)		1	2	18,7	11,6	1,76			
		2	2	21,5	11,6	1,46			
		3	2	24,9	10,0	1,38	1,51	16,0	
		4	2	24,0	10,0	1,25			
		5	2	20,0	10,9	1,37			
		6	2	21,0	10,9	1,86			
		1	4	20,6	11,7	0,72			Rupture de cisaillement à l'interface brique-coulis
	Plates à faible résistance : 21,55	2	4	21,55	11,7	0,69			
		3	4	21,67	16,5	0,86			
		4	4	21,22	16,5	0,86	0,785	9,8	
		5	4	21,22	16,5	0,84			
		6	4	19,78	17,37	0,74			
		1	6	20,46	14,18	0,52			
		2	6	20,46	14,18	0,63	0,61	—	
		3	6	25,0	18,02	0,69			
		4	6	25,0	18,02	0,58			
		1	8	22,14	17,37	0,405			Ecoulement de l'acier et rupture ultérieure par compression des murs en maçonnerie de briques
		2	8	18,86	13,04	0,44			
		3	8	20,86	13,04	0,47	0,43	7,87	
		4	8	19,66	13,56	0,43			
		5	8	19,66	13,56	0,38			
		6	8	20,46	14,18	0,46			
Dalles	Plates à résistance élevée : 88,33	1	10	22,7	22,1	0,50			
		2	10	22,7	22,1	0,50			
		3	10	22,7	22,1	0,54	0,48	12,6	Rupture de cisaillement
		4	10	19,2	19,2	0,46			
		5	10	19,2	19,2	0,38			
Poutres (% d'acier à forte élasticité = 0,88 pour tous les essais)	Faible résistance sur couche : 21,55	1	1,5	22,14	17,37	1,28			
		2	1,5	18,86	13,04	1,38			
		3	1,5	20,80	13,04	1,43	1,26	12,99	
		4	1,5	19,66	13,08	1,35			
		5	1,5	19,25	13,08	1,12			
		6	1,5	21,96	14,18	1,01			
		1	3,0	20,6	11,7	0,68			
		2	3,0	21,55	11,7	0,63			
	sur bord : 16,10	3	3,0	21,67	16,5	0,49	0,64	14,8	
		4	3,0	21,22	16,5	0,61			
		5	3,0	19,78	17,37	0,66			
		6	3,0	22,14	17,37	0,78			Rupture de cisaillement
		1	4,5	18,7	11,6	0,49			
		2	4,5	21,5	11,6	0,51			
		3	4,5	24,3	10,0	0,53	0,53	4,4	
		4	4,5	24,0	10,0	0,53			
		5	4,5	20,0	10,9	0,53			
		6	4,5	21,0	10,9	0,56			
		1	6,0	22,20	14,0	0,59			
		2	6,0	22,20	14,0	0,64			
		3	6,0	19,96	16,86	0,64	0,63	3,4	
		4	6,0	19,96	16,86	0,63			
		5	6,0	19,96	17,24	0,65			
		6	6,0	22,86	17,24	0,64			
		1	6,8	19,3	17,5	0,53			Rupture explosive
		2	6,8	22,5	14,0	0,55	0,56	—	Explosive par compression au sommet
		3	6,8	23,3	14,0	0,60			Lente compression au sommet

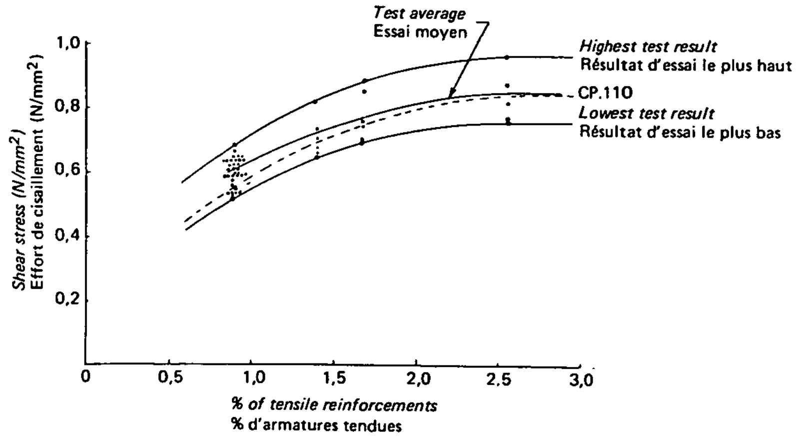
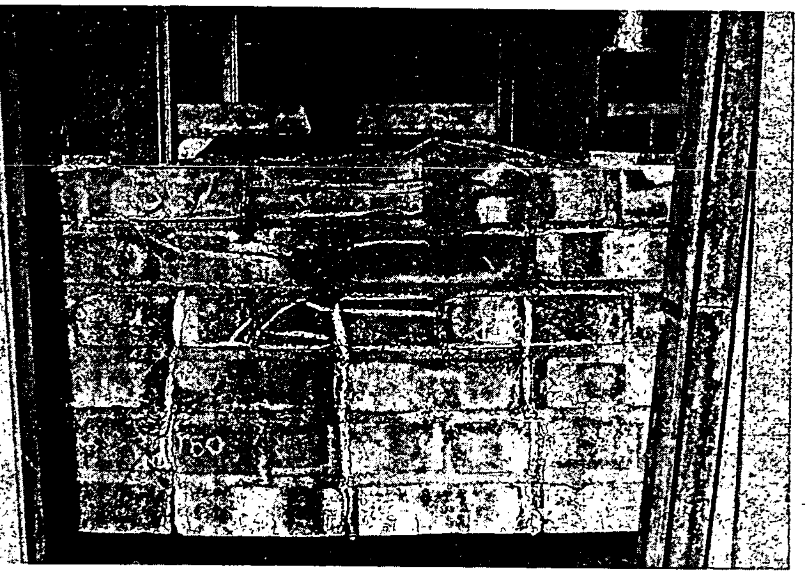


Figure 11
Shear strength in relation to percentage of tensile reinforcement

Résistance au cisaillement en fonction du pourcentage d'armatures tendues

Figure 12
Compressive failure of brickwork in a beam specimen

Rupture par compression de la maçonnerie dans une éprouvette de poutre



Effect of tensile reinforcement

Fig. 11 shows the effect of the percentage of tensile reinforcement on the shear strength of reinforced grouted brickwork flexural members. The brick strength (table 3) has no effect on shear strength; hence it has been ignored in plotting this curve. The shear strength increases with the increase in percentage of tensile steel, but not linearly.

Effect of shear reinforcement

The shear strength of beams built with medium and high strength bricks increased, since the shear reinforcement prevented the premature failure of the beam. The beams ultimately failed due to the yielding and crushing of the brickwork (fig. 13) in the maximum bending moment zone. In the case of low strength brick beams where the stirrups were provided at very close spacing or spacing equal to the effective depth of the beams, only a slight increase in shear strength was recorded (table 2). This is because the flexural and shear capacity of the beams may be of the same order and hence the failure load did not increase with shear reinforcement, except that it altered the failure mode.

The failure of beams with the shear reinforcement was slow and ductile compared to ultimate brittle failure of beams without it. It may be advisable to provide reinforcement, nominal or otherwise, against shear to stop sudden and catastrophic failure of reinforced brickwork beams.

Table 2 *Shear strenght of reinforced grouted brickwork beams with and without shear reinforcements*

$\frac{\text{shear span}}{\text{effective depth}}$ (a/d) = 6

No of specimens	Type and brick strength (N/mm²)	% steel	Shear strength (N/mm²)					
			No shear reinforcements		With shear reinforcements			
					8 mm Ø 360 mm		8 mm Ø 90 mm	
1	Low strength	0.88	0.59	0.63	0.72	0.66	0.67	0.65
2	Flat : 21.55		0.64		0.67		0.65	
3	On edge : 16.10		0.64		0.59		0.63	
4			0.63					
5			0.65					
6			0.64					
					With shear reinforcements 12 mm Ø 110 mm			
1	Medium strength	0.88	0.60	0.60	0.74	0.73		
2	Flat : 59.40		0.66		0.71			
3	On edge : 31.92		0.63		0.73			
4			0.53					
5			0.60					
6			0.60					
1	Flat : 88.33	0.88	0.59	0.61	0.78	0.76		
2	On edge : 26.4		0.64		0.75			
3			0.56					
4			0.59					
5			0.62					
6			0.64					

Influence des armatures tendues

La figure 11 montre l'influence du pourcentage des armatures tendues sur la résistance au cisaillement d'éléments en flexion. La résistance des briques (tableau 3) n'a pas d'effet sur la résistance au cisaillement ; il n'en a donc pas été tenu compte dans le tracé de la courbe. La résistance au cisaillement augmente en même temps qu'augmente le pourcentage des armatures tendues, mais de façon non linéaire.

Influence des armatures de cisaillement

La résistance au cisaillement des poutres construites avec des briques à moyenne et forte résistance a augmenté, étant donné que l'armature contre le cisaillement empêchait la ruine prématurée de la poutre. La ruine a fini par se produire en raison de l'écoulement et de l'écrasement de la maçonnerie (fig. 13) dans la zone de moment de flexion maximal. Dans le cas de poutres en briques à faible résistance où les étriers étaient très peu espacés ou selon un espace égal à la hauteur utile des poutres, seule a été enregistrée une légère augmentation de la résistance au cisaillement (tableau 2), et ceci parce que les capacités de résistance à la flexion ou au cisaillement peuvent être du même ordre ; donc la charge de rupture n'a pas augmenté avec les armatures de cisaillement ; par contre, elle a modifié le mode de ruine.

La rupture des poutres avec armature de cisaillement a été lente et ductile par rapport à la rupture fragile maximale des poutres sans ce type d'armatures. Il pourrait donc être judicieux d'assurer une armature, nominale ou autre, contre le cisaillement pour stopper une rupture soudaine et catastrophique des poutres en maçonnerie de briques armée.

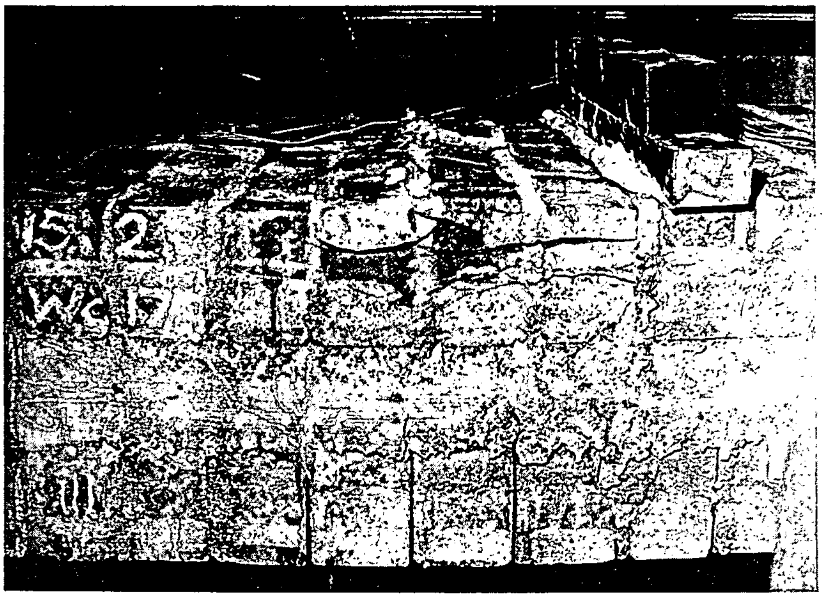


Figure 13
Final compressive failure of brickwork in a slab specimen

Rupture définitive par compression de la maçonnerie dans une éprouvette de dalle

Tableau 2 *Résistance à la compression des poutres en maçonnerie de briques armée et liée au coulis avec aciers contre le cisaillement ou sans*

$$\frac{\text{portée en cisaillement}}{\text{hauteur utile}} = (a/d) = 6$$

N° des éprouvettes	Type et résistance des briques (N/mm²)	% d'acier	Résistance au cisaillement (N/mm²)					
			Sans aciers contre le cisaillement		Avec aciers contre le cisaillement			
					8 mm Ø 360 mm		8 mm Ø 90 mm	
1			0,59		0,72		0,67	
2	Faible résistance		0,64		0,67	0,66	0,65	0,65
3	plates : 21,55		0,64	0,63	0,59		0,63	
4	sur bord : 16,10	0,88	0,63					
5			0,65					
6			0,64					
1			0,60		Avec aciers contre le cisaillement 12 mm Ø 110 mm			
2	Résistance moyenne		0,66					
3	plates : 59,40		0,63					
4	sur bord : 31,92	0,88	0,53	0,60	0,74			
5			0,60		0,71	0,73		
6			0,60		0,73			
1			0,59		0,78	0,76		
2			0,64		0,75			
3	plates : 88,33		0,56					
4	sur bord : 26,4	0,88	0,59	0,61				
5			0,62					
6			0,64					

development of a weak interface bond strength with the highly absorptive brick. However, the range of results suggests that this factor is not significant and for practical purposes may be ignored.

Effect of mortar grade

From table 4 it can be seen that the shear strength of reinforced grouted brickwork beams built in 1 : 1/2 : 4 1/2 grade II mortar is slightly lower than grade I mortar, 1 : 1/4 : 3. The strength difference is not very significant and may be assumed to be the same.

Effect of brick strength

The shear strength of beams is not very significantly affected by brick strength (table 3) or brickwork strength and for practical purposes can be taken as the same. However, the effect of brickwork strength becomes apparent when a beam is reinforced against shear and the failure is due to flexure.

In the case of low strength brick slabs, the shear strength was slightly lower than medium and high strength brick (table 3). This could be due to the

Characteristic shear strength for various percentages of tensile steel

In the limit state code, the variation in material strength is allowed for by a characteristic strength which is calculated by statistical principle.

The characteristic strength is defined by:

$$f_{kv} = f_{mv} - z.s. \qquad \text{(I)}$$

where

f_{kv} is the characteristic shear strength
f_{mv} is the mean shear strength
z is the coefficient = 1.64 (90 per cent probability)

At a shear span/effective depth ratio equal to 6, the shear strength (fig. 9) is not affected any more either by the a/d ratio or by the thinness of the section. Also at this ratio the sample, consisting of a

Table 3 *Effect of brick strenght on the shear strength of reinforced brickwork beams and slab*

$$\frac{\text{shear span}}{\text{effective depth}} \ (a/d) = 6$$

Type of specimen	Brick strength N/mm²	No of tests	1 : 1/4 : 3 mortar strength N/mm²	1 : 0.1 : 3 : 2 grout strength N/mm²	Nominal ultimate shear stress N/mm²	Average shear stress N/mm²	Coefficient of variation in %
Beams (with % high yield steel = 0.88)	Low strength flat : 21.55 on edge : 16.10	1	22.2	14.0	0.59	0.63	3.4
		2	22.2	14.0	0.64		
		3	19.96	16.86	0.64		
		4	19.96	16.86	0.63		
		5	22.86	17.24	0.65		
		6	22.86	17.24	0.64		
	Medium strength flat : 59.40 on edge : 31.92	1	24.58	15.61	0.60	0.60	7.2
		2	20.80	16.10	0.66		
		3	26.85	19.20	0.63		
		4	23.50	15.61	0.53		
		5	20.80	16.10	0.60		
		6	21.73	15.58	0.60		
	High strength flat : 88.33 on edge : 26.4	1	21.59	15.0	0.59	0.61	5.3
		2	21.59	15.0	0.64		
		3	21.17	12.64	0.56		
		4	21.17	12.64	0.59		
		5	19.04	17.68	0.62		
		6	19.04	17.68	0.64		
Slabs (% of high yield steel = 0.9)	Low strength flat : 21.55 on edge : 16.10	1	20.46	14.18	0.52	0.61	—
		2	20.46	14.18	0.63		
		3	25.0	18.02	0.69		
		4	25.0	18.02	0.58		
	Medium strength flat : 59.40 on edge : 31.92	1	21.66	15.58	0.62	0.71	10.28
		2	23.51	11.93	0.74		
		3	23:51	11.93	0.62		
		4	21.66	15.58	0.78		
		5	21.66	15.8	0.76		
		6	23.51	11.93	0.76		
	High strength flat : 88.33 on edge : 26.40	1	21.53	16.11	0.59	0.71	13.3
		2	21.53	16.11	0.72		
		3	21.53	16.11	0.69		
		4	26.85	19.18	0.65		
		5	24.0	15.61	0.72		
		6	24.0	15.61	0.87		

Influence de la résistance à la traction des briques

La résistance au cisaillement des poutres n'est pas affectée de façon significative par la résistance des briques (tableau 3) ou de la maçonnerie et l'on peut dans la pratique, les considérer comme similaires. Par contre, l'influence de la résistance de maçonnerie devient apparente lorqu'une poutre est armée contre le cisaillement et que la rupture est due à la flexion.

Dans le cas de dalles en briques à faible résistance, la résistance au cisaillement a été légèrement inférieure à celles des briques à moyenne et forte résistance (tableau 3). Cela pourrait être dû au développement d'une faible adhérence aux interfaces avec la brique à forte absorption. La gamme des résultats obtenus suggère cependant que ce facteur n'a pas grande importance et qu'on peut, en pratique, ne pas en tenir compte.

Influence du dosage du mortier

On peut voir, d'après le tableau 4, que la résistance au cisaillement de ce type de poutres construites avec du mortier de qualité II (1: 1/2: 4 1/2) est légèrement moins forte qu'avec du mortier de qualité I (1 : 1/4 : 3), mais cette différence n'est pas très significative et l'on peut considérer les résistances comme similaires.

Résistance caractéristique au cisaillement pour divers pourcentages d'aciers tendus

Dans le code relatif aux états limites, il est tenu compte de la variation dans la résistance d'un matériau par une résistance caractéristique que l'on calcule selon le principe statistique.

Tableau 3 *Influence de la résistance des briques sur la résistance au cisaillement des poutres et des dalles en maçonnerie*

$$\frac{\text{portée en cisaillement}}{\text{hauteur utile}} = (a/d) = 6$$

Type d'éprouvette	Résistance des briques (N/mm²)	N° des essais	Résistance du mortier 1 : 1/4 : 3 (N/mm²)	Résistance du coulis 1 : 0,1 : 3 : 2 (N/mm²)	Effort de cisaillement maximal nominal (N/mm²	Effort de cisaillement moyen (N/mm²)	Coefficient de variation (%)
Poutres % d'acier à forte élasticité : 0,88	Faible résistance plates : 21,55 sur bord : 16,10	1 2 3 4 5 6	22,2 22,2 19,96 19,96 22,86 22,86	14,0 14,0 16,86 16,86 17,24 17,24	0,59 0,64 0,64 0,63 0,65 0,64	0,63	3,4
	Résistance moyenne plates : 59,40 sur bord : 31,92	1 2 3 4 5 6	24,58 20,80 26,85 23,50 20,80 21,73	15,61 16,10 19,20 15,61 16,10 15,58	0,60 0,66 0,63 0,53 0,60 0,60	0,60	7,2
	Résistance élevée plates : 88,33 sur bord : 26,4	1 2 3 4 5 6	21,59 21,59 21,17 21,17 19,04 19,04	15,0 15,0 12,64 12,64 17,68 17,68	0,59 0,64 0,56 0,59 0,62 0,64	0,61	5,3
Dalles % d'acier à forte élasticité : 0,9	Faible résistance plates : 21,55 sur bord : 16,10	1 2 3 4	20,46 20,46 25,0 25,0	14,18 14,18 18,02 18,02	0,52 0,63 0,69 0,58	0,61	—
	Résistance moyenne plates : 59,40 sur bord : 31,92	1 2 3 4 5 6	21,66 23,51 23,51 21,66 21,66 23,51	15,58 11,93 11,93 15,58 15,8 11,93	0,62 0,74 0,62 0,78 0,76 0,76	0,71	10,28
	Résistance élevée plates : 88,33 sur bord : 26,40	1 2 3 4 5 6	21,53 21,53 21,53 26,85 24,0 24,0	16,11 16,11 16,11 19,18 15,61 15,61	0,59 0,72 0,69 0,65 0,72 0,87	0,71	13,3

Table 4 *Effect of mortar grades on the shear strength of reinforced grouted beams and slabs*

$$\frac{\text{Shear span}}{\text{Effective depth}} \quad (a/d) = 6$$

Type of specimen	Type and strength of brick N/mm²	No of specimens	Shear strength N/mm² Mortar grade 1 : 1/4 :3		1 : 1/2 : 4 1/2	
Beams	Low strength Flat : 21.55 Edge : 16.10	1 2 3 4 5 6	0.59 0.64 0.64 0.63 0.65 0.64	0.63	0.63 0.65 0.59	0.62
	Medium strength Flat : 59.40 Edge : 31.92	1 2 3 4 5 6	0.60 0.66 0.63 0.53 0.60 0.60	0.60	0.52 0.59 0.61	0.57
	High strength Flat : 88.33 Edge : 26.4	1 2 3 4 5 6	0.59 0.64 0.56 0.59 0.62 0.64	0.61	0.53 0.53 0.54 0.53 0.53	0.53
	Low strength Flat : 21.55 Edge : 16.10	1 2 3 4	0.52 0.63 0.69 0.58	0.61	0.60 0.72 0.73	0.68
Slabs	Medium strength Flat : 59.40 Edge : 31.92	1 2 3 4 5 6	0.62 0.74 0.62 0.78 0.76 0.76	0.71	0.60 0.64 0.78	0.67
	High strength Flat : 88.33 Edge : 26.40	1 2 3 4 5 6	0.59 0.72 0.69 0.65 0.72 0.87	0.71	0.71 0.55 0.66 0.62 0.58 0.67	0.63 Coeff. of variation = 9,4 %

Note : % of high tensile steel for
Beam = 0.88
Slab = 0.9

Table 5 *Effect of % of steel on the shear strength of reinforced grouted brickwork beams and walls*

$$\frac{\text{Shear span}}{\text{Effective depth}} \quad (a/d) = 6$$

No of specimen	Type and strength of brick (N/mm²)	Shear strength N/mm² Beams Steel % 0.88		Beams Steel % 1.38		Beams Steel % 1.68		Walls Steel % 0.9		Walls Steel % 2.54	
1 2 3 4 5 6	Low strength Flat : 21.55 On edge : 16.10	0.59 0.64 0.64 0.63 0.65 0.64	0.63	0.82 0.64 0.74	0.73	0.69 0.90 0.86	0.82	0.52 0.63 0.69 0.58	0.61	0.55* 0.96 0.77	0.87
1 2 3 4 5 6	High strength Flat : 88.33 On edge : 26.40	0.59 0.64 0.56 0.59 0.62 0.64	0.61	0.71 0.67 0.65	0.68	0.77 0.76 0.67	0.73	0.59 0.72 0.69 0.65 0.72 0.87	0.71	0.82 0.87 0.76	0.82

* Very inexperienced bricklayer from outside (result ignored).

Tableau 4 *Influence du dosage de mortier sur la résistance au cisaillement des poutres et dalles armées et liées*

$$\frac{\text{portée en cisaillement}}{\text{hauteur utile}} = (a/d) = 6$$

Type d'éprouvette	Type et résistance des briques (N/mm²)	N° des éprouvettes	Résistance au cisaillement (N/mm²) selon qualité du mortier 1 : 1/4 : 3		1 : 1/2 : 4 : 1/2	
Poutres	Faible résistance plates : 21,55 au bord : 16,10	1 2 3 4 5 6	0,59 0,64 0,64 0,63 0,65 0,64	0,63	0,63 0,65 0,59	0,62
	Résistance moyenne plates : 59,40 au bord : 31,92	1 2 3 4 5 6	0,60 0,66 0,63 0,53 0,60 0,60	0,60	0,52 0,59 0,61	0,57
	Résistance élevée plates : 88,33 au bord : 26,4	1 2 3 4 5 6	0,59 0,64 0,56 0,59 0,62 0,64	0,61	0,53 0,53 0,54 0,53 0,53	0,53
	Faible résistance plates : 21,55 au bord : 16,10	1 2 3 4	0,52 0,63 0,69 0,58	0,61	0,60 0,72 0,73	0,68
Dalles	Résistance moyenne plates : 59,40 au bord : 31,92	1 2 3 4 5 6	0,62 0,74 0,62 0,78 0,76 0,76	0,71	0,60 0,64 0,78	0,67
	Résistance élevée plates : 88,33 au bord : 26,40	1 2 3 4 5 6	0,59 0,72 0,69 0,65 0,72 0,87	0,71	0,71 0,55 0,66 0,62 0,58 0,67	0,63 coefficient de variation = 9,4 %

Nota : % d'acier à forte tension pour
— les poutres : 0,88
— les dalles : 0,9

Tableau 5 *Influence du % d'acier sur la résistance au cisaillement des poutres et dalles ou maçonnerie armée et liée*

$$\frac{\text{portée en cisaillement}}{\text{hauteur utile}} = (a/d) = 6$$

N° de l'éprouvette	Type et résistance des briques (N/mm²)	Résistance au cisaillement (N/mm²) — Poutres — % acier 0,88		% acier 1,38		% acier 1,68		Murs — % acier 0,9		% acier 2,54	
1 2 3 4 5 6	Faible résistance plates : 21,55 au bord : 16,10	0,59 0,64 0,64 0,63 0,65 0,64	0,63	0,82 0,64 0,74	0,73	0,69 0,90 0,86	0,82	0,52 0,63 0,69 0,58	0,61	0,55* 0,96 0,77	0,87
1 2 3 4 5 6	Résistance élevée plates : 88,33 au bord : 26,40	0,59 0,64 0,56 0,59 0,62 0,64	0,61	0,71 0,67 0,65	0,68	0,77 0,76 0,67	0,73	0,59 0,72 0,69 0,65 0,72 0,87	0,71	0,82 0,87 0,76	0,82

* Maçon très inexpérimenté, venu de l'extérieur (résultat ignoré).

large number of tests using different strength of bricks, different grades of mortar were available. Hence this a/d ratio was used as a datum for obtaining the characteristic shear strength. The distribution of the shear strength test data was found to be normal, hence the characteristic strength was obtained from the equation above and plotted in fig. 9.

Table 5 and fig. 11 show the range and average shear stress obtained in the test for different percentages of steel. The average shear stress of reinforced grouted brickwork beams for different percentages of tensile steel is higher than the characteristic strength of 20 N/mm^2 concrete (ref. 7). However, the limit state code must be based on characteristic strength. The characteristic strength thus obtained is shown in table 6, which formed the basis of BS 5628 draft code, Part 2. This proposal ignores the effect of brick strength, which has statistically no significant effect on shear strength.

Table 6
Characteristic shear strength of grouted cavity reinforced brickwork for various % of tensile steel

% of tensile steel	0.5	0.75	1.0	1.5	2.0	2.5
Characteristic shear stress f_{kv} . .	0.48	0.50	0.53	0.60	0.65	0.70

Ultimate bending moment

The theoretical and experimental failure moment of beams and slabs are compared in table 7. From more than a hundred tests, only the beams reinforced against shear failed due to initial yielding of steel in the maximum bending moment zone leading to the compressive failure of the brickwork. The theoretical (ref. 9) moment has been calculated by mathematically idealising the experimental stress-strain curves for brickwork and by using the crushing strength obtained from a prism of h/t ratio equal to 4.5 approximately.

Two types of prism specimens used to obtain the crushing strength, are shown in table 7. The compressive forces developed in the direction normal to the bed joint in the case of slabs and parallel to the bed joint (or normal to the collar joint) in the case of beams; the choice of these prisms reflected this state of stress. However, in the beam specimens the forces were not only resisted by the brickwork but also by the grout. The modulus of elasticity obtained from the test of the grout cylinders was very similar to the modulus of elasticity of brickwork, hence the calculation based on a transformed section using the compatibility of strain at time of failure did not significantly alter the calculated collapse moment.

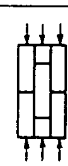
With the slab specimen the compressive zone was located within the brickwork and the grout was only in the tensile zone. Therefore grout did not contribute towards the ultimate strength of the slab. On the basis of the results a simplified formula for the calculation of the ultimate moment can be expressed by

$$M_{um} = 0.28\ f_m bd^2 \qquad (\textit{Equation 1})$$

$$\textit{or}\ M_{ut} = A_s.f_y.d\ \left(1 - \frac{0.553\ A_s\ f_y}{f_m bd}\right) \qquad (\textit{Equation 2})$$

A detailed account of the derivation of the formula is given elsewhere (ref. 9).

Table 7 *Comparison of experimental and theoretical ultimate moment for reinforced brickwork*

Brick strength (N/mm^2)	% steel	Test Nos. *	Average compressive strength (N/mm^2)	Specimen type	Ultimate moment in kNm			
					Experimental		Theoretical Cubic Parabolic	
Low : 21.55	0.9	1 2 3 4 5 6	11.17		26.78 28.90 30.80 26.30 25.53 31.08	28.24	27.89 27.50 27.11 29.01 28.49 29.01	28.25
Low : 21.55	0.88	7 8 9 10 11 12	12.92		183.0 176.0 186.6 194.7 192.7 192.7	185.7	172.5 170.0 170.0 172.5 172.5 172.5	171.67
Medium : 59.40	0.88	13 14 15	18.58	,,	196.0 197.2 204.0	199.0	171.0 173.10 175.78	173.3
High : 88.33	0.88	16 17	22.04	,,	200.0 193.5	196.75	176.9 176.9	176.9

* Note : 1 to 6 slab specimens
7 to 17 beam specimens.

.a résistance caractéristique est définie par

$$f_{kv} = f_{mv} - z.s. \qquad (11)$$

où
$_{kv}$ = résistance au cisaillement caractéristique
$_{mv}$ = résistance au cisaillement moyenne
: = coefficient = 1,64 (probabilité de 90 %)

Pour un rapport portée en cisaillement/hauteur utile égal à 6, la résistance au cisaillement n'est plus fig. 9) affectée ni par le rapport a/d, ni par la ninceur du profilé. De plus, pour ce rapport, il a été possible de disposer d'un échantillonnage consistant en un grand nombre d'essais avec des briques de ésistance différente et des mortiers de dosages lifférents. Ce rapport a/d a donc servi de données pour obtenir la résistance au cisaillement caractéristique. On a pu voir que la distribution des données l'essais de résistance au cisaillement était normale, a résistance caractéristique a donc été obtenue à partir de l'équation ci-dessus et la courbe tracée en igure 9.

Le tableau 5 et la figure 11 montrent la fourchette le contraintes de cisaillement et la contrainte noyenne obtenues dans l'essai pour différents pourcentages d'acier. La contrainte de cisaillement noyenne des poutres en maçonnerie armée et liée pour différents pourcentages d'acier tendu est upérieure à la résistance caractéristique du béton de 20 N/mm² (réf. 7). Cependant, le code relatif aux états limites doit se fonder sur la résistance caractéristique. Celle-ci, ainsi obtenue, est donnée au tableau 6 qui a servi de base au projet de code BS 5628, 2ᵉ partie. Cette proposition ne tient aucun compte de la résistance de la brique qui n'a statistiquement aucune influence significative sur la résistance au cisaillement.

Tableau 6
Résistance au cisaillement caractéristique des murs creux en briques liaisonnés au coulis pour divers % d'aciers tendus

% d'aciers tendus	0,5	0,75	1,0	1,5	2,0	2,5
Effort de cisaillement caractéristique f_{kv}	0,48	0,50	0,53	0,60	0,65	0,70

Moment de flexion maximal

Le tableau 7 donne une comparaison du moment de rupture théorique et expérimental des poutres et des dalles. D'après plus de 100 essais, seules les poutres armées contre le cisaillement ont connu une rupture due à l'élasticité initiale de l'acier dans la zone du moment de flexion maximal, entraînant la rupture par compression de la maçonnerie. Le moment théorique (réf. 9) a été calculé selon une idéalisation mathématique des courbes effort-déformation expérimentales pour la maçonnerie et en utilisant la résistance à l'écrasement obtenue à partir d'un prisme de rapport h/t égal approximativement à 4,5.

Le tableau 7 porte sur deux types d'éprouvettes prismatiques, utilisés pour obtenir la résistance à

Tableau 7 *Comparaison entre le moment maximal théorique et expérimental pour la maçonnerie de brique armée*

Résistance des briques (N/mm²)	% d'acier	Essai n° *	Résistance à la compression moyenne (N/mm²)	Type d'éprouvette	Moment maximal kNm			
					Expérimental		Théorique cubique parabolique	
Faible : 21,55	0,9	1 2 3 4 5 6	11,17		26,78 28,90 30,80 26,30 25,53 31,08	28,24	27,89 27,50 27,11 29,01 28,49 29,01	28,25
Faible : 21,55	0,88	7 8 9 10 11 12	12,92		183,0 176,0 186,6 194,7 192,7 192,7	185,7	172,5 170,0 170,0 172,5 172,5 172,5	171,67
Moyenne : 59,40	0,88	13 14 15	18,58	dito	196,0 197,2 204,0	199,0	171,0 173,10 175,78	173,3
Elevée : 88,33	0,88	16 17	22,04	dito	200,0 193,5	196,75	176,9 176,9	176,9

Nota : 1 à 6 : éprouvettes de dalles
7 à 17 : éprouvettes de poutres.

The theoretical results thus obtained from the above compare very favourably with the experimental results for beams and slabs built from low, medium and high strength bricks.

Summary and conclusions

On the basis of these tests the following conclusions can be drawn:

(i) The ultimate shear strength of reinforced grouted beams and slabs increases with:

(a) decreasing shear span/effective depth ratios

(b) an increase in the percentage of tensile reinforcement

(c) thinner sections.

(ii) The compressive strengths of bricks (low, medium and high) or brickwork and grades of mortar do not significantly affect the shear strength. The effect of brick and brickwork strength may be significant if the failure is due to flexure and not due to shear.

(iii) The shear reinforcement increases the shear strength of beams built with medium and high strength bricks but not of low strength brickwork beams. The shear failure is brittle while flexure failure is slow and ductile. Hence shear reinforcement should be provided in grouted reinforced brickwork beams.

(iv) The ultimate moment of the beam or slab built with low, medium or high strength bricks can be reliably predicted by the formulae proposed in this paper.

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l'écrasement. Les forces de compression se sont développées perpendiculairement au joint du lit dans le cas des dalles, et parallèlement au joint du lit (ou perpendiculairement au joint du collet) dans le cas des poutres; le choix des prismes a refleté cet état d'effort. Cependant dans les éprouvettes de poutres, ce n'était pas seulement la maçonnerie qui résistait à ces forces, mais également le coulis. Le module d'élasticité obtenu à partir de l'essai des cylindres de coulis a été très similaire à celui de la maçonnerie ; le calcul basé sur un profilé transformé utilisant la compatibilité de la déformation au moment de la rupture n'a donc pas modifié de façon significative le moment d'effondrement calculé.

Avec l'éprouvette de dalle, la zone de compression s'est située à l'intérieur de la maçonnerie elle-même et le coulis s'est trouvé dans la zone tendue seulement. Le coulis n'a donc pas contribué à la résistance maximale de la dalle. C'est sur la base de ces résultats que l'on peut utiliser pour le calcul du moment maximal la formule simplifiée suivante:

$$M_{um} = 0{,}28\ f_m b d^2 \qquad (1)$$

$$\text{ou } M_{ut} = A_s . f_y . d \left(1 - \frac{0{,}553\ A_s\ f_y}{f_m b d}\right) \qquad (2)$$

Il est rendu compte ailleurs (réf. 9), de façon détaillée, de la dérivation de la formule.

Les résultats théoriques ainsi obtenus concordent parfaitement avec les résultats expérimentaux relatifs aux poutres et aux dalles construites avec des briques de résistance faible, moyenne ou forte.

Résumé et conclusions

De ces essais, on peut tirer les conclusions suivantes:

1. La résistance au cisaillement maximale des poutres et des dalles armées et liaisonnées au coulis augmente avec:

a) des rapports portée en cisaillement/hauteur utile décroissants ;

b) une augmentation du pourcentage des armatures tendues ;

c) des éléments plus minces.

2. Les résistances à la compression des briques (faible, moyenne et forte) ou de la maçonnerie ou le dosage du mortier n'affectent pas de façon significative la résistance au cisaillement. L'influence de la résistance des briques et de la maçonnerie peut être significative, si la ruine est due à la flexion et non au cisaillement.

3. Les armatures contre le cisaillement augmentent la résistance au cisaillement des poutres construites avec des briques de résistance moyenne et forte mais non de celles construites avec des briques de faible résistance. La rupture de cisaillement est fragile tandis que la rupture par flexion est lente et ductile. Il faut donc assurer des armatures au cisaillement dans les poutres en maçonnerie armée et liaisonnée au coulis.

4. Il est possible de prévoir avec beaucoup de précision le moment maximal d'une poutre ou d'une dalle construite avec des briques de résistance faible, moyenne ou élevée grâce aux formules proposées dans cet article.

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IV–7. Behaviour of Reinforced Grouted Cavity Beams

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ABSTRACT

The paper describes the results of an investigation carried out on reinforced grouted cavity beams of different shear arm: effective depth ratios. The brick strength and the % of steel were kept constant throughout the testing. The calculated allowable moments based on the British Standards CP 111 and CP 110 (Limit state) are compared with the test results in this paper and it appears that the design based on CP 111 is rather conservative. Design based on ultimate load philosophy appears more realistic.

The paper also describes an approximate method which predicts favourably the ultimate shear strength of the test beams.

INTRODUCTION

The behaviour of reinforced concrete suggests that the ultimate shear strength of reinforced brickwork or reinforced grouted cavity walls could be affected by the shear span/effective depth ratio, percentage of tensile reinforcement, the shear reinforcement and, to some extent, the brick strength. For "ordinary" reinforced brickwork (i.e. with thin reinforcement in bed joints only), the ultimate shear increases significantly with decreasing shear arm/effective depth ratios[1,2,3]. However, there is a wide scatter of experimental results and Suter et al.[4] have recently suggested characteristic shear stress of 0.3 N/mm². The suggestion is based on test results of beams very different from thin reinforced retaining walls. Further, this is very much less than the result obtained on reinforced brick grouted beams carried out at BRE[5] for Structural Clay Products Ltd. In the light of these results it became necessary to examine the effect of all factors mentioned above which could influence the ultimate shear strength and thus the load carrying capacity of thin reinforced brick grouted beams as used in retaining walls.

The investigation described in this paper is mainly concerned with the effect of shear span/effective depth ratio on the behaviour and strength of reinforced brickwork beams with a constant % of steel. Only one type of brick was used. The effects studied were deflection, cracking and ultimate strength.*

MATERIALS

Bricks

Perforated Downing bricks with average crushing strength of 71.32 N/mm² were used for all tests. The coefficient of variation was 7.8%. The average water absorption was 4.2%. The suction rate was 0.2 Kg/m²/min.

Cement and Lime

Ordinary Portland Cement to BS 12 "Portland" Cement (Ordinary and rapid hardening) was used for the construction of the test specimen. The lime used in the mortar conformed to BS 890 "Building Limes".

Aggregate

Fine Aggregate

The sand available in Scotland could not meet the grading requirement of BS 1200 for reinforced brickwork. It was therefore necessary to obtain special sand from Leighton Buzzard conforming to the Standard. Local sand conforming to the BS grading zone 2 was used for grout.

Mortar

1:¼:3 (cement:lime:sand) mix was used for mortar. A mortar mix with water/cement ratio of 0.6 was found workable and kept constant for all tests. For each specimen three 100mm cubes were cast, cured in water, and tested at 28 days. The average strength was 28.38 N/mm².

Grout

The constituents of the grout were mixed by weight to give 1:0.1:3:2 (cement:lime:sand:pea gravel) mix by volume. The water/cement ratio was 1.2 and the slump 275 mm. Three 100 mm cubes/wall were tested at 28 days. The average strength was 20.34 N/mm².

Reinforcement

Hot rolled high yield deformed bars were used for the reinforcement. Two steel specimens were tested under tension and the resulting strain was measured by electrical strain gauges mounted on the specimen. The data were analysed and a least square linear regression method was applied to obtain the value of initial modulus of elasticity. The correlation coefficient in both cases was 0.998. The average modulus of elasticity was 217.56 kN/mm² (213.78 kN/m² and 221.34 kN/mm²), and the average ultimate strength was 525 N/mm² (500 and 550 N/mm²). The 0.2% proof stress[11] was 476 N/mm².

DETAILS OF BRICKWORK TEST SPECIMENS AND ARRANGEMENTS FOR TESTING

Test Specimens

The test specimens (Fig. 1) were 2445 × 660 × 275mm (l × b × d). The brickwork was tied by strip ties to BS

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* Since then (1973) a more comprehensive programme of research, investigating all factors, is being carried out at the University of Edinburgh under the sponsorship of the Building Research Establishment, U.K.

1243 spaced 400 × 300 mm and staggered. The reinforcement was seven 12 mm bars giving an area of 798 mm^2, or 0.88% of the effective area. The beams were tested to failure using symmetrical two point loading with loads 1000 mm apart.

The permissible bond stress for 20 N/mm^2 grade concrete is 1.7 + 30% = 2.2 N/mm^2. (CP 110: Part 1: 1972[6] Clause 3.11.6.2). Hence the anchorage length needed for 12 mm bars is

$$\frac{\pi r^2.f_y}{2\pi r.f_{bs}} = \frac{6 \times 410}{2 \times 2.2} = 0.56 \text{ m} = 560 \text{ mm}$$

and the minimum specimen length is thus 1000 + (2 × 560) = 2120 mm. The distance between supports changed with changes in a/d ratios (where d = 138 mm) and when this was 5 this distance becomes 1000 + (2 × 690) = 2380 mm. All specimens were made this length—actually 2445 mm—irrespective of a/d ratio and minimum bond achorage.

Compressive Strength

Ten prisms each six courses high (Fig. 2) were tested to obtain the ultimate strength. The average compressive strength[11] was 33.9 N/mm^2 with a coefficient of variation of 17.8%.

Modulus of Elasticity

The strain was measured by demec gauge on four prisms to obtain the modulus of elasticity. The modulus of elasticity varied from 18.3 to 22 kN/mm^2 with an average of 20.5 kN/mm^2.

Modulus of Rupture (Flexural Tensile) Strength of Brickwork

Three 8-courses high two brick wide wallettes were built and tested as a beam subjected to a central point load to obtain the flexural strength at 28 days (Fig. 3). This was necessary to determine the moment at which 1st crack appears in the beams.

The test results are shown in Table 1.

Bond Shear Strength

Tests were carried out as shown in Fig. 4 to obtain the bond shear strength of the grout and brickwork interface. The test results are shown in Table 2.

Instrumentation and Testing

Line loading (Fig. 1) was applied by 4 hydraulic jacks fixed to a loading frame, the load from each jack being measured by the load cells connected to a pen-chart recorder. The lateral displacement of beams was measured by means of dial gauges reading to 0.002 mm. The load was applied incrementally and the deflection was recorded for each beam. The loading was continued till beam failure occurred. Wherever possible the loads at which first and subsequent cracks appeared were noted.

TEST RESULTS AND DISCUSSION

Mode of Failure

In all the tests the initial visible cracks started at the interface of brick and mortar in the tension zone. From the deflection results it is apparent that the actual cracking took place much earlier than it became visible since there was marked changes in the load-deflection relationship. Typical crack propagation (Fig. 5) shows that the sudden failure at the interface of grout and top brickwork was due to shear compression failure and shear failure. With a lower shear span/depth ratio the failure, in some instances, may have coincided with the yielding of the steel. The test results are shown in Table 3.

Deflection

The load-deflection relationship for various beams is shown in Figs. 6 to 9. The load-deflection relationship is bi-linear and it appears that there is sudden change in deflection when the resulting moment exceeds the brickwork cracking moment. The load at which this deviation in deflection takes place can very well be predicted as shown in Figs. 6 to 9 by using the flexural strength of brickwork (Sec. 3.1.3) and uncracked section (full cross-section) of the beam. The deflection of different beams under similar loading conditions is not the same, which may be due to differences in the modulus of elasticity, workmanship and presence of hair-cracks at the brick/mortar interface affecting the stiffness of the beams.

Moment of Resistance

Uncracked Moment of Resistance

The moment of resistance before the crack became visible was calculated by using the full uncracked section and the flexural strength of brickwork from section 3.1.3. The moment of resistance (6.5 kNm) was very much less than the ultimate moment of resistance (48.8 kNm). However, even before the appearance of the first crack at the interface of brick and mortar, moment was higher than the permissible moment according to CP 111:1970. The factor of safety will be in the range of 1 to 2.6, if the design is based on completely uncracked section. (Table 3b, ii).

Ultimate Moment of Resistance

The ultimate moment of resistance (48.8 kNm) of the test beams has been calculated by assuming a parabolic stress block[11] and taking into account the actual yield stress of steel. This compares favourably with 48.1 kNm, the average test results of beams 2, 3, 4 and 6 (Table 3) where failure was due to shear and flexural tension. Similar results[11] can be obtained by Whitney theory using a rectangular stress block.

With higher shear span/depth ratios, 4 & 5, the failure moment was 78 to 52% less, due to premature shear failure, than the theoretical moment of resistance. It may be possible to increase the moment capacity of these beams by using shear connectors so that failure at the interface of brick and grout can be delayed.

Comparison of Results with CP 111[7] and CP 110 (Limit State)

The calculated allowable moments based on CP 111 and CP 110 are compared with the test results (Table 3). The factor of safety varies from 4 to 18 for beams with a/d ratio

5 to 2 when the shear stress (and hence the load and moment) is limited by the appropriate clauses of CP 111. When these clauses are ignored the factor of safety reduces and becomes 2.7 to 4.7 with the steel stressed to a very low value (97.5 N/mm²). Even then shear stress exceeds the allowable in CP 111 (Clause 321). The global safety factor varies from 1.1 to 1.92 if the design calculation for resistance moment is based on CP 110.

Ultimate Shear Stress

The ultimate shear stress increases with decreasing shear span/depth ratio (Table 4). As in most cases, the failure of the reinforced grouted beams was due to shear it is important to identify the parameters which affected the test results.

Shear Failure Theory

To calculate the ultimate shear stress, the brickwork beam can be treated as a tied arch, and the forces acting are shown in Fig. 11.

Hence,

$$H = \frac{M_{max}}{z} = \frac{Wa}{z} = \frac{Wa}{jd}; \quad M_{max} = \text{moment} \qquad \text{(i)}$$

The failure of arch[9] will take place if, (a) the compressive stress exceeds the compressive strength of brickwork; (b) the tie stress exceeds the tensile strength of steel; (c) the tie force H is greater than or equal to the ultimate shear strength of the brickwork and thus destroys the bond at the interface of brick and grout. In the present test the final failure is due to destruction of bond at the interface of brickwork and grout. Hence

$$H = V_{ult} \qquad \text{(ii)}$$

where V_{ult} is the ultimate shear force.

Between the support and loading point, i.e. within the shear arm, the brickwork and grout interface is subjected to precompression. Under such condition the ultimate shear strength[10] of brickwork can be represented by:

$$v_{ult} = t_b + \mu\sigma \qquad \text{(iii)}$$

where σ – compressive stress N/mm²; μ – coefficient of friction; t_b – initial bond shear N/mm²; v_{ult} – shear stress N/mm².

From equation (iii),

$$V_{ult} = t_b(a + l)b + \mu W \qquad \text{(iv)}$$

From equation (ii) and (iv),

$$\frac{Wa}{jd} = t_b(a + l)b + \mu W \qquad \text{(v)}$$

or,

$$\frac{Wa}{jd} - \mu W = t_b(a + l)\cdot b$$

or,

$$W\frac{a - \mu jd}{jd} = t_b(a + l)\cdot b$$

or,

$$\frac{W}{bd} = \frac{(a + l)t_b \cdot j}{a - \mu jd}$$

or,

$$v = j\cdot t_b \cdot \left[\frac{\frac{a}{d} + \frac{l}{d}}{\frac{a}{d} - \mu j}\right]$$

provided

$$\frac{a}{d} \geq 0.65 \qquad \text{(vi)}$$

where:

$j = \frac{z}{d} = \frac{\text{lever arm}}{\text{effective depth}}$ = 0.93 from parabolic stress block—Appendix I

μ = 0.7 (coefficient of friction between brickwork and grout or mortar)

t_b = 0.49 N/mm² (Initial bond shear: Section 3.14)

The theoretical ultimate shear stress was calculated from equation (vi) and compared with stress calculated from $\frac{W}{bd}$ and shown in Fig. 10.

From Fig. 10 it can be seen that there is very good agreement between the test results and theory. However, this needs to be conformed for $\frac{a}{d}$ values higher than 5 and also for other brick types.†

The shear stresses for brickwork reinforced in the bed joints were also calculated by this method, and the figures agreed closely with the results obtained by Suter and Hendry[4]. Since the initial bond strength for their case was not known, the initial bond strength of 0.354 N/mm² obtained in previous tests of Sinha and Hendry[8] on the same brick in 1:1:6 mortar was used.

Considering all these facts it would be reasonable to recognise the significant increase in nominal shear stress with decreasing $\frac{a}{d}$ values. Provision could be made in the Code to take this into account. For the present, the ultimate load design proposed in this report is an alternative for reinforced grouted cavity construction and the design moment and design shear stress can be obtained by dividing by a suitable factor of safety.

CONCLUSIONS

The ultimate shear stress increases significantly with decreasing shear span/depth ratio. Provision could be made in the Code to take this fact into account.

Design based on CP 111:1970:Part 2 is conservative for the type of brick and mortar used in tests because of low allowable flexural compressive stress and constant allowable shear stress. Design based on ultimate load philosophy as suggested in this report appears more reasonable.

† Recent work done by first author confirms this for other types of brick.

The theoretical shear stress calculated by the method suggested compares favourably with the present test results and also with results on "ordinary" reinforced brickwork.

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TABLE 1—Flexural Strength of Brickwork

No.	Flexural Strength N/mm²	Average Flexural Strength N/mm²
1	0.74	
2	0.70	0.78
3	0.90	

TABLE 2—Bond Shear Strength

Corresponding Wall No.	No.	Shear Stress N/mm²	Average Shear Stress N/mm²
12	1	0.473	
	2	0.287	0.49
6	1	0.630	
	2	0.550	

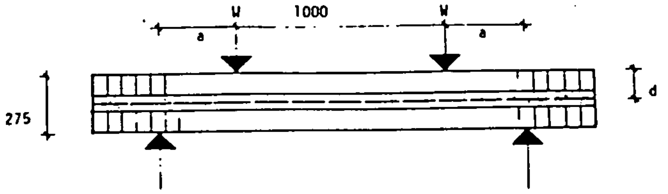

Figure 1. Test specimen and loading arrangement

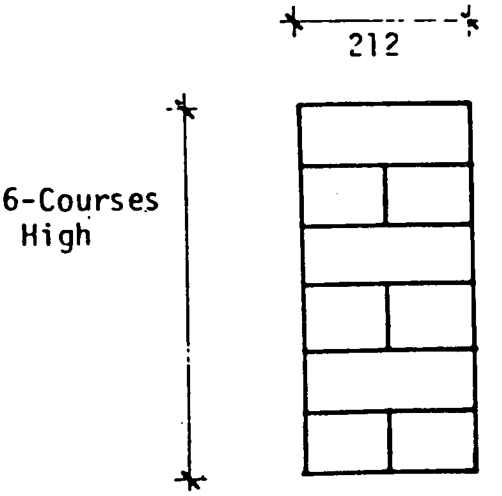

Figure 2. Test prism (dimensions in mm)

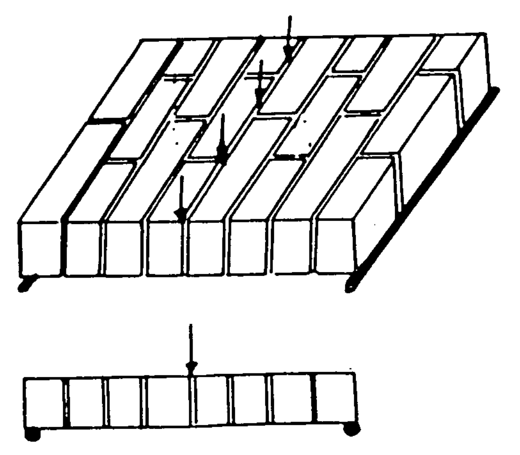

Figure 3. Test arrangement for determination of modulus of rupture.

TABLE 3—Comparisons on Test Results with Calculated CP.111 Performance (Elastic Design) & CP.110 Limit State

	a/d	2	3	4	5
Bending	(a) B.M. at Failure kNm	(1) 36.4 (2) 49.14 Av. 44.5 (3) 47.94	(4) 47.3 (5) 34.0 Av. 43.0 (6) 48.12	(7) 41.0 (8) 36.1 Av. 38.0 (9) 37.0	(10) 35.0 (11) 23.7 Av. 25.6 (12) 18.0
	(b) Permissible* B.M. from CP.111 (kNm)	(i) 9.4 (ii) 2.5	9.4 4.13	9.4 5.0	9.4 6.23
	(c) Ratio (a)/(b)	(i) 4.7 (ii) 17.8	4.6 10.4	4.0 7.6	2.7 4.1
Nominal shear stress $v = W/bd$	(d) Average shear stress at failure	1.79	1.18	0.82	0.46
	(e) Actual shear stress from b(i) Permissible shear stress b(ii)	0.374 0.1	0.25 0.1	0.187 0.1	0.15 0.1
	(f) Ratio (d)/(e)	4.8 17.9	4.7 11.8	4.4 8.2	3.1 4.6
Bending	(g) Design moment $\gamma_f = 1.4$ (CP.110)	23.2	23.2	23.2	23.2
	(h) Global safety factor, ratio (a)/(g)	1.92	1.85	1.6	1.1

* NOTE: (i) ignoring the shear stress of CP.110, Clause 3.4.5.
(ii) based on allowable shear stress of CP.110, Clause 3.4.5.

TABLE 4—Ultimate Shear Test Results (Age at test 28 days)

Beam No.	Mortar Strength N/mm²	Grout Strength N/mm²	a/d	Failure Mode	Shear stress v_1 N/mm² (from test load)	Average shear stress v_1 N/mm²	Shear stress due to dead wt v_2 N/mm²	Average ultimate shear stress $v = v_1 + v_2$ N/mm²
1	23.0	15.2	2	Ssu	1.45			
2	29.7	27.0	2	Ssu/FL	1.94	1.76	0.03	1.79
3	26.7	19.8	2	Ssu/FL	1.90			
4	29.7	23.0	3	Ssu/FL	1.23			
5	31.0	18.0	3	Ssu	0.91	1.14	0.035	1.175
6	31.8	16.6	3	Ssu/FL	1.28			
7	27.9	22.4	4	Ssu	0.86			
8	27.3	23.5	4	Ssu	0.73	0.78	0.041	0.821
9	28.4	28.2	4	Ssu	0.74			
10	32.9	22.3	5	Ssu	0.38			
11	28.9	19.2	5	Ssu	0.56	0.41	0.045	0.46
12	29.2	14.0	5	Ssu	0.29			

Notes: Ssu — Sudden shear failure
FL — Flexural failure

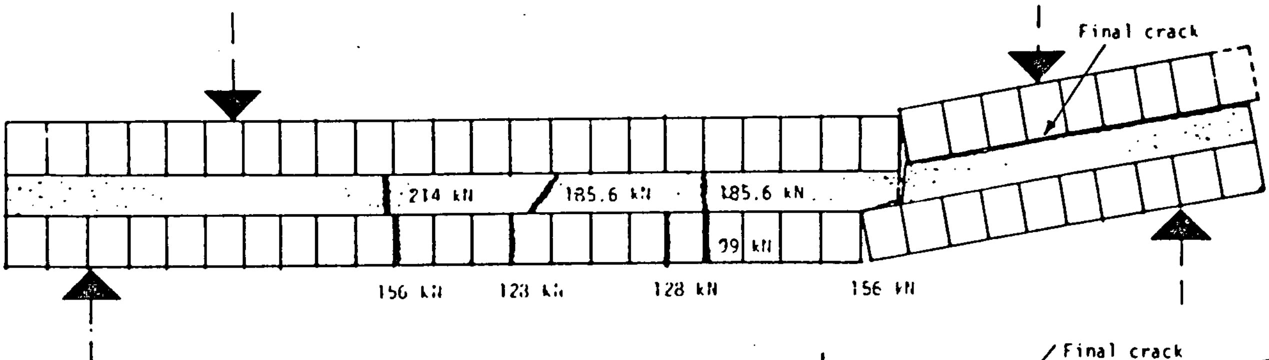

Figure 5a. Typical failure

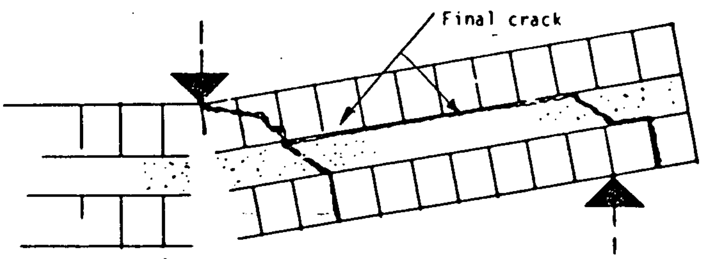

Figure 5b. Alternative failure pattern (e.g. wall 10: a/d = 5)

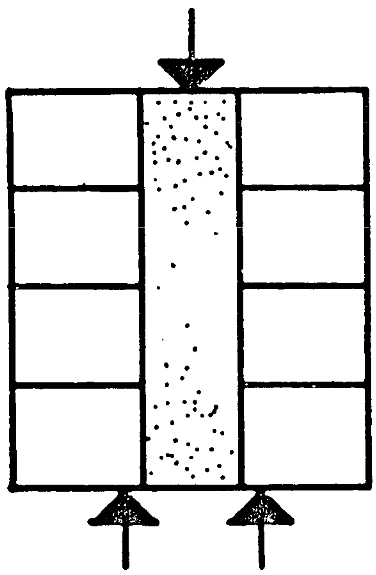

Figure 4. Bond shear test arrangement.

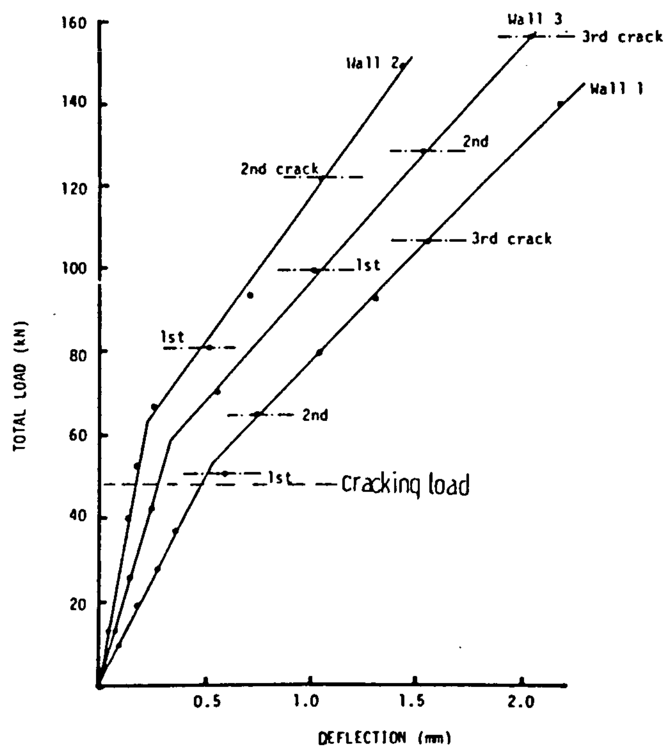

Figure 6. Load/deflection graphs for walls with a/d = 2

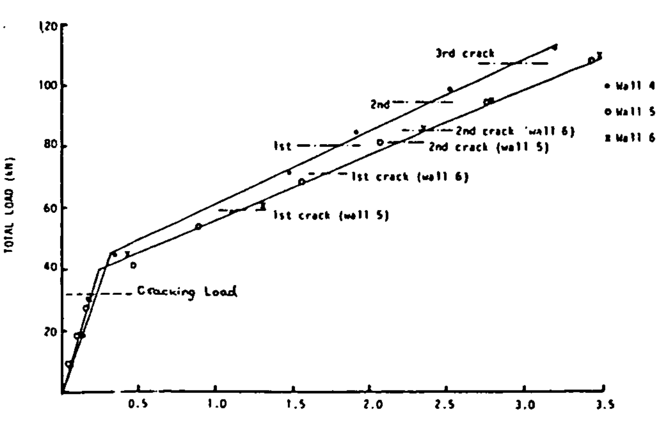

Figure 7. Load/deflection graphs for walls with a/d = 3

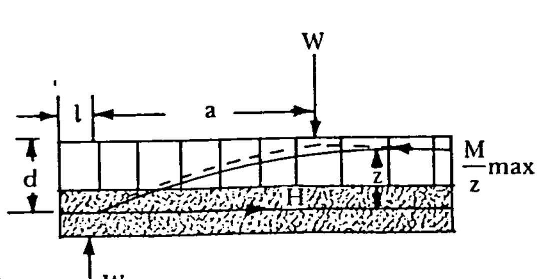

Figure 11.

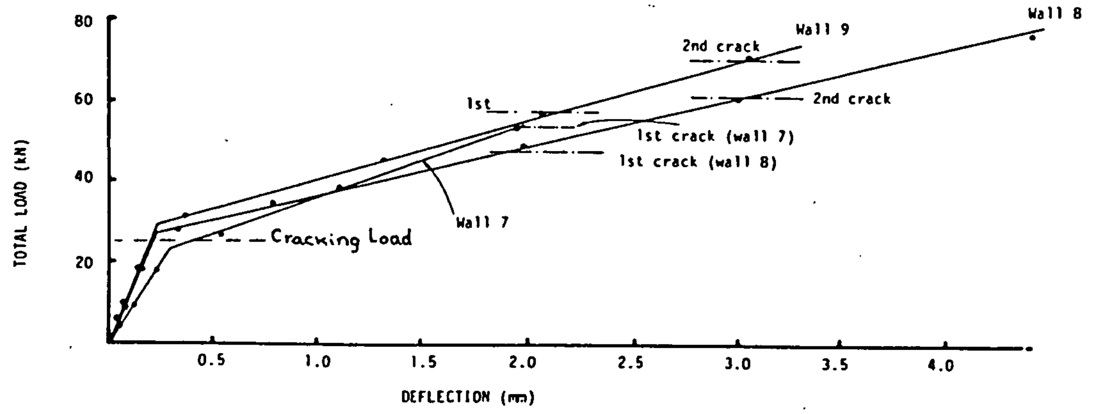

Figure 8. Load/deflection graphs for walls with a/d = 4

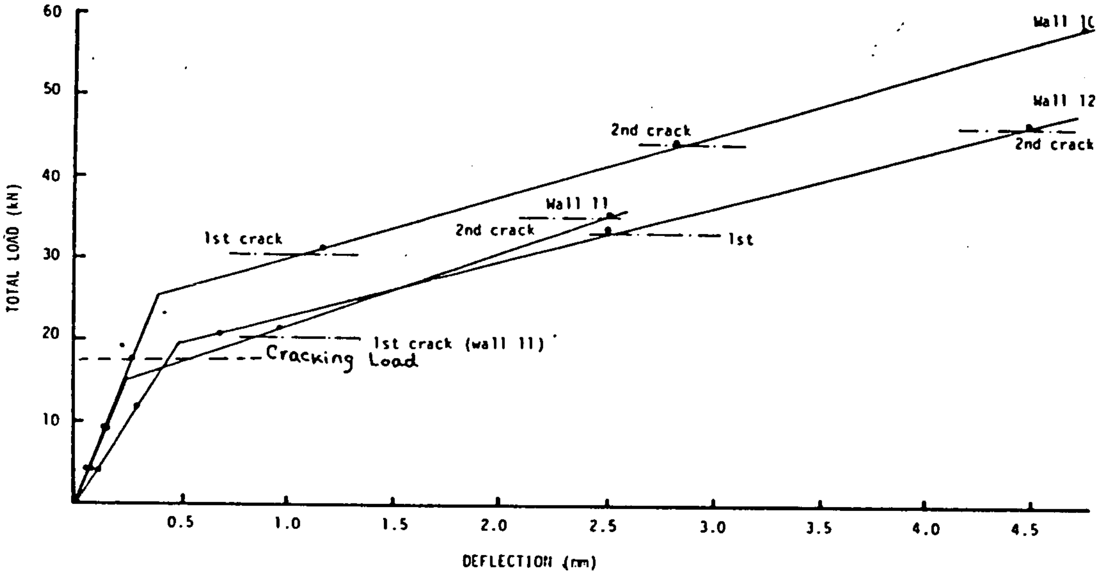

Figure 9. Load/deflection graphs for walls with a/d = 5

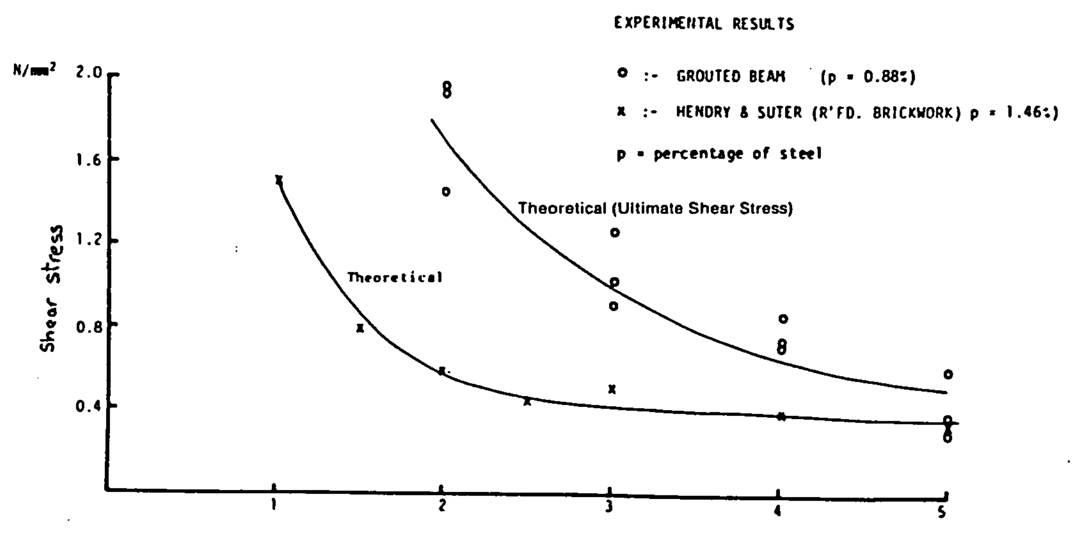

Figure 10. Influence of a/d on the shear strength of beams

Factors affecting the Shear Strength of Reinforced grouted Brickwork Beams and Slabs

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B.P. Sinha * and R.C. de Vekay +

The paper describes the statistical analysis carried out on the test results to identify the factors which affect the shear strength of the reinforced grouted beams and slabs. It appears from this analysis that the shear strength of the reinforced grouted cavity brickwork beams and slabs is strongly affected by:

i) shear arm/effective depth ratio
ii) % of tensile reinforcements
iii) shear reinforcements
iv) thinness of section

The grades of mortar ($1 : \frac{1}{4} : 3$ - cement : lime : sand) and ($1 : \frac{1}{2} : 4\frac{1}{2}$ - cement : lime : sand) slightly affect the strength, but not very significantly. The brick strength does not affect the shear strength of reinforced brickwork.

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INTRODUCTION

Reinforced brickwork has been used to a limited extent during the past. The failure of reinforced brickwork is almost invariably due to shear and it is very difficult to provide shear reinforcement in brickwork. Further, the necessity to put the main reinforcement in the bed joint limits the size of reinforcement to less than the joint thickness. These limitations of reinforced brickwork can be overcome by the use of brick grouted cavity construction. Basically, it consists of two skins of brickwork as used in an ordinary cavity wall; the cavity being used to accomodate the main and shear reinforcements. The cavity is finally grouted to form a monolithic construction and also to give an additional protection to the reinforcements from the weather. This form of construction would have a wide field of application and may prove economical because of elimination of formwork. Although, reinforced brickwork was the subject of investigation (1,2,3) in the past, no comprehensive (4,5) tests were carried out in case of grouted cavity construction. Because of this lack of experimental data relating to grouted cavity construction a comprehensive programme of research was undertaken.

This paper describes very briefly the experimental work done to investigate shear strength of grouted cavity reinforced brickwork beams and analyses the results statistically to establish the factors which affect the strength. The major variable considered in this programme were:

i. Shear span/effective depth ratio: The investigation covers a wide range of a/d ratio varying from 1:5 to 10.

ii. Brick strength: Three types of bricks, low (21.55 N/mm^2), medium (59.38 N/mm^2) and high strength (88.33 N/mm^2) were used for the construction of the beams and slabs.

iii. Mortar Grade: Two grades of mortar 1:¼:3 (cement:lime:sand) and 1:½:4½ (cement:lime:sand) were used.

iv. % tensile reinforcement: Three different percentages of the tensile reinforcements varying from 0.88 to 1.68 were used for the beams. The percentages of tensile reinforcements used for the walls were 0.9 and 2.54.

v. Effect of extra ties: In case of the walls, the failure was always noticed at the interface of the grout and the brickwork. The normal spacing (900mm horizontally and 400mm vertically staggered) of the wall ties were used originally. The number of ties were increased with the idea to stop the interface failure and thus to establish the effect of the extra number of ties.

vi. Shear reinforcements: The variable considered was the spacing of the vertical stirrups and its effect on shear strength of beams built with various types and strength of bricks.

DETAILS OF THE TEST SPECIMENS AND TEST ARRANGEMENTS

Two types of specimens, a beam and a thin slab, as shown in Fig. 1 were used for the tests. The two leaves of the cavity wall forming both sets of specimens were 80 - 90mm apart and were tied by galvanised fish tail ties spaced at 450 - 300mm staggered. In the case of the slab the reinforcement was in the centre of the cavity. For every specimen, full anchorage length for the reinforcements were provided as per concrete code (6). A 1:0.1:3:2 (cement:lime:sand:pea gravel) mix by volume was used for the grout with a constant water cement ratio of 1.2.

Both types of specimens were tested under two point loading in a specially designed loading frame providing a pin and a roller support as shown in fig. 2. The load was applied by means of two hydraulic jacks operated by a pump, the load being measured by the load cells connected to a pin chart recorder and a digital voltmeter. The load was applied at stages till failure.

The majority of the test specimens for investigating the effect of shear arm/effective depth ratios on the ultimate shear strength were made from low strength bricks. Since this variable has no significant effect on the shear strength after a/d > 5, the effects of all other variables were investigated by keeping this constant at 6.

METHOD OF STATISTICAL ANALYSIS

Fully replicated analyses of variance were carried out on the results in Tables 1 to 6 as follows:-
One way analysis of results in Table 1 with the shear arm/depth ratio as the variate.
Two way analysis with brick strength as one variate and the other variates being beams versus walls (Table 2), percentage steel (Table 4), effect of extra ties (Table 5) and effect of shear reinforcement (Table 6).

Lastly, a three way analysis of the variates mortar strength, brick strength and 'beam versus wall' was carried out on the results in Table 3.

RESULTS AND DISCUSSION

The test results are given in Tables 1 to 6 and the results of the statistical analyses in Table 7. They are listed as either significant at the 95%, 99% or 99.9% probability level or not significant (NS), i.e. less than 95% probability. For data on physical engineering properties it is seldom of value to consider probabilities lower than 95% and to look carefully at these at 95% especially interactions. The results demonstrate clearly, as expected, the effect of shear arm/depth ratio and the improvements resulting from increasing the percentage of steel and adding shear reinforcement.

The analysis also indicated that it was possible to develop a higher shear stress in walls than in beams of the same a/d ratio and materials. This is probably because the shear plane in walls does not pass through the brickwork but will be at the interface between brick and concrete or concrete and steel.

The result that brick strength had little overall effect on shear strength is also reasonable since few of the specimens failed by shearing or crushing of bricks most by interfacial shear. The interaction terms indicate, however, that there is a significant but small effect of brick strength on walls as opposed to beams and that with shear reinforcement in a beam it is possible to exploit higher brick strengths.

The significant effect of the mortar strength was probably due to its influence on the shear resistance of the brickwork in beams and on the crushing resistance of the brickwork in walls.

CONCLUSIONS

On the basis of the statistical analysis the following conclusions can be drawn from the test results:

i) Shear strength is affected strongly by shear arm/depth ratio and by percentage of tensile reinforcements.

ii) Shear strength is strongly affected by addition of shear reinforcement to beams.

iii) For the same shear arm/effective depth ratio of 6 and for approximately same % of steel the walls were significantly stronger than the beams.

iv) The shear strength is affected by the difference between grade i and ii mortars, prepared in laboratory condition, albeit weakly. This does imply, however that good quality - control of the mortar used on site will be necessary.

v) The use of stronger bricks will not improve the strength of simple beams designed to fail in shear although it can result in small improvements in the strength of walls of similar design. Higher brick strengths may be exploited in designs where the shear strength is also increased by some form of shear reinforcement.

vi) The addition of extra wall ties did not enhance the strength significantly.

ACKNOWLEDGEMENTS

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TABLE 1: Effect of $\frac{\text{shear span}}{\text{Effective depth}}$ ratios on the shear strength of reinforced grouted brickwork beam and slab (wall)

Brick Strength N/mm²	No of Spec-imens	Shear span ratio / Effective depth	1:1/4:3 Mortar Strength N/mm²	1:0.1:3:2 Grout Strength N/mm²	Nominal Ultimate Shear Stress N/mm² V/bd	Average Shear Stress N/mm²	Coeff. of variation in %	Remarks
Low Strength flat : 21.55	1	2	18.7	11.6	1.76	1.51	16.0	Shear failure at the interface of brick and grout
	2	2	21.5	11.6	1.46			
	3	2	24.9	10.0	1.38			
	4	2	24.0	10.0	1.25			
	5	2	20.0	10.9	1.37			
	6	2	21.0	10.9	1.86			
	1	4	20.6	11.7	0.72	0.785	9.8	
	2	4	21.55	11.7	0.69			
	3	4	21.67	16.5	0.86			
	4	4	21.22	16.5	0.86			
	5	4	21.22	16.5	0.84			
	6	4	19.78	17.37	0.74			
% of steel 0.9	1	6	20.46	14.18	0.52	0.61	-	
	2	6	20.46	14.18	0.63			
	3	6	25.0	18.02	0.69			
	4	6	25.0	18.02	0.58			
	1	8	22.14	17.37	0.405	0.43	7.87	Yielding of steel & Subsequent Compressive failure of b.w.
	2	8	18.86	13.04	0.44			
	3	8	20.86	13.04	0.47			
	4	8	19.66	13.56	0.43			
	5	8	19.66	13.56	0.38			
	6	8	20.46	14.18	0.46			
High Strength flat: 88.33	1	10	22.7	22.1	0.50	0.48	12.6	Shear failure
	2	10	22.7	22.1	0.50			
	3	10	22.7	22.1	0.54			
	4	10	19.2	19.2	0.46			
	5	10	19.2	19.2	0.38			
Low Strength on bed: 21.55	1	1.5	22.14	17.37	1.28	1.26	12.99	shear failure
	2	1.5	18.85	13.04	1.38			
	3	1.5	20.80	13.04	1.43			
	4	1.5	19.66	13.08	1.35			
	5	1.5	19.25	13.08	1.12			
	6	1.5	21.96	14.18	1.01			
on edge: 16.10	1	3.0	20.6	11.7	0.68	0.64	14.8	"
	2	3.0	21.55	11.7	0.63			
	3	3.0	21.67	16.5	0.49			
	4	3.0	21.22	16.5	0.61			
	5	3.0	19.78	17.37	0.66			
% of steel 0.88	6	3.0	22.14	17.37	0.78			
	1	4.5	18.7	11.6	0.49	0.53	4.4	"
	2	4.5	21.5	11.6	0.51			
	3	4.5	24.3	10.0	0.53			
	4	4.5	24.0	10.0	0.53			
	5	4.5	20.0	10.9	0.53			
	6	4.5	21.0	10.9	0.56			
	1	6.0	22.20	14.0	0.59	0.63	3.4	"
	2	6.0	22.20	14.0	0.64			
	3	6.0	19.96	16.96	0.64			
	4	6.0	19.96	16.86	0.63			
	5	6.0	19.96	17.24	0.65			
	6	6.0	22.86	17.24	0.64			
	1	6.8	19.3	17.5	0.53	0.56	-	Explosive Failure
	2	6.8	22.5	14.0	0.55			Explosive compressive at top
	3	6.8	23.3	14.0	0.60			Slow compressive at t[illegible]

$\frac{\text{Shear span}}{\text{effective depth}}$ (a/d) = 6

Type of Specimen	Brick Strength N/mm²		No of tests	1:1/4:3 Mortar strength N/mm²	1:0.1:3:2 Grout strength N/mm²	Nominal Ultimate shear stress N/mm²	Average shear stress N/mm²	Coefficient of variation in %
BEAM with % High Yield steel = 0.88		Low strength	1	22.2	14.0	0.59	0.63	3.4
	flat	21.55	2	22.2	14.0	0.64		
	on edge	16.10	3	19.96	16.86	0.64		
			4	19.96	16.86	0.63		
			5	22.86	17.24	0.65		
			6	22.86	17.24	0.64		
		Medium strength	1	24.58	15.61	0.60	0.60	7.2
	flat	59.40	2	20.80	16.10	0.66		
	on edge	31.92	3	26.85	19.20	0.63		
			4	23.50	15.61	0.53		
			5	20.80	16.10	0.60		
			6	21.73	15.58	0.60		
		High strength	1	21.59	15.0	0.59	0.61	5.3
	flat	88. 33	2	21.59	15.0	0.64		
	on edge	26.4	3	21.17	12.64	0.56		
			4	21.17	12.64	0.59		
			5	19.04	17.68	0.62		
			6	19.04	17.68	0.64		
SLAB or WALL % of High Yield steel = 0.9		Low strength	1	20.46	14.18	0.52	0.61	-
	flat	21.55	2	20.46	14.18	0.63		
	on edge	16.10	3	25.0	18.02	0.69		
			4	25.0	18.02	0.58		
		Medium strength	1	21.66	15.58	0.62	0.71	10.28
	flat	59.40	2	23.51	11.93	0.74		
	on edge	31.92	3	23.51	11.93	0.62		
			4	21.66	15.58	0.78		
			5	21.66	15.8	0.76		
			6	23.51	11.93	0.76		
		High strength	1	21.53	16.11	0.59	0.71	13.3
	flat	88.33	2	21.53	16.11	0.72		
	on edge	26.40	3	21.53	16.11	0.69		
			4	26.85	19.18	0.65		
			5	24.0	15.61	0.72		
			6	24.0	15.61	0.87		

TABLE 3: EFFECT OF MORTAR GRADES ON THE SHEAR STRENGTH OF REINFORCED GROUTED BEAMS AND SLABS (wall)

$\frac{\text{Shear span}}{\text{Effective depth}}$ a/d = 6

Type of Specimen	Type & Strength of brick N/mm2	No of Specimens	Shear Strength N/mm² Mortar Grade 1:1/4:3		1:½:4½	
	Low Strength	1	0.59			
	Flat:21.55	2	0.64		0.63	
	Edge:16.10	3	0.64	0.63	0.55	0.62
		4	0.63		0.59	
		5	0.65			
BEAM		6	0.64			
	Medium Strength	1	0.60			
	Flat:59.40	2	0.65		0.52	
	Edge:31.92	3	0.63	0.60	0.59	0.57
		4	0.53		0.61	
		5	0.60			
		6	0.60			
	High Strength	1	0.59			
	Flat: 88.33	2	0.64		0.53	
	Edge: 26.4	3	0.56	0.61	0.53	0.53
		4	0.59		0.54	
		5	0.62		0.53	
		6	0.64		0.53	
	Low Strength	1	0.52			
	Flat: 21.55	2	0.63		0.60	
	Edge: 16.10	3	0.69	0.61	0.72	0.68
		4	0.58		0.73	
	Medium Strength	1	0.62		0.60	
	Flat: 59.40	2	0.74		0.64	0.67
	Edge: 31.92	3	0.62	0.71	0.78	
		4	0.78			
		5	0.76			
		6	0.76			
SLAB	High Strength	1	0.59		0.71	
or	Flat: 88.33	2	0.72		0.55	
WALL	Edge: 26.40	3	0.69		0.66	
		4	0.65	0.71	0.62	0.63
		5	0.72		0.58	Coeff. of variation = 9.4%
		6	0.97		0.67	

Note: % of High tensile steel for

Beam = 0.38

Slab = 0.9

TABLE 4: EFFECT OF % OF STEEL ON THE SHEAR STRENGTH OF REINFORCED GROUTED BRICKWORK BEAMS AND WALLS

$\frac{\text{Shear span}}{\text{Effective depth}}$ a/d = 6

No of Specimen	Type and strength of brick N/mm^2) / % of steel	Shear Strength N/mm^2 BEAM 0.88		1.38		1.68		WALL 0.9		2.54	
1	Low Strength	0.59		0.82		0.69		0.52		0.55*	
2	Flat: 21.55	0.64		0.64	0.73	0.90	0.82	0.63		0.96	0.87
3		0.64	0.63	0.74		0.86		0.69	0.61	0.77	
4	On edge: 16.10	0.63						0.58			
5		0.65									
6		0.64									
1	High Strength	0.59		0.71		0.77		0.59		0.82	
2		0.64		0.67	0.68	0.76	0.73	0.72		0.87	0.82
3	Flat: 88.33	0.56	0.61	0.65		0.67		0.69	0.71	0.76	
4	on edge: 26.40	0.59						0.65			
5		0.62						0.72			
6		0.64						0.87			

* very inexperienced bricklayer from outside

TABLE 5: EFFECT OF EXTRA TIES ON THE SHEAR STRENGTH OF GROUTED REINFORCED BRICKWORK WALL (SLAB)

$\frac{\text{Shear span}}{\text{Effective depth}} \quad \frac{a}{d} = 6$

No of Specimens	Type and strength of brick (N/mm^2)	Normal spacing of ties	Extra number of ties
1	Low Strength	0.52	0.69
2		0.63 0.61	0.65 0.65
3	Flat: 21.55	0.69	0.61
4		0.58	
1		0.59	0.81
2	High Strength	0.72	0.82 0.84
3	Flat: 80 33	0.69 0.71	0.89
4		0.65	
5		0.72	
6		0.87	

NOTE: % of steel : 0.9

TABLE 6. SHEAR STRENGTH OF REINFORCED GROUTED BRICKWORK BEAMS WITH AND WITHOUT SHEAR REINFORCEMENTS

$\frac{\text{shear span}}{\text{effective depth}}$ a/d = 6

No of Specimens	Type and Brick Strength (N/mm^2)	% Steel	Shear Strength N/mm^2: No Shear reinforcements		with shear reinforcements 8mm @ 360mm		8mm @ 90mm	
1	Low Strength		0.59		0.72		0.67	
2			0.64	0.63	0.67	0.66	0.65	0.65
3	Flat: 21.55	0.88	0.64		0.59		0.63	
4			0.63					
5	On edge: 16.10		0.65					
6			0.64					
					with shear reinforcements 12mm @ 110mm			
1	Medium strength		0.60		0.74			
2			0.66		0.71	0.73		
3	Flat: 59.40	0.88	0.63	0.60	0.73			
4			0.53					
5	On edge: 31.92		0.60					
6			0.60					
1			0.59		0.78	0.76		
2			0.64		0.75			
3	Flat : 88.33	0.88	0.56	0.61				
4			0.59					
5	On edge: 26.4		0.62					
6			0.64					

TABLE 7 RESULTS OF VARIANCE ANALYSES

Table No.	Experiment	Variates and interactions	Degrees of freedom	Variance ratio	Significance %
1)	Effect of shear arm/ depth ratio				
1a)	wall	a/d	4/22	70	99.9
1b)	beam	a/d	4/18	69	99.9
2)	Effect of brick strength all at same a/d ratio	Brick strength	2/28	1.3	NS
		Beam/wall	1/28	11.2	99
		interaction	2/28	3.6	95
3)	Effect of mortar grades all at same a/d ratio	Mortar strength	1/45	4.4	95
		brick strength	2/45	0.8	NS
		beam/wall	1/45	23	99.9
		beam/wall v brick	2/45	3.6	95
		All other interaction			NS
4)a)	Effect of % steel- beams	% steel	2/18	16.3	99.9
		Brick strength	1/18	4.3	NS
		interaction			NS
b)	Effect of % steel- walls	% steel	1/11	12.62	99
		Brick strength	1/11	1.25	NS
		interaction			NS
5)	Effect of extra ties on walls	ties	1/12	4.2	NS
		Brick strength	1/12	11.4	99
		interaction			NS
6)	Beams with and without shear reinf.	Shear reinf.	1/20	40	99.9
		Brick strength	2/20	0.04	NS
		Interaction	2/20	6.5	99

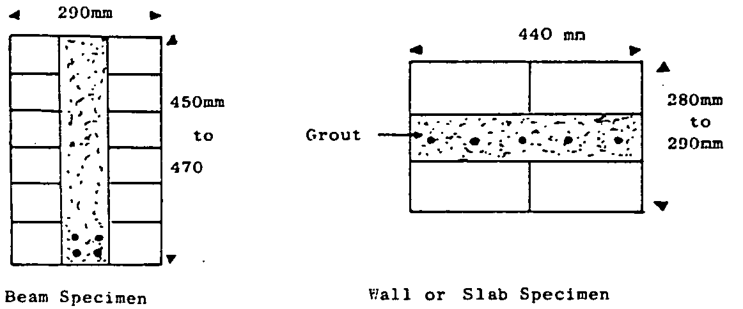

Fig. 1 Cross-sectional dimension of the test specimens

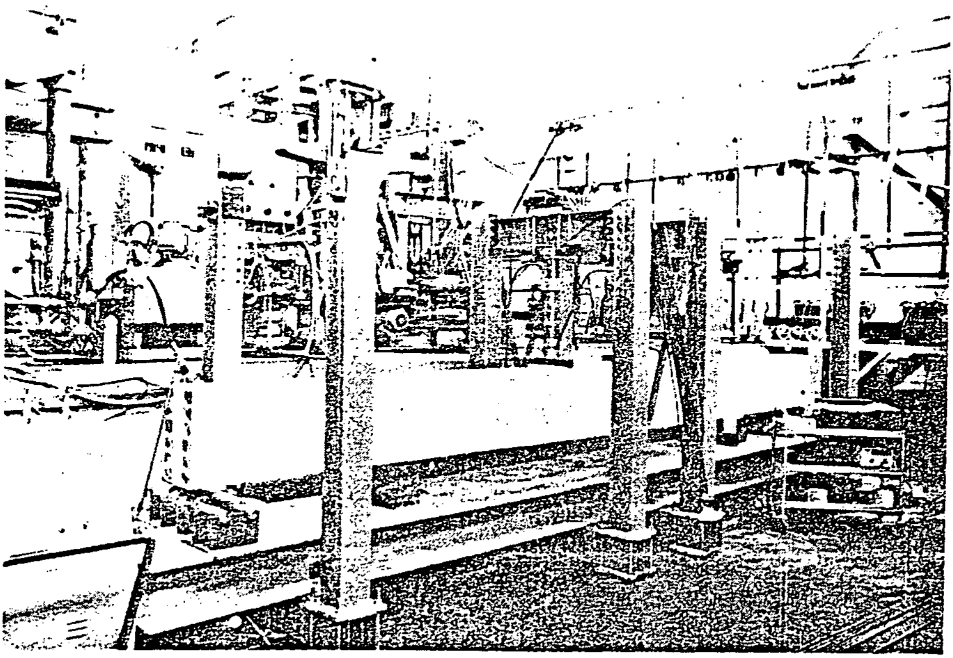

Fig.2 Showing the test arrangement and a beam of 6.3m span under test.

Prestressed Masonry

(Group 4 - Papers 34 to 44)

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Development and investigation of the ultimate load behaviour of post-tensioned brickwork beams

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Synopsis
This paper describes the development of post-tensioned brickwork beams and their behaviour up to failure under short-term loading. The brickwork section was specially designed for ease of construction and grouting of the tendons. The test results of 15 post-tensioned beams, with spans ranging from 2 m to 6 m, are given. The ultimate moments obtained from the tests are compared with the theoretical moments derived from a simplified cubic parabolic stress/strain relationship of the brickwork, and it is shown that the ultimate moment of post-tensioned brickwork can be closely predicted by this method.

Notation

a is the shear span
A_{ps} is the area of steel
b is the breadth of beam
F_c is the compressive force at failure
d is the effective depth
f_m is the compressive strength of brickwork prisms
f_{su} is the tensile stress in steel at failure
h is the overall depth of section
M_u is the ultimate moment of resistance
n is the neutral axis depth
p is the prestressing force
F_t is the tensile force at failure
ε_m is the strain in brickwork at failure
ε_{sa} is the strain in steel due to prestress
ε_{se} is the additional strain in steel due to applied loads
ε_{su} is the strain in steel at failure
λ_1, λ_2 are constants

Introduction

Very limited work[1,2] has been done up to the present in prestressed brickwork. No doubt, simple post-tensioning has been used for mainly compression[3,4] members, e.g. walls, with a view to increasing the shear resistance or to cater for wind loading but not for a member that carries the load primarily due to bending, as in a beam. Although the use of brickwork has been well established for compression members, this is a limited field of application. To remedy this situation, an attempt has been made to use reinforced brickwork in recent years. It is well documented that reinforced brickwork works out cheaper[5] than other structural materials used in similar situations. However, the failure of reinforced brickwork is due to shear and thus the strength of brickwork is not economically exploited. Further, reinforced brickwork cracks at early stages of loading and to keep cracks at an acceptable limit, the steel stress has to be kept low—an inefficient use. These disadvantages can be overcome with the development of prestressed brickwork. At present no British Standard or Code of Practice exists in this country for the design of prestressed brickwork. A comprhehensive research and development programme has therefore been undertaken, and this paper describes the preliminary work done so far on full-scale post-tensioned brickwork beams.

Materials

All materials were tested according to the relevant British Standard.

Mortar

A 1:¼:3 mix (cement:lime:sand) by volume was used for the construction of the specimens. Three 100 mm cubes were made over each construction cycle. The average compressive strength of the mortar for individual specimens is given in Table 1.

Grout

Two different mixes were used: for beams 1—7 a 1:1 mix (sand:cement) by weight was used; in beams A1—A8 a 1:2·5 (cement:sand) mix was used. In both types a plasticiser was used to reduce the effects of shrinkage and shorten the setting time. Three 100 mm cubes were cast during each grouting operation and tested at 7 days. The average compressive strength of the grout is given in Table 1 for each of the specimens.

Bricks

Three-hole 67·59 N/mm² bricks were used throughout. Compressive tests were carried out in all three directions and the results are given in Table 2. The average 24 h water absorption of the bricks was 5·67 %.

Reinforcement

10·9 mm diameter, stabilised strand was used with a yield stress of 1736 N/mm², a 0·2 % proof stress of 1597 N/mm², and Young's modulus 199 kN/mm².

Development of the post-tensioned beam section

In the early stages of the development, various sections, depending on the orientation of the bricks, including grouted-cavity construction for the post-tensioned beams, were considered. At one stage it was thought to pass the tendons through the perforations in the bricks. However, it was soon realised that it would be extremely difficult for the bricklayer to align the perforations in long-span beams, and also the tendons may not have adequate protection from the weather, even after grouting. It was felt that in selecting a section the following factors must be taken into account:

—effective utilisation of as much ceramics as possible
—ease in grouting the tendons
—ease of construction
—same bonding arrangement as used in a brick wall, hence no special skill required of craftsmen
—provision of cavity so that tendons can be placed at required depth

Two sections (Fig 1) were suggested by the second author in accordance with the above five constraints. The first series of beams 1—7 utilised the section shown in Fig 1(a) in which normal English bond was used, except in the second course. The second course consisted of bricks laid on edge with the perforations pointing towards the face of the beam. Except for three holes needed for grouting, all the perforations were filled with mortar during construction. The second series of beams A1—A8 were

TABLE 1—Experimental results and its comparison with the theoretical predicted moments

Beam	Span (m)	Shear span / Effective depth	Mortar strength (N/mm²)	Grout strength (N/mm²)	PS force (kN)	Ultimate load (kN)	1 Experimental ultimate BM (kNm)	2 Theoretical BM (kNm)	Ratio Experimental / Theoretical
1	6·00	9·87	22·16	32·75	167·0	50·1	60·7	49·7	1·221
A7	6·00	10·80	18·30	16·30	125·0	44·6	53·5	40·9	1·308
A8	6·00	10·80	18·30	16·30	147·0	43·3	51·8	42·1	1·230
2	4·43	6·92	21·15	20·1	116·0	56·3	51·3	47·5	1·080
3	4·43	6·92	21·15	20·1	145·0	64·6	57·9	49·0	1·182
A5	4·43	7·57	18·10	17·5	140·0	72·2	46·1	41·4	1·114
A6	4·43	7·57	18·10	13·0	130·0	52·8	48·8	40·9	1·193
4	3·20	4·61	21·15	48·0*	157·0	91·7	55·2	48·7	1·133
5	3·20	4·61	20·67	14·6	311·0	129·4	77·9	69·6	1·119
A3	3·20	5·04	21·60	17·5	116·0	76·8	46·0	40·9	1·125
A4	3·20	5·04	21·60	17·5	140·0	72·2	46·1	42·0	1·100
6	2·00	2·34	20·67	21·9	335·0	217·4	67·8	70·7	0·959
7	2·00	2·34	20·67	32·8	133·5	158·9	50·3	48·7	1·032
A1	2·00	2·57	21·60	17·5	125·0	154·0	47·9	41·3	1·160
A2	2·00	2·57	18·10	13·0	128·0	145·0	45·3	41·4	1·094

* cured in air

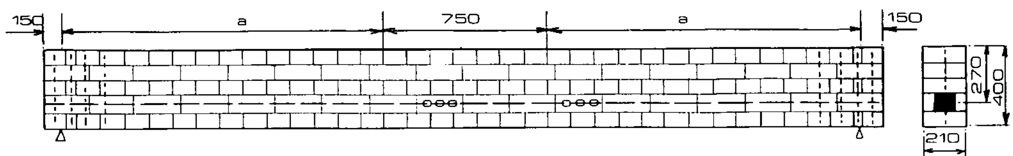

Fig 1(a). Beams 1-7

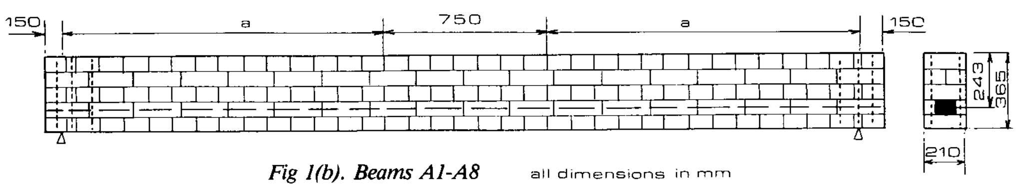

Fig 1(b). Beams A1-A8 all dimensions in mm

TABLE 2—Compressive strength of bricks

	flat	on edge	on end
Average compressive strength (N/mm²)	67·60	27·7	26·5
Range (N/mm2)	44·16-79·47	23·44-40·29	23·07-33·80
Standard deviation (N/mm²)	10·20	5·66	5·69
Coefficient of variation(%)	15·0	20·4	21·47

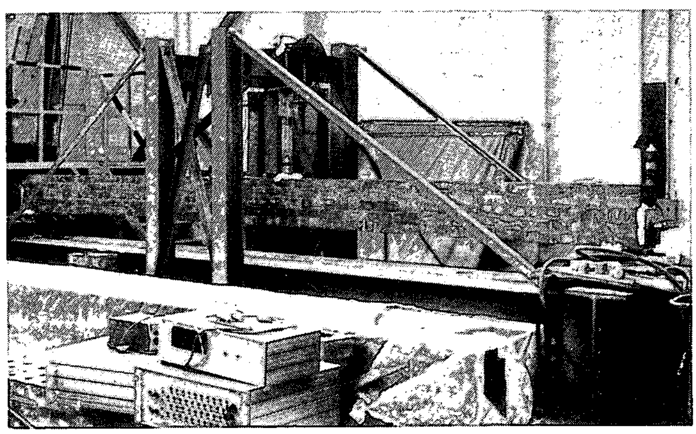

Fig 2. Test arrangement

built as shown in Fig 1(b) in normal English bond. The cavity receiving the tendon was formed in the second course by splitting the bricks lengthwise and placing them flush with the face of the beam.

Constructional details
All beams were built on the floor of the testing laboratory by an experienced bricklayer. To stop horizontal splitting of the bed joints the ends of the beams were reinforced to resist the anchorage forces which develop in the 'lead in length'[6]. In beams 1—7, four 8 mm dia. rods (Fig 1(a)) were placed along the centreline of the beam at a pitch of approximately 100 mm and then mortared solid. In beams A1—A8, three pairs of 8 mm dia. rods were built in during construction, the rods being placed either side of the cavity and passing through the perforations (Fig 1(b)) in the bricks, which were filled solid with the mortar used for bricklaying.

In beams 5 and 6 four strands were used; in the remaining beams two strands were used. The strands were placed so that the resultant prestressing force was acting on the 'kern' limit of the section. The specimens were left curing for 21 days before post-tensioning. 25 mm-thick mild steel anchor plates were then attached to the beams. The beams were prestressed and then grouted immediately. A grout pump was needed for grouting the tendons in the case of beams 1—7. Later the grouting process was greatly simplified for beams A1—A8. The grout was poured through the holes at the base of the beam and vibrated with a poker vibrator. Testing was carried out 7 or 8 days later.

Test arrangements and instrumentation
The beams were tested in a specially designed, two point loading rig (Fig 2) capable of testing spans up to 6·5 m. The supports for the beams consisted of a pin and a roller supports. The loads were applied in stages

to failure using hydraulic jacks. The applied loads were measured at jacking point using load cells attached to a digital voltmeter and penchart recorder which enabled the failure load to be determined accurately. The prestressing force in each tendon was measured using small, specially designed (Fig 3) load cells placed between the anchorages and anchor plates and also strain gauges attached to the strand.

The deflections of the beams were measured using dial gauges reading to 0·002 mm of the ends and 0·01 mm at midspan. Strains in the brickwork were measured using a 'demec' gauge. In some instances crack widths were measured using an 'ultra lomar' microscope.

Small specimen tests for the determination of compressive strength of the brickwork

Brickwork prisms (Fig 4) representing the top three courses of the beams were built and tested to failure to obtain the compressive strength. The average compressive strength of the prisms was 11·47 N/mm². The compressive strength thus obtained was used to predict the failure moment of the beam.

Results and discussion

Deflection and cracking

Typical deflection of beams 1, A7 and A8 of span 6·0 m are given in Fig 5. The deflection for almost all the beams was linear up to the point at which decompression of the prestressing force occurred. At this stage, tensile cracks appeared, thus reducing the stiffness of the section resulting in a rapid increase in deflection without a corresponding increase in load. The deflection of beams A7 and A8 was slightly higher than beam 1, owing to the effective depth of beam 1 being 30 mm greater than the other two. In most cases the recovery of deflection was between 65 to 90 % after failure.

Fig 6 shows the typical propagation of cracking in the beams. Visible cracking first occurred at between 55 and 70 % of the ultimate load. In practice these cracks would not be allowed to occur, as all the prestressing force would not be neutralised under working loads.

Mode of failure

With the exception of one beam of a/d ratio = 2·34, most of the beams failed primarily by yielding of the steel resulting in the compressive failure of the brickwork (Fig 7) in the maximum bending moment zone. Beam no. 6 (Fig 8) failed in shear, cracking occurred in the midspan of the beam as well as considerable diagonal cracking in the shear span. The diagonal cracks in this beam progressed at an angle of approximately 45°; the failure was sudden when the cracks reached the top course. In three beams, flexural failure and horizontal cracking at the interface of the brick and mortar in the top course occurred simultaneously. From these results it appears that, unlike reinforced brickwork, shear is not such an important problem.

One of the ends of beam A3 remained intact after testing and was tested again as a beam of span 1·5 m with a central point load. This 'new' beam

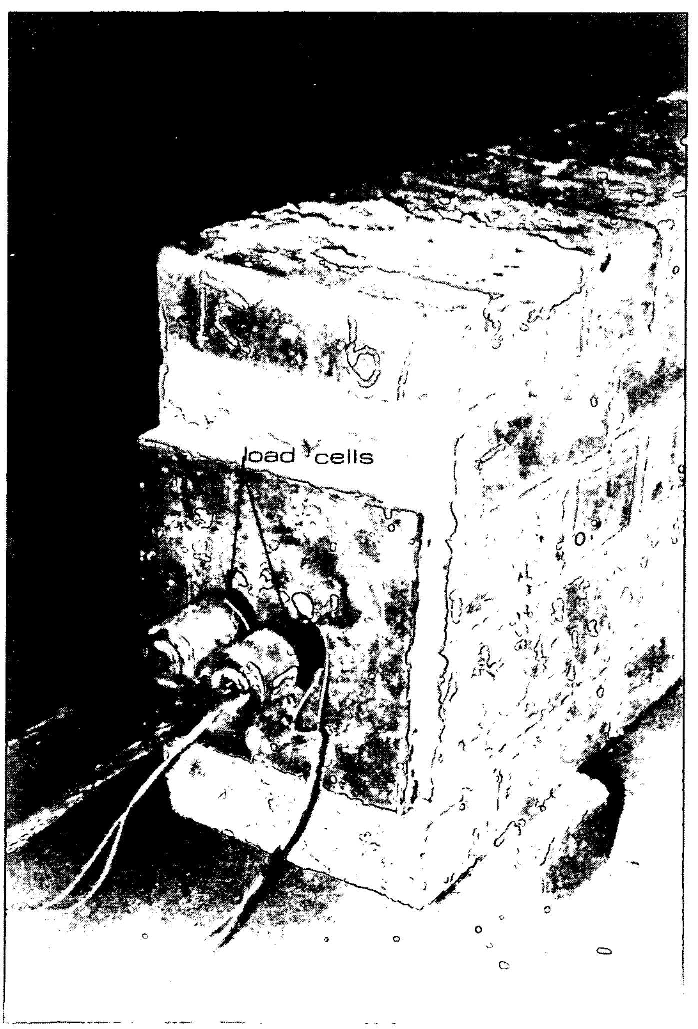

Fig 3. Load cells to measure the prestressing forces

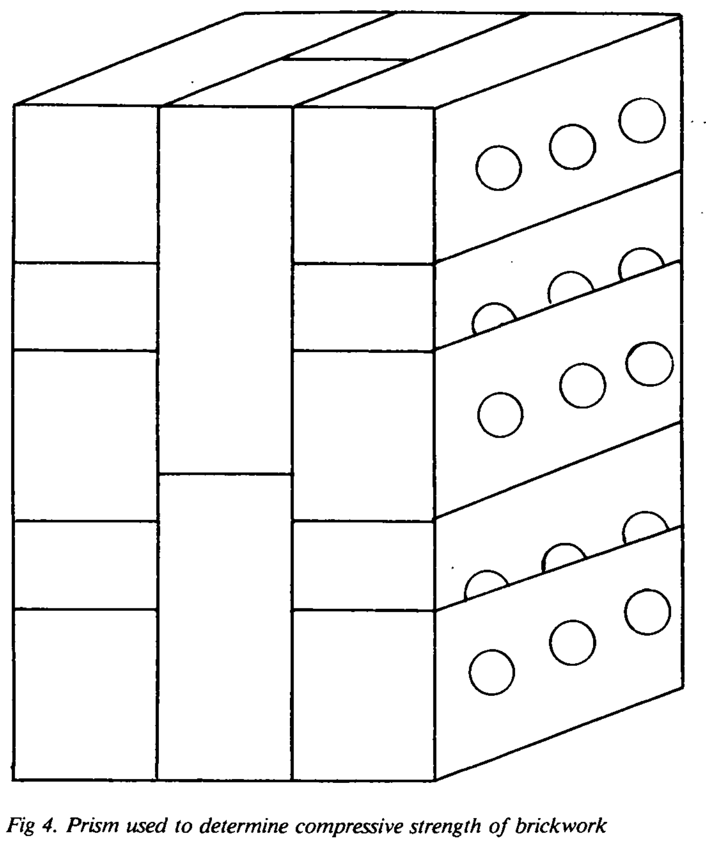

Fig 4. Prism used to determine compressive strength of brickwork

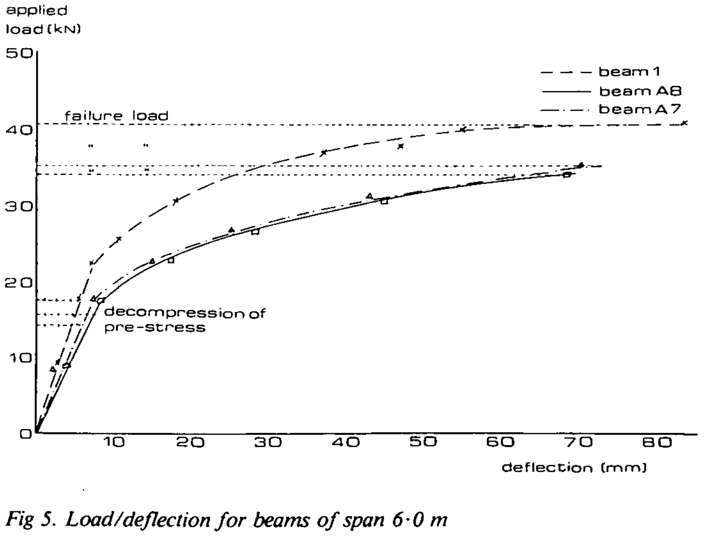

Fig 5. Load/deflection for beams of span 6·0 m

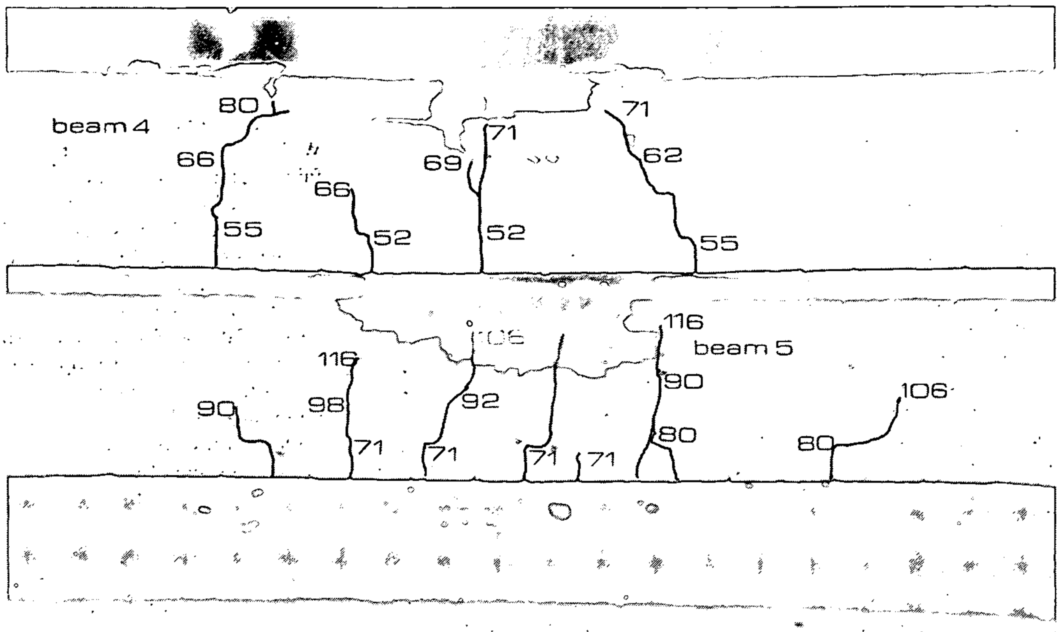

Fig 6. Propagation of cracks in beams 4 and 5 (cracking load in kN)

failed in flexure, as did the original. The ultimate moment was 50·1 kNm which was slightly greater than that achieved by the original beam. This indicates that the loadcarrying capacity of the section has not been reduced except in the localised zone where crushing of brickwork took place because of yielding of the steel.

Calculation of ultimate moment of resistance

A simplified stress block as suggested by Sinha[7] was used for the calculation of the compressive forces. From Fig 9 (c) the total compressive and tensile force can be given by:

$$F_c = \lambda_1 . b . n . f_m \qquad \ldots (1)$$

$$F_t = f_{su} . A_{ps} \qquad \ldots (2)$$

$$F_c = F_t \qquad \ldots (3)$$

$$n = \frac{f_{su} . A_{ps}}{\lambda_1 . f_m \, b} \qquad \ldots (4)$$

$$\varepsilon_{su} = \varepsilon_{sa} + \varepsilon_{se} \qquad \ldots (5)$$

Assuming full bond between steel and grout at failure, the steel strain is

$$\varepsilon_{se} = \varepsilon_m \left(\frac{d-n}{n}\right) \text{ where } \varepsilon_m = 0{\cdot}003^7 \qquad \ldots (6)$$

From stress/strain relationship for steel and knowing ε_{su}, f_{su} may be obtained. This process involves a certain amount of trial and error to find n, such that $F_c = F_t$

$$M_u = f_{su} . A_{ps} (d - \lambda_2 . n) \qquad \ldots (7)$$

The theoretical moment thus calculated was compared with the experimental results in Table 1. There appears to be good agreement between the theoretical and experimental results. Further work is under progress to establish the shape of the stress block and to check the validity of this method for different strengths of brick.

Summary and conclusions

(i) The section chosen appears quite efficient, and no difficulty was encountered in post-tensioning or handling of the specimen.

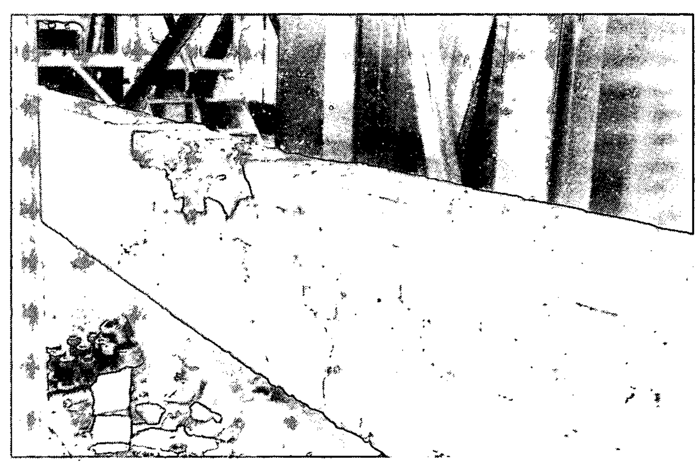

Fig 7. Typical compressive failure of brickwork in maximum bending moment zone (beam 1)

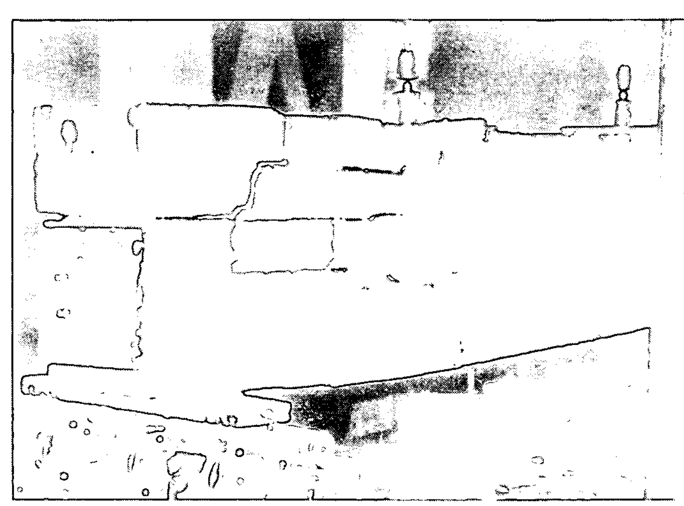

Fig 8. Shear failure of beam 7 (a/d ratio-2·64)

(ii) The post-tensioned brickwork beams failed in flexure in these tests, not in shear; hence the *a/d* ratio has no effect on the ultimate moment.
(iii) The ultimate moment of resistance may be reliably predicted by using the method proposed in this paper.

Acknowlegements
The work described in this paper has been supported by the Science and Engineering Research Council and Structural Clay Products Ltd. and carried out in the Department of Civil Engineering & Building Science, University of Edinburgh.

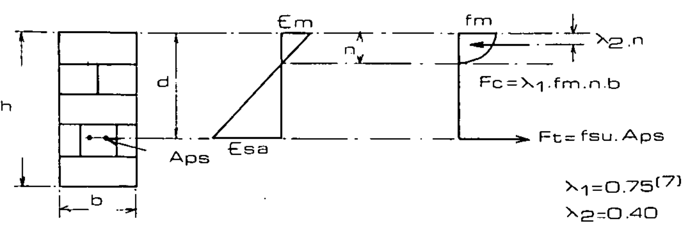

Fig 9(a). Beam section *Fig 9(b). Strain distribution at failure* *Fig 9(c). Stress distribution at failure*

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Because of increased printing and postal costs Council have approved the selling prices given below for Institution publications from 1 March 1982. The members' rate is available only to those sending remittance with order.

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The scheme of Informal Study Groups is now in full operation. Each works under the direction of a Convener appointed by the Council. The Convener is personally responsible for organising the work of his Group, conducting correspondence, producing and exchanging draft documents, etc. Experience has shown that Study Groups vary in the way they operate. Some have a continuing function, while others have objectives that are met in a given period. The general procedure for the working of a Study Group is set out on this page. Some such simple rules are clearly necessary, but it is the hope of the Council that the strength of a Study Group will lie in its informality and the opportunity it provides for members in all parts of the world to exchange ideas and experience. Members interested in contributing to one or other of the Groups now working are invited to get in touch with the appropriate Convener at the address given. Full details of the purpose and work programme of each Study Group will be found in the issue of *The Structural Engineer* under the reference given.

Problems of Covering Large Areas

Convener: S. B. Tietz, BSc, CEng, FIStructE, FICE, S. B. Tietz & Partners, 10-14 Macklin Street, London WC2B 5NF
The Structural Engineer, September 1970, p. 342

Inflated and Pneumatic Structures

Joint Conveners: F. Newby, MA(Cantab), CEng, FIStructE, HonFRIBA, 231 Gower Street, London NW1, and for correspondence M. G. T. Dickson, BA, MS, CEng, MIStructE, University of Bath, Claverton Down, Bath BA2 7AY
The Structural Engineer, October 1972, p. 404

History of Structural Engineering

Convener: R. J. M. Sutherland, BA, CEng, FIStructE, FICE, Harris & Sutherland, 38-42 Whitfield Street, London W1P 5RF
The Structural Engineer, March 1973, p. 110

Model Analysis as a Design Tool

Convener: F. K. Garas, PhD, CEng, MIStructE, MICE, Head of Structures Research Laboratory, Taylor Woodrow Construction Ltd., Taywood House, 345 Ruislip Road, Southall, Middlesex UB1 2QX
The Structural Engineer, February 1977, p. 63

Foundations on Shrinkable Clays: Load Independent Movements due to Vegetation and Seasonal Climatic Changes

Convener: D. Gonsal, CEng, MIStructE, MICE, Chief Assistant Engineer, Chief Engineer's Department, London Borough of Camden, Old Town Hall, 213 Haverstock Hill, London NW3
The Structural Engineer, October 1977, p. 410

Qualitative Analysis of Structural Behaviour

Convener: J. S. Armitage, BSc, DLC, CEng, MIStructE, MICE
CIRIA, 6 Storeys Gate, Westminster, London SW1P 3AU
The Structural Engineer, November 1978, p. 309

The Influence of Creep on Structural Behaviour

Convener: G. L. England, PhD, DSc(Eng), CEng, MIStructE, MICE, Reader in Engineering Mechanics, Department of Civil Engineering, University of London King's College, Strand, London WC2R 2LS
The Structural Engineer, August 1979, p. 244

Offshore Structural Audits

Convener: R. J. M. Bennett, BSc, CEng, MIStructE, MICE, Bennett & Rudd, Consulting Engineers, 61 Manor Place, Edinburgh EH3 7EG
The Structural Engineer, February 1981, p. 38

Materials and Components

Convener: K. Thomas, MSc, CEng, FIStructE, FICE, Timber Research and Development Association, Stocking Lane, Hughenden Valley, High Wycombe, Bucks. HP14 4ND
The Structural Engineer, Part A, September 1982, p. 269

General procedure

Object
The object of the Study Group scheme is to create opportunities for members of the Institution to exchange ideas and work on deepening and developing their knowledge of structural engineering, thus stimulating a greater interest in and promoting the art and science of structural engineering.

Right to set up
Any corporate member shall have the right to propose setting up a Study Group, provided that an outline of the work of the proposed Study Group is submitted, it does not duplicate the work of other Groups, its interests lie in the field of structural engineering and it conforms to any rules that may be formulated by Council. For each Study Group approved, Council will appoint a Convener, who shall be a member of the Institution. A Study Group so established shall be disbanded on completion of its work or as decided by Council.

Right to participate
Every member of the Institution may of right take part in the work of any Study Group; non-members may also participate, with the approval of the Convener.

Organisation
The internal organisation of each Study Group shall be at the discretion of the Group, although only Institution members may serve on a Group's management or steering committee unless Council approval is obtained.

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No Study Group shall impose any financial commitment on the Institution, other than the cost of correspondence from the Secretariat and of notices and announcements in The Structural Engineer, *without the express prior consent of Council. In this context it must be understood that 'commitment' includes any activity that is expected to be financially self-supporting. Neither shall any Study Group organise any activity calling for the collection of admission fees or other money without the express permission of Council. A Study Group is part of the Institution, which is responsible for any losses incurred and to whom any surplus funds are accountable. The Institution may assist financially in some work at the discretion of the Council.*

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The Convener of each Study Group shall report at least annually to the Institution on the Study Group's progress. To prevent conflict or confusion with events organised by Council the Institution requires prior consultation on any proposed public activity organised by one or more of its Study Groups. When a Study Group concludes its work a final report detailing the nature and results of its study shall be prepared. This report and resources, together with the Group's records, shall be the property of the Institution which has the right to publish or otherwise make them available to members. All members shall be entitled to consult the report, which shall be placed in the Institution's library.

Title of the paper

THE BEHAVIOUR OF POST-TENSIONED BRICKWORK BEAMS

Author

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Organization/Enterprise

University of Edinburgh, Scotland

Key words

Brickwork, prestressed, shear, post-tension

Summary

The research paper describes the development of post-tensioned brickwork beams and their ultimate load behaviour under short term loading. The brickwork section was specially designed for ease of construction and grouting of tendons. The span of the post-tension beams varied from 1.5 m to 6.3 m. The variables considered in this investigation were:

i) Brick strength

ii) Mortar strength (Grade 1, 1:$\frac{1}{4}$:3 or Grade 2, 1:$\frac{1}{2}$:4$\frac{1}{2}$ cement:lime:sand)

iii) Shear span/effective depth ratio

A method for the calculation of the ultimate moment is also outlined. The theoretically predicted moment; taking into account the non-linear stress-strain relationship of brickwork, is in good agreement with the experimental results. From the results it appears that brick and mortar strengths do not significantly affect the ultimate load carrying capacity of beams with a low % of steel. The ultimate shear strength of prestressed brickwork beams is affected by the shear span/effective depth ratios in cases where premature failure happended due to shear.

Titre du texte

LA PERFORMANCE DES POUTRES EN BRIQUES PRE-CONT...

Auteur

B.P. Sinha et R.F. Pedreschi

Organisation/Entreprise

Université d'Edimbourg, Ecosse

Mots-clés

Poutres en brique, pré-contrainte, cisaillement, se tendre en suivant

Sommaire

L'article décris le développement des poutres en briques pré-contrainte et leurs performances maximales sous les charges de la petite durée. Le coupe de brique était formé pour la construction et liasonnant au coulis des cordes en aciers facile. Les portées des poutres étaient variées entre 1.5 m à 6.3 m.

Les variables qu'étaient considerés en cette étude étaient en suivant:

i) La résistance de brique

ii) La résistance du mortier (qualité 1, 1:$\frac{1}{4}$:3, ou qualité 2, 1:$\frac{1}{2}$:4$\frac{1}{2}$ ciment: oxide du calcium: sable)

iii) Rapport porteé en cisaillement/hauteur utile

Une mode du calcul pour le moment maximal est donnée. Les résultats experimentaux et théoriques ont bon accord pour le moment maximal quand les caractéristiques non-lineaire du matériaux étaient pris en compte. On apércois des résultats que la résistance de brique et du mortier n'ont pas d'influences sur les charges maximales valuers pour les poutres avec un petit pourcentage d'acier. Le résistance au cisaillement d'une poutre en brique pré-contrainte est affecté par le rapport portée de cisaillement/hauteur utile où la ruine prématuré est arrivée parce que de la force de cisaillement.

THE BEHAVIOUR OF POST-TENSIONED BRICKWORK BEAMS

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INTRODUCTION

With the increasing cost of high energy input building materials, it has become necessary to seek its cheaper alternative for the housing sector. Brickwork, being labour intensive and low energy input material is most suited for this need. Its use as a compression member to carry the load is well established and increasing both in the developed and developing countries. It has not been used as widely as concrete for the flexural members in the housing sector. Interest in reinforced and prestressed brickwork is growing rapidly. However, the reinforced brickwork flexural members crack in the very early stages of loading and to keep it to any acceptable limit, the stress in steel must be kept low, which results in the inefficient use of steel. Further, the premature failure of a reinforced brickwork beam usually happens due to shear and it is not very easy to reinforce against it; thus the brickwork is also not used to its optimum. These disadvantages to a great extent can be overcome by prestressing which results in economic use of both materials. Not enough work (1,2) has been done to exploit prestressed brickwork and hence a comprehensive investigation was undertaken so that the construction industry can take advantage of the development. The paper outlines the R and D work done to study the behaviour of such beams.

SCOPE OF THE INVESTIGATION

To study the behaviour of prestressed beams, the following variables were considered:

i) Brick strength: Three-hole bricks, having compressive strengths of 88 N/mm^2, 67 N/mm^2 and 34 N/mm^2 were used. In a few tests, single deep frog bricks (22 N/mm^2) were used. The compressive strengths were determined according to BS.3921 (3).

ii) Mortar mix or grade: Two grades of mortars, grade 1 (1:$\frac{1}{4}$:3 cement:lime:sand) and grade II (1:$\frac{1}{2}$:4$\frac{1}{2}$) which are likely to be used in practice were considered. The brick strength was kept constant while invesitgating the effect of mortar.

iii) Shear span/effective depth ratio: After the neutralisation of the initial pre-stress, the behaviour of a prestressed beam will be very akeen to an ordinary reinforced brickwork beam. The reinforced brickwork beam usually fails in shear, which is significantly affected by the shear span/effective depth ratio, it was essential, therefore to assess the effect of this parameter on the prestressed brickwork beams.

Grade 1 mortar was used for the investigations listed under headings (i) and (iii) above. The brick strength and steel percentage were kept the same while investigating the effect of shear span/effective depth ratio on the strength of prestressed beams. The beams were prestressed using 10.9 mm diameter stabilised seven wire strand, having a 0.2% proof stress of 1580 N/mm^2. Except for the effect of shear span/effective depth ratio, the steel percentage was kept constant at 0.274.

DEVELOPMENT OF TEST SPECIMENS AND ITS CONSTRUCTION

It was felt that for practical utility of this development; the beam sections must be as such that no special skill is required of the bricklayer and it could be built on the site or at a factory with the help of semi-skilled or skilled labour. The cross-section as shown in fig. 1 was developed with the following advantages:

i) effective utilisation of as much ceramics as possible.
ii) ease of grouting tendons
iii) ease of construction
iv) same bonding pattern as used in brickwall
v) provision of cavity so that tendons can be placed at required depth
vi) elimination of shuttering or any form work.

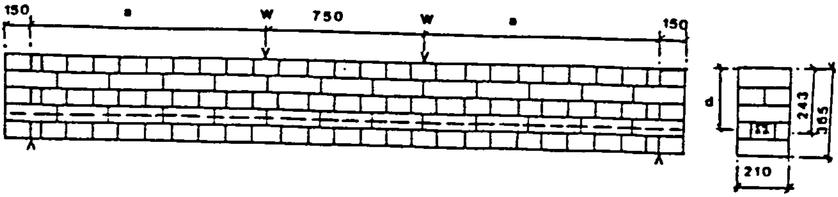


Fig. 1 Typical beam details

TEST ARRANGEMENTS AND INSTRUMENTATIONS

The beams were tested under two point loading in a specially designed test rig (fig. 2) which provided pin and roller support. The loads were applied by jacks and measured at the jacking points with the help of load-cells and a pen-chart recorder. The loads were applied at stages until failure. The deflection was measured by dial guage. Strains were recorded by a 'demec' gauge. The crack width was measured by 'ultra lomar microscope' capable of measuring widths down to 0.02 mm.

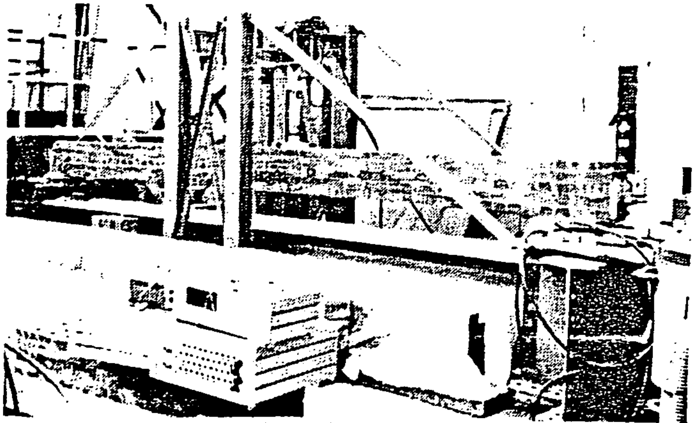


Fig. 2 Test Arrangement

RESULTS & DISCUSSION

The results of the tests are given in Tables 1, 2 and 3.

i) Effect of Brick Strength: From the result in Table 1, it is apparent that the brick strength does not significantly affect the ultimate moment of prestressed brickwork beams, having similar percentage of steel or initial prestress. This is also confirmed by the theoretical analysis. With low percentage of steel, the failure is initiated by the yielding of steel which leads to flexural failure of brickwork. Although brick-strength is considerably different for this particular area of steel the section is under reinforced and consequently the internal forces in the beam are governed by the reinforcement, the only influence of the brickwork strength being to shift the position of the neutral axis depth slightly.

ii) Effect of Mortar Grade: It can be seen from Table 2 that the grade of mortar only marginally affects the ultimate moment carrying capacity of the prestressed brickwork beams. The compressive strength of grade I ($1:\frac{1}{4}:3$) mortar was approximately 2.8 times higher compared to grade II mortar ($1:\frac{1}{2}:4\frac{1}{2}$); the increase in the ultimate moment capacity of the beam made from grade I mortar was 12.5% only. This may be because the mortar strength does not significantly affect the compressive strength of brickwork.

iii) Effect of shear span/effective depth ratio on shear strength: All the beams with varying shear span/effective depth (2 to 11) made from 67 N/mm^2 brick having constant 0.274% steel exhibited flexural failure, hence the effect of shear span/effective depth ratios on shear strength could not be ascertained. With similar percentage of steel both beams made from high strength (88 N/mm^2) and low strength (34 N/mm^2) failed due to flexure.

The beams made from low strength (34 N/mm^2) and medium (67 N/mm^2) were prestressed to the maximum limit, hence further work using these bricks to clarify this could not be done. It was then decided to use high strength brick with double the area of prestressing steel (0.548%) resulting in higher prestress. All the beams with shear span/effective depth varying from 2 to 11 failed prematurely due to shear. Fig. 3 shows that $\frac{\text{shear span}}{\text{effective depth}}$ ratio (a/d) has significant effect on the shear strength of these beams which is analogous to the behaviour of the reinforced brickwork beams. The degredation of the moment due to shear failure was between 16% of the predicted failure moment for beams of a/d = 2.0 and 7% of the predicted failure moment for beams of a/d = 11.

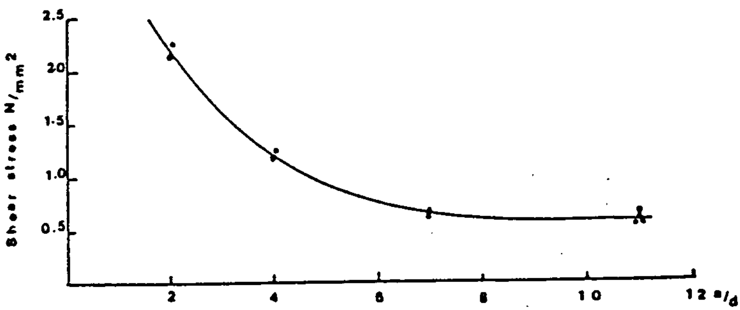


Fig.3 Effect of a/d ratio on ultimate shear stress.

DEFLECTION AND CRACKING

The typical deflection of the beam is shown in fig. 4.

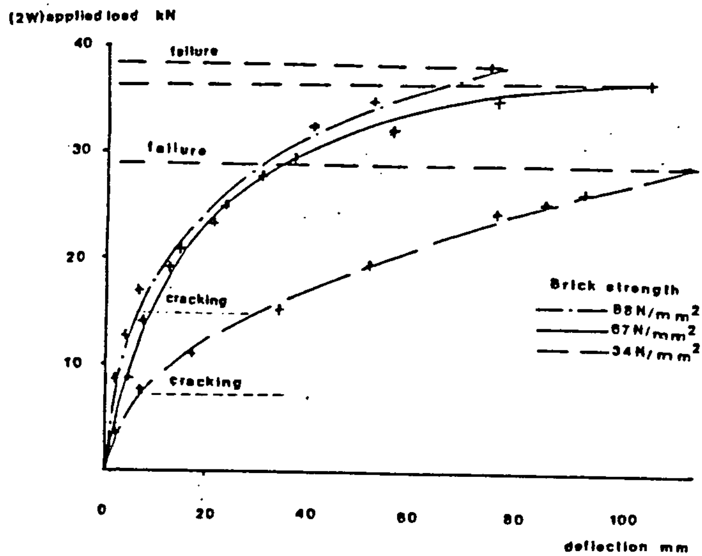
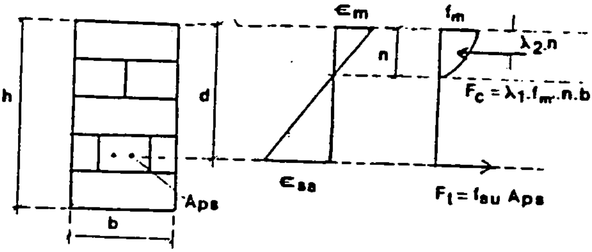


Fig.4 Typical load/deflection

Initially, the load-deflection relationship was linear. The deflection increased rapidly after the initial prestress was reduced to zero and the moment exceeded the cracking moment of the brickwork. Visible cracking first occurred at 55-70% of ultimate load, i.e. above working load. The load at which this increase in deflection takes place can very well be predicted as shown in fig. 4 by using the flexural strength of brickwork together with additional strength due to prestress and uncracked section (full cross-section) of the beam. The recovery of deflection was 65 to 90% after flexural failure in most cases.

CALCULATION OF ULTIMATE MOMENT OF RESISTANCE

For calculation of the ultimate design moment, the stress-strain curve obtained from a three-course prism (4) made from different bricks, was mathematically idealised in non-dimensional form as a cubic parabola and the failure strain was assumed as is 0.3%. The detailed derivation is given elsewhere (5).



(a) beam section (b) strain distribution (c) stress distribution

Fig. 5 Conditions at failure

From fig. 5, the total compressive and tensile forces can be calculated as:

$$F_c = \lambda_1 . b . n . f_m \qquad (i)$$

$$F_t = f_{su} . A_{ps} \qquad (ii)$$

$$F_c = F_t \qquad (iii)$$

$$n = \frac{f_{su} . A_{ps}}{\lambda_1 . f_m . b} \qquad (iv)$$

Assuming full bond between steel, grout and brickwork at failure, the strain in the steel can be calculated from:-

$$\epsilon_{sa} = \epsilon_m \left(\frac{d-n}{n}\right) \quad \text{where} \quad \epsilon_m = 0.003 \quad (v)$$

The steel strain at the time of failure

$$\epsilon_{su} = \epsilon_{sa} + \epsilon_{sp} \qquad (vi)$$

With the known strain ϵ_{su}, the stress f_{su} can be obtained from the stress-strain relationship of the steel. Some trial and error is involved to find the neutral axis depth 'n' unless the beam is under-reinforced, to give $F_c = F_t$. The ultimate moment can then be calculated:-

$$M_u = f_{su} . A_{ps} \quad (d - \lambda_2 n) \quad (vi)$$

The moment thus calculated is compared in Table 3 with the experimental results and it appears that there is fairly good agreement between them.

SUMMARY AND CONCLUSIONS

On the basis of these tests the following conclusions can be drawn:

i) The section chosen appears quite efficient and no difficulty was encountered by the bricklayer in construction and in post-tensioning or handling of the specimen.

ii) The compressive strength of bricks does not significantly affect the ultimate moment of the prestressed brickwork beams.

iii) The strength or the grade of mortar marginally affects the failure moment of the prestressed brickwork beams.

iv) The ultimate shear strength of prestressed brickwork beams increases with decreasing shear span/effective depth ratio in cases where failure happened due to shear.

v) The ultimate moment of the beam built with low, medium and high strength bricks can be predicted by the method proposed in this paper, using average compressive strength of three-course brickwork prism made in similar bonding pattern as the test beam.

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NOTATIONS

a	is the shear span
A_{ps}	is the area of steel
b	is the breadth of beam
f_m	is the compressive strength of brickwork prism
F_c	is the compressive force at failure
d	is the effective depth
h	is the overall depth of the section
f_{su}	is the tensile stress in steel at failure
F_t	is the tensile force at failure
n	is the neutral axis depth
Mu	is the ultimate moment of resistance
ϵ_m	is the strain in brickwork at failure
ϵ_{sa}	is the additional strain in steel due to applied load
ϵ_{sp}	is the strain in steel due to prestress
ϵ_{su}	is the strain in steel at failure
λ_1, λ_2	are constants

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Investigation of the ultimate load behaviour of prestressed brickwork beams built with perforated bricks

B P Sinha,* R F Pedreschi** and R C De Vekey***

Abstract – Perforated bricks account for 30 percent of the total production of bricks in the UK and their use is increasing. Little performance data exists on their use in prestressed brickwork beams and the authors have undertaken a series of full-scale tests to examine the influence of brick strength, mortar grade and shear span/effective depth ratio on the ultimate load behaviour of prestressed brickwork beams. This paper reports on part of their work, an evaluation of the effect of perforation pattern and area on the ultimate load behaviour.

Brickwork is strong in compression, hence structural forms have evolved which have utilised this to a large extent. In recent years, efforts have been made to use brickwork in other forms, particularly as a beam [1]. As brickwork is very weak in tension, this could only be achieved by incorporating steel to take the tension, developed due to loading in a flexural member as in the case of reinforced brickwork, or by prestressing. Reinforced brickwork fails generally due to shear, [2,3] hence the compressive strength of brickwork is not utilized to its optimum. Further, the reinforced brickwork flexural members crack very early and to keep those cracks within acceptable limits the stress in steel must be kept low – and inefficient use.

These disadvantages can be overcome by applying the technique of prestressing, developed for reinforced concrete, to brickwork. In practice [4], brickwork has been prestressed in very limited cases to enhance the ability of a wall to resist lateral loads [5,6] or to increase shear resistance of walls. This does not represent a radical departure from the traditional form of construction in which masonry is largely used as a compression member. So far, except for the construction of a brickwork water tank [7], no serious use has been made of prestressed masonry as a structural material carrying the load primarily in bending. This is mainly due to lack of data of the behaviour of prestressed brickwork. A comprehensive investigation [8] in full-scale was therefore undertaken to study the influence of the following variables:

- brick strength,
- mortar grade,
- shear span/effective depth ratio

on the ultimate load behaviour of prestressed brickwork beams. The majority of the bricks used in the investigation were three-hole bricks with a percentage of perforations equal to 11% of their volume.

Perforated bricks account for 30 percent of the total production of bricks in the UK, but although their use is increasing due to certain advantages – saving of fuel and the conservation of resources – no performance data is available regarding their use in prestressed brickwork beams. In view of this a supplementary investigation was carried out to evaluate the effect of different percentages of perforations on the ultimate load behaviour of prestressed brickwork beams.

Experimental details

Materials

All materials were tested according to the relevant British Standards.

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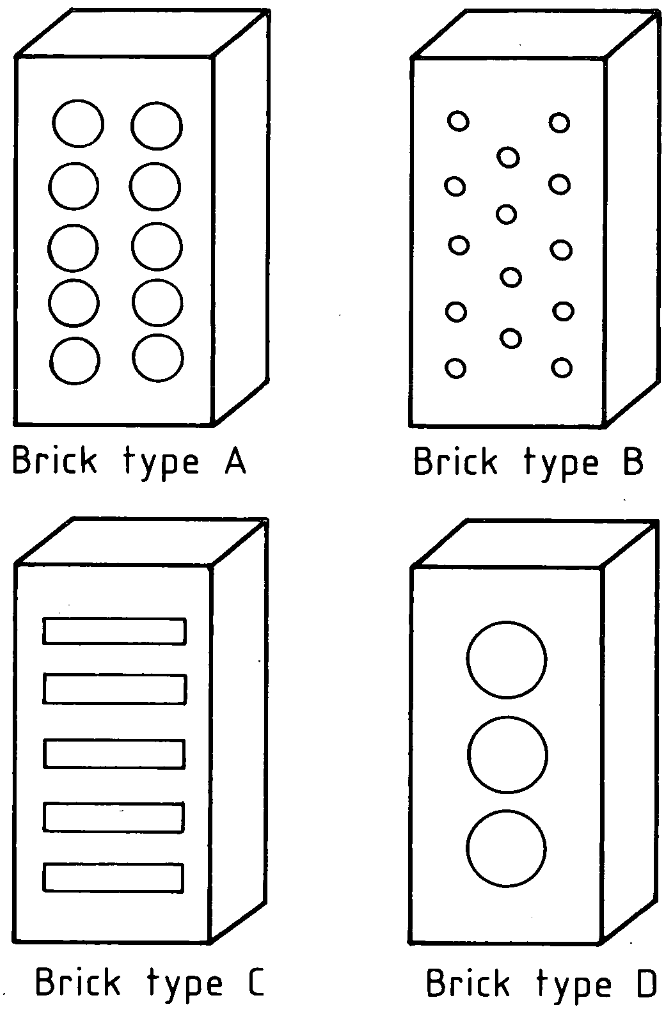

Fig 1 Types of bricks

Bricks: Four different types of bricks were used. Three bricks, designated A, B and C, all had areas of perforations greater than 26% with different patterns of perforations (Fig 1). The fourth brick, D, was three hole with perforations equal to 11% of its volume. Compressive strength tests were carried out in all three orthogonal directions. The 24-hour water absorption test was also carried out. The results are given in Table 1.

Mortar: 1:0.25:3 (cement:lime:sand) mortar was used for all the tests. 100mm control cubes were taken and tested at 28 days. The resulting compressive strength of mortar for each beam is presented later in Table 4.

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Table 1 Properties of bricks

Type of Brick	% of perforation with respect to volume	Compressive strength									24 hr. water absorption in %
		Flat			Edge			End			
		Average comp. strength N/mm²	S.D. N/mm²	Coeff. of variation %	Average comp. strength N/mm²	S.D. N/mm²	Coeff. of variation %	Average comp. strength N/mm²	S.D. N/mm²	Coeff. of variation %	
A (10-hole)	23.14	70.19	5.65	8.05	29.54	4.33	14.66	20.11	3.20	15.91	5.41
B (14-hole)	21.32	74.33	8.03	10.08	26.22	3.50	13.35	10.32	2.18	21.12	21.32
C (Slotted)	20.01	64.08	11.23	17.52	51.84	7.07	13.64	12.50	1.99	15.9	20.01
D (3-hole)	12.17	82.03	5.85	7.13	53.17	9.43	17.73	40.23	6.94	17.25	4.17

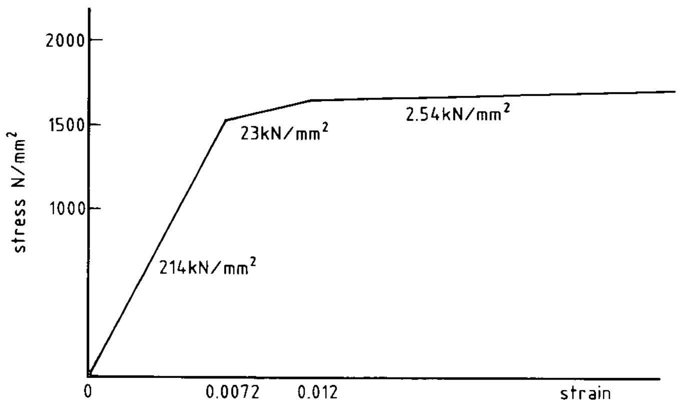

Fig 2 Idealised stress-strain curve for prestressing steel

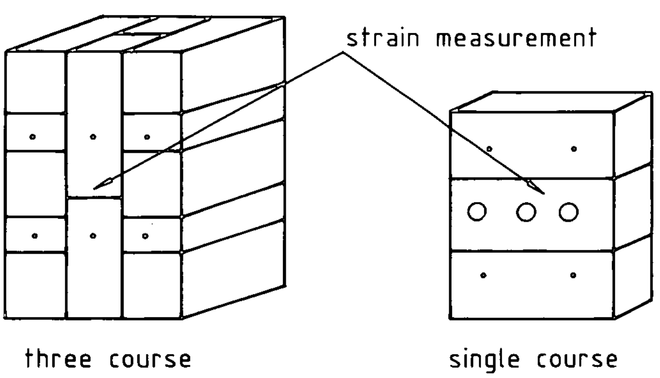

Fig 3 Brickwork test prisms

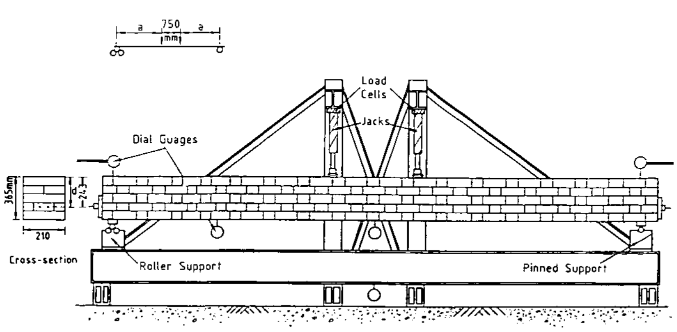

Fig 4 Showing the cross-section of the beam and test set up

Grout: The same sand and cement as used for the mortar was used for the grout. A 1:2 (cement:sand) mix was used. A platiciser ('conbex') was added to the mix to allow a reduction of the w/c ratio and hence the shrinkage of the grout. 100 mm cubes were taken during each grouting operation and tested at 7 days. The compressive strength of the grout is given later in Table 4 for each beam.

Prestressing strand

Seven wire, stabilised, prestressing strands were used throughout. The nominal diameter was 10.9 mm and the cross-sectional area was 72 mm². The strand was tested and the 0.2% proof stress was 1642 N/mm² and the ultimate tensile strength was 1708 N/mm². The experimental stress/strain relationship was mathematically idealised in trilinear form as shown in Fig 2.

Brickwork properties

To determine the strength and deformation properties of the brick, two test specimens were chosen (Fig 3). The first of these represents the top three courses of a prestressed brickwork beam and the other, the top course. The prisms were capped and levelled using a rich mortar mix. Plywood sheets (6 mm thick) were then placed between the prisms and the platens of the test machine. During the tests strain measurements were taken at six points on the three course prisms and four points on the single course prisms, (Fig 3) using a 'demec' gauge. The load was applied at small, equal increments. At each increment the load was held constant while strain measurements were taken. At approximately 70% of the failure load the increment was halved. Axial loading was ensured at the beginning of the test by checking the strain measurements for uniform strain.

Construction of beams and testing procedure

The section of the beam was chosen so that no special skill was required of the bricklayer. It was felt that the following factors must be taken into account in designing the sections:

- effective utilisation of as much ceramics as possible
- ease of grouting tendons
- ease of construction
- same bonding pattern as used in brick wall
- provision of cavity so that tendons may be placed at required depth.

The resulting section and typical beam details are given in Fig 4.

All beams were built by an experienced bricklayer on the floor of the testing laboratory. They were then allowed to cure for 21 days. The tendon was placed in the preformed cavity and mild steel anchorage plates were attached. After prestressing the beams were grouted. This was facilitated by building teams in an inverted position, enabling the grout to be poured through the perforations on the top course (as built). The beams were then left to cure for a further 7 days prior to testing.

Beams were tested in a two point loading rig (Fig 4) which provided a pin and a roller support. The loads were applied by means of hydraulic jacks and measured at the jacking points using load cells attached to a digital voltmeter and penchart recorder. The loads were applied in increments up to failure. The deflections were measured using dial gauges. Strains were recorded on the brickwork using a 'demec' gauge

Table 2 Compressive strength of brickwork prisms

Type of Brick	Compressive strength N/mm²			
	Single course prisms Test results	Single course prisms Average	Three course prisms Test results	Three course prisms average
A (10-hole)	16.06		14.46	
	14.24	15.16	13.16	14.50
	15.18		15.87	
B (14-hole)	9.12		10.38	
	9.30	9.0	14.27	12.68
	8.59		13.30	
C (Slotted)	28.44		17.39	
	27.32	27.89	14.06	15.55
	27.91		15.21	
D (3-hole)	–	32.0 (from ref. 10)	–	20.8 (from ref. 10)

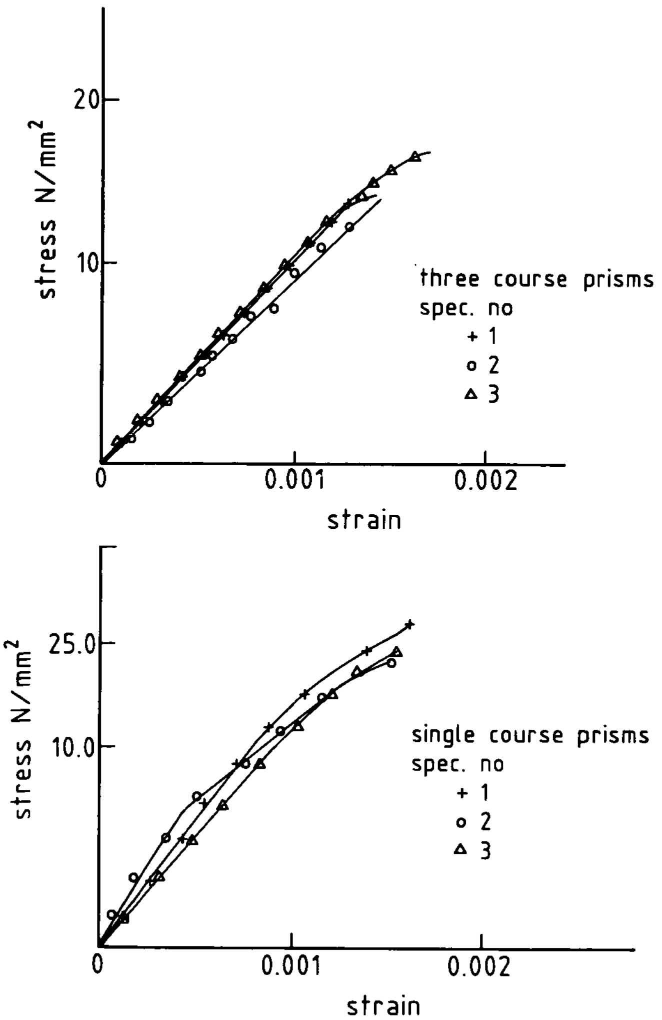

Fig 5 Stress-strain relationship for brick type A

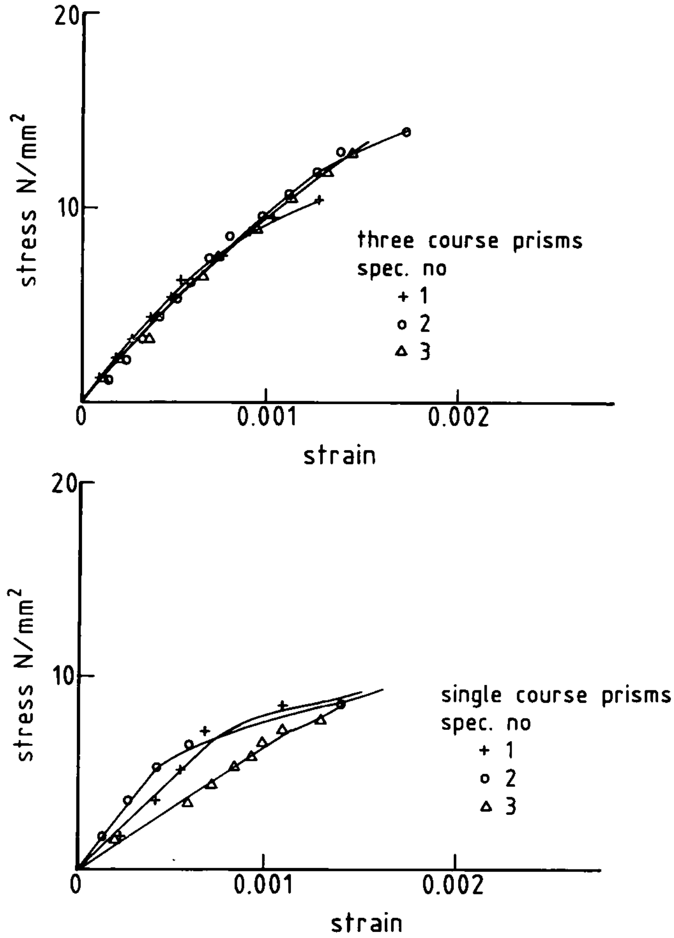

Fig 6 Stress-strain relationship for brick type B

and crack widths were measured using an 'ultra lomara microscope' capable of measuring widths down to 0.02mm.

Results and discussion

Compressive strength of brickwork

Table 2 presents the compressive strengths of the brick prisms tested. For brick types A, C and D the greatest compressive strength was attained by the single course prisms. This is particularly marked in the cases of bricks C and D where the single course prisms have between 60 and 80 percent greater compressive strength. The reverse is true for brick type B in which the compressive strength of the three course prisms was 41 percent greater than the single course prisms. Brick type B was the brick with the greatest number of perforations, 14 holes (Fig 1). During the tests on the single course prisms it was noticed that cracking occurred around the perforations in the bricks. For brick types A, C and D, once this cracking occurred the prism was able to sustain further load. However in the case of brick type B, once these cracks formed total collapse of the prism occurred very shortly afterwards.

Stress/strain relationship of brickwork

The experimental stress/strain relationships for brick types A, B and C are given in Figs 5–7. The stress/strain relationships for brick type D has been presented elsewhere [11]. Initially the stress/strain relationship was linear, after which the strain increased more rapidly than the stress. The stress/strain relationships for brick type A for both prism types are very similar. The single course prisms built in brick type C failed at much higher strain than the three course prisms. Incidentally, the failure strain recorded in both the prisms for type C bricks were the highest of all bricks used in this test.

The results of the stress/strain relationships are summarised in Table 3. Also given in this table is the stress/strain relationship for brick type D [10]. The strains were measured up to 90-95% of the ultimate load. The stress/strain relationship for each prism type was mathematically extrapolated to produce the ultimate strain for each prism tested. In all cases (Table 3) it may be seen that the single-course prism underwent greater strains than the three-course prisms; the greater variation between ultimate strains for

Table 3 Stress/strain relationship of brickwork

Prism type	Brick type	Average compressive strength N/mm^2	Ultimate strain	x_1	x_2	x_3	x_4	r^2	λ_1	λ_2
Three	A	14.50	0.00145	0.00	1.03	0.13	−0.16	1.00	0.52	0.34
Single	A	15.16	0.00160	0.01	1.52	−0.58	0.05	0.99	0.58	0.36
Three	B	12.68	0.00142	−0.02	1.45	−0.42	−0.01	0.99	0.58	0.35
Single	B	9.00	0.00147	−0.02	2.59	−3.75	2.19	0.92	0.59	0.38
Three	C	15.55	0.00217	−0.01	1.70	−0.93	0.24	0.98	0.60	0.36
Single	C	27.89	0.00381	0.09	1.10	−0.11	+0.0028	0.97	0.51	0.33
Three	D	20.48	0.00205	−0.011	−1.74	−1.00	0.025	0.99	0.61	0.37
Single	D	32.56	0.00326	−0.05	2.37	−2.09	0.78	0.99	0.65	0.39

Table 4 Summary of tests on beams built from perforated bricks (0.274% steel, span 6.2m)

Beam	Brick type	Mortar strength N/mm^2	Grout strength N/mm^2	P S force kN	Ultimate moment kNm	Shear stress at failure N/mm^2	failure mode	Experimental/theoretical moments single	three
10/1		23.8	15.0	151	50.9	0.39	flexure	1.11	1.13
10/2	A	24.6	15.0	150	52.3	0.40	flexure	1.14	1.16
10/3		23.8	15.0	134	57.1	0.44	flexure	1.23	1.25
14/1		23.3	18.0	154	54.5	0.42	flexure	1.52	1.25
14/2	B	24.3	18.0	150	52.6	0.41	shear	1.47	1.21
14/3		23.8	18.0	130	51.3	0.40	shear	1.46	1.20
5/1		24.1	15.2	152	61.0	0.47	flexure	1.16	1.32
5/2	C	23.5	15.2	151	57.1	0.44	shear	1.09	1.24
5/3		20.7	15.2	136	55.2	0.42	shear	1.04	1.19

Note: The shear stress at failure has been calculated on the basis of V/bd irrespective of mode of failure.

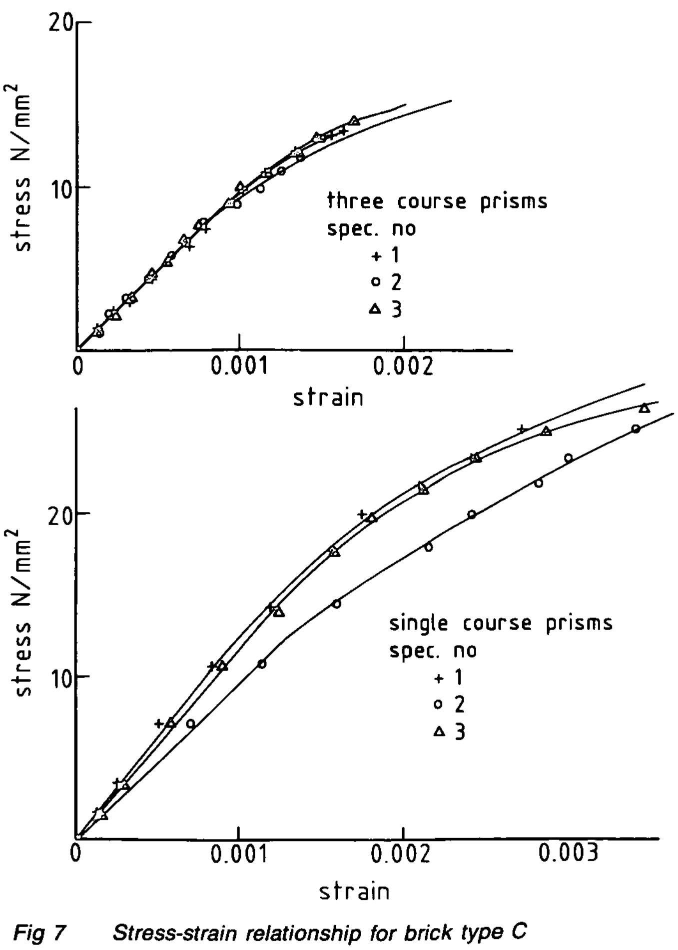

Fig 7 Stress-strain relationship for brick type C

prism types appears to coincide with the greatest variation in compressive strengths, namely in brick types C and D.

The stress/strain relationships for each prism and brick type have been mathematically idealised in the form of a non-dimensional polynomial, such that:

$$\frac{\sigma}{\sigma_m} = x_1 + x_2 \frac{\epsilon}{\epsilon_m} + x_3 \left(\frac{\epsilon}{\epsilon_m}\right)^2 + x_4 \left(\frac{\epsilon}{\epsilon_m}\right)^3 \quad (1)$$

The column in Table 3 presents the correlation coefficient for an idealised curve and is a measure of how well the curve fits the experimental results; for perfect correlation $r^2 = 1.0$.

Using Equation (1) with the relevant data from Table 3 it is possible to determine the stress block factors λ_1 and λ_2 for each prism tested. λ_1 is the ratio of the areas under the non-dimensional stress/strain curve to the area of an enclosing rectangle, and is also the ratio of the average compressive stress in the compression zone to the compressive strength of brickwork. λ_2 is the position of the resultant thrust of the compressive forces and is the ratio of the depth of the centroid from the extreme fibre to the neutral axis depth. From Table 3 it may be seen that λ_1 for the three-course prism in brick type A and the single-course prisms in brick type C is only slightly greater than 0.5. Referring to Figs 5 to 7 it can be seen that as the stress/strain relationships for these two sets of prisms are more or less linear up to failure then λ_1 would equal 0.5 and λ_2 would equal 0.33. The stress block factors for these highly perforated bricks are slightly lower. They lie within the range obtained [11,12] for solid and three-hole bricks and hence the idealised stress-strain relationship obtained with a large number of test results, including these, were used for the prediction of m-ϕ relationship:

$$\frac{\sigma}{\sigma_m} = 2.26 \frac{\epsilon}{\epsilon_m} - 2.09 \left(\frac{\epsilon}{\epsilon_m}\right)^2 - 0.83 \left(\frac{\epsilon}{\epsilon_m}\right)^3 \quad (2)$$

Fig 8 A typical flexural failure

Fig 9 Shear failure of beam

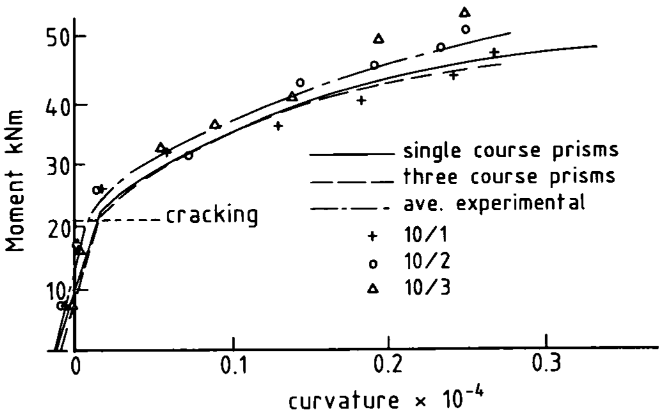

Fig 10 Moment-average curvature relationship (beams of brick type A)

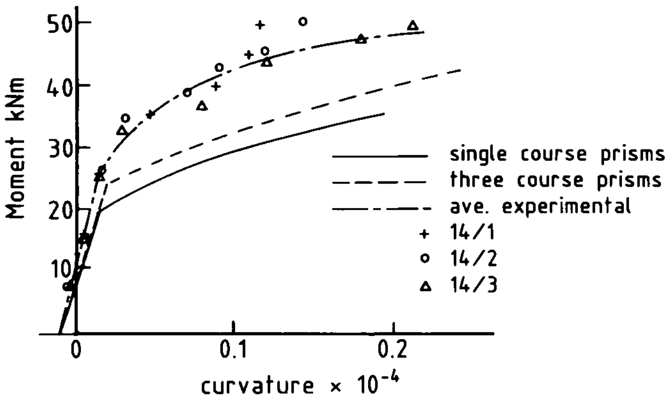

Fig 11 Moment-average curvature relationship (brick type B)

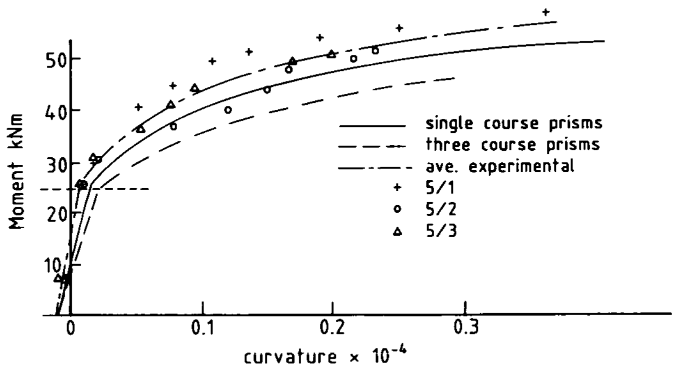

Fig 12 Moment-average curvature relationship (beams of brick type C)

Similarly, the ultimate flexural strength of the beam was predicted using the average [11,12] stress block characteristics for brickwork.

Ultimate strength of beams built from perforated bricks

Table 4 presents the results of the tests on the beams built from brick types A, B and C. All beams were tested over a 6.2 m span and had 0.274 percent of steel. Previous work [8,10] had shwon that beams built from brick type D with the same prestress and steel area failed in flexure. From Table 4 it can be seen that the only series of beams that consistently failed in flexure were those built in brick type A. A typical flexure failure is shown in Fig 8. Yielding of steel occurred followed by crushing of the compression zone in the constant moment region of the beam.

For the other two groups, two of the three beams failed in shear with splitting along the top bedjoints running into the supports, as in Fig 9. It should be noted, however, that there was little reduction in the ultimate moment of these beams over those that failed in flexure.

From Table 4, considering where a flexural failure occurred, it can be seen that with brick types A and C the single-course prisms predict higher moments than the three-course prisms, which are in closer agreement with the experimental results. This trend was also noted in the case of brick type D beams with the same steel content [10]. For brick type B the three-course prisms provide a closer estimate of the flexural strength.

Moment-curvature relationship

Figures 10 to 12 show the average m-ϕ relationships, obtained experimentally for the three different brick types A, B and C respectively. The curvatures were obtained from strain measurements in the brickwork, and prestressing strand, using the following expression,

$$\phi_{ave} = \frac{\epsilon_t + \epsilon_{st}}{d} \qquad (3)$$

where ϕ_{ave} is the average curvature, ϵ_t is the compressive strain in the outermost fibre of the beam and ϵ_{st} is the additional strain in the steel. The general characteristics of the m-ϕ relationship were similar for all beams. Initially, there is a negative curvature caused by the prestressing. As the load increases the curvature eventually becomes positive, at this stage there is a linear variation in curvature with moment. Further loading causes the section to crack after which the curvature increases much more rapidly with moment up to failure of the beam.

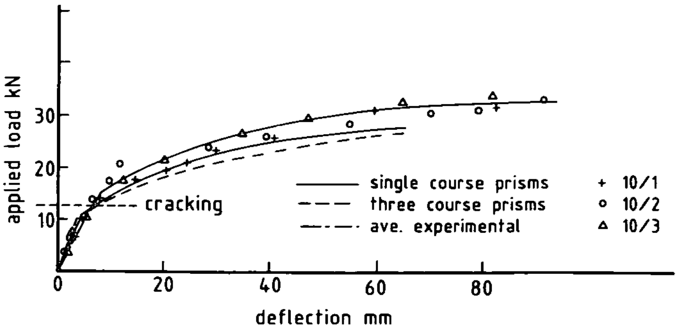

Fig 13 Load-deflection relationship (beams of brick type A)

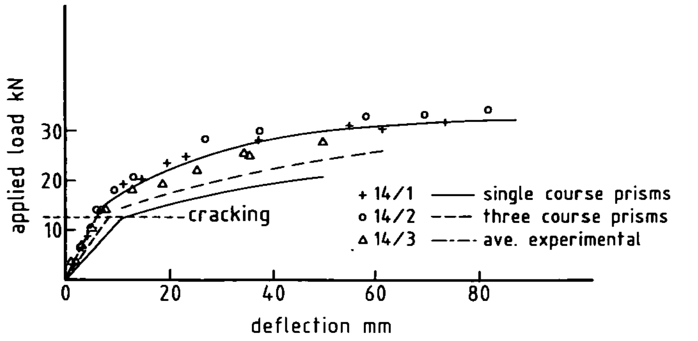

Fig 14 Load-deflection relationship (beams of brick type B)

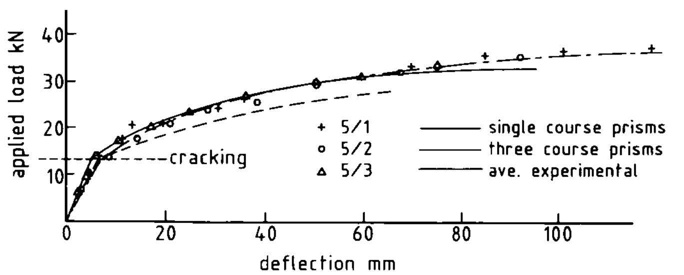

Fig 15 Load-deflection relationship (beams of brick type C)

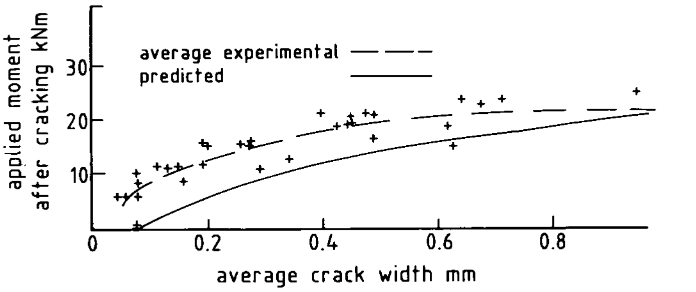

Fig 16 Relationship between moment and average crack width

The experimental m-ϕ relationships were compared with theory using a method developed elsewhere [10] and which allows for nonlinear material behaviour, cracking and tension stiffening. The compresive strengths for each brick type from both three-course and single-course prisms were used in conjunction with Equation (3) to predict the m-ϕ relationship.

For brick types A and C the compressive strength from the single-course prism more accurately predicts the curvatures than the three-course prisms and very good correlation is attained (Figs 10 and 12). Both the single and three-course prisms tend to overestimate the curvatures for brick type B and in this case better correlation is obtained from the three-course prisms.

Load deflections

Using the m-ϕ relationship it is possible to calculate the load/deflection response of prestressed brickwork beams. The experimental load/deflection response shows similar characteristics to the moment/curvature relationships, as shown in Figs 13 to 15. There is a linear variation of deflection with load up to the point where cracking occurs, after which the deflections increase much more rapidly with load.

The predicted deflections show very good agreement with the experimental results, especially for brick types A and B. For these two groups of beams the single-course prisms predict the deflections more accurately than the three-course prisms. The predicted deflections for the beams built from brick type B overestimate the experimental deflections which is a direct consequence of the overestimated curvatures (Fig 11).

The process of calculating the m-ϕ relationship and the load deflection response has been carried out by means of a computer program which is described elsewhere [10].

Cracking

Cracking occurred in the beams once decompression of the prestress had taken place and the flexural tensile strength of the brickwork was exceeded. Crack widths were measured in the constant moment zone of the beams (Fig 16). Once cracking starts the crack width increases very rapidly with further loading. The experimental average crack widths were compared with average crack widths obtained from the following expression [10].

$$N_{ave} = (N_j + 0.41)\, b_j\, \epsilon_{smb} \qquad (14)$$

where N_j is the number of joints between cracks, and is dependent on the neutral axis depth after cracking and the cover to the strand, in this case, equal to 2.0; b_j is the distance between the joints; and ϵ_{smb} is the average strain at the level of the crack. Equation (4) overestimates the crack widths slightly and so provides a safe estimate.

Comparison of results with three-hole bricks

It would be difficult to compare the results exactly, because the compressive strength of bricks with various degrees of perforations was different. The area of prestressing steel was constant, but the effective prestress was also slightly different because of loss of prestress before testing. However, this difference was about 7% and can be neglected as it would not affect the ultimate moment.

Table 5 gives the result of this comparison with a three-hole brick with similar compressive strength. It appears that the type and degree of brick perforations are not detrimental to the ultimate moment carrying capacity of the beam. In most cases failure was initiated due to the yielding of steel. As the section was under-reinforced the compressive forces that may develop are dictated by the area of steel. The high degree of quality control during manufacture of the steel ensures that its properties do not vary greatly. Thus the compressive forces in an under-reinforced beam will also not be subject to large variations. Hence, although the brick strength may be prone to considerable variations, the flexural strength of under-reinforced prestressed brickwork beams will not undergo the same degree of variation. The slight variation in ultimate moment for different strength of bricks may be due to strain hardening of steel. The differences may also be due to variation in compressive strength of brickwork resulting in different lever arms.

Table 5 Compaction of ultimate moment of various bricks having different degrees of perforations

Mortar 1:¼:3

Brick strength & type N/mm²	% Perforation	Average effective prestress kN	Ultimate moment kNm
70.19 A-10 hole	23.14	145.0	53.4
74.33 B-14 hole	21.32	144.7	52.8
64.08 C-slotted	20.17	146.3	57.8
68.0* three hole	11.0	138.2	53.54

* ref. 10

Conclusions

The ultimate load-carrying capacity of the prestressed beam with constant percentage area of steel is not affected by the different types and degree of perforations of the constituent bricks for perforations ranging from 11 to 23.14 percent of their volume.

The results of the single-course prism tests gave the best prediction of ultimate strength and flexural behaviour for brick types A, C and D but underestimate the ultimate strength of beams built with brick type B.

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Acknowledgements

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International Survey

Durability of cement-based materials

New high-performance cement-based materials are more durable than ordinary concrete, but still have shortcomings that limit their use, according to materials scientist Leslie J Struble at a Materials Research Society meeting. Struble summarized what is known about the durability of these new cements.

She said that durability refers to how well a material performs over the long term, how it responds to its environment and its ability to resist corrosion. Strength and toughness refer to a material's ability to withstand crack growth. One specialty material that scientists are investigating is known as MDF, or macro-defect-free cement. MDF is a mixture of cement, water and organic polymer – processed to provide high strength and toughness. Researchers in the Centre for Advanced Cement-Based Materials, a National Science Foundation centre at Northwestern University and the University of Illinois are investigating this material.

A problem with MDF, said Leslie Struble, is that it weakens on exposure to water. "It is not very permeable, but the water does get absorbed by the polymer," she said. "When it gets wet, the organic polymer becomes weak and rubbery. This change causes the MDF to lose strength."

Another high-strength specialty material is known as DSP, or densified with small particles. DSP, a mixture of cement, water and submicron-size silica particles, is ten times as strong as ordinary concrete. Although not permeable, it is susceptible to microcracks, Struble said. "If the crack gets big enough, unwanted chemicals can come in from the outside. You do not want concrete to crack."

"These problems, if not prevented, seriously limit the usefulness of these materials. Considerable research is required before the long-term performance of these cement-based materials can be assured."

In addition to specialty materials, scientists are investigating construction materials. Ultra-high strength concrete for construction can be made in the lab to be much stronger than ordinary concrete, but it still is brittle. Several researchers also are looking at fundamental properties of DSP and MDF and its components, polyvinyl alcohol and calcium aluminate. "At the Centre for Advanced Cement-Based Materials, we're trying to understand in detail how the two components of MDF interact chemically and physically," said J. Francis Young, director of the centre at the University of Illinois at Urbana-Champaign.

Building terminology

Several new parts have been published to BS 6100 *Glossary of buildings and civil engineering terms*.

BS 6100: Section 1.3 *Parts of construction works:* Subsection 1.30: 1991 *External works:* Gives definitions for external works including fencing and other barriers, landscape features, types of plants used in landscape work, and associated operations.

BS 6100: Section 1.5 *Operations; associated plant and equipment:* subsection 1.5.5:1991 *Plant equipment* gives definitions for tools, plant and equipment used in construction.

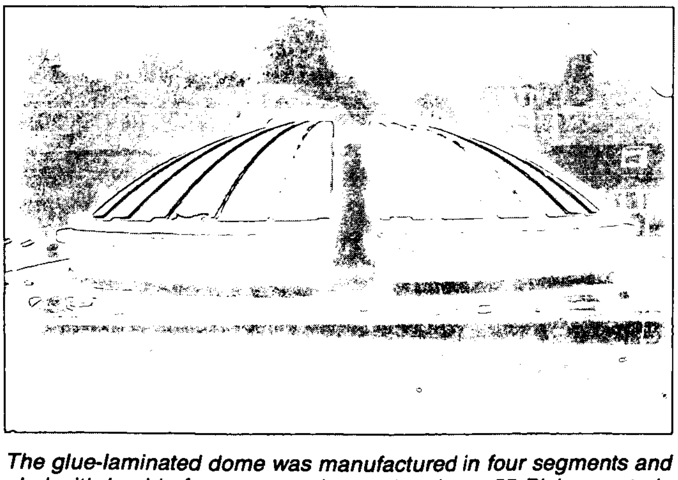
BS 6100: Part 6 *Concrete and plaster:* Section 6.6 *Products, applications and operations:* Subsection 6.6.1:1991 *Concrete and mortar* gives definitions for terms relating to products, properties, operations such as testing, and equipment relating to concrete and mortar, including reinforcement.

Glue-laminated dome in London

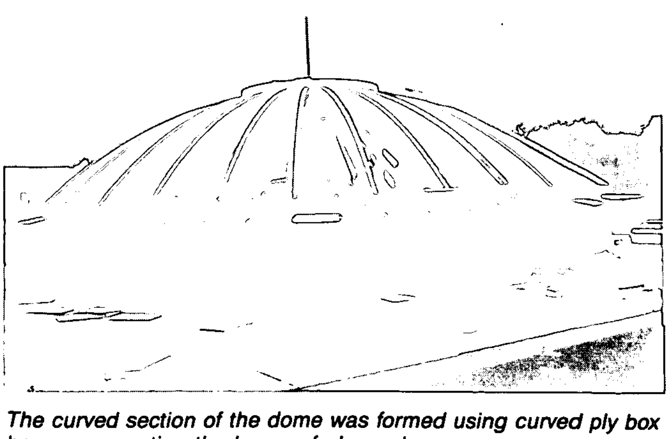
A recently-erected structure in the City of London is an impressive glue-laminated dome. Prefabricated in four sections off site, the 9.6m diameter lead-clad dome was designed and constructed by John Hirm Construction for erection on the eighth floor of the building.

The dome, with its 1.4m-high perimeter wall was built in four sections. A 300mm × 9mm glulam timber beam of the required radius forms the bottom tension ring. The perimeter walls were constructed on this beam using 100mm × 50mm CLS hemlock studs. Springing from the bottom glue-laminated ring are 32 rafters, 150mm × 50mm, supporting a 2m diameter top compression ring.

Plywood web beams with curved top flanges span from the top of the perimeter wall to the compression ring, thus supporting the three layers of 6mm plywood which in turn form the curved dome surface.



The glue-laminated dome was manufactured in four segments and clad with lead before transporting to the site at 55 Bishopsgate in the City of London



The curved section of the dome was formed using curved ply box beams supporting the layers of plywood

Deformation and cracking of post-tensioned brickwork beams

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Synopsis
The paper presents a theoretical method for calculating the moment-curvature and thus the load-deflection relationships of post-tensioned brickwork beams. The theoretical method takes into account the various sources of non-linearity in brickwork behaviour, e.g. stress strain relationship, tensile cracking, and tension stiffening. A theoretical expression for predicting the crack width in terms of crack spacing and average strain at the level of crack is also described.

The theoretical results are compared with the test results of post-tensioned brickwork beams built from different brick strengths (low, medium, and high) with various % of steel. A very good correlation is obtained between the experimental and theoretical results.

Notation

σ_1 or σ_2	is the stress
p	is the prestressing force
A	is the cross-sectional area
e	is the eccentricity of prestress
z	is the sectional modulus
C	is the compressive force
b	is the width of the section
d	is the effective depth of prestressing steel
h	is the total depth
n	is the neutral axis depth
E'	is the initial modulus of brickwork
E	is the modulus of elasticity of brickwork
E_s	is the modulus of elasticity of steel
ϕ	is the curvature
f_s	is the stress in steel at crack
f'_s	is the stress in steel away from crack
T_s	is the tensile force in steel
T_m	is the tensile force in masonry
A_{ps}	is the area of steel
f_r	is the modulus of rupture of brickwork
M_{cr}	is the cracking moment
k	is the tension stiffening factor
b_j	is the distance between vertical cross-joints
f_{scr}	is the steel stress at crack at cracking moment
$F_m(\varepsilon)$	is the stress/strain relationship of brickwork
$F_s(f_s)$	is the stress/strain of prestressing strand
h_{cr}	is the initial height of crack
l_a	is the lever arm
n_j	is the no. of joints
S_m	is the average crack spacing
ε	is the strain in brickwork
ε_1, ε_2	is the strain top and bottom fibre of beam
ε_{p1}, ε_{p2}	is the prestress strain in the brickwork
ε_s	is the strain in strand caused by applied loads
εs_{am}	is the average additional strain in strand after cracking
εs_{mb}	is the average strain after cracking at any level of beam.

Introduction

In recent years, both research workers and designers[1,2] have shown a growing interest in prestressed brickwork. This trend has been further reflected by the new Code of Practice for the use of reinforced and prestressed masonry[3]. For the design of prestressed brickwork beams, it may be economically advantageous to allow the prestress force to be neutralised and limited cracking to occur under working load, in a similar fashion to class 3 members in prestressed concrete design[4]. However, at present, there is a complete dearth of information regarding deformation and cracking of prestressed brickwork beams. Hence a study of the deformation and cracking characteristics was undertaken as a part of an investigation into the behaviour of post-tensioned brickwork beams[5].

A theoretical method for calculating load-deflection was developed, capable of taking into account the various sources of non-linearity in brickwork behaviour such as stress-strain relationship, tensile cracking, and tension stiffening. A theoretical expression for predicting the crack width in terms of crack spacing and average strain at the level of the crack was also developed and compared with the experimental results.

Theoretical determination of m-ϕ and load-deflection relationships

The method employs the actual stress/strain relationships for both the brickwork and the prestressing steel to calculate the moment and curvature in the prestressed brickwork beams. The moment-curvature relationship is then used to calculate the deflection. The experimental stress/strain relationship of brickwork is mathematically idealised in the form of a polynomial as shown in Fig 1. The experimental stress/strain relationship of the prestressing strand is idealised in a trilinear form (Fig 2). The main assumptions made are:

—The strain distribution through the section is linear throughout the loading history.
—Full bond exists between the steel, grout, and brickwork.
—The stress/strain relationship of brickwork in tension is linear.

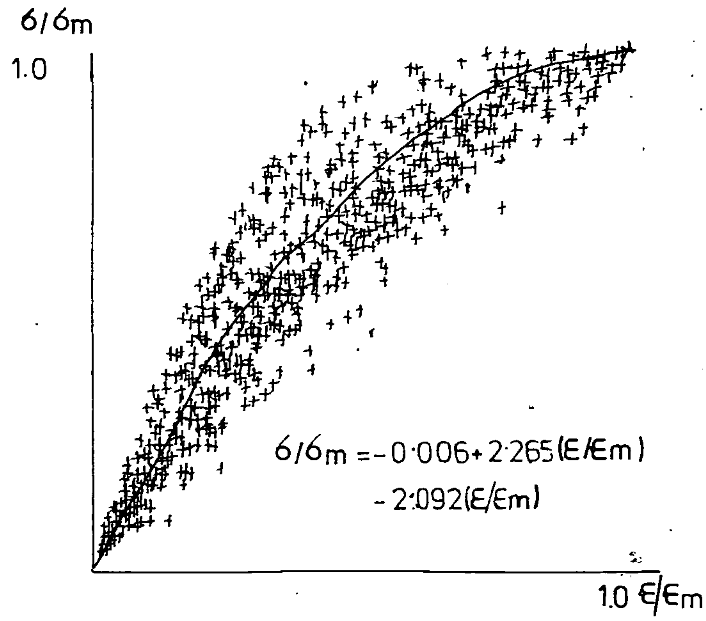

Fig 1. Non-dimensional stress/strain relationship for brickwork

A similar approach has been used to study prestressed concrete beams[6,7] where concrete behaviour was assumed linear elastic up to cracking. However, in this method the actual non-linear stress-strain relationship for brickwork has been used for the complete loading history. The applied loading is considered in three stages:

—prestressing
—prestressing to cracking
—post-cracking to ultimate load

Prestressing

Fig 3 shows the distribution of stresses and strain in a prestressed brickwork beam due to prestressing. Initially, linear elastic behaviour is assumed and the stresses in the outermost fibres are therefore

$$\sigma_1' \text{ or } \sigma_2' = P/A \pm \frac{Pe}{z} \qquad \ldots (1)$$

Assuming an initial value of elastic modulus, E', the corresponding extreme fibre strains, ε_{p1} and ε_{p2} can be found. The total compression force in the section is then

$$C = b. \int_n^h F_m(\varepsilon)\, d(x) \qquad \ldots (2)$$

$$\text{where } \varepsilon = \varepsilon_{p1} + (\varepsilon_{p2} - \varepsilon_{p1})\frac{x}{h} \qquad \ldots (3)$$

$F_m(\varepsilon)$ is the stress/strain relationship in compression.

$$\text{Let } r_t = \frac{\varepsilon_{p1}}{\varepsilon_{p2}} \qquad \ldots (4)$$

The resultant compressive force in the section must be equal to the applied prestress force, i.e.

$$C = P \qquad \ldots (5)$$

If eqn. (5) is not satisfied, using the initial assumption in eqns. (1) and (2), the initial value of elastic modulus is modified:

$$E = E'\left(\frac{C}{P}\right) \qquad \ldots (6)$$

The strain ratio, r_t is always kept constant but the magnitude of ε_{p1} and ε_{p2} is changed according to the revised value of E. Eqns. (2), (5), and (6), are applied until eqn. (5) is satisfied. The curvature due to prestress is then:

$$\phi_p = \frac{\varepsilon_{p1} - \varepsilon_{p2}}{h} \qquad \ldots (7)$$

$m - \phi$ relationship up to cracking

Fig 4 shows the distribution of strain and stress prior to cracking. Cracking will take place once decompression of prestress and the flexural tensile strength of the brickwork at the extreme fibre is exceeded. The actual strain distribution may be considered as the sum of the strains due to prestress and applied load (Fig 5). The additional strain necessary to cause decompression of the bottom fibre must be equal and opposite to that initially applied. At this stage in the loading, there is zero strain at the soffit. Further load will cause tension to develop. The ultimate tensile strain of the brickwork ε_r was obtained from the modulus of rupture, f_r

$$\varepsilon_r = \frac{f_r}{E'} \qquad \ldots (8)$$

Hence the total applied strain required to cause cracking in the extreme fibre, from prestressing

$$\varepsilon_{cr} = \varepsilon_{p2} + \varepsilon_r \qquad \ldots (9)$$

The total compressive force in the section is

$$C = b \int_o^n F_m(\varepsilon)\, dx \qquad \ldots (10)$$

$$\text{where } \varepsilon = \varepsilon_1 - \left(\varepsilon_1 \frac{x}{n}\right) \text{ and } n = \left(\frac{\varepsilon_1}{\varepsilon_1 + \varepsilon_2}\right)h$$

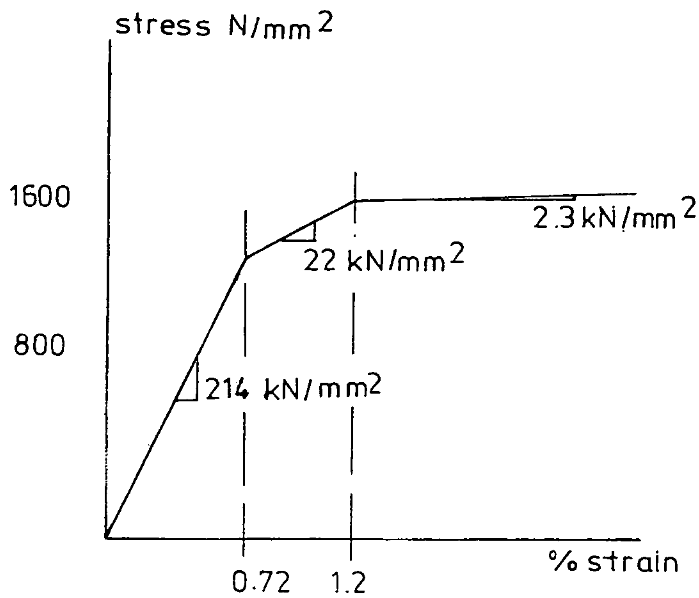

Fig 2. Idealised stress/strain relationship for strand

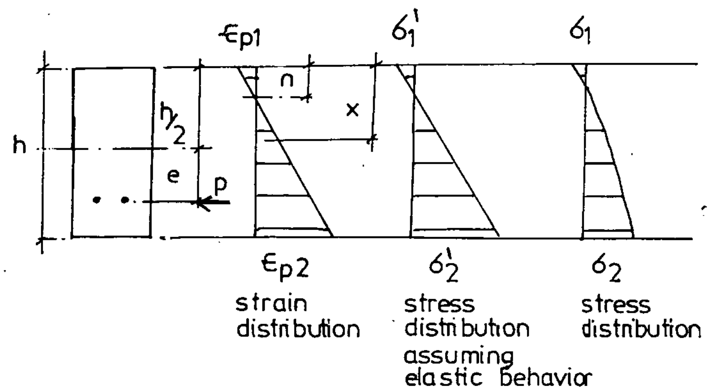

Fig 3. Stress conditions in beam due to prestress

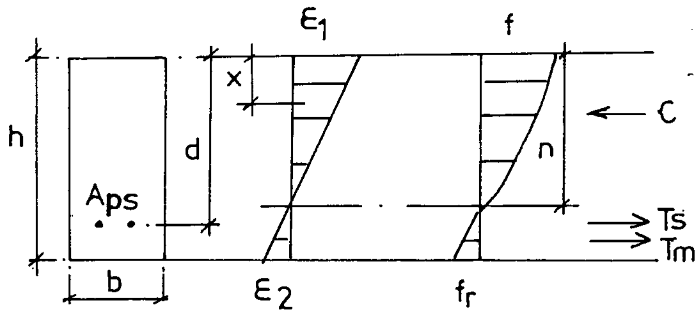

Fig 4. Conditions prior to cracking

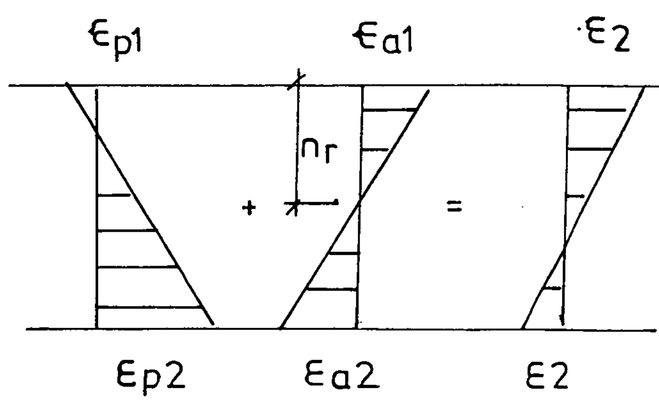

Fig 5. Strains in beam at cracking

The total strain in the steel ε_s is equal to the strain due to prestress, ε_{ps} and the strain due to applied loads, ε_{sa}.

$$\varepsilon_s = \varepsilon_{ps} + \varepsilon_{sa} \quad \ldots(11)$$

The force in the steel is then

$$T_s = A_{ps} F_s(\varepsilon_s) \quad \ldots(12)$$

The tensile force in the masonry is

$$T_m = (h-n)f_r \frac{b}{2} \quad \ldots(13)$$

for equilibrium,

$$C = T_s + T_m \quad \ldots(14)$$

The centroid of compression may be found by taking moments about the soffit,

$$l_a = \frac{[\cdot b \int_0^n F_m(\varepsilon)(h-x)dx]}{C} \quad \ldots(15)$$

The cracking moment is then

$$M_{cr} = C.l_a - T_s(h-d) - T_m(h-n)/_3 \quad \ldots(16)$$

The curvature immediately prior to cracking is

$$\phi = (\varepsilon_1 - \varepsilon_2)/h \quad \ldots(17)$$

Because of the non-linear stress/strain relationship adopted for brickwork, the modular ratio between the brickwork and steel will change with the applied loading from prestressing up to cracking, n_r in Fig 6.

To take account of this the strain at the soffit level of the beam is applied in increments up to ε_{cr}. An initial value of n_r is obtained based on the uncracked transformed section. From this the strain in the uppermost fibre is found. Eqns. (10) – (13) are then applied and if (14) is not satisfied n_r is modified and the strain in the top fibre recalculated. The process is repeated until equilibrium is achieved. The moments and curvatures are then found for each increment from eqns. (16) and (17).

Moment curvature relationship after cracking at a crack

After cracking, the crack is assumed to extend up to the neutral axis depth. Because of the tensile strength of the brickwork, the depth of crack penetration will actually be slightly lower. However, the influence of this on the moment is minimal and ignored in this analysis. At the point of cracking, there are two possible conditions which the beam may exhibit, cracked and uncracked. The uncracked state was defined in the previous section. As the tensile strength of the brickwork has been exceeded in the cracked state, there must be an increase in steel stress to accommodate the same moment. Eqn. (14) becomes

$$C = T_s \quad \ldots(18)$$

or

$$b \int_0^n F_m(\varepsilon)\,dx = A_{ps} F_s(\varepsilon_{sa} + \varepsilon_{po}) \quad \ldots(19)$$

where $\varepsilon = \varepsilon_1 - (\varepsilon_1 \cdot x/n)$

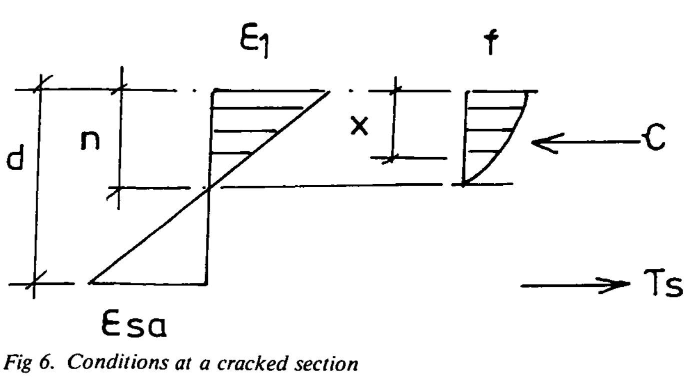

Fig 6. Conditions at a cracked section

The moment at cracking is given as

$$M_{cr} = C \cdot l_a - T_s(h-d) \quad \ldots(20)$$

The strains ε_{sa} and ε_1 in Fig 6 are not readily known but are obtained by simultaneous solution of eqns. (19) and (20).

The $M-\phi$ relationship from cracking up to ultimate is obtained by applying compressive strains to the top fibre in increments up to the ultimate compression strain ε_m. Eqn. (19) is solved to find the neutral axis depth and the moment is calculated using eqn. (20), replacing M_{cr} with M.

Effect of tension stiffening

An empirical expression based on experimental results was obtained to determine the reduction in steel strain due to tension stiffening[5] between cracks. The average additional strain in the strand is expressed as

$$\varepsilon_{sam} = \varepsilon_{sa} - k \frac{f_r\, bd}{E_s A_{ps}} \quad \ldots(21)$$

The factor k varies with the stress in the strand after cracking and can be obtained from[5]

$$k = 1 - 0{\cdot}97 f_{scr}/f_s \quad \ldots(22)$$

where f_{scr} is the additional stress in the strand immediately before cracking and f_s is the additional stress in the strand after cracking.
The average curvature is then obtained from

$$\phi_{av} = \frac{(\varepsilon_1 + \varepsilon_{sam})}{d} \quad \ldots(23)$$

Hence, the moment-average curvature relationship from cracking up to ultimate load is obtained by reducing the additional strains from the $m-\phi$ relationship using eqns. (21) and (22) and then recalculating the average curvature with eqn. (23).

Calculation of deflection from the m – ϕ *relationship*

Once the moment and average curvature relationship was obtained the load-deflection response of the prestressed brickwork beams was determined using the finite difference[5] method. Because of the large

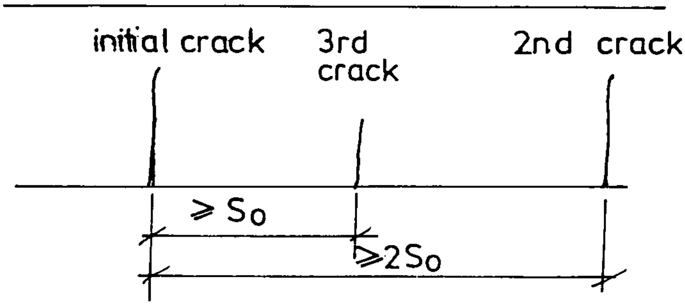

Fig 7. Spacing of cracks

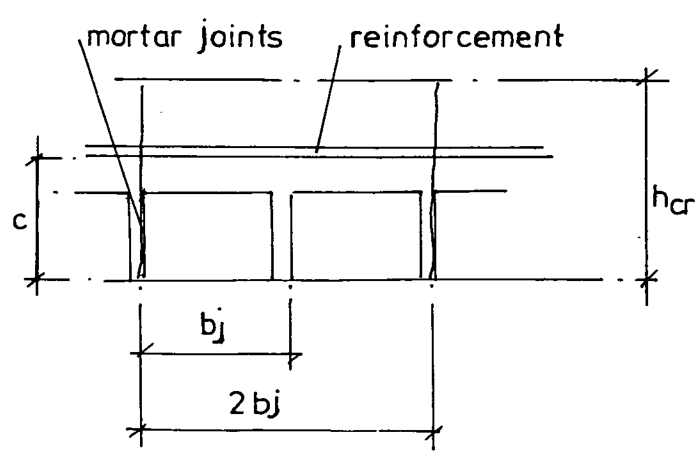

Fig 8. Crack spacing in brickwork beam

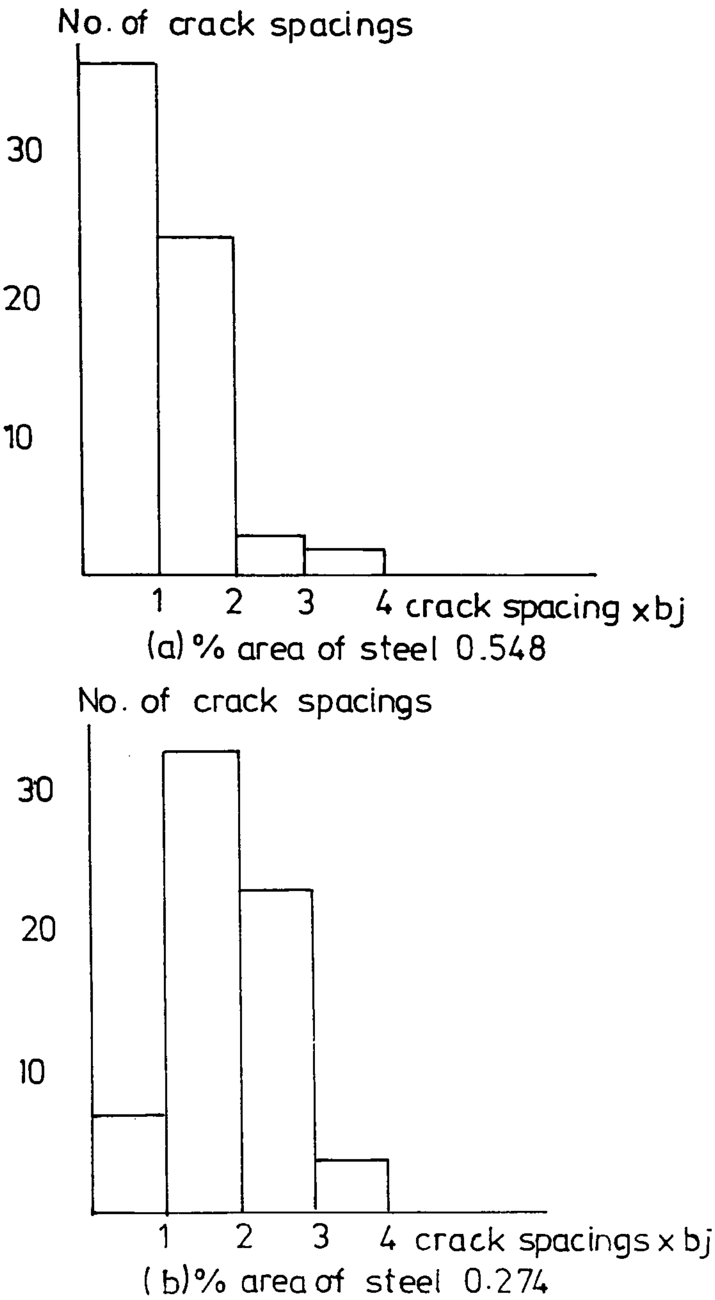

Fig 9. Distribution of crack spacing

number of iterative operations involved in eqns. (1) – (23), a computer program[5] was written to deal with them.

Cracking of post-tensioned brickwork beams

In reinforced concrete members it has been shown that the crack width is closely related to the strain in the steel[8]. As a consequence of this a number of expressions for predicting crack widths of the form

$$W_m = S_m \varepsilon_{sm} \qquad \ldots (24)$$

have evolved. S_m is the average spacing of cracks. When the first crack forms in a flexural member the stresses in the concrete in the immediate zone of the crack will be relieved. There will, therefore, be a certain minimum distance away from the initial crack before a subsequent crack can form S_o (Fig 7). If the second crack forms at a distance less than $2S_o$, it is not possible for a third crack to form between the first two. If, on the other hand, the second crack occurs at a distance greater than $2S_o$, a third crack may form between these two. The average final spacing is then likely to be between S_o and $2S_o$.

TABLE 1—Details of test beams and failure moments

Beam	Brick strength (N/mm²)	Effective prestress (kN)	% area of steel	Ultimate moment (kN)
B1	88 (high)	133	0·274	59·2
B2	88 (high)	115	0·274	56·4
B3	88 (high)	133	0·274	61·5
B4	88 (high)	144	0·274	58·4
B5	88 (high)	133	0·274	59·2
B6	88 (high)	152	0·274	58·8
BA1	88 (high)	221	0·441	72·6
BA2	88 (high)	221	0·441	79·9
BA3	88 (high)	216	0·441	74·8
BA4	88 (high)	196	0·441	74·5
BB1	88 (high)	275	0·548	87·2
BB3	88 (high)	309	0·548	103.0
BB4	88 (high)	280	0·548	92·9
A7	67 (med.)	134	0·274	53·4
A8	67 (med.)	142	0·274	51·8
A9	67 (med.)	134	0·274	54·1
A10	67 (med.)	152	0·274	51·7
A11	67 (med.)	129	0·274	56·3
C1	34 (low)	75	0·274	45·9
C2	34 (low)	61	0·274	42·5

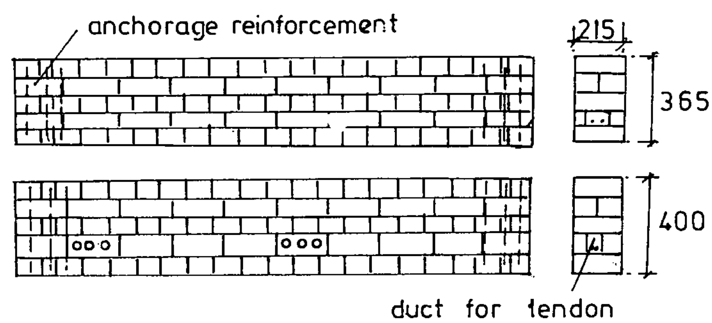

Fig 10. Bonding pattern of beams

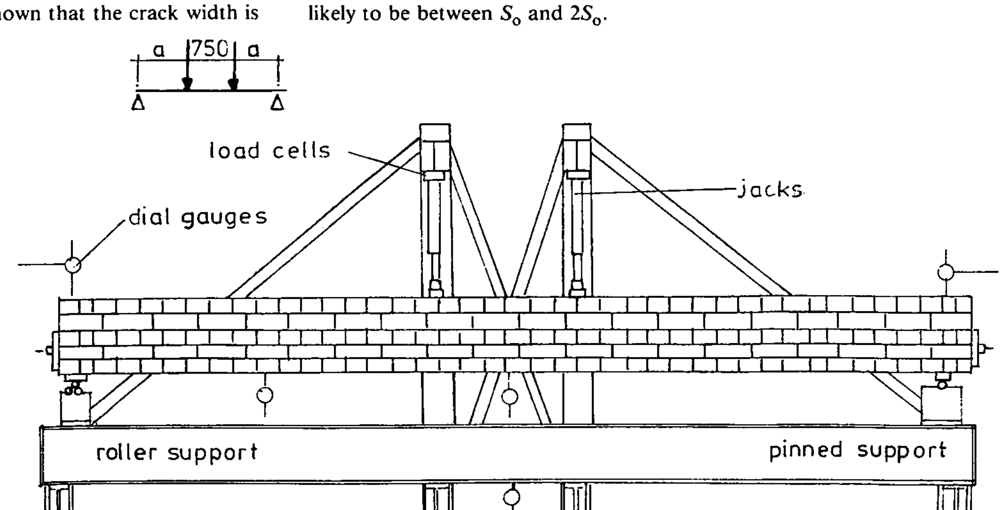

Fig 11. Layout of test rig

For reinforced concrete Beeby[9] has shown that the minimum crack spacing will be between two limits, these being defined by the initial crack height h_{cr} and the cover to the steel. In reinforced and prestressed brickwork beams, the situation is different. In concrete the tensile strength will be relatively uniform along the length of the beam, whereas in brickwork the flexural tensile strength will vary. The tensile strength of the masonry unit or the mortar itself will be considerably greater than that of the brick/mortar interface. Hence cracks are more likely to form at the interface, rather than through the brick or mortar. This, then, simplifies the problem of crack spacing, as cracks are more likely to form at intervals coincident with the vertical mortar joints (Fig 8). The cracks may not form at every mortar joint, but at multiples of the distance between two adjacent joints, b_j. If the cover to the reinforcement is greater than b_j, the minimum crack spacing must be greater than b_j and if h_{cr} is greater than 2_{bj}, but less than 3_{bj}, the minimum crack spacing will be 2_{bj}. The minimum crack spacing can then be obtained by comparing the upper and lower limits of crack spacings based on the initial height and cover to the reinforcement, with the range of possible spacings determined from the brickwork bonding pattern. The prediction of the minimum crack spacing then relies on determining at which point the stresses in the brickwork are no longer affected by the crack. From this point onwards, moving away from the crack, the stresses will be relatively uniform and the next crack may tend to form at the next weakest joint, which may not necessarily be the first joint at which cracking is possible. Fig 9 shows the distribution of actual crack spacings obtained experimentally for two series of beams with different percentages of steel. For the first group, with 0·584 % steel, 92 % of the crack spacings lie within b_j or 2_{bj} with the greatest proportion being b_{bj}, whereas for beams with 0·274 % steel, 84 % of the results lie within 2_{bj} and 3_{bj}. Using the previous assumptions, the predicted minimum crack spacings were b_j and 2_{bj} for the beams with 0·548 % and 0·274 % steel, respectively. Fig 9 shows that 40 and 42 % of the experimental crack spacings are one b_j greater than the predicted minimum crack spacing. Taking an average, then, it is likely that 41 % of the actual crack spacings are greater than the predicted minimum crack spacing. Eqn. (24) can therefore be modified for prestressed brickwork beams

$$W_m = (N_j + 0{\cdot}41)\, b_j\, \varepsilon_{smb} \qquad \ldots.(25)$$

where N_j represents the crack spacing in terms of the minimum number of cross joints between cracks and ε_{smb} is the average additional strain at the level which the crack is being considered.

Brief experimental details

To compare with the theoretical analysis, a series of prestressed brickwork beams were built, the details of which are given in Table 1. Typical sections and brick bonding patterns for the beam are shown in Fig 10. The beams were tested over a span of 6·2 m in the rig shown in Fig 11. Strains in the prestressing strand were measured with electrical resistance gauges and on the brickwork with 'demec' gauges. Deflections were measured at the supports and the midspan using mechanical dial gauges; crack widths were measured using 'ultra lomara' microscope. The applied load was measured with load cells connected to a pen-chart recorder.

Comparison with experimental results

Moment-curvature relationship

The moment-curvature relationship was obtained experimentally using the measured strains in the top fibre of the brickwork beams and from the electrical resistance gauges on the prestressing strand, and using eqn. (23). Fig 12 shows the experimental moment-average curvature relationship for the beams built from high strength brick with steel areas varying from 0·274 % to 0·548 %. For the beams with lower steel areas (Figs 12(a) and (b)) it can be seen that the m-ϕ relationship takes a well-defined three-phase form. The three phases correspond to the initial uncracked section, cracked section with stress in steel below yield point, and the third phase which flattens out indicating that the steel is yielding. The third phase is not apparent in Fig 12(c) for the beams with the highest steel percentage, suggesting that these beams were in fact over-reinforced.

From Fig 12(a), (b), and (c), it can be seen that there is very good agreement between the experimental theotretical M-ϕ relationships, throughout the full loading history of the beams.

Load-deflection response

The experimental load-deflection response of the beams built with high strength brick is given in Fig 13(a), (b), (c). It can be seen that, as with the M-ϕ relationship in Fig 12, cracking occurs at higher loads in the beams with greater steel areas and also prestress forces, as would normally be expected. Also the slope of the load-deflection response after cracking is steepest in the beams with greatest percentage of steel, indicating that these beams have stiffer uncracked sections.

Fig 14 (a), (b) shows the load-deflection response for beams built with medium and low strength brick. Both these beams had the same

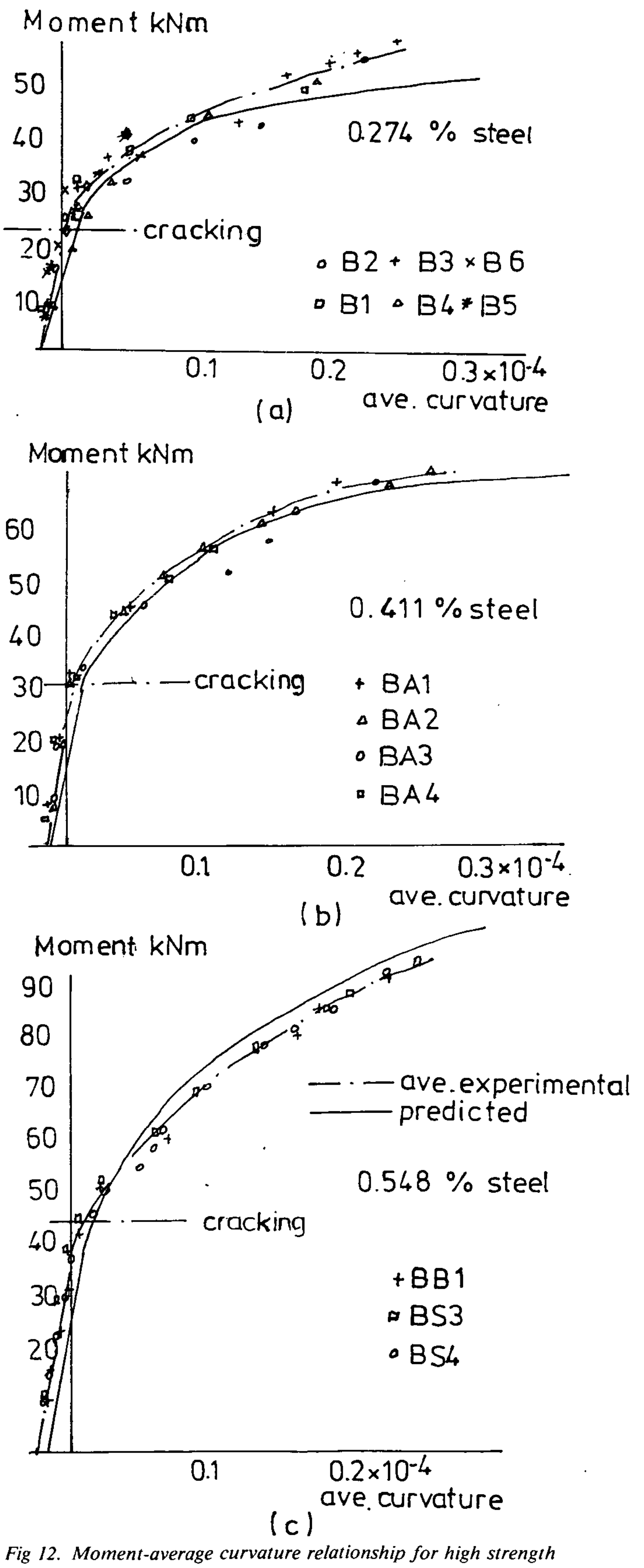

Fig 12. Moment-average curvature relationship for high strength brick beams

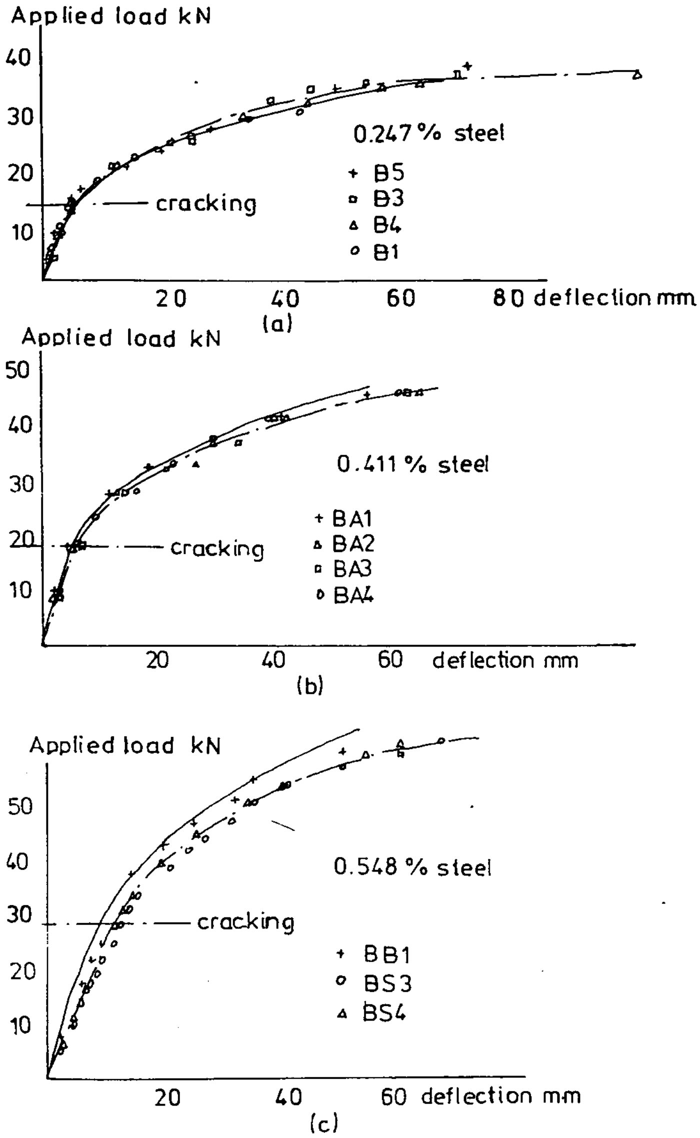

Fig 13. Load-deflection response, high strength brick

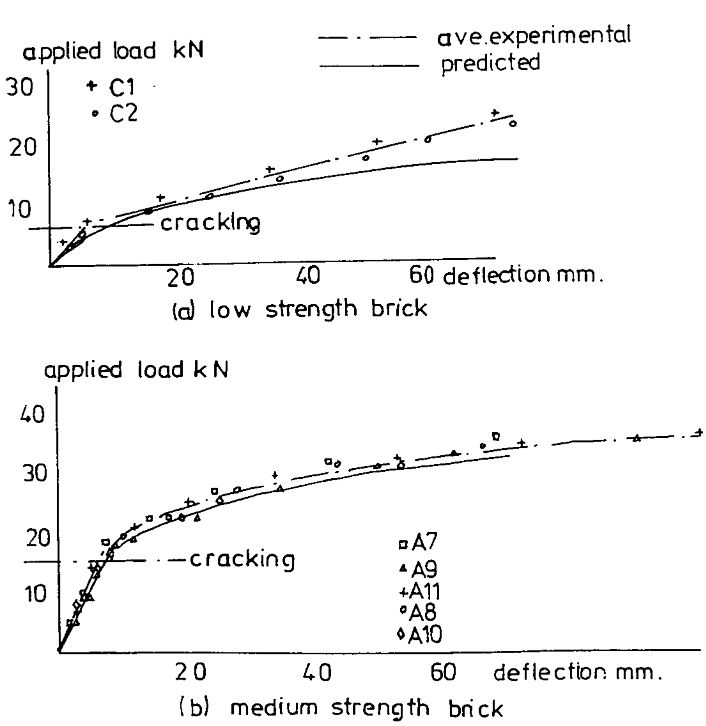

Fig 14. Load-deflection response, low and medium strength brick

TABLE 2—Predicted minimum crack spacing

% area of steel	Cover (mm)	h_{cr} (mm)	minimum crack spacing (mm)	experimental most common crack spacing
0·274	116	220-255	220	220
0·44	90	192	110	220
0·548	90	171	110	110

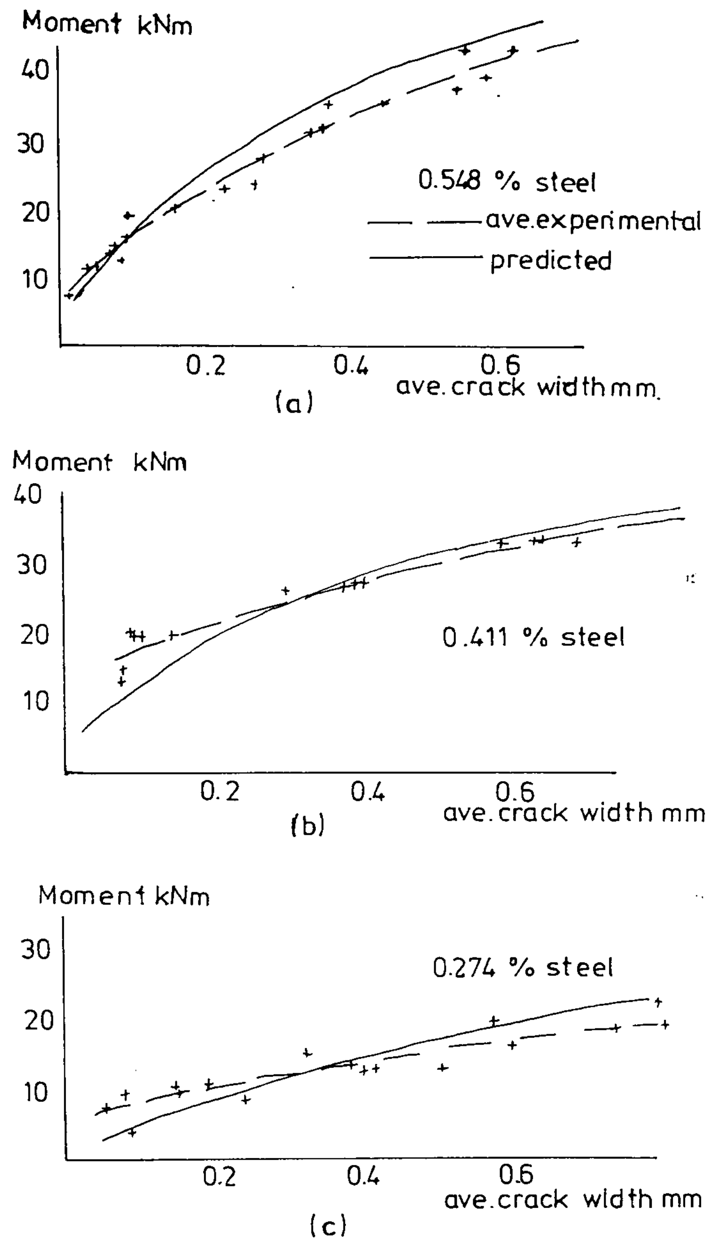

Fig 15. Comparison between predicted and experimental average crack widths for high strength brick beams

percentage steel as the high strength brick beams in Fig 13(a). However, the low strength brick beams had reduced prestress force and this is reflected by cracking occurring at lower loads. There does not appear to be a significant difference between the load-deflection response of the medium and high strength brick beams with the same steel area and degree of prestress.

The results shown in Figs 13 and 14 show that there is excellent agreement between the experimental and theoretically predicted load-deflection behaviour. This is no doubt due to the accurate prediction of the M-ϕ relationship shown in Fig 12 (a), (b), (c).

Cracking

Using the assumptions given earlier the most likely minimum spacing of

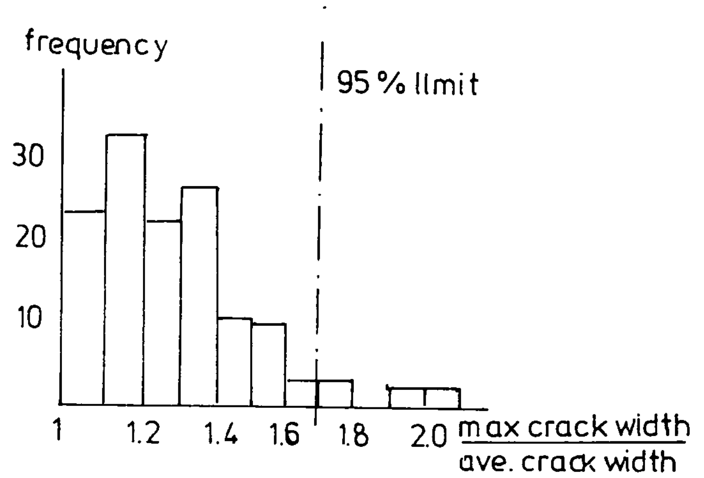

Fig 16. Distribution of maximum crack widths

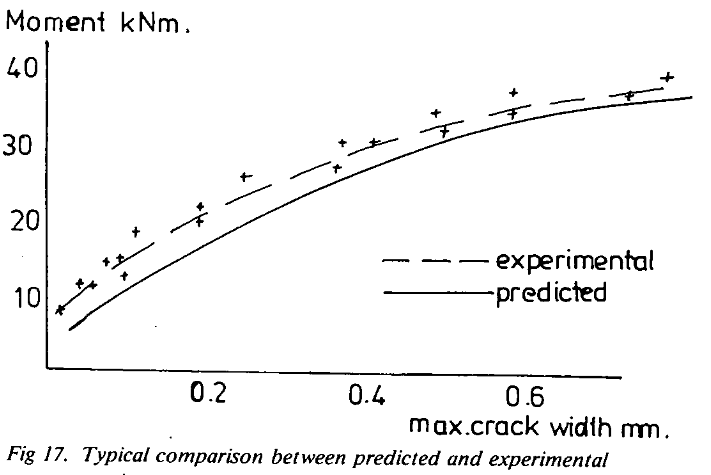

Fig 17. Typical comparison between predicted and experimental maximum crack widths for high strength brick beams

the cracks was predicted. The results are summarised in Table 2 for the different areas of steel. In the experiments, all cracks initiated at the brick/mortar interface and consequently the experimental crack spacings were all multiples of b_j. In the third column of Table 2 there is a range of initial crack heights for the beams with 0·274 % steel. In this group of beams the brick strength varied from high-low and the initial crack height varied as a result of this. From this table it can be seen that the crack spacing decreased as the percentage of steel decreased.

Using eqn. (25) and the data from the fourth column of Table 2, the average crack spacing was predicted. Fig 14 shows the comparison between the experimental and predicted crack widths for the beams built with high strength bricks and varying steel areas. Here it may be seen that the beams with the higher steel areas form smaller crack widths for a given increment of load. Because of the greater steel area a smaller strain is required to result in the same increase in tensile force.

It can be seen in Fig 15(a), (b), (c) that, although there is a considerable degree of scatter in the experimental results, eqn. (25) provides an accurate prediction of the average crack spacing.

Up to this point the analysis has concentrated on the average width of cracks. In practice a number of actual crack widths will exceed this. One crack particularly wider than the average may cause more concern than a few cracks of average width. In design, therefore, the maximum likely crack width should be taken into account. Fig 16 shows the frequency that the maximum crack width exceeded the actual crack width by a given amount. The maximum crack width was most often between 1·1 and 1·2 times the average crack width. Also shown in this figure is the 95 % confidence limit, i.e. the ratio of the maximum crack width to average crack width that will be exceeded in only 5 % of all cases and is equal to 1·7. Using this expression eqn. (25) may be rewritten

$$W_{\text{max.}} = 1{\cdot}7\,(N_j + 0{\cdot}41)\,b_j\,\varepsilon_{\text{smb}} \qquad \ldots(26)$$

Fig 17 shows a typical relationship between maximum crack width and moment for high strength brick beams. In all cases it can be seen that the predicted maximum crack width is slightly greater than the experimental and hence would provide a safe estimate of the maximum crack width in design.

Conclusions

(1) The m-ϕ relationship for prestressed brickwork beams with low percentage of steel exhibits a distinct three-phase behaviour corresponding to uncracked, cracked with steel in elastic range, and steel yielding. Increase in steel area tends to reduce or eliminate entirely the third phase.

(2) The load-deflection response of the beams built with low steel percentage is not significantly affected by brick strength (high or medium).

(3) The m-ϕ relationship and thus the load-deflection response of prestressed brickwork beams can be closely predicted by the proposed method which takes into account the non-linear stress-strain relationship of brickwork and tension stiffening.

(4) On neutralisation of prestress, and the flexural strength of brickwork, cracks in the tensile zone appear in prestressed brickwork beams at the brick/mortar interface. The average and maximum crack width thus formed can be predicted accurately by the equation given in this paper.

Acknowledgements

The work described in this paper has been supported by the Science & Engineering Research Council, Structural Clay Products Ltd., St. Neots, and George Armitage & Sons Ltd.

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Informal Study Groups

The scheme of Informal Study Groups is now in full operation. Each works under the direction of a Convener appointed by the Council. The Convener is personally responsible for organising the work of his Group, conducting correspondence, producing and exchanging draft documents, etc. Experience has shown that Study Groups vary in the way they operate. Some have a continuing function, while others have objectives that are met in a given period. The general procedure for the working of a Study Group is set out on this page. Some such simple rules are clearly necessary, but it is the hope of the Council that the strength of a Study Group will lie in its informality and the opportunity it provides for members in all parts of the world to exchange ideas and experience. Members interested in contributing to one or other of the Groups now working are invited to get in touch with the appropriate Convener at the address given. Full details of the purpose and work programme of each Study Group will be found in the issue of *The Structural Engineer* under the reference given.

History of Structural Engineering

Convener: R. J. M. Sutherland, BA, CEng, FIStructE, FICE, Harris & Sutherland, 38-42 Whitfield Street, London W1P 5RF
The Structural Engineer, March 1973, p. 110

Model Analysis as a Design Tool

Convener: F. K. Garas, PhD, CEng, FIStructE, MICE, Taylor Woodrow Construction Ltd., Taywood House, 345 Ruislip Road, Southall, Middlesex UB1 2QX
The Structural Engineer, February 1977, p. 63

Qualitative Analysis of Structural Behaviour

Convener: R. Bishop, CEng, MIStructE, MIHT, Civil Engineering Department, Polytechnic of Central London, 35 Marylebone Road, London NW1 5LS
The Structural Engineer, November 1978, p. 309

The Influence of Creep on Structural Behaviour

Convener: G. L. England, PhD, DSc(Eng), CEng, MIStructE, MICE, Reader in Engineering Mechanics, Department of Civil Engineering, University of London King's College, Strand, London WC2R 2LS
The Structural Engineer, August 1979, p. 244

Offshore Structural Audits

Convener: R. J. M. Bennett, BSc, CEng, MIStructE, MICE, Scott Bennett Associates, Consulting Engineers, 61 Manor Place, Edinburgh EH3 7EG
The Structural Engineer, February 1981, p. 38

Materials and Components

Convener: K. Thomas, MSc, CEng, FIStructE, FCIoB, 3 The Paddocks, Church End Green, Haddenham, nr. Aylesbury, Bucks. HP17 8AG
The Structural Engineer, Part A, September 1982, p. 269

The Design of Steel Portal Frames

Convener: L. J. Morris, BSc(Eng), PhD, ACGI, DIC, CEng, FIStructE, Simon Engineering Laboratories, University of Manchester, Manchester M13 9PL
The Structural Engineer, Part A, June 1983, p. 170

Vibration Problems in Structures

Convener: J. W. Smith, BSc(Hons), PhD, CEng, MIStructE, Department of Civil Engineering, University of Bristol, Bristol BS8 1TR
The Structural Engineer, Part A, June 1983, p. 170

Tall Buildings—Their Design and In-Service Performance

Convener: R. M. Lawson, BSc(Eng), PhD, ACGI, CEng, MIStructE, MICE CIRIA, 6 Storeys Gate, London SW1P 3AU
The Structural Engineer, October 1984, p. 297

Economical Cover and Operation of Large Spaces

Convenors: S. B. Tietz, BSc(Eng), CEng, FIStructE, FICE, S. B. Tietz & Partners, 10-14 Macklin Street, London WC2B 5NF; M. G. T. Dickson, BA, MS, CEng, MIStructE, University of Bath, Claverton Down, Bath BA2 7AY
The Structural Engineer, April 1985, p. 106

General procedure

Object
The object of the Study Group scheme is to create opportunities for members of the Institution to exchange ideas and work on deepening and developing their knowledge of structural engineering, thus stimulating a greater interest in and promoting the art and science of structural engineering.

Right to set up
Any corporate member shall have the right to propose setting up a Study Group, provided that an outline of the work of the proposed Study Group is submitted, it does not duplicate the work of other Groups, its interests lie in the field of structural engineering and it conforms to any rules that may be formulated by Council. For each Study Group approved, Council will appoint a Convener, who shall be a member of the Institution. A Study Group so established shall be disbanded on completion of its work or as decided by Council.

Right to participate
Every member of the Institution may of righ take part in the work of any Study Group; non-members may also participate, with the approva of the Convener.

Organisation
The internal organisation of each Study Grou shall be at the discretion of the Group, althoug only Institution members may serve on Group's management or steering committe unless Council approval is obtained.

Finance
No Study Group shall impose any financial commitment on the Institution, other than the cost o correspondence from the Secretariat and o notices and announcements in The Structura Engineer, *without the express prior consent o Council. In this context it must be understoo that 'commitment' includes any activity that i expected to be financially self-supporting Neither shall any Study Group organise any ac tivity calling for the collection of admission fee or other money without the express permissio of Council. A Study Group is part of the Institu tion, which is responsible for any losses incurre and to whom any surplus funds are accountable The Institution may assist financially in som work at the discretion of the Council.*

Publications
No publication other than a newsletter shall b made by any Study Group without the prior con sent of Council. In any case copyright shall be in vested with the Institution and royalties, wher applicable, shall be assigned to the Institution.

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AN INVESTIGATION OF THE STRUCRURAL PERFORMANCE OF PRESTRESSED BRICKWORK AND CONCRETE BEAMS

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1. Introduction

In the housing sector of many developing countries, reinforced concrete lintels and beams are used to span the openings. In developing countries, cement not only is in short supply, but also requires high energy for production. Thus with increasing demand for housing and increasing pressure on finite natural resources, one has to look for cheaper alternatives. A cheaper way of replacing this will be the use of reinforced or prestressed brickwork floor slabs, lintels and beams. In some parts of India, reinforced brickwork floor and roof slabs are used frequently in the private sector. However, to minimize cracking of the reinforced brickwork, the steel stress needs to be kept low. Besides this, the failure of reinforced brickwork is invariably due to shear, hence both the steel and brickwork are not used to their optimum. These defects can be overcome by prestressing. The prestressed brickwork beams or lintels do not require very highly skilled mason, formwork nor the degree of sophistication required for concrete, hence may be cheaper and viable in the developing countries. Hence, a R & D project was undertaken a few years ago to study the behaviour up to ultimate of prestressed brickwork beams [1,2] with following variables:

i) brick strength
ii) mortar strength
iii) Prestressing steel
iv) shear arm/effective depth ratio

so that this type of construction can be used in practice. Any such new development in brickwork to be acceptable will have to complete on economic ground and on comparable structural performance with concrete, which is a well established construction material for flexural members. Therefore, an investigation was carried out to study the structural behaviour of prestressed concrete and brickwork beams [2] of similar strength and cross-sectional area up to failure. This paper summarises the theoretical and experimental results of this investigation.

2. Theoretical Analysis

For theorectical analysis of the load-deflection relationship and the ultimate moment of these beams, the stress strain relationship and the ultimate strengths of materials are required. These were obtained as explained below.

2.1 Brickwork

The ultimate strength and the stress-strain relationship of brickwork was obtained by testing prisms as shown in Fig. 1. The stress-strain relationship is represented by a third degree polynomial [4] as given in fig. 2. The stress-strain relationship in tension is assumed linear up to the modulus of rupture of the brickwork.

2.2 Concrete

The stress-strain relationship given in B S 8110 [5] as shown in fig. 3 was used for concrete. Linear stress-strain relationship in tension was assumed also for concrete; cubes and cylinders were tested to obtain the strength.

2.3 Prestressing tendons

The tendons were tested under axial tension and the resulting strain were measured with the help of electrical strain gauges. The stress-strain relationship [6] was mathematically idealised in tri-linear for fig. 4.

An interactive computer programme [7] was developed which utilises the idealised linear and non-linear material behaviour of prestressing steel brickwork and concrete and predicts the ultimate moment and the load-deflection relationship up to failure.

3 Test specimen and test arrangements

3.1 Beam details

The cross-sections of the concrete and brickwork beams are given in fig. 5. The compressive strength of concrete and brickwork is given in Table 1. Seven wire stablished steel strands were used for prestressing and the area of steel was 0.274% of the cross-sectional area of brickwork and concrete.

3.2 Test Arrangements

The beams 6.2 m long, were tested under two point loading as shown in fig. 5. The central deflection in each beam was measured with the dial gauge. The strain in the tendon was measured by electrical strain gauges. A 'demec' gauge was used to measure the strains in brickwork and concrete. The load was applied in stages to failure. The load was measured at the jacking point with the help of load cells connected to a pen-chart recorder.

4. Results and discussion

The test results are shown in Table 1.

4.1 Cracking Moment

From the Table it can be seen that the cracking moment of prestressed concrete beam is 12.9% higher than the brickwork beams for same degree of prestress. However, this is not very significant and is due to higher value of modulus of rupture of concrete compared to brickwork. The experimental cracking moment is slightly higher than predicted by the theory. The theory predicts the moment just before the start of cracking when both the prestress and tensile stress of the material has been neutralised due to the applied loading whereas in the experimental result the cracking moment has been taken as moment at which first visible crack appeared on the beam.

4.2 Ultimate Moment

The experimental ultimate moments of beams of brickwork and concrete are of the same order (Table 1) and practically there is no difference. From these results, it is evident that concrete and brickwork beam of similar section and with same effective prestress will fail at the same ultimate moment provided the failure is due to flexure.

In such cases, the steel stress will be the governing factor than the compressive strength of the materials. The ultimate moments predicted by the theory for brickwork and concrete beams are only 8 to 11% lower than experimental values, hence the agreement between experimental and theoretical results are good and the theory can safely be used for design.

4.3 Load-deflection relationship

The load-deflection relationship is shown in fig. 6. There is good agreement between average experimental and theoretical results. From experimental results it appears tht the concrete beam is slightly stiffer than brickwork beam after cracking. The load-deflection relationship before decompression is exactly similar. In this respect, concrete beam has no advantage over brickwork beam.

Considering, the working load moment is equal to the decompression moment for both brickwork and concrete the central deflections will be of the order of $\frac{\text{span}}{1550}$ & $\frac{\text{span}}{1250}$ for brickwork and concrete respectively.

The difference is due to the fact that the decompression moment of brickwork is 25.5 KNm which is slightly less than of concrete (28.8KNm). At 25.5 KNm moment, the central deflection of concrete and brickwork beam are the same.

5. Conclusions

1. The ultimate moment of resistances of prestressed concrete and brickwork beam of similar cross-section area, prestressing force and percentage area of steel are similar provided the failure is in flexure.

2. The cracking moment of concrete beam is slightly higher than brickwork beam with the same degree of prestress.

3. The theory predicts the cracking and ultimate moments including the deflection up to failure which compares very favourably with experimental results, hence it can be used in practice for the design of such beams.

4. From the tests carried out so far, it may be inferred that the short-term structural behaviour of prestressed brickwork and concrete beams are similar and the latter has no advantage over the former.

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The financial support of the Science and Engineering Research Council and George Armitage and Sons plc, Wakefield, U.K. is thankfully acknowledged. The author is thankful to his past and present students especially Dr Pedreschi and Miss Uduehi for carrying out the beam tests.

Table 1: Summary of Experimental and Theoretical Results

Type of Beam	Effective Prestressing Force kN		Cracking Moment kNm			Ultimate Moment kNm		
		Average	1st Exp. Visible Crack	Exp. Average 1st Visible Crack	Theory	Exp	Exp. Average	Theory
Brickwork (Prism Strength = 33N/mm²)	133 115 133 144 133 152	13.5	24.0 22.0 29.5 26.0 23.0 28.5	25.5	24.9	52.9 56.4 61.5 58.4 59.2 58.8	57.9	53.5
Concrete (Cube Strength = 35N/mm²)	137 132 136	13 5	21.3 31.8 33.2	28.8 * (32.5)	31.0	58.8 58.1 59.1	58.7	52.9

*neglegting low result

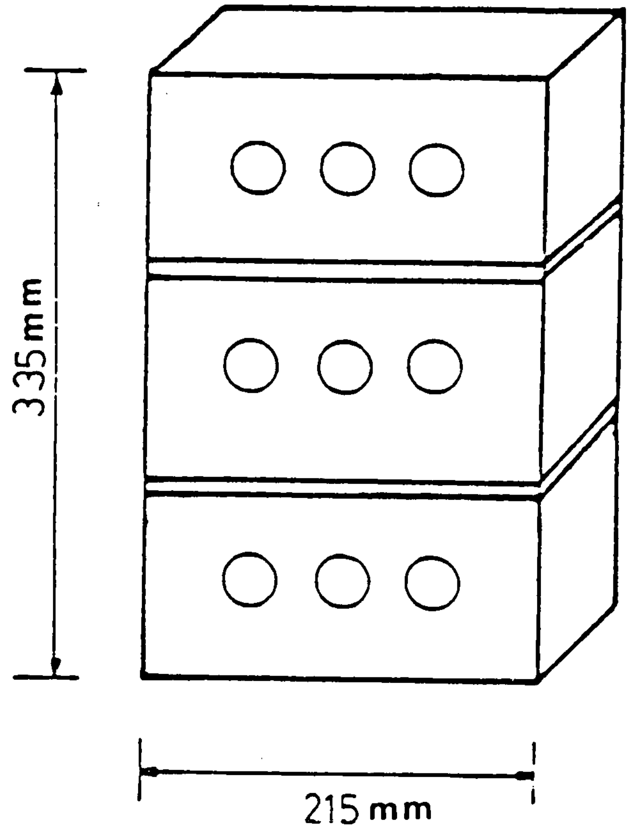


Fig. 1 - Test Prism

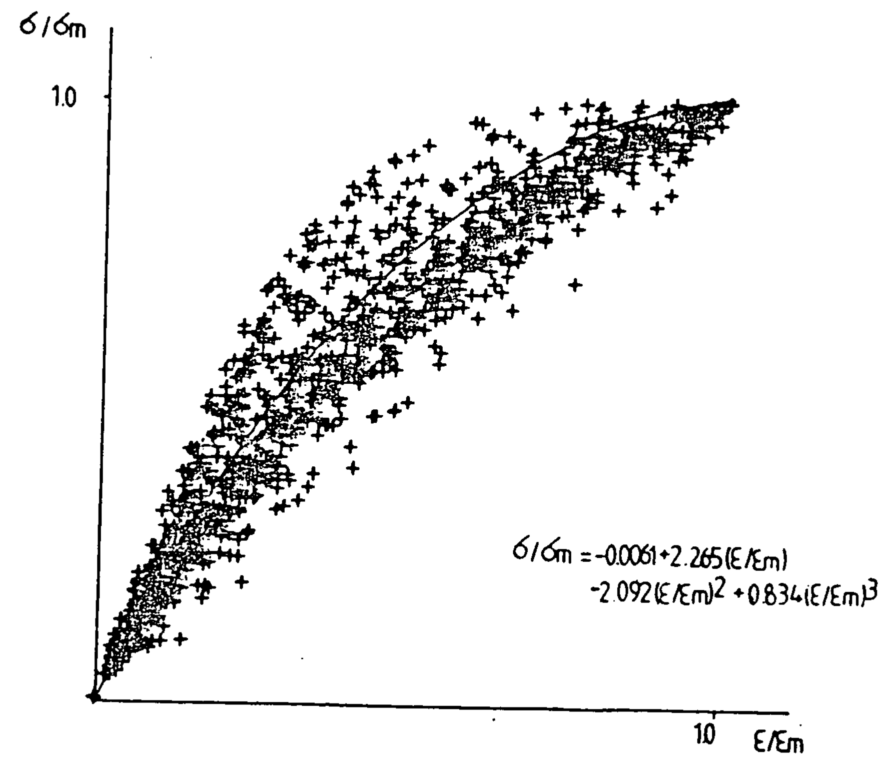
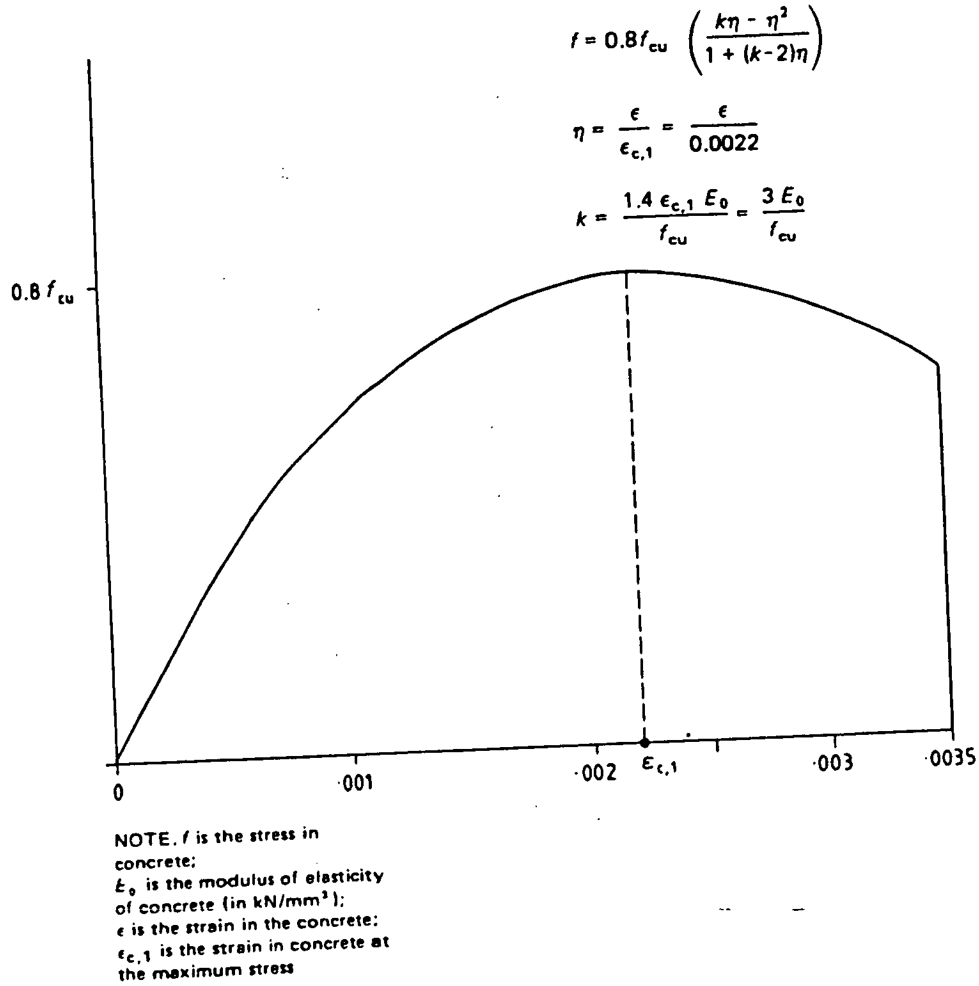


Fig.2 *Nondimensional stress/strain relationships.*



.Fig. 3 Stress-strain relationship of concrete

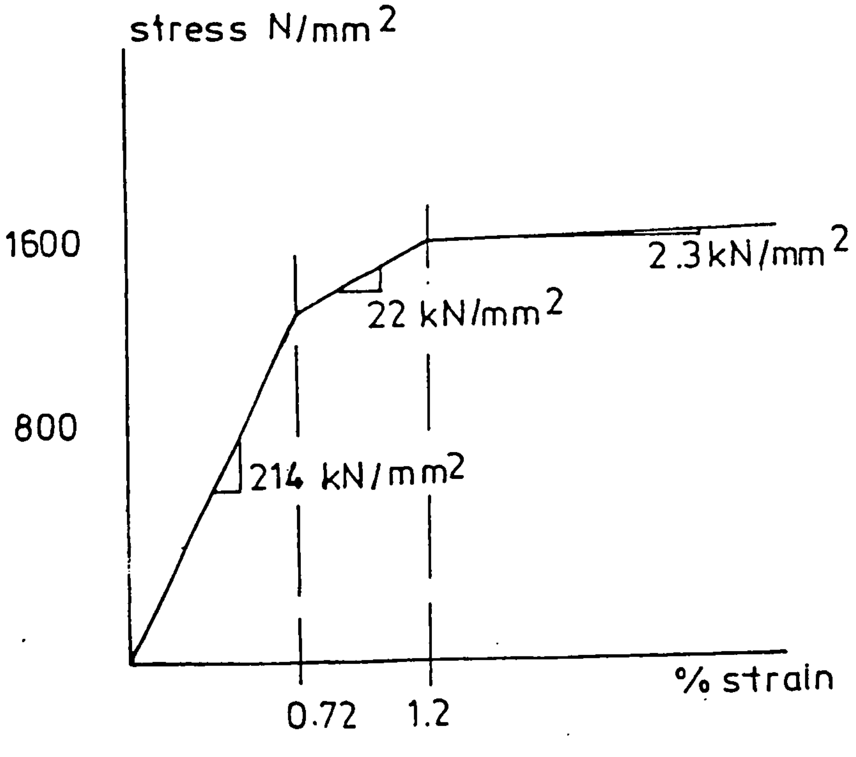


Fig. 4 Idealised stress-strain curve for prestressing steel

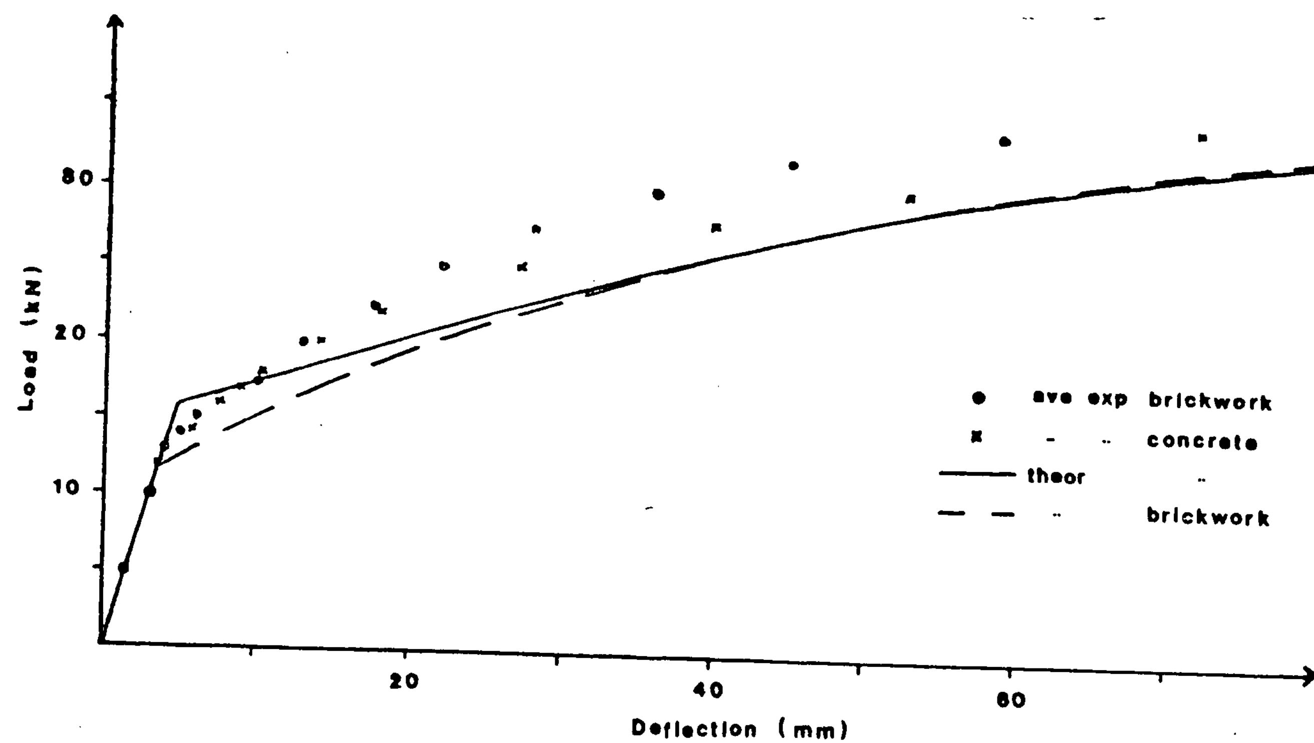


Fig. 6 Load-deflection relationship of the prestressed concrete and brickwork beam

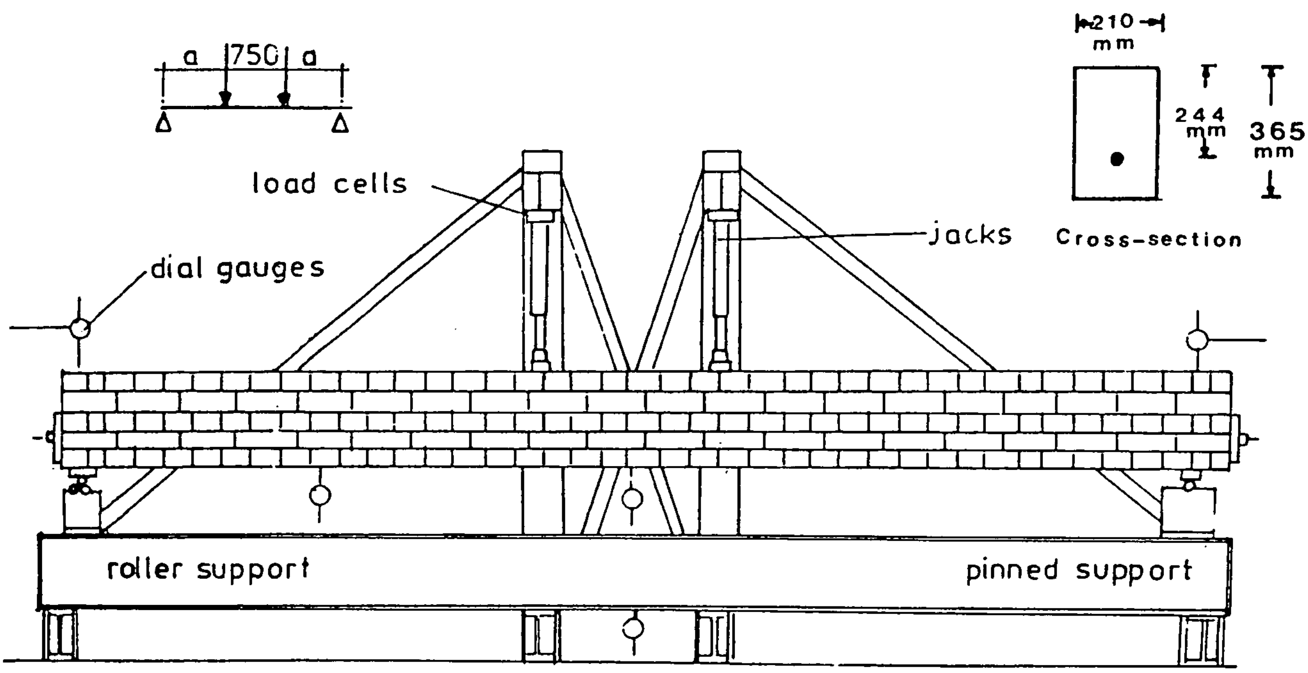


Fig. 5 Showing the cross-section of the beam and the test arrangement

BEHAVIOUR OF PARTIALLY PRESTRESSED BRICKWORK BEAMS

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ABSTRACT The paper summarises the result of an investigation on the behaviour of full-scale partially prestressed brickwork beams. 10 full-scale beams of span 6.2 m were tested to study the ultimate moment, the moment-deformation relationship, and mode of failure. The variables considered were the percentage of steel and brick strength.

The experimental ultimate moment and moment-deformation relationship are compared with theoretical analysis using the material properties obtained from the brickwork prism tests.

INTRODUCTION

The technique of prestressing is generally associated with concrete. Prestressed concrete has established itself as a major structural material, which is used widely in the construction industry. The principle of prestressing which is so widely used for concrete can also be applied to brickwork. Recently, a comprehensive research program (1) has been carried out to study the behaviour of fully prestressed brickwork beams. From this investigation it is very clear that the brickwork can be fully prestressed, but the transfer stress has to be kept low to prevent splitting in the anchorage zone. In addition, the prestressing steel has to be kept at 'kern' limit to avoid the development of tensile stresses due to prestressing. As a result of this constraint, the width of the crack may be much larger than allowed for a class 3 prestressed concrete member [2] at service load. The width of the crack can be controlled by 'partial prestressing'. Partial prestressing of a section is achieved in two ways: i) either by reducing the level of initial prestress applied to the entire tensile reinforcement required for ultimate load, or ii) by prestressing part of the tensile reinforcement to a maximum allowable stress level and leaving the rest non-stressed [3]. As no work has been done so far in this field, a comprehensive investigation of the behaviour of partially prestressed brickwork beam was undertaken to study the effects of the following variables:

i) percentage of steel
ii) ratio of prestressing steel to ordinary reinforcement
iii) mortar strength or grade 1:¼:3 and 1:½:4½ (Cement: Lime:Sand).
iv) brick strength

on ultimate moment, deflection and cracking.

This paper only summarises the result of the preliminary investigation on 10-full-scale partially prestressed brickwork beams.

MATERIALS

Mortar: A 1:¼:3 (Cement:Lime:Sand) mix by volume was used throughout the construction of the beams. The average compressive strength of the mortar for each individual beam is given in Table 2.

Concrete: A 1:2½:2 (Cement:Sand:Pea gravel) mix was used for all the beams except 5 and 6. The mix used for the beams 5 and 6 was 1:2 (Cement:Sand).

In both mixes a plasticiser (Conbex) was used to reduce the effects of shrinkage and to shorten the setting time. Three 100 mm cubes were cast during each concreting operation and tested at 7 days. The average compressive strength of the concrete is given in Table 2 for each of the test specimens.

Bricks: 3-hole perforated bricks were used throughout the test programme. The average compressive strength of high and medium strength bricks was 82.0 N/mm^2 and 58.9 N/mm^2 respectively.

Prestressing Reinforcement: 10.9 mm diameter, stabilised steel strand was used for prestressing. The average ultimate stress was 1708 N/mm^2, with 0.2% proof stress of 1640 N/mm^2. The modulus of elasticity was 214 kN/mm^2.

Non-stressed Reinforcement: 12 mm diameter, high strength deformed bars were used throughout, with an ultimate stress of 670 N/mm^2, a yield stress of 535 N/mm^2 and Young's Modulus 200 kN/mm^2.

The stress-strain relationship of prestressing steel and reinforcement was idealised in tri-linear form as shown in Fig. 1 and 2 for theoretical analysis.

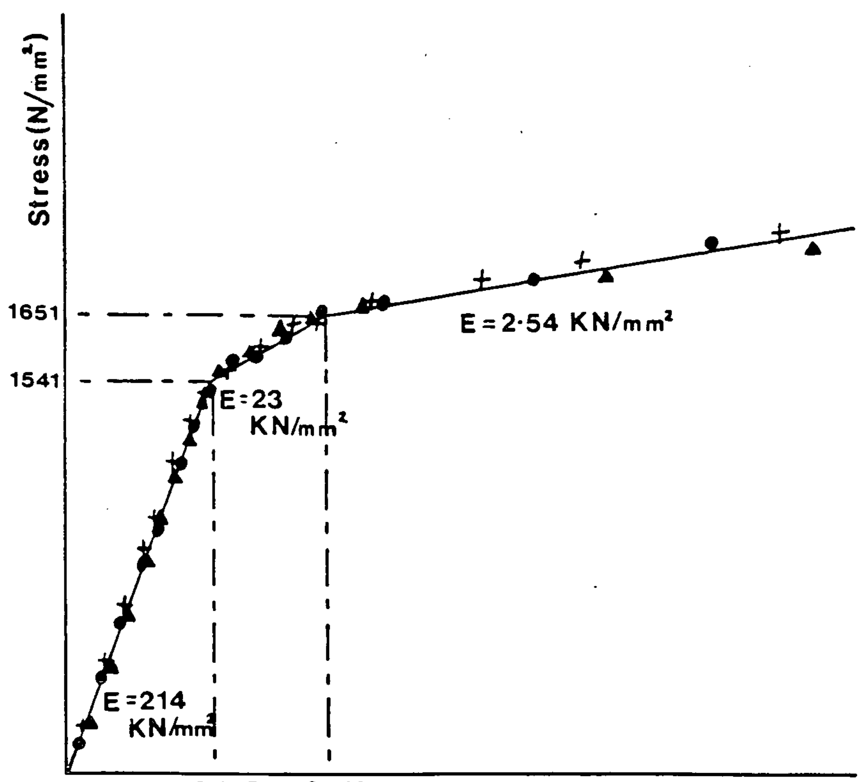


Fig. 1 Idealised tri-linear stress/strain relationship for prestressing steel

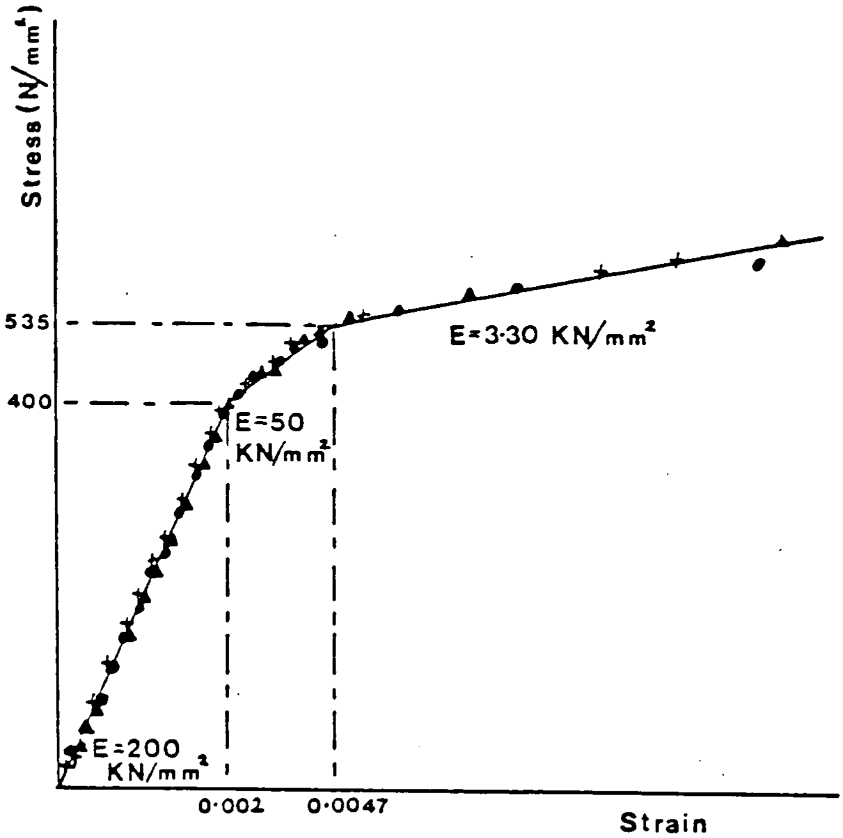
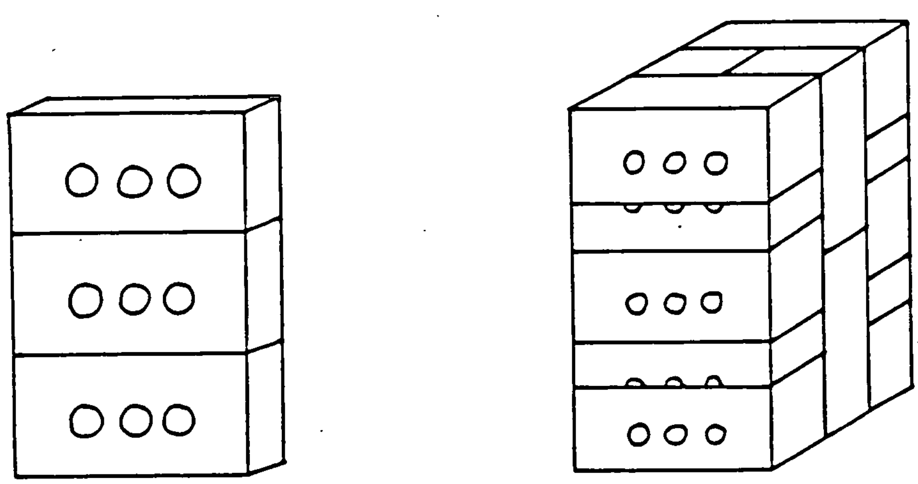


Fig. 2 Idealised tri-linear stress/strain relationship for reinforcement

PROPERTIES OF BRICKWORK

Brickwork prism specimen: The strength and stress-strain relationship, are both required for theoretical analysis. Two types of prisms as shown in Fig. 3 were selected to obtain the strength and material properties of the brickwork. Prism B represents the top three courses of the brickwork resisting the compressive force developed in the beam during early stages of loading. During the testing of the beam, it was observed that the cracks developed after the neutralisation of the prestress and penetrated deeper than the level of prestressing steel. As a result, only the topmost course of brickwork resisted the compressive force, hence single course prism was also tested to obtain the strength and material properties. The test results are given in Table 1.



Single Course
Prism Type A

Three Course
Prism Type B

Fig. 3 Brickwork Prisms

Brick Strength N/mm^2	Mortar grade	Prism type	Test No.	Ultimate Compressive Strength N/mm^2 Test Results	Average	Ultimate Compressive Strain x 10^{-5} Test Results	Average
82.0	1:¼:3	Single Course	1	28.9	28.8	305	292
			2	24.2		261	
			3	29.9		313	
			4	30.0		281	
			5	31.8		337	
			6	28.0		255	
82.0	1:¼:3	Three Course	1	16.6	19.4	189	213
			2	19.2		196	
			3	21.5		211	
			4	23.6		257	
			5	17.5		224	
			6	18.0		201	
58.9	1:¼:3	Three Course	1	10.8	11.8	253	270
			2	13.9		294	
			3	10.8		264	

Table 1 Properties of Brickwork Prisms

Stress-strain relationship: Both types of brickwork prisms were tested under uni-axial compression and the resulting strain was measured with a 'demec' gauge. The experimental stress-strain relationship was mathematically idealised and the relationship was obtained by a third-degree polynomial (Fig. 4) of the form:

$$f/_{f_m} = 1.95(\varepsilon/_{\varepsilon_m}) - 1.24(\varepsilon/_{\varepsilon_m})^2 + 0.29(\varepsilon/_{\varepsilon_m})^3$$

The equation was very similar to the one proposed earlier [4] . From the stress-strain relationship, the stress block factors $\lambda_1 = 0.63$ and $\lambda_2 = 0.37$ were obtained.

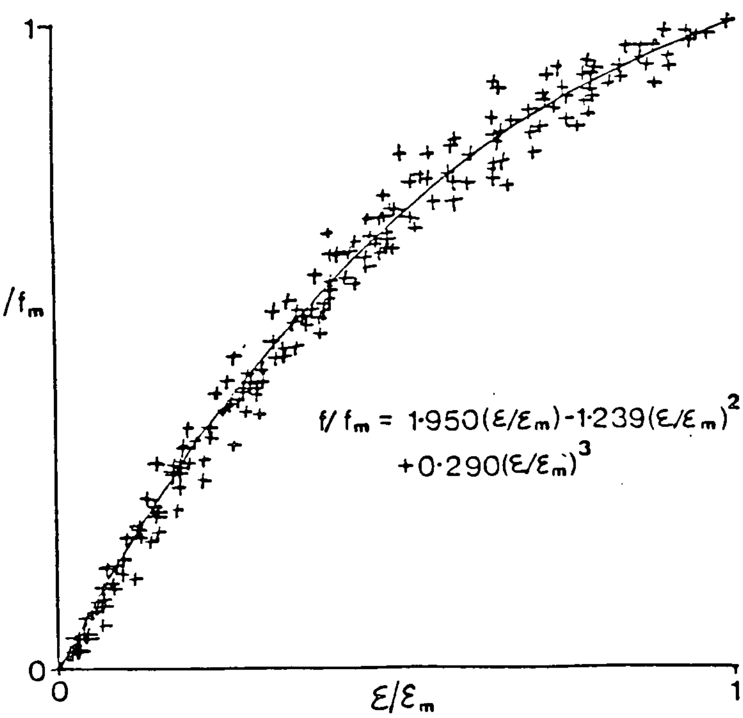


Fig. 4 Non-dimensional stress/strain relationships for brickwork

CHOICE OF BRICKWORK SECTION FOR PARTIALLY PRESTRESSED BEAMS

Any development in brickwork to be of practical use, needs to take into account the available skill and the available shape of the unit. Ignoring these two major constraints may lead to constructional difficulties associated with useless costly development. In addition the section must offer certain other competitive advantages over other forms of construction such as:

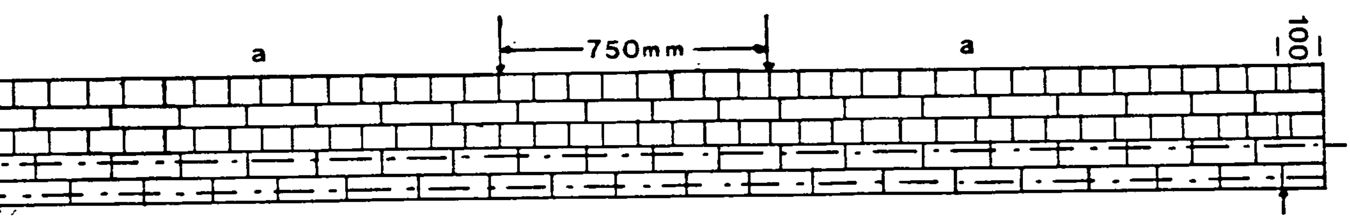
i) effective utilisation of as much ceramics as possible

ii) ease of grouting

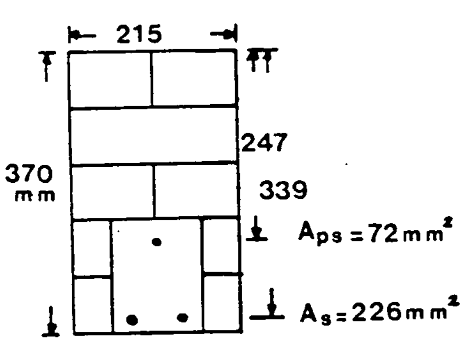
iii) provision of cavity for placement of the reinforcements at required depth

iv) elimination of formwork

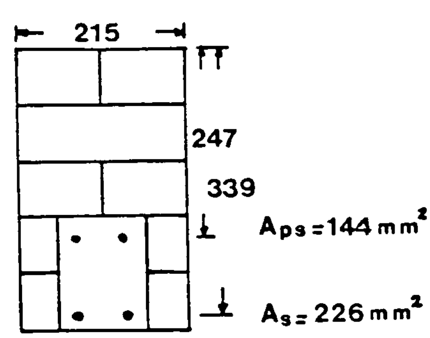
Keeping these in mind, the section developed for the beam is shown in Fig.5 . The top three courses of the beam were built in normal English bond and the bottom two courses receiving the reinforcement was formed by splitting the bricks lengthwise and placing them flush with the face. The cavity allowed positioning of the prestressing steel at the 'kern' limit and the non-stressed reinforcement at any suitable depth. The area of the cavity to be filled with concrete was 18% approximately of the total cross-sectional area.



BEAM EVELATION



Section
Beams 1-8
p=0.47%



Section
Beams 9-10
p=0.61%

Fig. 5 Beam elevation, brick bonding arrangement and sections

CONSTRUCTION OF THE BEAMS

All test beams were built on the floor of the laboratory by an experienced bricklayer. To prevent horizontal splitting of the bedjoints the ends of the beams were reinforced with 6 mm mild steel rods to resist the anchorage forces which develop in the 'lead in length'.

The beams were cured for 21 days before post-tensioning. 25 mm thick mild steel anchorage plates were attached to the beams. The beams were prestressed to 70% of the tendon's ultimate strength. Immediately after prestressing, the non-stressed steel was put in position and then the concrete was poured to fill the cavity. The beams were tested after 7 days of concreting.

The amount of prestressing steel and non stressed-steel for various beams are shown in Fig. 5.

TEST ARRANGEMENTS AND INSTRUMENTATION

The test set-up is shown in Fig. 6. The test rig (Fig. 7) provided pin and roller support. Load was applied by jacks connected to a hydraulic pump. The loads at the jacking point were measured with load-cells connected to a pen-chart recorder. Strains up to failure were measured in the constant moment zone at various depths of the beams by a 'demec gauge'. Steel strains were measured by the electrical resistance gauges. Crack width and depth were also recorded at each load interval. Central deflection of each beam was measured with dial gauges reading to 0.02 mm.

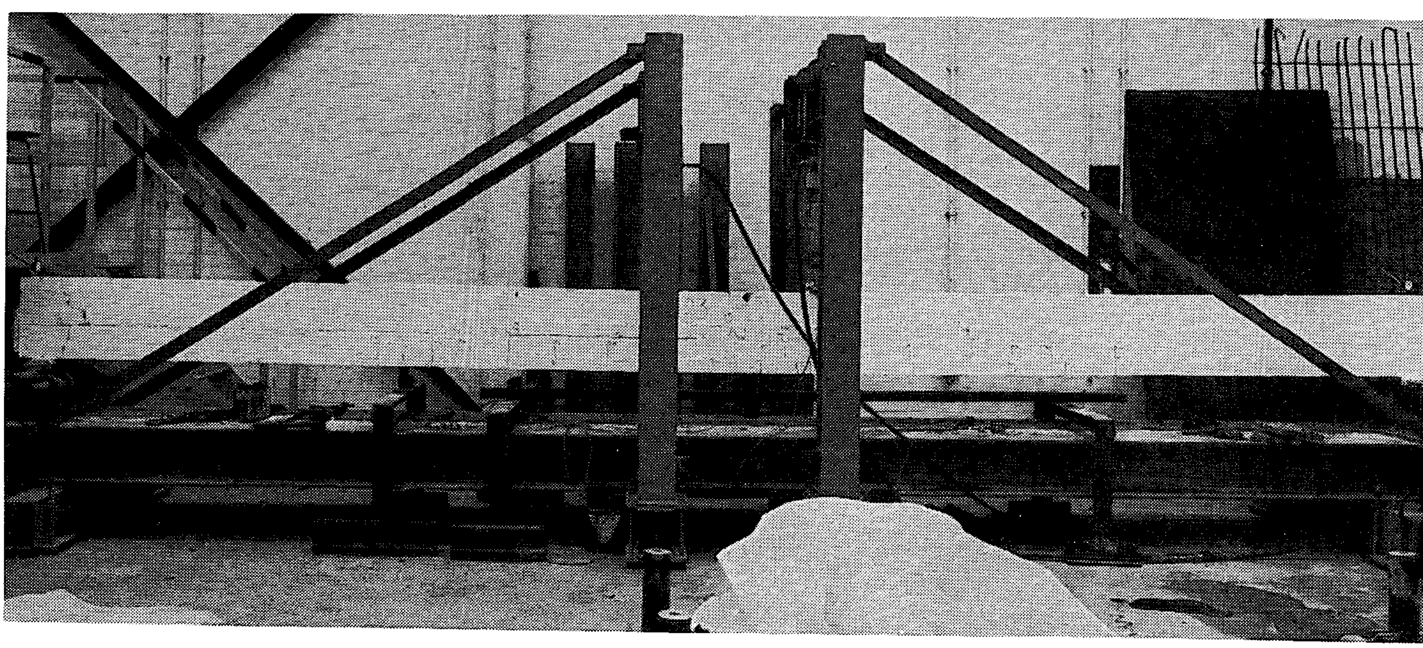


Fig. 6 Test set-up, showing the failure of a beam in shear

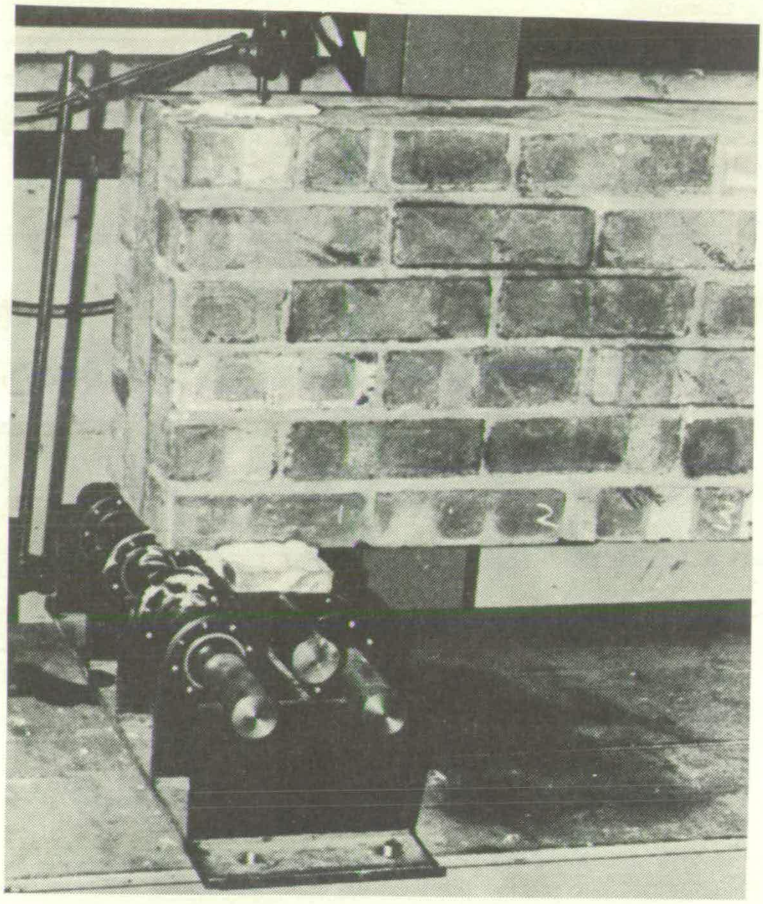


Fig. 7 Showing Roller Support

THEORETICAL ANALYSIS

Determination of moment-curvature relationship and deflection: This direct method uses the experimental idealised stress-strain curves of brickwork (Fig. 4) and both prestressing and reinforcing steel (Figs 1 & 2) to calculate the moment and curvature of the partially prestressed beams. The moment-curvature for the whole loading history is used to calculate the deflection. The applied loading is considered in three stages (1,5):

i) prestressing
ii) prestressing to cracking
iii) post cracking to ultimate load

Due to the large number of iterative operations involved in obtaining the moment-curvature relationship and deflection of the beam, a computer program was written to cope with these. The detailed derivation of this method is given elsewhere [1].

Calculation of ultimate-moment of resistance: The moment of resistance of the beam was also calculated from the stress block in addition to the direct method of calculation described above. At the time of failure, in any beam, the prestressing force is completely neutralised in maximum moment zone and the behaviour of partially prestressed beams then will be very akin to a reinforced brickwork beam.

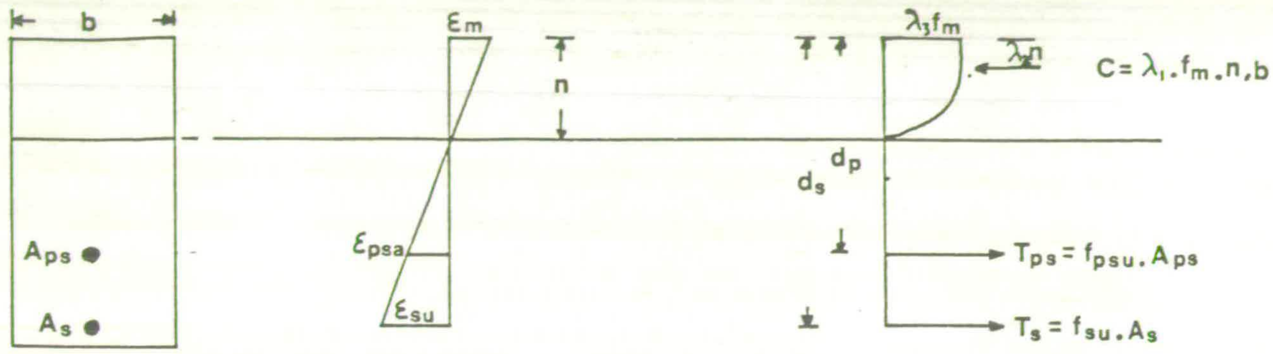


Fig. 8a Beam Section Fig. 8b Strain Distribution at Failure Fig. 8c Stress distribution at Failure

From Fig. 7 the total force of compression (6,7) and tension will be given by:

$$F_c = \lambda_1 . b . n . f_m \qquad -(i) \; ; \quad \text{since } \lambda_3 = 1 \; (7)$$

$$F_t = A_{ps} f_{psu} + f_{su} . A_s \qquad -(ii)$$

$$F_c = F_t \qquad -(iii)$$

$$n = \frac{A_{ps} . f_{psu} + A_s f_{su}}{\lambda_1 . b . f_m}$$

$$\varepsilon_{psu} = \varepsilon_{sp} + \varepsilon_{psa} \qquad -(iv)$$

where ε_{psa} = strain due to applied load
ε_{sp} = strain due to prestress

Assuming full bond between the steel and concrete at failure, the strains in the prestressing and non-stressed reinforcement respectively are given by:

$$\varepsilon_{psa} = \varepsilon_m . \frac{(d_p - n)}{n} \qquad -(v)$$

$$\varepsilon_{su} = \varepsilon_m \left(\frac{d_s - n}{n}\right) \qquad -(vi)$$

where ε_m is the ultimate strain derived from the brickwork prisms test.

Once ε_{psu} and ε_{su} are known, f_{psu} and f_{su} may be obtained from the respective stress-strain relationships for the steel. This method for the calculation of ultimate moment involves a process of trial and error to calculate n, such that $F_c = F_t$.

Once this is satisfied, moment is given by:

$$M_{su} = f_{psu} . A_{ps} [d_p - \lambda_2 . n] + f_{su} . A_s [d_s - \lambda_2 n] \qquad -(vi)$$

The theoretical moment thus calculated was compared with the experimental results in Table 3.

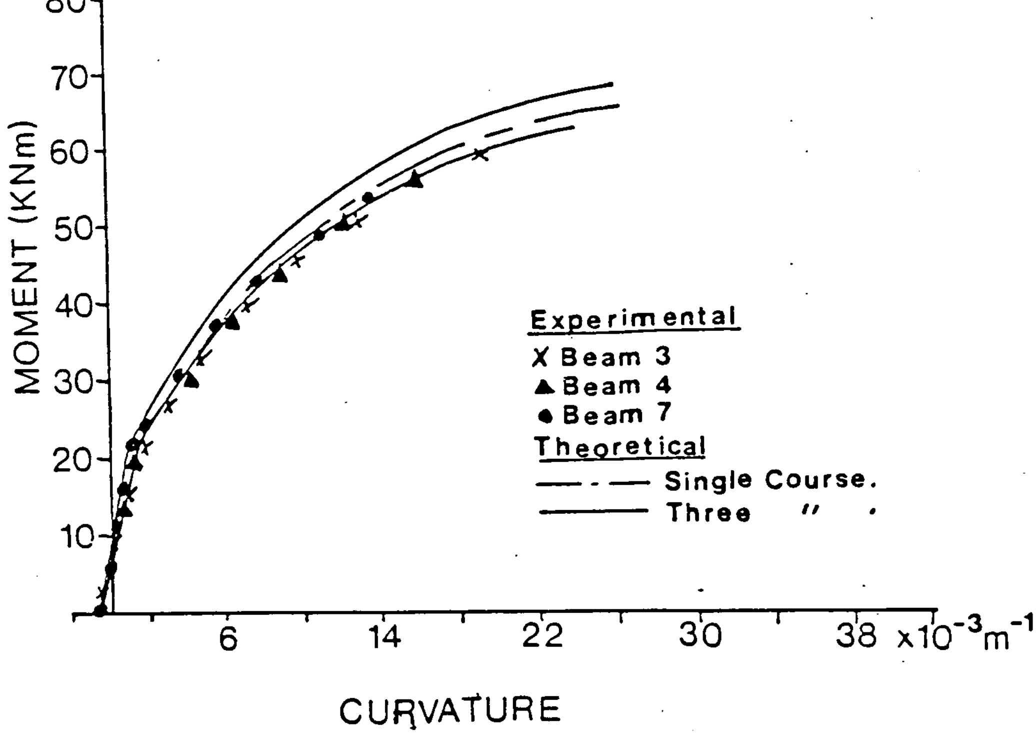


Fig. 10 Moment-curvature relationship for beams of 0.47% steel

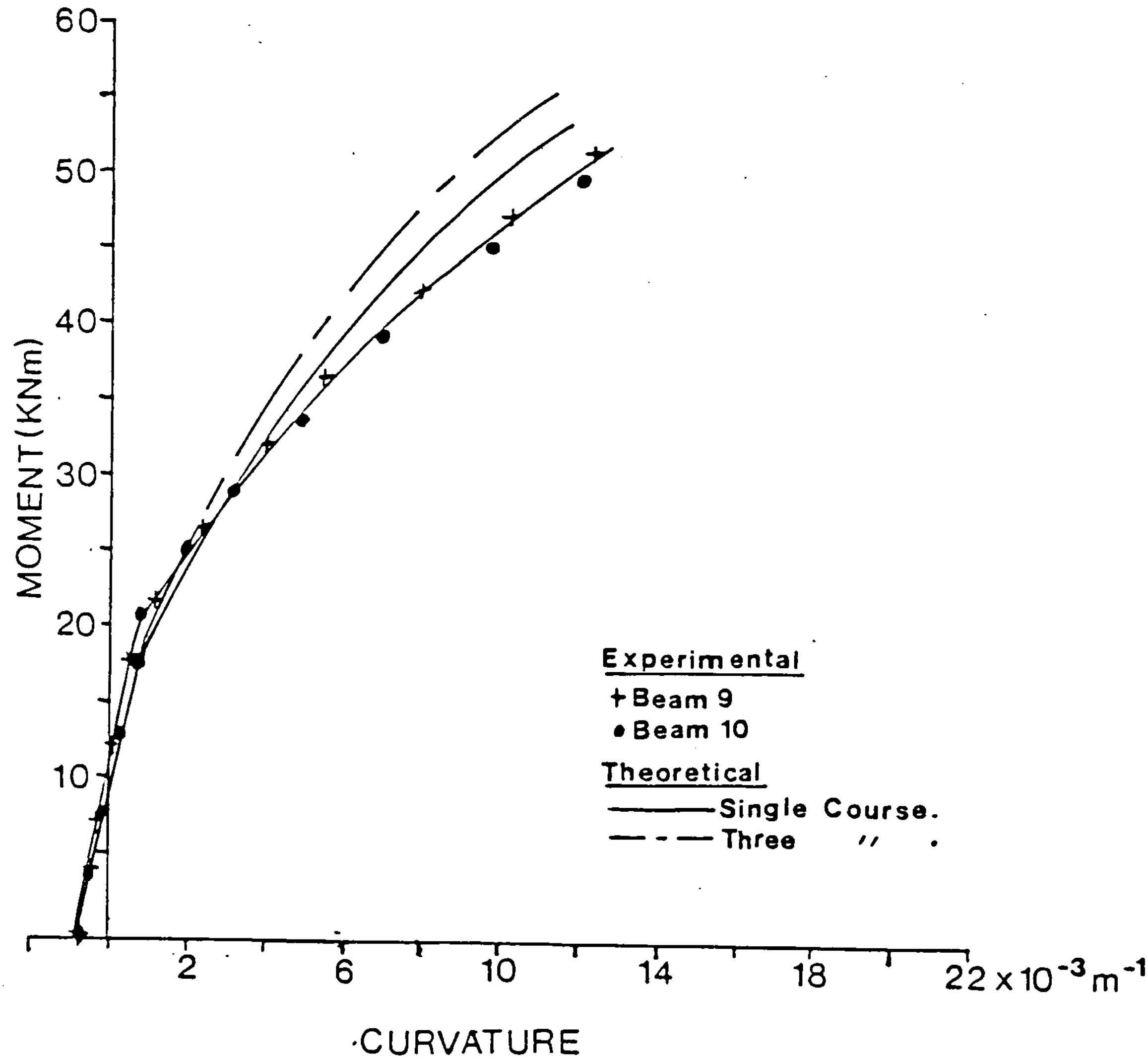


Fig. 11 Moment-curvature relationship for beams of 0.61% steel.

RESULTS AND DISCUSSION

The test results for all the beams and their mode of failure is given in Table 2.

Moment-Curvature Relationship: Typical moment-curvature relationship for the tested beams are shown in figures 9 to 11, the beams (figs.9,10) with a 0.47% of steel which failed in flexure show three distinct phases: linear up to cracking, cracking up to yield stress of steel and post yield phase when it becomes parallel to x-axis. The beams with 0.47% and 0.61% steel area which failed prematurely due to shear, the third phase was completely absent (fig.11).

As expected, the curvature for the beams 9-10 with higher percentage of steel (0.61%) was lower than for beams 1-4 with percentage of steel equal to 0.47.

From figs 9-11, it can be seen that there is very good agreement between the experimental and theoretical values of m-ϕ derived by direct method from both types of brick prisms, but the single course prism results giving slightly better agreement.

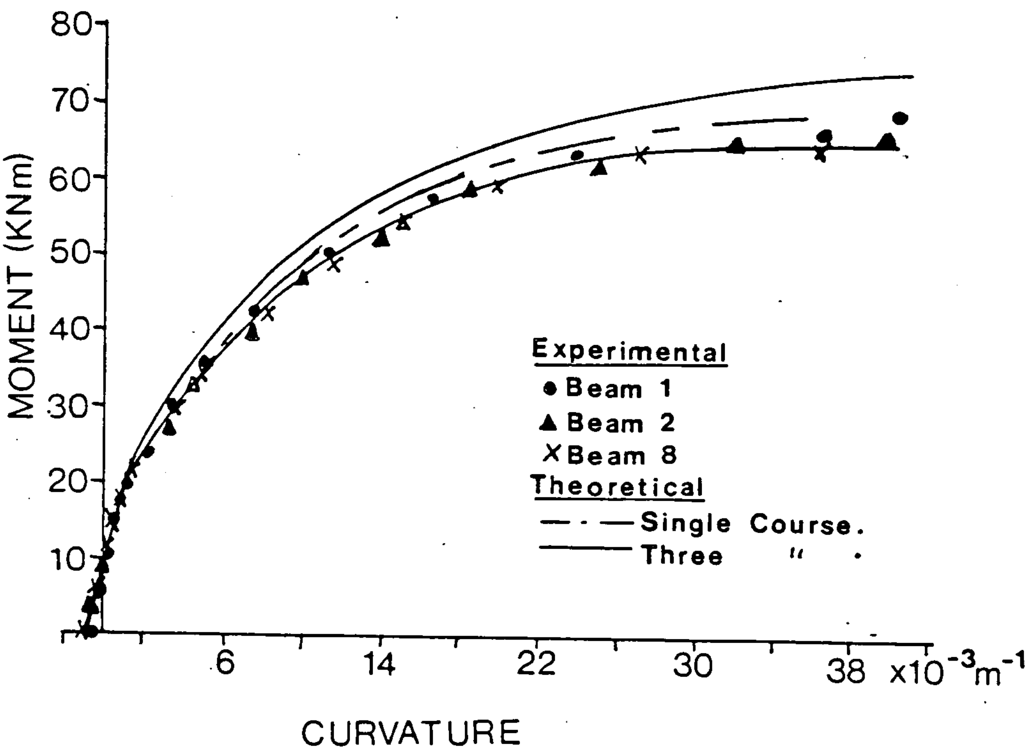


Fig. 9 Moment-curvature relationship for beams of 0.47% steel

Deflection: Figs 12, 13 and 14 show typical moment-deflection relationships for the tested beams, Fig. 11 indicating three distinct phases as with the moment-curvature relationship. Comparing the predicted deflections with those experimentally derived the agreement is good, especially for the deflections using the results of the single course prism tests.

The recovery of deflection after release of the load was between 23 and 46% for beams failing in shear.

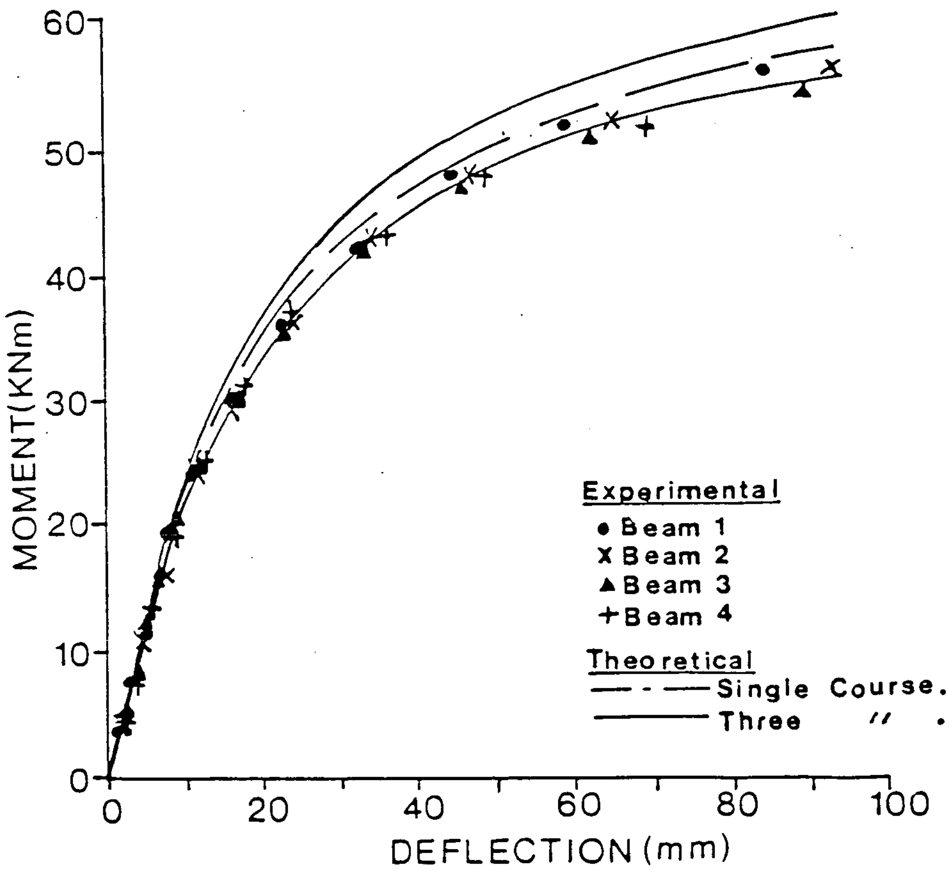


Fig. 12 Moment-deflection relationship for beams of 0.47% steel, span 6.2m.

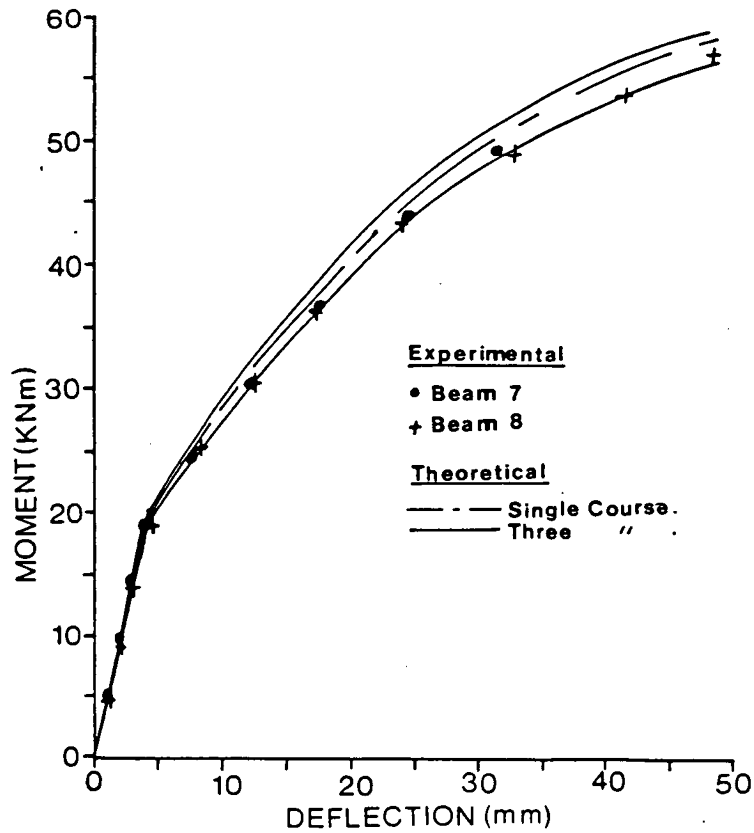


Fig. 13 Moment-deflection relationship for 0.47% steel, span 5.2 m

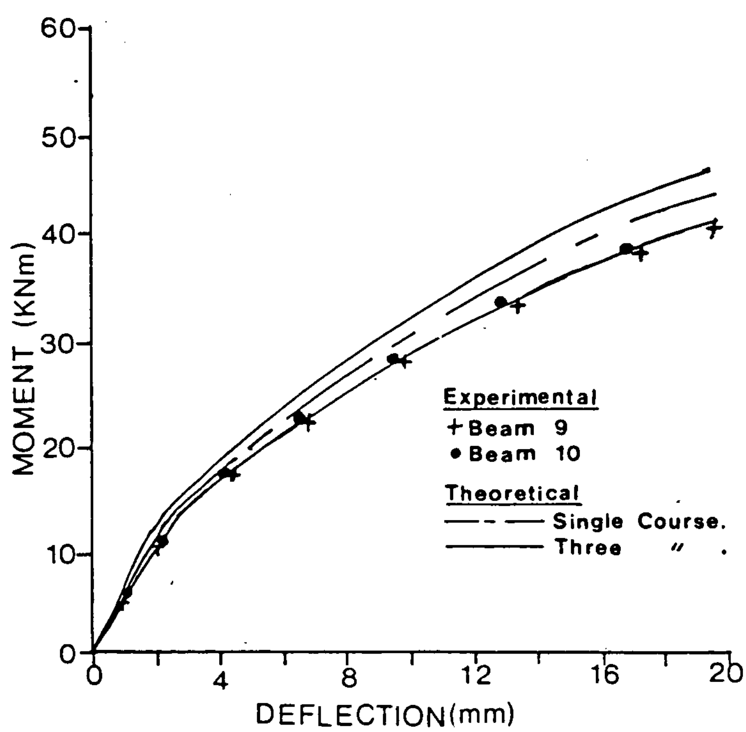


Fig. 14 Moment-deflection relationship for beams of 0.61% steel, span 5.2 m

am	Brick Strength N/mm^2	Mortar Strength N/mm^2	Grout Strength N/mm^2	Span m	Effective Prestress kN	Experimental Ultimate Moment kNm	Ultimate Shear Stress, V_u* N/mm^2	Failure Mode
1	82.0	25.8	25.8	6.2	70.4	67.60	0.39	Tension
2	82.0	23.2	24.7	6.2	68.2	66.70	0.39	Tension
3	82.0	16.9	19.8	6.2	68.5	61.33	0.36	Shear
4	82.0	19.8	21.7	6.2	67.8	58.13	0.34	Shear
5	58.9	24.9	25.3	6.2	66.8	59.43	0.35	Tension
6	58.9	29.9	36.5	6.2	66.8	59.43	0.35	Tension
7	82.0	17.0	21.4	5.2	66.9	52.30	0.31	Shear
8	82.0	30.7	21.2	5.2	69.3	73.09	0.42	Tension
9	82.0	26.9	-	5.2	69.9	59.25	0.36	Shear
.0	82.0	35.1	21.4	5.2	67.7	44.42	0.31	Shear

$V_u = \frac{V}{bd}$. Ultimate shear stress is calculated as the loading at failure, irrespective of the failure mode.

Table 2 Summary of Beam Test Results

Ultimate Moment and Mode of Failure: Beams 1, 2 and 8 (p = 0.47%) of the high strength brick all failed in tension, with yielding of the steel reinforcement leading to crushing of the brickwork (average ultimate moment = 69.1 kNm). Other beams 3, 4 and 7 in this series failed in shear, with a reduction in average ultimate moment of 17% (Table 2). The shear failures of these beams occurred with longitudinal splitting along the concrete/brickwork interface from the support to the loading point (Fig. 6).

The medium strength brick beams 5 and 6 both failed in tension (table 2) with a 14% reduction in ultimate moment compared with the average moment of the high strength brickwork beams failing in tension.

Shear failure occurred with shear cracks propagating from the support along the concrete/brickwork interface to the loading point (fig. 6) all the shear failures occurring suddenly with no warning. But unlike reinforced brickwork there was no 'total' collapse and the beams were still able to carry some load after failure.

Table 3 compares the experimentally and theoretically derived ultimate moment. The experimental results of beams which failed in flexure are only compared with the theory, since it assumes the crushing of the compression zone at ultimate failure. From table 3 it can be seen that the methods presented predict the moments to a very satisfactory degree of accuracy. Thus using either method presented the ultimate moment of a partially prestressed brickwork may be calculated.

Beam No.	Experimental Ultimate Moment, kNm	Moment predicted using stress block factors				Moment predicted by direct metho			
		SINGLE COURSE		THREE COURSE		SINGLE COURSE		THREE COURSE	
		kNm	Exp./theo.	kNm	Exp./theo.	kNm	Exp./Theo.	kNm	Exp./The
1	67.6	66.8	1.01	61.1	1.11	73.6	0.92	68.2	0.99
2	66.7	66.8	0.99	61.1	1.09	73.6	0.91	68.2	0.98
5	59.4	-	-	54.0	1.10	-	-	53.3	1.12
6	59.4	-	-	54.0	1.10	-	-	53.3	1.12
8	73.1	66.8	1.09	61.1	1.20	73.6	0.99	68.2	1.07

Table 3 Comparison of experimental and theoretical ultimate moments

SUMMARY AND CONCLUSIONS

i) The section used in this investigation proved satisfactory and no problems were encountered in prestressing, concreting and handling of the specimens.

ii) The ultimate moment of a partially prestressed brickwork beam can reliably be predicted by the methods proposed in this paper.

iii) The direct method proposed in this paper which takes the non-linear behaviour of materials into account predicts accurately the load deflection relationships of the partially prestressed brickwork beams up to failure.

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NOTATION

a	Shear span
b	Breadth of beam section
d_p	Depth of prestressing steel
d_s	Depth of non-stressed steel
f_m	Ultimate compressive strength of brickwork
f_{psu}	Stress in prestressing steel at failure
f_{su}	Stress in 'non-stressed' steel at failure
n	Neutral axis depth
A_{ps}	Area of prestressing steel
A_s	Area of non-stressed steel
E	Young's Modulus
F_c	Compressive force at failure
F_t	Tensile force at failure
M_u	Ultimate bending moment
ε_m	Ultimate compressive strain of masonry
ε_{psa}	Strain in prestressing due to prestress
ε_{pse}	Strain in prestressing steel at failure due to applied loading
ε_{psu}	Strain in prestress steel at failure
ε_{su}	Strain in 'non-stressed' steel at failure
λ, λ_2	Stress block factors

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A COMPARATIVE STUDY OF REINFORCED, FULLY AND PARTIALLY PRESTRESSED BRICKWORK BEAMS

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KEYWORDS

Beams, Brickwork, Prestressed, Reinforcement

ABSTRACT

Brickwork is very strong in compression, but very weak in tension. As a result, it cannot be used as a flexural member such as a beam, which carries the load predominantly due to bending; resulting in both tension and compression throughout the section. There are ways of countering this low tensile strength by using reinforcement or by prestressing or by a combination of both as in the case of partial prestressing, so that it can be used economically and effectively as a flexural member. With increasing pressure on finite resources, one has to look for cheaper alternatives to high energy input materials like steel and concrete, which are used, at present, for flexural members. Instead the flexural member built from brickwork may replace them, at least in the housing and public building sectors of both developed and developing countries. The brickwork beam, which does not require formwork nor the degree of sophistication needed for other materials may prove cheaper and viable in such a situation. Therefore, an R & D programme was undertaken to examine the behaviour of of such beams up to failure.

This paper summarises the results of tests on 12 full-scale reinforced prestressed and partially prestressed brickwork beams which were built and tested to study the load-deformation relationship up to failure. The experimental results were compared with the theoretical results. The theoretical results were obtained by using the non-linear properties of brickwork and steel.

Une Etude Comparative des Poutres en Briques Armées, Précontraintes et Partiellement Précontraintes.

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Université et d'Edimbourg, Ecosse.

Mots - clés

Poutres, Briques, Précontraintes, Armature.

Sommaire

Le briquetage est très résistant à la compression mais peu résistant à la traction. Par conséquent, on ne peut pas s'en servit comme membre flexible tel qu'une poutre, qui supporte la charge grace siotout a la flexion, ce qui produit á la fois la traction et la compression dans toute la section.

On peut compenser cette faible résistance à la traction en se servant de l'armature ou de la précontrainte ou d'une combinaison des deux, telle que la précontrainte partielle, afin de l'employer de façon économique et efficace comme membre flexible. A une époque où les ressources finies sout de plus en plus recherchées, il faut trouver des matériaux dont la production consomme moins d'énergie que l'acier ou le béton dout on se sert actuellement comme membres flexibles. On pourrait les remplacer par les poutres en briques, au moins dands les secteurs de la construction des logements et des travaux public dans les pays developpés aussi bien que dans les pays en voie de développement. La poutre en brique, qui n'a besoin ni de coffrage ni du degré de sophistication exigé par d'autres matériaux, pourait se révéler moins chere et viable dans cette situation. Donc on a entrepris des travaux de recherche et de développement afin d'examines le comportement de les poutres jusqu'à la defaillance.

Cette communication résume les résultats des expériences effectuées sur 12 poutres en briques de grandeur naturelle armées, préconstraintes et partiellement précontraintes, construites et essayées pour étudier le rapport charge-déformation jusqui'à la défaillance. Les résultats des expériences furent compare's avec les résultats théoriques. On a obtenu les résultats théoriques en se servant des propriétés non-linéaires de la brique et de l'acier .

INTRODUCTION

In many developing countries brickwork is the only indigenous material used for the construction of houses. To span openings reinforced concrete or steel is used. These materials are not only expensive but in short supply in these countries. With an acute housing shortage and constraint upon resources it is essential that cheaper alternatives should be tried. Reinforced or prestressed brickwork beams or slabs may offer a cheaper alternative for spanning openings. While utilising a labour intensive, widely available material, reinforced and prestressed brickwork flexural members do not require the sophisticated techniques involved with other conventional materials. Hence an investigation was undertaken at Edinburgh University to study the comparative behaviour of reinforced, partially and fully prestressed beams subjected to lateral loading.

The techniques of reinforcing[1] or full prestressing[2] have been successfully applied to brickwork beams, however partial prestressing has received little attention. Partial prestressing of a section is achieved by prestressing only part of the tensile reinforcement to a maximum allowable limit and leaving the rest non-tensioned[3]. Reinforced and fully prestressed members where either all of the reinforcement is non-tensioned or all the reinforcement is fully prestressed.

MATERIALS All materials used in the test programme conformed to relevant British Standard.

MORTAR A 1:1/4:3 (Cement:lime:sand) mix by volume was used. 100mm cubes were taken, the average compressive strength of the mortar at 28 days for each beam is given in Table 1.

GROUT For beam series A a grout mix of 1:2 (Cement:Sand) by volume was used. A concrete mix of 1;2½:2 (Cement:sand:pea gravel) was used in beam series B,C and D.

A plasticiser, 'Conbex', was added to the mixes to reduce the effect of shrinkage and shorten the setting time. 100mm cubes were taken during each grouting operation and tested at 7 days. The average compressive strength of the grout for each beam is given in Table 1.

BRICKS 3-hole perforated class B engineering bricks were used. The average compressive strength in the bed-joint direction was 96.6 N/mm².

REINFORCEMENT

7 wire stabilised steel strand was used for prestressing, average ultimate stress, f_{pu} = 1700 N/mm² and a 0.2% proof stress, f_{py} = 1640 N/mm².

High stress deformed bars were used for the non-stressed reinforcement, average ultimate stress, f_{su} = 670 N/mm² and a 0.2% proof stress, f_{sy} = 470 N/mm².

The experimental stress/strain relationships of the steel were idealised in tri-linear form[4] for use in subsequent theoretical predictions.

BRICKWORK PROPERTIES

COMPRESSIVE STRENGTH AND STRESS/STRAIN RELATIONSHIP

An understanding of the stress/strain characteristics of the brickwork are necessary for accurate predictions of the behaviour of brickwork beams. The ultimate compressive strength and stress/strain properties of brickwork were obtained from the prism illustrated in fig.1, which represents the top course (compression zone) of the brickwork beam section.

Twenty prisms were loaded in uniaxial compression and measurements of strain were taken at increments of loading up to failure using a 'demec' gauge. The average compressive strenght (f_m) was 33.5N/mm² and the ultimate strain(ε_m) was 0.00356.

The experimental stress/strain relationships were mathematically idealised in the form of a third degree polynomial, such that:

$$f/f_m = x_1(\varepsilon/\varepsilon_m) - x_2(\varepsilon/\varepsilon_m)^2 + x_3(\varepsilon/\varepsilon_m)^3 \qquad (1)$$

where $x_1 = 1.96$, $x_2 = 1.60$ and $x_3 = 0.64$

The stress block factors, $\lambda_1 = 0.61$ and $\lambda_2 = 0.37$ are given by:

$$\lambda_1 = \int_0^1 [x_1(\varepsilon/\varepsilon_m) - x_2(\varepsilon/\varepsilon_m)^2 + x_3(\varepsilon/\varepsilon_m)^3]\, d(\varepsilon/\varepsilon_m) \qquad (2)$$

$$\lambda_2 = 1 - \frac{\int_0^1 [(\varepsilon/\varepsilon_m)\{x_1(\varepsilon/\varepsilon_m) - x_2(\varepsilon/\varepsilon_m)^2 + x_3(\varepsilon/\varepsilon_m)^3\}]\, d(\varepsilon/\varepsilon_m)}{\lambda_1} \qquad (3)$$

CONSTRUCTION OF THE BEAMS AND TESTING PROCEDURE

The section used for the reinforced and partially prestressed brickwork beams is shown in fig.1. The section used for the fully prestressed[2] brickwork beams and the development of these sections are dealt with elsewhere[4,5].

All test beams were built 'upside down' on the floor of the laboratory by an experienced bricklayer. They were allowed to cure for 21 days before prestressing. The tendon and non-stressed reinforcement were placed in the cavity and 25mm thick mild steel plates were attached to the ends of the beams. The tendons were stressed to 70% of their ultimate strength, after stressing the cavity was grouted. For the reinforced beams the cavity was grouted at 21 days. The beams were cured for a further 7 days prior to testing. All beams were designed for approximately the same ultimate moment.

The beams were tested in a two point loading rig (fig.2) which provided a pin and roller support (simply supported) over a span of 6.2m. Loading was applied by hydraulic jacks and measured using load cells connected to a digital volt meter and pen chart recorder. The load was applied incrementally up to failure. At each loading brickwork strains in the constant moment zone at various depths were recorded using a 'demec' gauge. Steel strains were measured using electrical resistance strain gauges. Central deflection was measured with a dial gauge reading to 0.01mm.

EXPERIMENTAL RESULTS AND DISCUSSION

MODE OF FAILURE

Both the fully and partially prestressed brickwork beams exhibited typical flexural failures characteristic of an under-reinforced section, yielding of the tensile reinforcement leading to crushing of the brickwork in compression (fig.3). In the partially prestressed brickwork beams yielding of the non-tensioned reinforcement occurred at a moment equal to 75% of the ultimate, the prestressing steel yielded at approximately 90% of the ultimate moment. The maximum strain in both types of reinforcement at failure was approximately 2%. The tensioned steel in the fully prestressed beams (series A) reached its proof stress at 95% of the failure moment.

The maximum steel strain in the reinforced brickwork beam (series D) at failure was 1%, indicating that the steel had yielded even though failure was due to secondary shear. Inclined flexural cracks in the shear span propagated along the brick/mortar interface to the loading point, eventual failure occurred suddenly without warning (fig.4). Therefore by prestressing flexural cracking is delayed and hence the effective shear resistance of the section is increased and secondary shear failure is avoided.

CRACKING AND ULTIMATE MOMENT

CRACKING MOMENT

As soon as the extreme tensile fibre stress exceeded the flexural strength of the brickwork, visible cracking appeared in the reinforced brickwork beams, and the initial cracks penetrated to a height of 150-200mm. The average cracking moment was 9.3kNm (Table 1). Prestressing enhanced the performance of the beams by raising the threshold of the cracking moment, first cracking appeared in the fully prestressed beams at an average moment of 26kNm (Table 1), an increase of 280%. The cracking moment of the partially prestressed beams ranged between 13.7 and 17.4kNm depending upon the level of pre-compression, an increase of 147% and 187% in comparison with the reinforced brickwork beams. The initial crack height in all the prestressed members was approximately 100mm. Therefore, the onset and initial height of flexural cracks in brickwork beams can be controlled by prestressing.

ULTIMATE MOMENT

All beams were designed as under-reinforced and to fail at a similar ultimate moment. The experimental results are presented in Table 1. Although, all beams were designed for similar ultimate moment the reinforced brickwork beams exhibited a decrease in failure moment of up to 10% due to the premature secondary shear failure. By delaying flexural cracking and thereby increasing the effective shear resistance of the section the full flexural capacity was realised in the prestressed brickwork beams.

The test results are compared in Table 1 with a theoretical approach which utilises the actual stress/strain relationships of the materials, developed and described in detail elsewhere(2). For the fully and partially prestressed brickwork beams the theory generally underpredicts the ultimate moment although this difference is less than 10%. The theoretical prediction of the ultimate moment for the reinforced brickwork beams overestimates in comparison with the test result due to the secondary shear failure, even though the stress in the reinforcement had exceeded the proof stress. The calculation of ultimate moment assumes that the maximum compressive strain in the brickwork at failure is equal to 0.356%, compressive stress 33.5N/mm², whereas the maximum compressive strain measured at failure was only 0.28%, compressive stress equal to 28.9kN/mm². Once the reinforcement has yielded there is little increase in stress with strain, and so in order to balance a tensile force similar to that at failure at a stress of only 28.9N/mm² the depth of the compression zone will be greater than at ultimate, thereby reducing the lever arm and hence the ultimate moment.

MOMENT-CURVATURE AND LOAD-DEFLECTION

The experimental results for average moment-curvature and load-deflection are presented in figs. 5 & 6 respectively. The experimental curvatures were obtained from the brickwork strain readings taken in the constant moment zone, curvature equal to the slope of the strain profile at each loading.

Also in figs. 5 & 6 the experimental points are compared with theoretical relationships for moment-curvature and load-deflection. The method used was developed by Pedreschi[2] to predict the moment-curvature and load-deflection relationships of fully prestressed brickwork beams. It utilised the experimental idealised stress/strain relationship[4] for the prestressing and reinforcing steels. Although the theory was developed for fully prestressed beams there is excellent agreement with the predicted and experimental values for all test beams, and therefore is equally applicable to either reinforced, fully prestressed or partially prestressed brickwork beams.

In fig.5 there is an initial negative curvature caused by the prestress, the higher the prestress the larger the curvature. Upon loading the moment-curvature relationship has three distinct phases:

(i) linear up to cracking

(ii) cracking up to yielding of the steel, and

(iii) post-yield phase where it eventually becomes parallel to x-axis

The load-deflection curves (fig.6) show similar characteristics to the moment-curvature relationships. Unlike the moment-curvature relationships the load-deflection curves all start from the origin since it was not possible to measure the deflection due to self weight and so the load-deflection corresponds to applied loading.

The deformation of the partially prestressed brickwork beams lies between the boundaries represented by the fully prestressed and reinforced brickwork beams (figs.5,6). Prior to cracking the slope of the moment-curvature and load-deflection relationships for each type of beam was equal. With increasing loading each beam cracks and so the deflection at any particular loading will be greatest in the beams with the least prestress. For example at a bending moment of 26kNm the deflection due to applied load for beam series A,B,C and D was 4.5, 12.5, 14.0 and 17.0mm respectively. By prestressing the deflection has been reduced by 74% for beam series A, 26% for series B and 18% for series C in comparison with the reinforced brickwork beams. The rate of increase in deformation with further loading was less for the beams with the largest areas of non-tensioned reinforcement, due to the extra stiffness of the section resulting from the non-tensioned steel. Therefore by appropriate selection of the level of prestress the deflection of any brickwork beam may be controlled to within the limits defined by the reinforced and fully prestressed beams.

The average moment and deflection for each beam type at a measured maximum crack width of 0.2mm is given in Table 1. By prestressing, the deflection in comparison with the reinforced brickwork beams has been reduced by 26% in the fully prestressed beam and by between 6% and 10% in the partially prestressed beams. Conversely the moment has increased by up to a factor of 2, from 14.3kNm to 29.0kNm, depending upon the level of prestress. The deflection of all four types of beam satisfies the serviceability limit state of deflection of span/250(6) (24.8mm) and hence for design the limit state of cracking becomes the controlling factor. The factor of safety, ratio of ultimate moment to the moment at a maximum of crack width is 0.2mm, for the reinforced brickwork beam is 3.55. For the fully prestressed and the partially prestressed beams the factor of safety is 1.82, 2.49 for series B and 3.33 for series C respectively. Although the safety factor is adequate for all beams the prestressed brickwork beams provide the most economical use of the materials by raising the magnitude of the moment at the serviceability limit state and therefore keeping the factor of safety to a minimum.

SUMMARY AND CONCLUSION

(i) Prestressing increases the cracking moment of a brickwork beam, hence cracking can be avoided under service loading by suitably prestressing the section.

(ii) All test beams were under-reinforced and designed to reach same ultimate moment, but the reinforced beams primarily failed due to yielding of steel leading to secondary shear failure. This resulted in 10% reduction in the ultimate moment compared to the fully and partially prestressed beams. Thus prestressing enhances the effective shear capacity of the beam.

(iii) The ultimate moment, moment-curvature and load-deflection relationships of reinforced, fully and partially prestressed brickwork beams can be accurately predicted using the experimentally idealised stress/strain relationships for the brickwork, prestressing and reinforcing steels.

(iv) The deflection of a brickwork beam can be controlled to within the limits defined by the fully prestressed and reinforced brickwork beams by appropriate selection of the level of the prestress. The deflection is least in the fully prestressed brickwork beam and greatest in the reinforced brickwork beam for all loads up to failure.

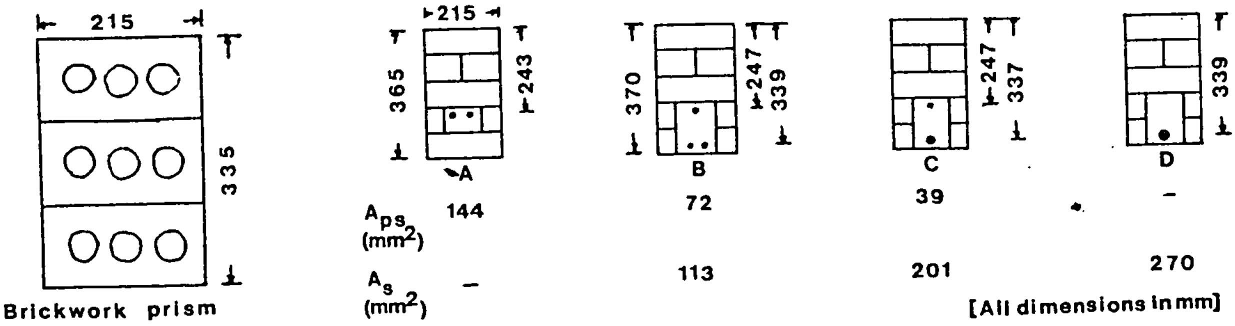

Fig.1 Brickwork Prism and Details of Beam Sections

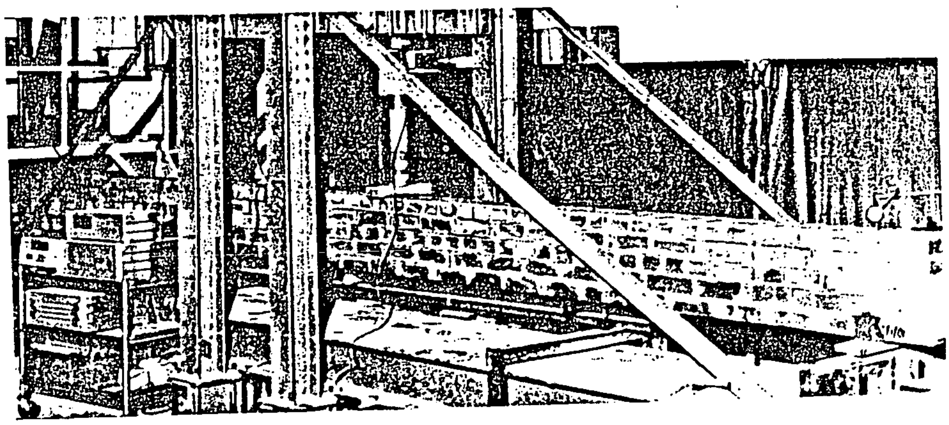

Fig.2 Test set-up

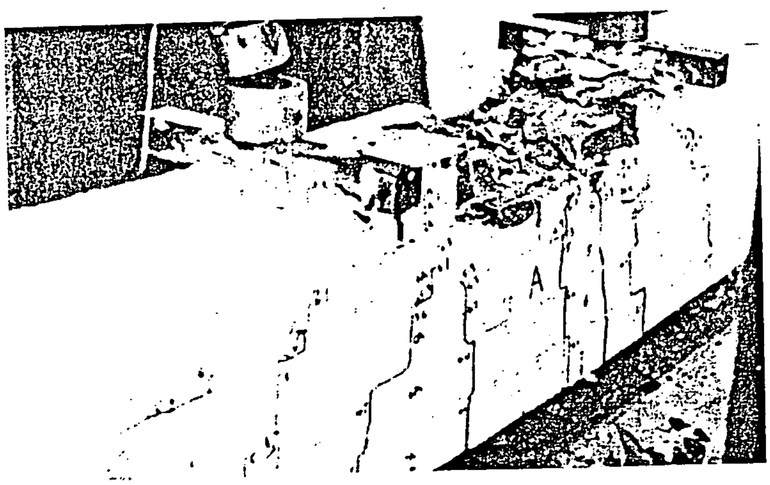

Fig.3 Typical Flexural failure of fully and partially prestressed brickwork beams

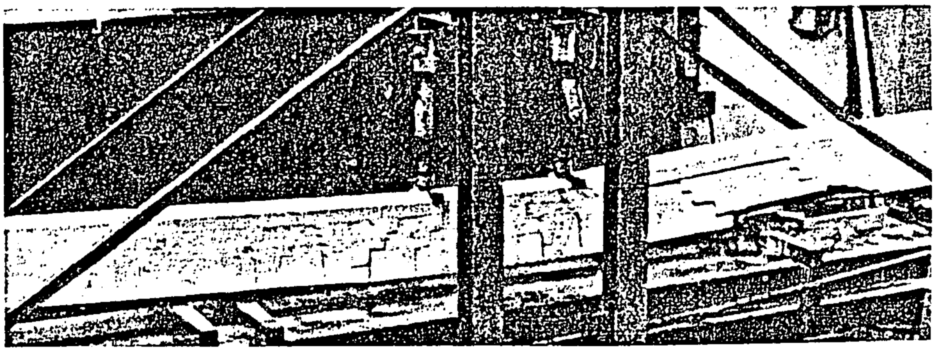

Fig.4 Typical secondary shear failure of reinforced brickwork beams

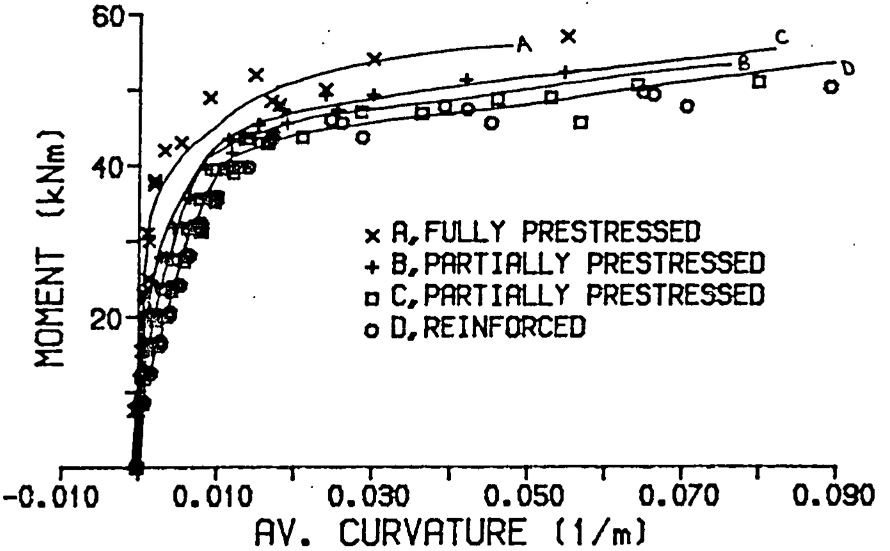

Fig.5 Moment-curvature relationships for test beams

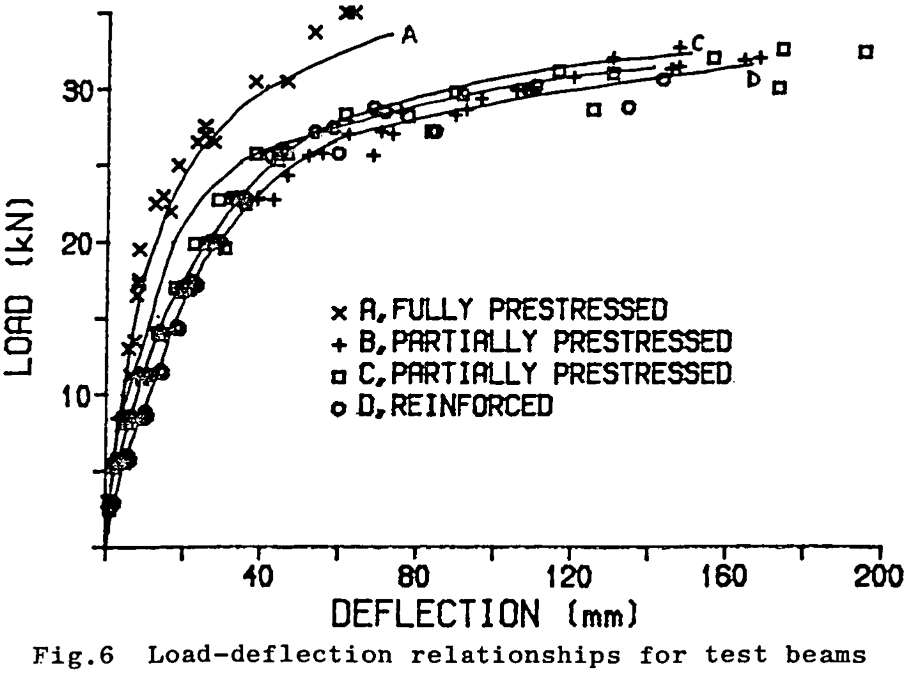

Fig.6 Load-deflection relationships for test beams

Table 1 Summary of Beam Test Results

Beam No.	Strength N/mm^2		Effective Prestress kN	Moment kNm			$\frac{M_{ult}}{M_{th}}$	Moment at 0.2mm crack kNm	Deflection at 0.2mm crack (mm)
	Mortar	Grout		Cracking	Ult. Expt. M_{ult}	Theory M_{th}			
A1	15.8	17.8	133	26.1	52.9	54.3	0.97		
A2	15.8	17.8	115	23.7	56.4	54.3	1.04	29.0	7.1
A3	16.6	13.4	152	28.2	58.8	54.3	1.08		
B1	19.2	27.5	67	17.4	52.8	50.4	1.05		
B2	19.9	20.0	67	17.4	53.2	50.4	1.06	21.5	8.6
B3	16.4	26.1	61	16.7	54.6	50.4	1.09		
C1	19.9	20.3	36	13.9	56.6	54.9	1.03		
C2	20.1	25.2	35	13.7	55.3	54.9	1.01	16.5	9.0
C3	17.4	20.6	42	14.6	52.9	54.9	0.96		
D1	16.9	23.0	-	9.3	52.2	53.9	0.97		
D2	19.7	18.8	-	9.3	48.9	53.9	0.91	14.3	9.6
D3	21.8	17.5	-	9.3	51.3	53.9	0.95		

A1 to A3 - Fully Prestressed

B1 to B3 & C1 to C3 - Partially Prestressed

D1 to D3 - Reinforced

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PREDICTING THE FLEXURAL STRENGTH OF PRESTRESSED BRICKWORK BEAMS

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Abstract—The accuracy of prediction of the ultimate strength of prestressed brickwork beams is considered. A large series of tests on brickwork prisms are used to derive the compressive strength and strain characteristics of brickwork loaded parallel to the bedjoint. The data are then used to compare the predicted ultimate moment with a series of full-scale beam tests. The paper considers the use of the prisms to establish the ultimate moment, neutral axis and ultimate compressive strain at failure.

NOMENCLATURE

Aps	area of prestressing strand
b	breadth of beam
C	compressive force in beam
d	effective depth of beam
fm	stress in masonry at failure
fsu	stress in steel at failure
Mu	ultimate moment capacity of beam
n	neutral axis depth of beam
p	force in steel due to prestress
pu	ultimate tensile force in steel
Ts	force in steel at failure
λ_1	ratio of stress block area to enclosing rectangle
λ_2	centroid of stress block
λ_3	ratio of strength of masonry in prism to strength in beam
ε_{ps}	strain in steel due to prestress
ε_{sa}	strain in steel due to applied loads
ε_{su}	total strain in steel

1. INTRODUCTION

In conventional load-bearing brickwork applications compressive forces develop normal to the bedjoint, hence the compressive strength properties in this direction have been well researched and documented. More recently, however, the use of brickwork in flexural applications has led to a growth in the use of reinforced and prestressed masonry. In this application, compressive forces may occur primarily in the direction parallel to the bedjoint. Brickwork being anisotropic, has compressive strength characteristics that are greatly influenced by the orientation of the applied forces to the bedjoint. Generally speaking the behaviour of prestressed brickwork exhibits many similarities to prestressed concrete. However, unlike concrete the adoption of a suitable test specimen has considerably more difficulties, there being no ready equivalent to the concrete cube or cylinder. A suitable test specimen in addition to using the same brick and mortar combination should also reflect the bonding pattern of the full-scale structure and be tested in the same direction as the applied forces. Some research on brickwork prisms tested parallel to the bedjoint has already been reported.[1,2] The results from brickwork prisms when they are used in the prediction of the flexural strength of prestressed brickwork beams should model as accurately as possible the behaviour of the compression zone. This paper presents the results of a series of prism tests on a range of brick types and mortar grades from which the compressive strength, ultimate strain and stress block characteristics were obtained. Two different formats of prism were used, each modelling the bonding pattern of prestressed brickwork beams. This work formed part of an experimental study into the behaviour of full-scale prestressed brickwork beams. A theoretical analysis of the beams was carried out using the compressive characteristics from the prism tests and compared with the experimental behaviour of the beams. The ultimate flexural moment, maximum compressive strain and neutral axis depth at failure are considered. This paper is primarily concerned with the theoretical analysis of prestressed brickwork beams. Comparisons with current design methods are currently underway and will be dealt with elsewhere.

2. BRICKWORK PRISMS

2.1. *Bricks*

Seven different bricks were used. The compressive strength tested on bed varied from 22–82 N mm^{-2}. The format of the bricks is shown in Fig. 1. The bricks were tested in compression in each of the three orthogonal directions shown in Fig. 2. The results are given in Table 1. The values are based on the gross cross-sectional properties. Brick types 1–3, with three holes, had an average area of perforation of 14–15%. Brick type 4 was a single-frogged common brick. Bricks 5–7 were all

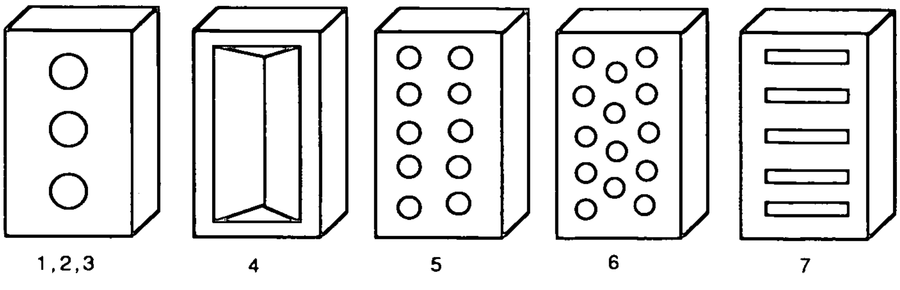


Fig. 1. Bricks used in study.

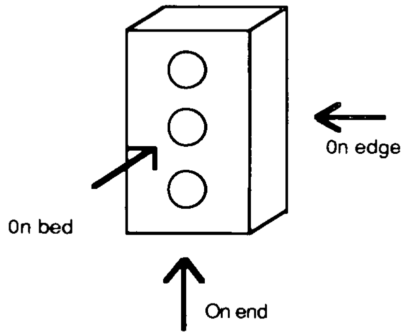


Fig. 2. Direction of tests on bricks.

perforated in excess of 20%. The results show, as has been previously reported,[3] that the compressive strength of bricks tested on bed tends to be greater than either of the two other orthogonal directions.

2.2. *Brickwork prism tests*

A number of brickwork prisms were built using the five brick types. Two formats of prism were built (Fig. 3). Both of these represent the upper courses of a prestressed brickwork beam. A 1:¼:3 (cement:lime:sand) mix was used for the mortar. Additional tests were carried out using brick type 2 and a 1:½:4½ mix. The procedure for testing was as follows. The prisms were cured for 28 days and then levelled and capped with a rich mortar mix. Three-millimetre plywood sheets were placed between the prism and the platens of the test machine. Generally strain measurements were taken using a "demec" gauge at four points on the single-course prisms and six points on the three-course prisms. In a number of single-course prisms, strain measurements were taken at six points to compare the strain distribution with the three-course prisms. An initial preload was applied and the strains were recorded. Small adjustments were made to ensure axial loading. The load was then applied in increments to failure. For brick types 1–4 a total of six specimens were tested for each prism type with strains recorded in three of these. For brick types 5–7, three specimens of each were tested. The results are summarized in Table 2. Further information can be found elsewhere.[4,5] From Table 2 there are considerable differences between the strengths and ultimate strains obtained with each prism type for a given brick and mortar combination. With the exception of brick types 4 and 6, the single-course prisms exhibited both greater compressive strengths and higher ultimate strains. The behaviour of single-course prisms in brick type 4 may be attributed to the presence of the frog. Although this was filled with mortar, differences in elastic properties of the bricks and mortar cause cracking around the interface between the brick and mortar thereby reducing the effective cross-sectional area and introducing an eccentricity into the loading. Tests on a similar

Table 1. Characteristics of bricks

Brick type	Compressive strength (N mm^{-2})						Absorption (% by wt)	
	On bed		On edge		On end			
	Average	S.D.	Average	S.D.	Average	S.D.	Average	S.D.
1	82.03	5.85	53.17	9.43	40.23	17.25	4.17	0.46
2	67.58	12.20	26.36	5.71	23.23	25.40	5.71	0.90
3	34.18	2.79	11.48	3.54	10.67	30.37	7.20	0.30
4	22.72*	3.36	16.95	2.24	15.81	14.71	23.85	0.24
5	73.21	8.26	29.52	4.11	20.11	3.20	5.42	0.82
6	70.68	7.29	26.22	3.50	10.32	2.18	3.90	0.36
7	64.07	10.65	51.84	6.71	12.50	1.88	3.43	1.12

* Frog filled.

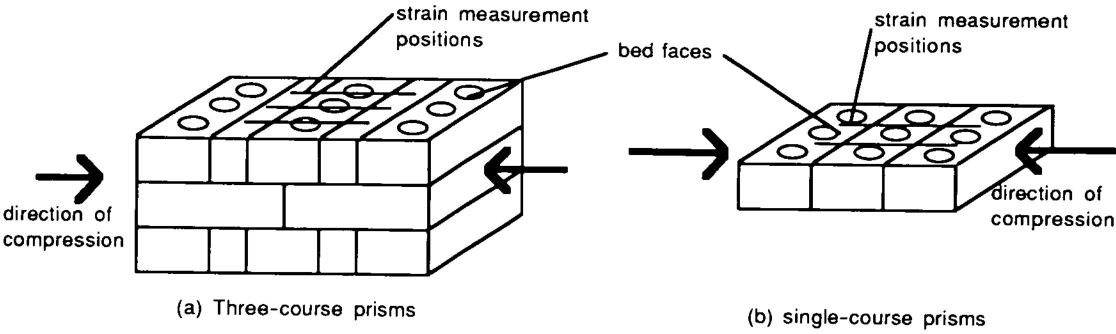


Fig. 3. Prism test pieces.

Table 2. Brickwork prism results

Brick type	Compressive strength (N mm^{-2})		Av. ultimate compressive strain		Stress block factors λ_1		Stress block factors λ_2	
	Prism type		Prism type		Prism type		Prism type	
	Three	Single	Three	Single	Three	Single	Three	Single
1	20.48	32.56	0.0021	0.0033	0.61	0.65	0.37	0.39
2	12.36	23.70	0.0017	0.0026	0.65	0.67	0.39	0.38
3	9.36	9.37	0.0015	0.0043	0.57	0.70	0.35	0.40
4	10.99	6.92	0.0018	0.0020	0.54	0.61	0.36	0.36
2*	13.48	16.93	0.0026	0.0030	0.68	0.69	0.39	0.39
5	14.50	15.16	0.0015	0.0016	0.52	0.58	0.34	0.36
6	12.68	9.00	0.0014	0.0015	0.58	0.59	0.35	0.38
7	15.55	27.89	0.0022	0.0038	0.60	0.51	0.36	0.33
Average					0.59	0.63	0.36	0.37

* 1:½:4½ mortar (Grade II).

prism with the frog removed are more fully explained elsewhere.[4] It was observed during the tests on the three-course prisms that splitting along the bedjoint tended to occur (Fig. 4) prior to ultimate compressive failure. Once this happened the strain distribution across the prism became very erratic (Fig. 5), whilst the strains in the single-course prisms were uniform up to higher levels of stress. The compressive stress at the point of splitting is given in Table 3. By expressing this as a percentage of the average failure stress of the three-course prisms and comparing with the difference in strength between the two prism formats it can be seen that the greatest difference is found when splitting occurs earlier in the loading cycle. This is illustrated in Fig. 6 where there appears to be a linear relationship between splitting stress and the percentage difference in the compressive strength of the single- and three-course prisms. This suggests that if premature splitting could be avoided, then there would be closer agreement between prism results.

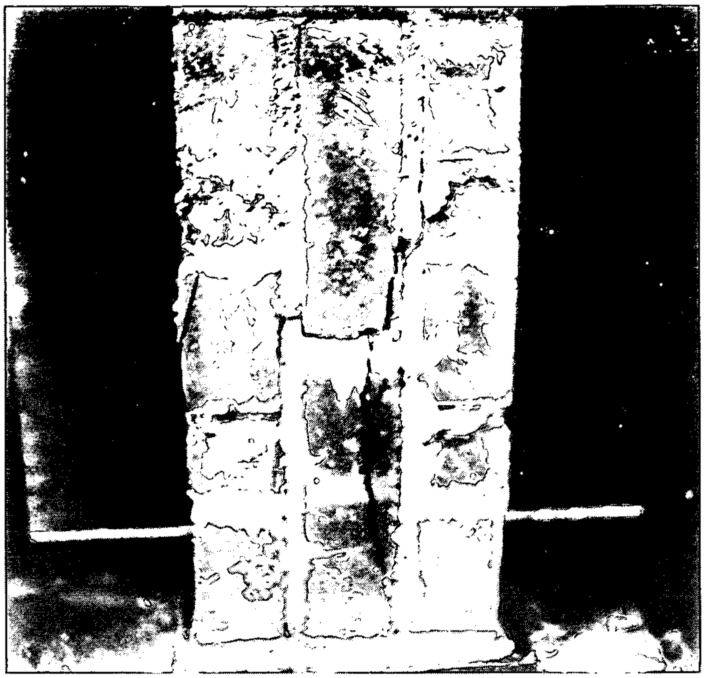
Returning to Table 2, the last four columns present the stress block factors. Stress block factors have been used to describe the shape of concrete and brickwork flexural members and form part of the ultimate load theory for such members (Fig. 7). The variation in the stress block factors is not nearly as great as the variation in compressive strengths and ultimate strains. The figures shown are in reasonable agreement with the results[1] of a

Table 3. Stress at splitting of bedjoints in three-course prisms

Brick type	Av. compressive stress at point of splitting (N mm^{-2})	Splitting stress as a % of failure stress	Difference in strength of prism types as % of lower result
1	12.88	60	59
2	9.89	80	92
2*	12.13	90	26
3	8.42	90	—
4	6.48	59	58
5	13.92	96	46
6	9.90	78	41
7	8.24	53	79

* 1:½:4½ mortar.

(a)



(b)

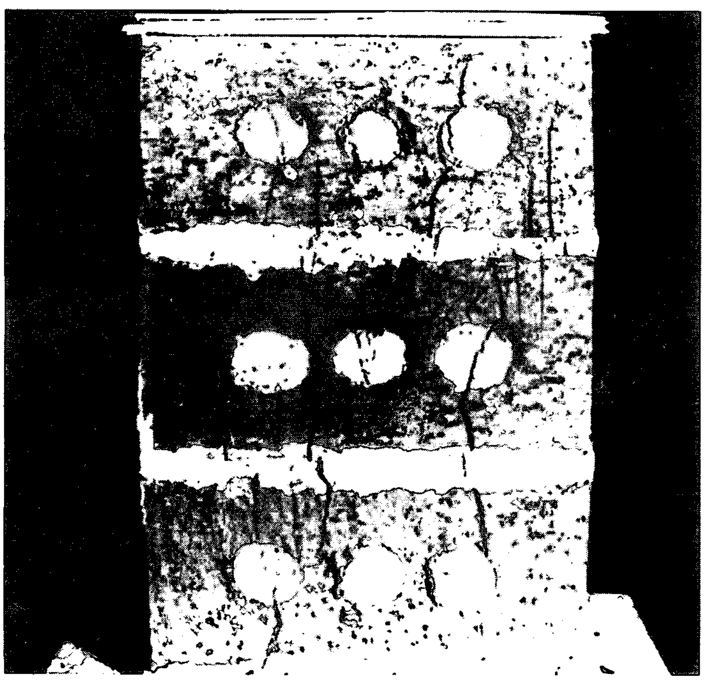


Fig. 4. (a) Failure of three-course prism. (b) Failure of single-course prism.

very large number of tests where average values of $\lambda_1 = 0.64$ and $\lambda_2 = 0.38$ were found.

3. FULL-SCALE BEAM TESTS

As part of a major research programme[4,5] a large series of full-scale prestressed brickwork beams were tested to failure. In addition to brick type and mortar grade, other variables included the degree of prestress, percentage area of steel and the influence of shear span/effective depth ratio. The cross-section of the beams is given in Fig. 8. The beams were tested under two-point loading in spans between 2.0 and 6.2 m.

A 10.9-mm-diameter, seven-wire prestressing strand was used for all the beams. The percentage

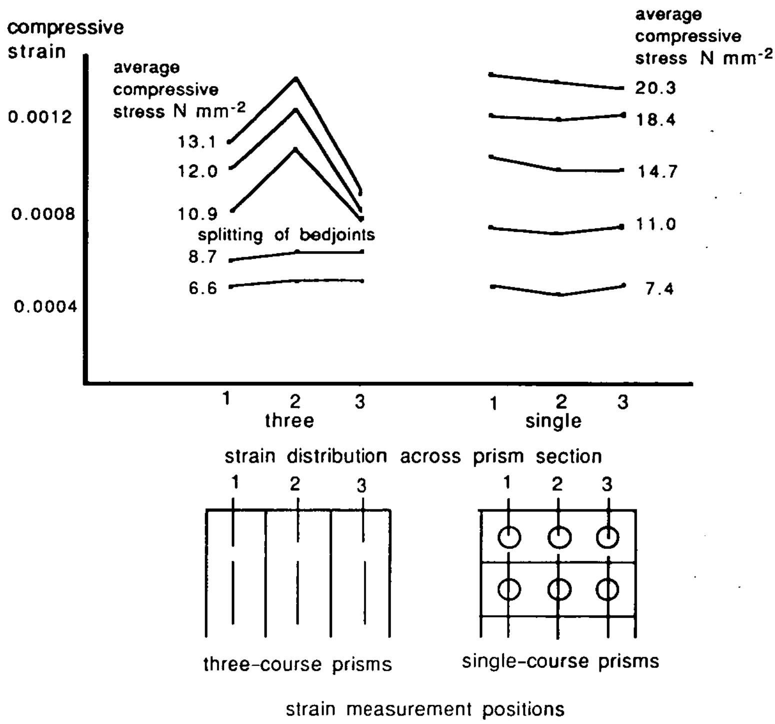


Fig. 5. Comparison of typical strain distributions across prisms at different levels of average compressive stress.

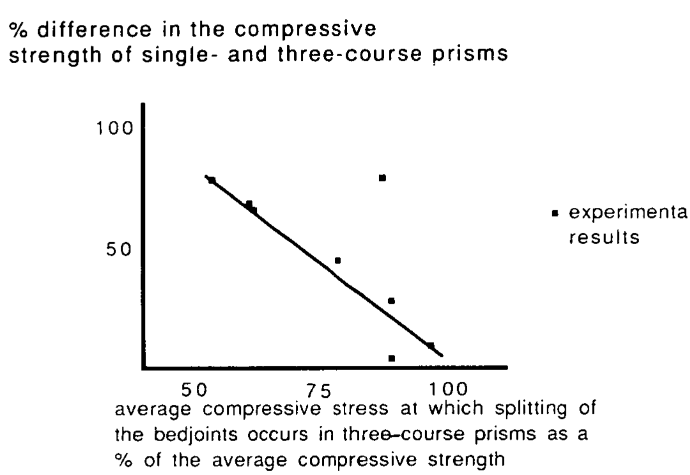


Fig. 6. Relationship between splitting stress of three-course prisms and the difference in strength of single- and three-course prisms.

area of prestressing varied from 0.255 to 0.548% using two, three or four strands per beam. Generally all strands were stressed up to 70% of their ultimate load. However, on some beams, those with weaker bricks, the prestress was reduced to 35%. Strain gauges placed on the strand allowed monitoring of the loss prestress between stressing and testing, usually 1–2 weeks. The average loss varied between 11.8 and 25% for 6.2-m-long beams and 2.0-m-long beams, respectively. Table 4 summarizes the results of those beams that failed in flexure whether they were over or under-reinforced. Strains were measured on the surface of the beams at various points as well as on the prestressing strand itself, enabling the neutral axis depth and extreme fibre strains to be found.

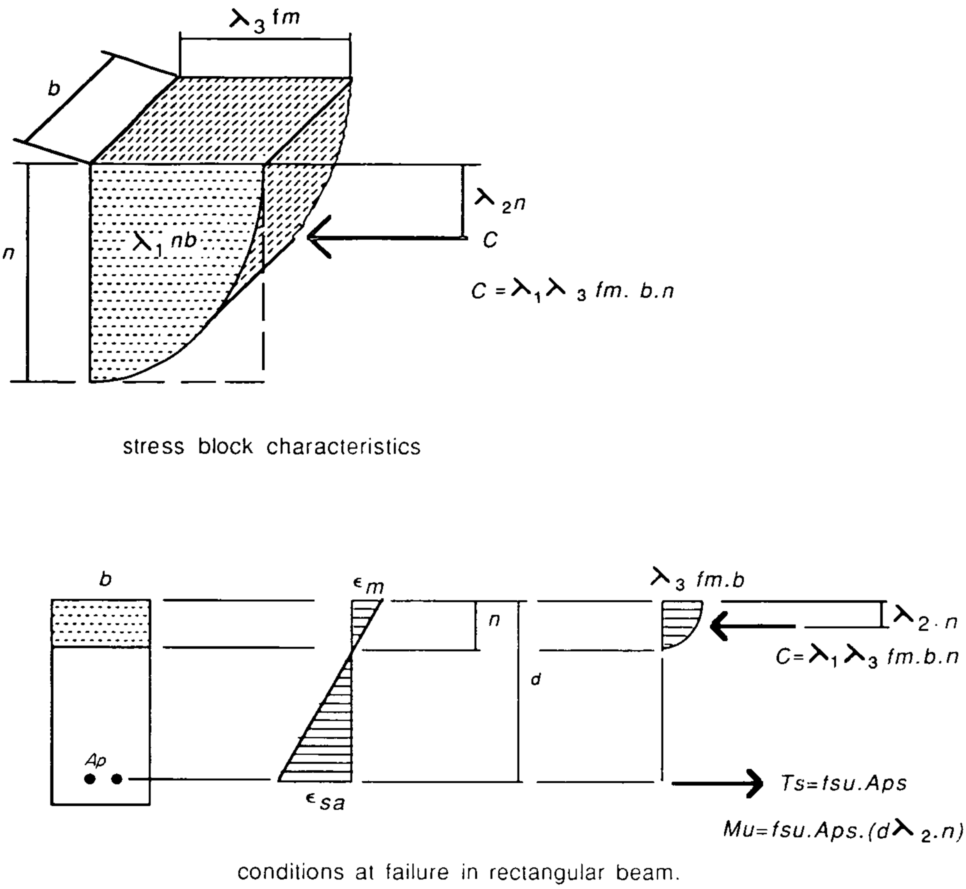


Fig. 7. Stress block characteristics and conditions at failure in prestressed brickwork beams.

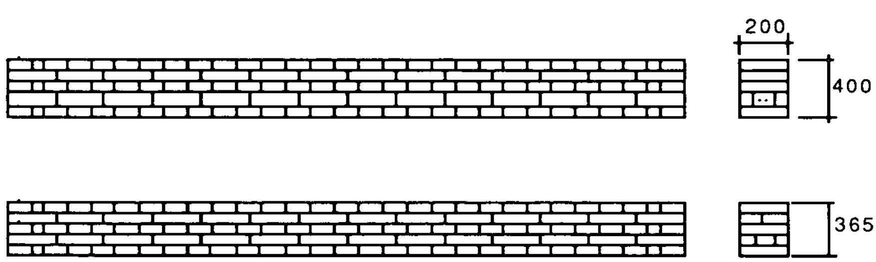


Fig. 8. Bonding patterns of beams.

4. COMPARISON BETWEEN EXPERIMENTAL RESULTS AND THEORETICAL ANALYSIS USING THE PRISM RESULTS

It is not possible to measure the compressive stress block directly on a full-scale beam. There are, however, three quantities that can be measured: ultimate moment, neutral axis depth and compressive strain at failure. If there is agreement in these three quantities between the experimental and theoretical results, then it may be concluded that the compressive stress characteristics from the prism tests provide an accurate description of the compression zone. Each of these quantities is now considered in turn.

4.1. *Ultimate flexural strength*

The ultimate flexural strength of the prestressed brickwork beams was calculated using conventional ultimate load theory[6,7] using the stress block characteristics given in Table 2 for each prism type and the conditions in Fig. 7. The stress–strain relationship for the prestressing strand was obtained by test and idealized as shown in Fig. 9.

An initial value for the neutral axis depth was assumed and the compressive force in the cross was given as

$$C = \lambda_1 fm \cdot b \cdot n$$

λ_3 was taken as equal to 1.0.

Table 4. Comparison between experimental and theoretical results for full-scale beam tests

Beam	Brick type	Prestress (kN)	% steel	Ult. moment (kNM) (a)	Predicted moments Single-course (b)	a/b	Three-course (c)	a/c
B1	1	133	0.274	52.9	54.3	0.97	49.4	1.15
B3	1	133	0.274	61.5	54.3	1.13	49.3	1.25
B4	1	144	0.274	58.5	54.4	1.07	49.7	1.21
B5	1	133	0.274	59.2	54.4	1.09	49.3	1.20
B6	1	152	0.274	58.8	54.4	1.08	49.5	1.19
BA3	1	216	0.441	74.8	72.5	1.03	58.1	1.29
BS1	1	194	0.548	87.2	94.8	0.92	62.1	1.40
BS2	1	202	0.548	92.6	95.0	0.98	67.3	1.38
A1	2	124	0.274	47.9	52.2	0.92	38.2	1.26
A3	2	124	0.274	46.0	52.2	0.88	38.6	1.19
A4	2	149	0.274	46.1	52.4	0.88	41.0	1.12
A7	2	134	0.274	53.4	52.3	1.02	39.6	1.35
A9	2	142	0.274	51.8	52.5	0.99	41.8	1.24
A10	2	134	0.274	54.1	52.3	1.03	39.6	1.37
A11	2	152	0.274	51.7	52.4	0.99	41.0	1.26
	2	129	0.274	56.3	52.2	1.07	38.7	1.45
1	2	152	0.255	61.4	58.7	1.05	44.5	1.38
2	2	141	0.255	49.5	58.7	0.84	44.5	1.11
3	2	124	0.255	49.4	58.7	0.84	44.5	1.11
4	2	134	0.255	56.9	58.8	0.97	47.9	1.19
5	2	140	0.255	56.0	58.8	0.95	46.7	1.21
6	2	303	0.51	77.1	90.9	0.84	62.1	1.25
7	2	287	0.51	67.8	90.1	0.75	61.5	1.10
AM1+	2	132	0.274	48.6	49.0	0.99	44.5	1.10
AM2+	2	127	0.274	45.8	47.8	0.96	42.2	1.09
AM4+	2	142	0.274	48.6	49.0	0.99	44.5	1.09
C1	3	75	0.274	45.9	36.11	1.29	26.7	1.75
V2	3	61	0.274	42.5	37.2	1.14	28.4	1.50
C3	3	119	0.274	54.1	40.25	1.34	33.4	1.62
D1	4	61	0.274	35.5	24.1	1.48	27.8	1.28
D2	4	72	0.274	25.8	24.8	1.04	28.7	0.90
10/1	5	151	0.274	50.96	45.9	1.11	45.0	1.13
10/2	5	150	0.274	52.3	45.9	1.14	45.9	1.16
10/3	5	133	0.274	57.1	45.4	1.23	45.7	1.25
14/1	6	154	0.274	54.5	35.9	1.52	43.6	1.25
14/2	6	150	0.274	52.6	35.8	1.47	43.5	1.21
14/3	6	130	0.274	51.3	35.1	1.46	42.8	1.20
5/1	7	152	0.274	61.0	52.6	1.16	46.2	1.32
5/2	7	151	0.274	57.1	52.4	1.09	46.0	1.24
5/3	7	136	0.274	55.2	53.1	1.04	46.4	1.19

The additional strain in the strand was obtained

$$\varepsilon_{sa} = \varepsilon_m \frac{(d-n)}{n},$$

the total strain being

$$\varepsilon_{su} = \varepsilon_{sa} + \varepsilon_{ps}.$$

The stress *fsu* in the prestressing strand was found from Fig. 9 and the total tensile force

$$Ts = fsu \cdot Aps.$$

The compressive force, C, was compared with the tensile force, Ts. This process was repeated, modifying the neutral axis depth until $C = Ts$.

The ultimate moment capacity Mu was then obtained

$$Mu = fsu \cdot Aps \cdot (d - \lambda_2 \cdot n).$$

The experimental and theoretical results are compared in Table 4. Generally the single-course prisms provide a closer estimate of the ultimate flexural moment than the three-course prisms. With the exception of brick type 2 in the single-course prism format, all the predicted results tended to

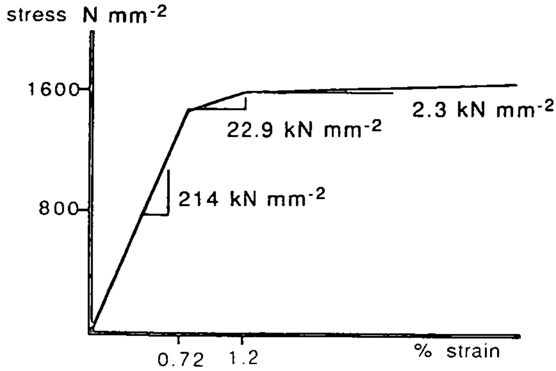


Fig. 9. Stress–strain relationship for prestressing strand.

Table 5. Comparison between single- and three-course prisms in the prediction of the flexural strength of prestressed brickwork beams

Brick type	Average % difference between predicted and experimental ultimate moment	
	Single-course prisms	Three-course prisms
1	6.8	25.0
2	8.8	24.6
2*	2.0	9.3
3	25.7	70.1
4	26.0	19.0
5	16.0	18.0
6	54.0	22.0
7	9.7	25.0
Average	13.6	27.4

* 1:½:4½ mortar.

underestimate the experimental results. The overestimate using brick type 2 was approximately 5% on average. The results are presented in a clearer manner in Table 5. Brick types 1,2,2 (in a 1:½:4½ mortar) and 7 are within 10% of the experimental results and taken across the whole range tested within 14% using the single-course prisms. There are two exceptions, brick types 4 and 6; a better estimate is provided using the three-course prisms, although the presence of the single frog in brick type 4 should be borne in mind. Using the three-course prisms the predicted results were on average within 27.4% of the experimental results. It is interesting to note also that the prism result which produced the greatest compressive strength also provided the closest estimate of the ultimate strength.

4.2. *Neutral axis depth at failure*

From the strain measurements taken at various points on the surface of the beam it was possible to determine the neutral axis depth throughout the loading history of the beam. Typical results are shown in Fig. 10 for beams built using brick type 1. Two beams with differing percentage areas of steel are shown. For each it can be seen that there is a linear distribution of strain. The neutral axis depth is initially outside the section, the section being completely in compression. As loading increases it rises up through the section towards the outer compression fibres. As the beam approaches failure, the neutral axis depth becomes constant for the remaining two or three increments of load (around 80–85% of ultimate). Table 6 compares the average neutral axis depth at failure with that

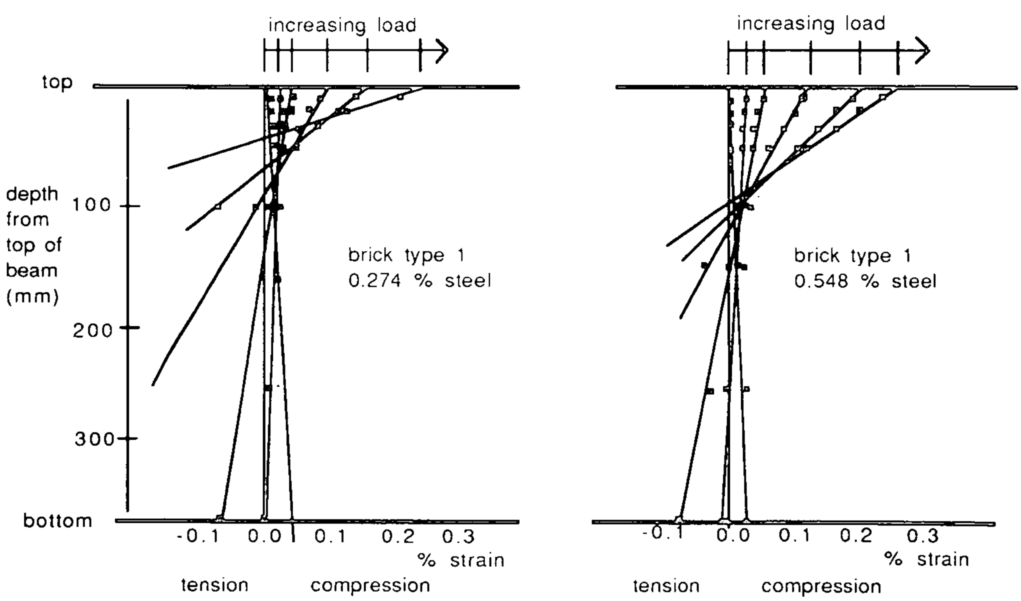


Fig. 10. Typical strain distribution in beams at different levels of load.

Table 6. Comparison between experimental and predicted neutral axis depths and compressive strain at failure

Brick type	% steel	Exp.	Neutral axis depth (mm) predicted: Single	Exp./pred.	Three	Exp./pred.	Compressive strain at failure: Exp.	Prism result: Single	Prism result: Three
1	0.274	61	51	0.84	78	1.27	0.0031		
1	0.441	93	89	0.95	112	1.20	0.0028	0.0033	0.0021
1	0.548	112	105	0.94	127	1.14	0.0035		
2	0.274	78	68	0.87	107	1.37	0.0037	0.0026	0.0017
2*	0.274	93	93	1.00	109	1.17	0.0042	0.0030	0.0026
2	0.255	67	68	1.01	107	1.60	0.0030	0.0026	0.0017
3	0.274	107	127	1.18	124	1.19	0.0028	0.0043	0.0015
4	0.274	108	122	1.13	108	1.01	0.0021	0.0020	0.0018
5	0.274	78	110	1.41	117	1.50	0.0028	0.0016	0.0015
6	0.274	80	152	1.90	123	1.54	0.0019	0.0015	0.0014
7	0.274	80	77	0.96	101	1.26	0.0031	0.0038	0.0022

* 1:½:4½ mortar.

predicted using the theoretical analysis. For brick types 1, 2 and 7 very close agreement between the experimental and predicted neutral axis depths is obtained with the single-course prism. In general, but again, with the exception of brick types 4 and 6, the single-course prisms provide a closer estimate than the three-course prisms, the three-course prisms tending to over-estimate the neutral axis depth.

4.3. *Compressive strain at failure*

The compressive strains in the outer fibres of the beam were also measured. Figure 11 shows a typical relationship between applied bending moment and top fibre, compressive strain. Initially there is a linear relationship up to the occurrence of cracking after which the compressive strain increases more rapidly to failure. Although it was not possible to measure the strains at failure, the strains were measured up to 95% of the ultimate load. The average failure strain was found either by extrapolating the curve to the average failure moment or by stopping the curve at the average failure moment. The results are given in Table 6 where they are compared with the average compressive strain obtained from the prism tests. In general the ultimate compressive strain at failure was in the range 0.0028–0.0042 suggesting a median value of 0.0035. However, with brick types 4 and 6 the strains were considerably lower. In all cases the ultimate strains obtained from the single-course prisms were closer to the results from the full-scale tests than the three-course prisms.

5. DISCUSSION

In predicting the flexural strength of prestressed brickwork beams using small-scale brick prisms, difficulties may arise. Prisms built from the same

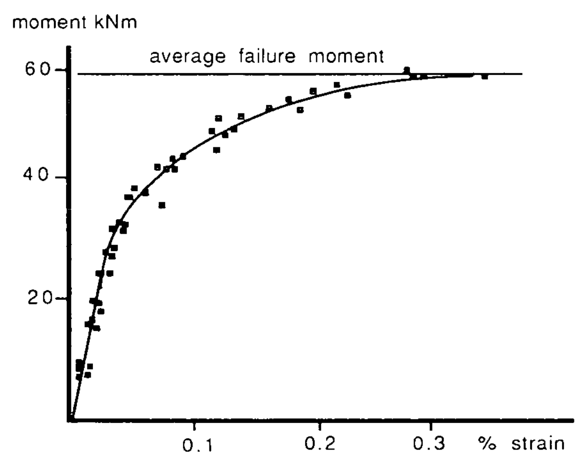


Fig. 11. Typical relationship between moment and top fibre strain.

brick and mortar and following the same bonding pattern but in a different format may nevertheless lead to completely different results in terms of compressive strength and strain at failure, with obvious consequences for the accuracy of any theoretical analysis. The presence of bedjoints parallel to the direction of the applied load may result in non-uniform strain distributions at high levels of stress due to transverse tensile failure across the bedjoint. In testing prisms under axial compression it is important to maintain uniform strain measurements otherwise the stress distribution will be unknown. Non-uniform distribution of strains in prisms with bedjoints parallel to the axial force has also been found by other researchers,[2] even when using loading apparatus that applied a constant strain.

During the full-scale beam tests it was noted in some instances that splitting of uppermost bedjoint

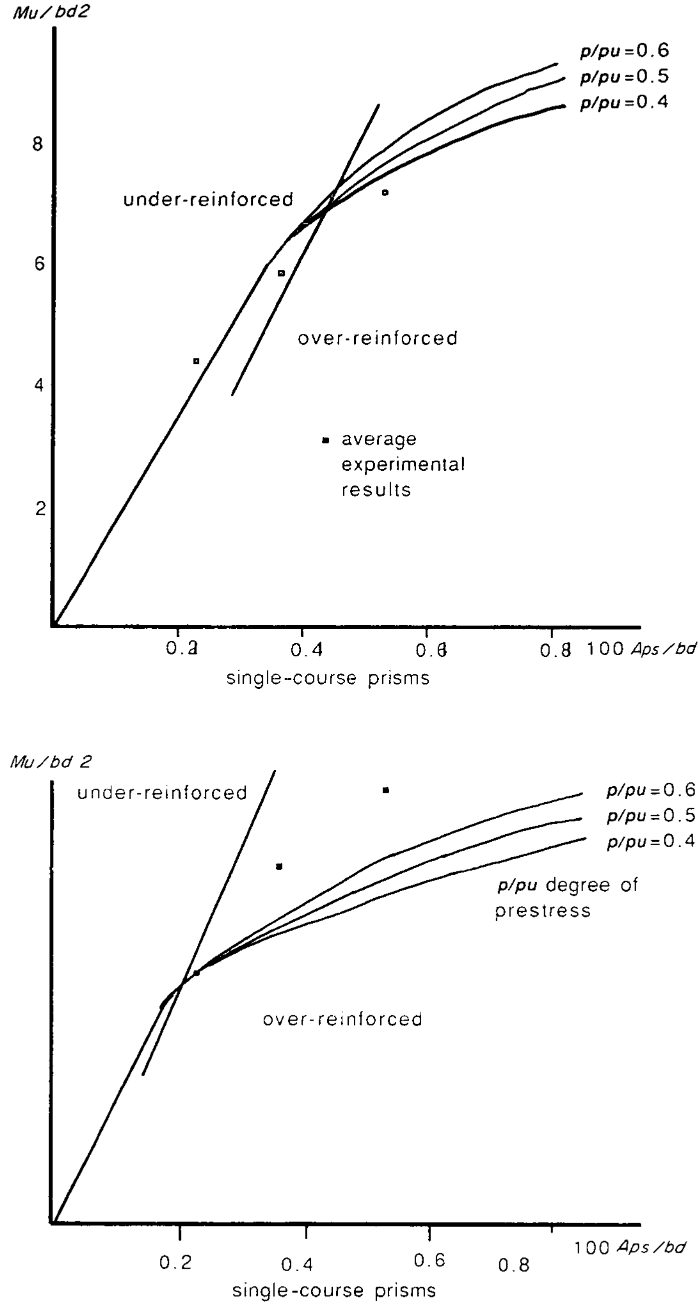


Fig. 12. Relationship between ultimate moment and percentage steel (type 1 bricks).

occurred prior to failure.[4] The difference between splitting in this instance and in the three-course prisms, is that the strain distribution is controlled by the flexural action of the beam, providing a well-defined linear distribution, increasing from zero at the neutral axis depth to a maximum at the outermost compressive fibres. At failure then it is possible to consider the compression zone as a series of courses in which the bedjoint between them has split, with each course subjected to progressively greater stresses moving away from the neutral axis position. This implies that prism formats such as the single-course prism should provide a closer representation of the compressive characteristics parallel to the bedjoint. In considering the effectiveness of a particular prism format to model the compression zone, the three aspects of performance that can be compared experimentally, viz. the ultimate moment, neutral axis depth and extreme fibre strain, with tests on full-scale beams, have tended to support the use of single-course prisms based on the results in this work. It is also worth noting that the compressive strain at failure and the neutral axis depth combine and become particularly important when considering the balanced section, and the percentage of steel that marks this point. If the prism results predict greater neutral axis depths or lower compressive strains at failure, then the full-scale tests indicate that the percentage steel for balanced design will be reduced. This is illustrated in Fig. 12. The influence of percentage of steel on ultimate flexural moment and neutral axis depth, calculated using both the single- and three-course prism results is shown. The percentage steel for balanced design is approximately 0.25% using the three-course prisms and 0.45% with the single-course prisms.

It has also been shown[8] that the stress–strain relationship obtained from tests on single-course prisms can be used to calculate accurately the curvatures and deflections in full-scale prestressed brickwork beams.

6. CONCLUSIONS

(1) The accuracy of prediction of the flexural strength of post-tensioned brickwork beams is greatly influenced by the format of the prisms used to derive the properties of the compression zone.

(2) The presence of bedjoints parallel to the direction of the applied compressive forces may give rise to uneven and erratic strain distributions, particularly at higher levels of stress which can result in a less accurate description of the compressive characteristics of brickwork.

(3) For the prism formats tested in this paper a close estimate of the ultimate strength of prestressed brickwork beams was obtained using single-course prisms for most bricks considered.

(4) The compressive strain in the extreme fibres at failure of prestressed brickwork beams varied from 0.0019 to 0.0042; in all cases these were closer to the ultimate strains obtained from single-course prisms.

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STRUCTURAL PERFORMANCE OF PRESTRESSED BRICKWORK POCKET-TYPE RETAINING WALLS

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Abstract—This paper describes the tests carried out on 10 full-scale post-tensioned brickwork pocket-type retaining walls. The effect of the percentage area of steel, prestressing force and brickwork strength on deflection, cracking and ultimate moments has been considered. A method for predicting the ultimate moment and the deflection of beams from prestressing to failure is described. Experimental results were compared with the theoretical prediction and good correlation was found between them.

NOMENCLATURE

f_m	compressive strength of brickwork
E_m	modulus of elasticity of brickwork
ε_{ma}	brickwork strain due to applied load
ε_{mp}	brickwork strain due to prestress
ε_{mu}	ultimate strain in brickwork
ε_{pb}	tendon strain
ε_p	tendon strain due to prestress
A_{ps}	area of prestressing steel
f_{pb}	stress in tendon
b	breadth of section
d	effective depth of section
d_c	depth of the neutral axis
λ_1, λ_2	constants
M_u	ultimate moment

1. INTRODUCTION

The lateral pressure or load acting on a free standing wall or on a cantilever retaining wall causes tension towards the loaded face. Masonry, either blockwork or brickwork, having very low tensile strength can only be used, provided the tension is eliminated altogether or resisted by some other means.

For brickwork, three possible techniques can be used; building a massive gravity-type structure; reinforcing; or prestressing. During the Victorian era, the massive gravity type structure was most common for building the retaining walls, so that the resultant of all the forces remained within the "kern" limit of the section and thus resulting in the elimination of tension throughout the structure. In the present situation, it would be uneconomical both in terms of the cost of labour and material. In the early part of the 20th century, reinforced brickwork[1] has been used extensively for the retaining walls. Pocket-type retaining walls in which the reinforcements are provided at regular intervals in the specially created voids are very common in the U.S.A. Some comparative cost analyses[2,3] in the U.K. have shown that this type of retaining wall, from 1 to 6 m high, is cheaper than reinforced concrete. The third technique, which may be viable, is prestressing of brickwork. Prestressing will eliminate cracking, increase the shear strength and reduce the deflection of the brickwork retaining wall. Consequently, more slender structures can be built, compared to those produced with reinforced brickwork. The technique of prestressing brickwork pockets is simpler than concrete and the expensive wedges, anchor plates and barrels can be removed[4] after the grout has reached sufficient strength. Thus, they can be used over and over again, which may prove cheaper than any other constructional solution.

2. SCOPE OF THE INVESTIGATION

As no performance data of prestressed brickwork pocket-type retaining walls were available, this investigation was undertaken to study the effect of:

(i) the percentage steel area;
(ii) the prestressing force; and
(iii) the brickwork strength, on the cracking and ultimate moments and the load deflection relationship of these walls.

3. TEST ARRANGEMENTS

Ten brickwork pocket-type walls with various degrees of effective prestress, having different percentage area of steel and brickwork strength, were tested in this study. The grade 1 mortar (1:1/4:3)—cement:lime:sand) was used for building all the test specimens. The walls were tested as simply supported slabs (Fig. 1) in a specially designed frame which provided pin and roller supports. Two line loads were applied approximately at one-third of span. The loads were applied at small increments at stages till failure, and measured at each jacking point by load-cells connected to a digital voltmeter and a pen-chart

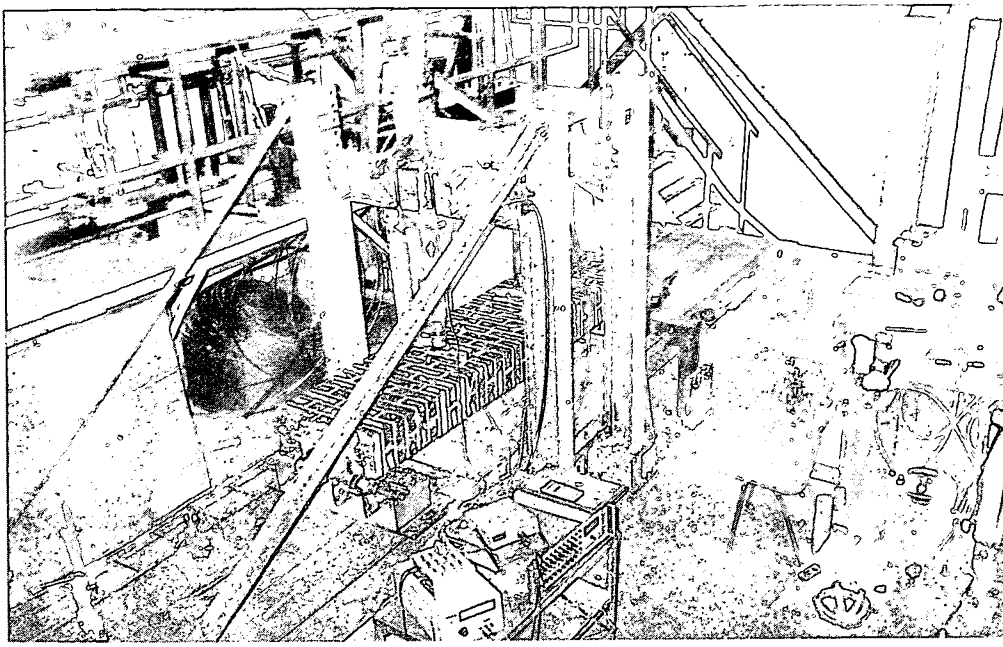

Fig. 1. Test arrangement.

recorder. Brickwork strains were measured by a "demec" gauge in the constant bending zone. The steel strain was measured by electric strain gauges. The deflections of the walls were measured by the dial gauges reading to an accuracy of 0.002 mm.

4. THEORETICAL ANALYSIS

A direct computational method, which takes into account the non-linear behaviour of materials, tensile cracking and tension stiffening, was developed to predict cracking and ultimate moments, the moment-curvature, and thus the deflection of the walls throughout the loading history, i.e. from prestressing, prestressing to cracking, post-cracking to failure. The direct method is fully described elsewhere.[5] The material properties required are the stress–strain relationship and ultimate strengths of brickwork and steel and also the ultimate brickwork strain. The compressive strength, the ultimate strain and the stress–strain curve of brickwork were obtained from a uni-axial test of the six-course brickwork prisms (Fig. 2). The stress–strain relationship of brickwork was idealized by a third degree polynomial as shown in Fig. 3 for use in the computational method or for the calculation of the ultimate moment described in Sec. 5. The average values of the characteristics of the stress block factors, λ_1 and λ_2, were 0.68 and 0.38, respectively. The ultimate strain varied from 0.002 to 0.0024, depending on the type of bricks. The stress–strain curve of steel obtained in uniaxial tension was idealized into a tri-linear relationship as shown in Fig. 4.

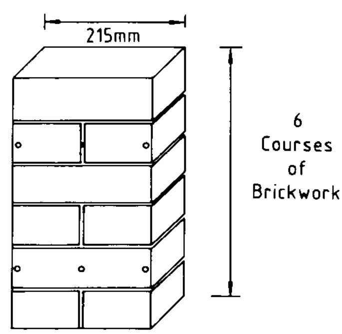

Fig. 2. Test prism.

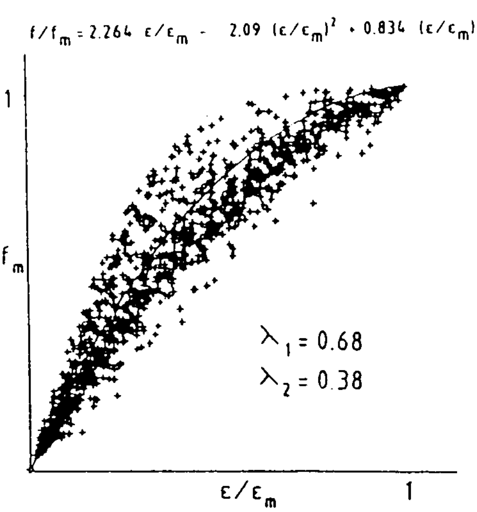

Fig. 3. Idealized stress–strain curve of brickwork.

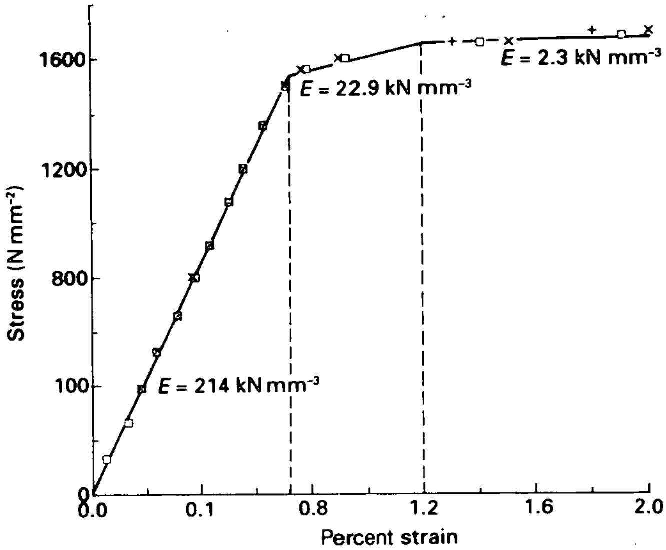

Fig. 4. Idealized stress–strain curve of steel.

5. CALCULATION OF THE ULTIMATE MOMENT OF RESISTANCE

The computer program based on the direct method can evaluate the ultimate moment of the pocket-type retaining wall. In a design practice, however, an approximate method using the characteristics of the stress block factors can easily be applied to calculate the ultimate moment. For all practical purposes, it is assumed that the final failure happens by crushing of brickwork at an ultimate strain of ε_{mu}. Near failure, the shape of the stress distribution diagram corresponds to the idealized stress–strain relationship (Fig. 3) and can be defined by two stress block factors, $\lambda_1 f_m$ and $\lambda_2 d_c$ which represent the average compressive stress and the distance of the centre of compression from the extreme fibres as shown in Fig. 5. At failure [from Fig. 5(ii)] for a fully bonded tendon, the steel strain, ε_{pb}, consists of strain due to applied load and due to prestress and given by

$$\varepsilon_{pb} = \varepsilon_{ma} + \varepsilon_{mp} + \varepsilon_{pe'}$$

where ε_{mp}

$$= \frac{\text{prestressing stress in brickwork at tendon level}}{E_m}. \tag{1}$$

From Fig. 5(ii)

$$\varepsilon_{ma} = \varepsilon_{mu}\frac{d-d_c}{d_c}. \tag{2}$$

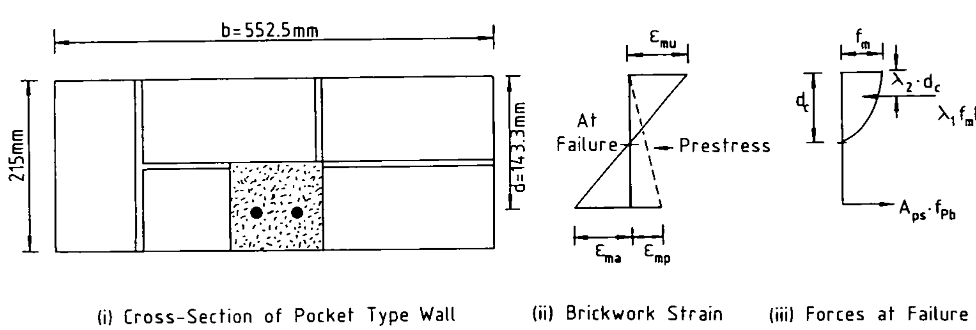

Fig. 5. Cross-section of the beam, strain distribution and the forces at failure.

Substituting the value of ε_{ma} in Eq. (1) and re-arranging we get

$$d_c = \frac{\varepsilon_{ma}}{\varepsilon_{mu} + \varepsilon_{pb} - \varepsilon_{mp} - \varepsilon_{pe}} d. \qquad (3)$$

At failure, the forces of compression and tension must be equal [see Fig. 5(iii)]

hence $\lambda_1 f_m b d_c = A_{ps} f_{pb}$

or

$$d_c = \frac{A_{ps} f_{pb}}{\lambda_1 f_m b}. \qquad (4)$$

Substituting the value of d_c in Eq. (3)

$$f_{pb} = \frac{\lambda_1 f_m b d}{A_{ps}} \left(\frac{\varepsilon_{mu}}{\varepsilon_{mu} + \varepsilon_{pb} - \varepsilon_{mp} - \varepsilon_{pe}} \right). \qquad (5)$$

At failure, the values of two unknowns f_{pb} and ε_{pb} are determined by the trial and error method, so that the conditions of Eq. (5) and the stress–strain relationship of steel given in Fig. 4 are both satisfied. Having found f_{pb} and ε_{pb}, the depth of the neutral axis d_c can be found from Eq. (4) and the ultimate moment can be obtained by

$$M_u = A_{ps} f_{pb}(d - \lambda_2 d_c). \qquad (6)$$

The theoretical ultimate moments given by Eq. (6) and the direct method are compared with the experimental results in Table 1 and Fig. 8.

6. RESULTS AND DISCUSSIONS

The test results are given in Table 1 and Figures 6–10.

6.1. *Cracking moment*

From Table 1 and Fig. 6, it can be seen that the cracking moment increases with the increase in

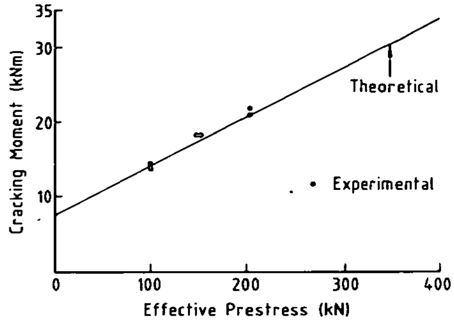

Fig. 6. Comparison of theoretical and experimental cracking moments.

prestress, which is not surprising. It can also be seen that the theoretical and experimental results are in good agreement. The brickwork strengths have no influence on the cracking moment. The cracking occurs after neutralization of the prestress at the soffit of the wall between brick and mortar (Fig. 7). The interface tensile bond strength does not depend on brick strength, hence the cracking moment ought to be similar. At zero prestress (Fig. 6), the cracking moment is much smaller, the condition is akin to reinforced pocket-type walls of similar cross-section.

6.2. *Ultimate moment*

The theoretical predicted ultimate moment (Fig. 8 and Table 1) of prestressed pocket-type walls agrees well with the experimental results for various strengths of brickwork and also for various percentage areas of steel in which brickwork strength was kept constant. Initially, the ultimate moment increases linearly with the increase in the percentage area of steel. With higher percentage area of steel, the section seems slightly over-reinforced. As a result, the stress in the steel at failure is slightly lower than yield stress, hence there is a deviation from the linear relationship (Fig. 8).

Table 1. Comparison of experimental cracking and ultimate moments with theoretical values

				Cracking moment (kNm)			Ultimate moment (kNm)		
Wall No.	Percentage area of steel	Effective prestress (kN)	Brickwork strength (N mm^{-2})	Experimental	Theoretical	Direct computer method	Experimental	Theoretical	Direct computer method
1	0.18	104.0	22.0	14.0	14.5	14.4	33.10	30.72	32.42
2	0.18	106.5	22.0	14.1	14.8	14.5	33.98	30.75	32.50
3	0.18	127.5 101.5	34.10	16.3	16.3	16.4	31.50	32.32	30.99
4	0.18	75.6	34.10	13.3	12.0	12.2	34.60	32.38	30.40
5	0.18	101.0	37.60	14.5	13.6	14.0	32.12	32.18	32.68
6	0.18	101.0	37.60	14.0	13.6	14.0	32.70	32.18	32.61
7	0.27	148.0	37.60	18.0	17.0	18.0	44.93	47.80	47.40
8	0.27	148.0	37.60	18.0	17.0	18.0	50.57	47.80	47.40
9	0.36	206.0	37.60	22.0	21.0	20.9	65.34	62.0	60.40
10	0.36	206.0	37.60	21.0	21.0	20.9	62.90	62.20	60.40

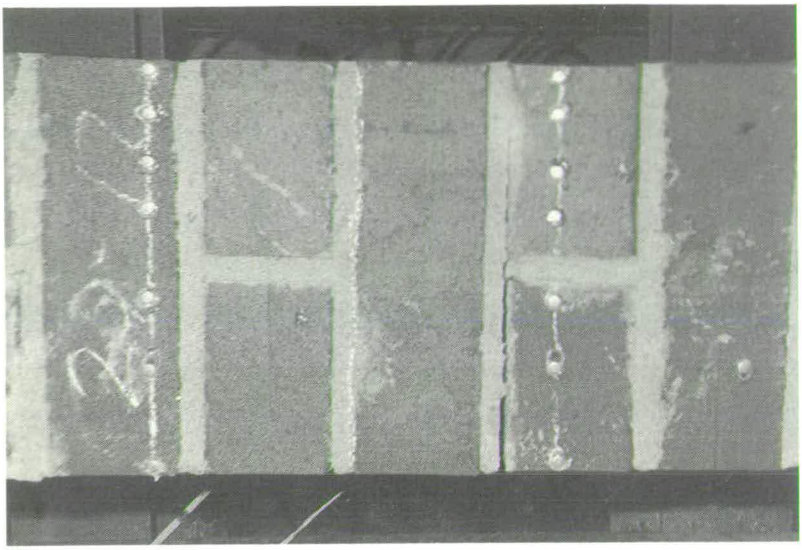

Fig. 7. Showing the crack between brick and mortar interface.

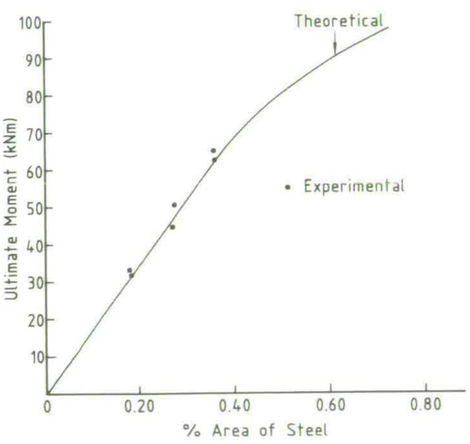

Fig. 8. Comparison of theoretical and experimental ultimate moments.

Before compressive failure occurs, however, there is every likelihood that the section may fail in shear. This needs experimental confirmation. From Fig. 9 it can be seen that within the range of the test, the brickwork strength does not significantly affect the ultimate moment of the wall with 0.18% area of steel. The failure of these walls (brickwork strengths 22.0–37.6 N mm^{-2}) was due to the yielding of steel, hence the ultimate moment is mainly dependent on the forces developed in the steel rather than the brickwork. However, if low strength brickwork with the same percentage area of steel is used, there will be some drop in the ultimate moment as predicted by the theory. The section with the same percentage area of steel (0.18%) becomes over-reinforced if the brickwork compressive strength is less than 15 N mm^{-2}. The

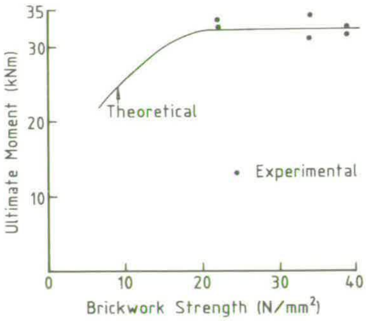

Fig. 9. Showing the relationship between the ultimate moment and the brickwork strength.

failure is, then, dictated by the compressive force developed in the brickwork.

6.3. *Deflection*

Some typical load–deflection relationships are shown in Fig. 10. The load–deflection relationship is linear up to cracking. The deflection increases more rapidly after cracking. It is clear from Fig. 10 that at every stage of loading, the deflection of the wall with a high percentage area of steel and prestressing force is lower compared to those having a lower percentage area of steel and prestressing force. The deflection due to the applied load has to counteract the large upward deflection due to higher prestress, hence the resultant deflection will be less compared to walls subjected to lower prestress within the linear range. As the crack develops later with higher prestress, the wall again exhibits higher stiffness for similar lateral load than the walls with lower effective prestress. Some typical experimental and theoretical deflection results are compared in Figs 11 and 12. A good correlation is found between them.

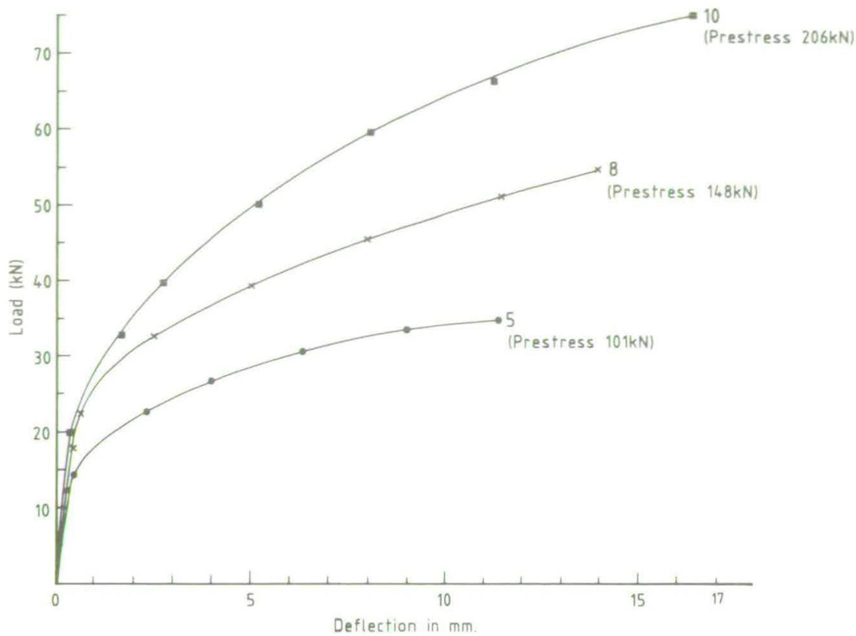

Fig. 10. Load–deflection relationship of pocket-type retaining wall with varying degree of prestress.

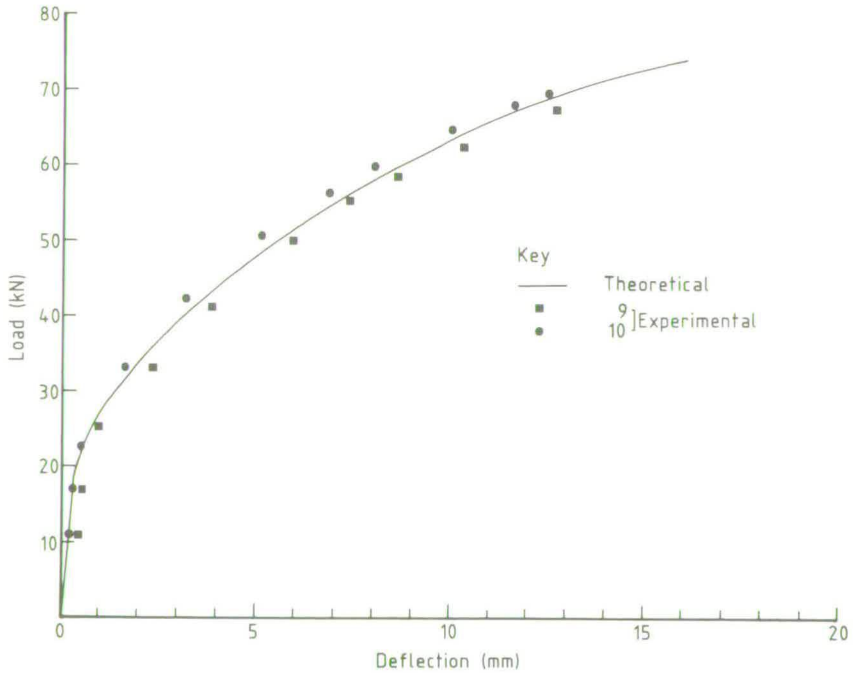

Fig. 11. Comparison of experimental and theoretical deflection result.

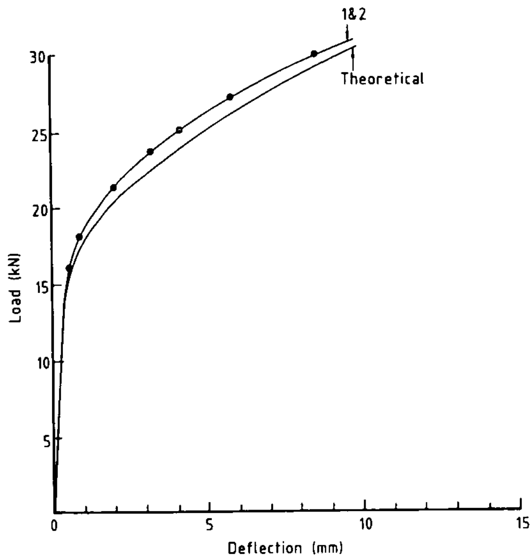

Fig. 12. Comparison of experimental and theoretical deflection result.

7. CONCLUSIONS

(1) The ultimate bending moment increases linearly with the increase in the percentage area of steel within the range of the test. However, for a particular brickwork strength there is an upper limit of the percentage area of steel beyond which the section becomes over-reinforced and the relationship between them deviates from the linear relationship, provided premature failure does not happen in shear.

(2) As expected, the cracking moment increases with the increase in prestressing force.

(3) The brickwork strength has practically no influence on the cracking moment. Similarly, it does not exert any influence on the ultimate moment, if the failure is in flexure.

(4) The stiffness of the wall increases due to the increase in the effective prestress and the steel area. As a result, the deflection is less throughout the loading history up to failure for the prestressed pocket-type walls with high prestress and area of steel.

(5) The load–deflection relationship, the cracking and the ultimate moments of the prestressed pocket-type retaining walls can be reliably predicted by the theoretical method described in this paper using the material properties from the small scale tests.

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The Shear Strength of Brickwork Beams Prestressed Normal to the Bed Joints

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ABSTRACT
Eight full size beams with a brick bonding arrangement very similar to that used for pocket type walls, 2½ bricks wide and one brick thick, were prestressed normal to the bedjoints and tested to fail in shear. The brick type, mortar strength, shear span/effective depth ratio and percentage area of steel were kept constant. The only variable was the prestressing force. The results show that shear stress is enhanced by prestressing and a relationship with prestress/unit area is given. With sufficient prestress the ultimate moment of the section was close to the theoretical value.

1. INTRODUCTION

Brickwork can be prestressed either parallel or normal to the bed-joints. The shear strength of prestressed beams when prestress is applied parallel to the bed-joint has been thoroughly investigated[1] and documented. The BS5628:Part 2[2] recognises the enhancement of shear strength due to the prestress, if the prestressing force is applied normal to the bed-joint. The characteristic shear strength of prestressed brickwork in the BS[2] is given by:

$$f_v = 0.35 + 0.6g_B$$

where g_B is the design load/unit area due to the prestress at right angles to the bed-joint. It seems that this equation was derived from the walls tested in combined axial compression and shear, which may not be valid for prestressed walls. As performance data to support the contention of the BS[2] was not available, an investigation was undertaken to establish the behaviour and shear strength of prestressed brickwork beams, with the prestress normal to the bed joints.

All the beams tested in this investigation were built with a brick bonding arrangement very similar to that used for a pocket type wall, and it was intended that they fail in shear. The brick type, mortar strength, shear span/effective depth ratio and percentage area of steel were kept constant. The only variable was the prestressing force.

Table 1
Compressive Strength of Mortar and Grout
(Mean of 3 x 100mm cubes)

Beam no.	Average compressive strength N/mm²	
	Mortar 28d	Grout 7d
3	22.5	29.3
4	18.3+	11.8+
5	33.9	23.8
6	29.9	25.0
7	29.9	19.7
8	28.6	15.6

+ cured in air

2. EXPERIMENTAL DETAILS

2.1 Materials

Three-hole 84N/mm² engineering class B bricks and a mortar mix of 1:¼:3 (cement:lime:sand) by volume, were used for the construction of the walls and the prisms. A 1:2½:2 (cement:sand:aggregate) mix by volume was used for grouting. A plasticiser "Conbex" was added to the mix to reduce the shrinkage. The mean compressive strengths of the mortar and grout cubes are given in Table 1.

Six, 7-wire stabilised prestressing strands of 10.9mm nominal diameter, having ultimate strength equal to 1708N/mm², were used.

2.2 Specimens

Six-course high brickwork prisms were built along with the beams. The prisms were tested in uni-axial compression at 28d. The load was applied gradually until failure and strains were measured up to 95% to 98% failure stress. The results are given in Table 2. The ultimate strains were extrapolated from the stress-strain relationships. The average compressive strength of 18 prisms tested in this programme was 25.2N/mm² with the range of 23.0 to 30.3N/mm². The overall extrapolated failure strain was 0.0024.

Eight full-scale pocket-type beams, built vertically as if they were walls, were constructed for testing. They were two and a half bricks wide and one brick thick as shown in Figure 1. The beams were left for curing for 21d. Before prestressing, 35mm

Table 2
Compressive strength of 6-course brickwork prisms (28d)

Beam no. with prism	Compressive strength N/mm²	
		Average
3 - 1	25.6	
3 - 2	25.6	25.3
3 - 3	24.6	
4 - 1	23.3	
4 - 2	25.1	26.2
4 - 3	30.3	
5 - 1	25.2	
5 - 2	27.9	26.5
5 - 3	26.5	
6 - 1	24.4	
6 - 2	26.0	25.5
6 - 3	26.0	
7 - 1	23.0	
7 - 2	24.0	23.3
7 - 3	23.0	
8 - 1	24.9	
8 - 2	25.1	24.8
8 - 3	24.4	

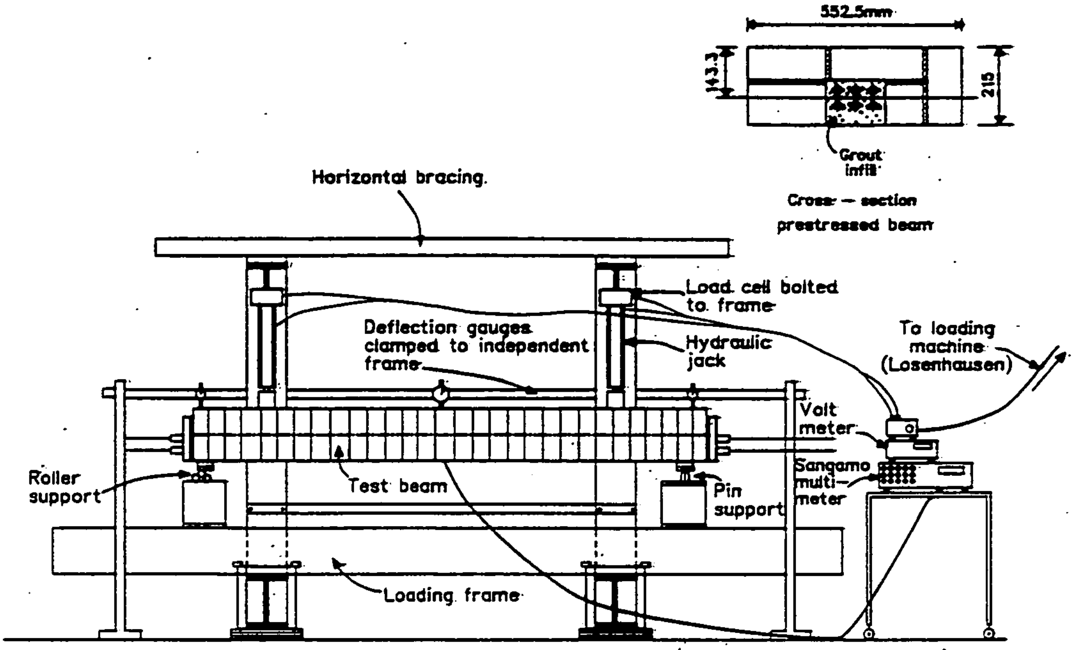

Figure 1-Showing the cross-section of the beam and the test arrangement

Table 3
Effective prestress after losses

Beam No.	Applied prestress kN	"lock off" loss %	Actual prestress before grouting kN	Effective prestress after 7d kN
3	203.5	50.0	101.5	83.4
4	168.5	43.8	94.8	78.0
5	270.0	32.0	183.6	142.8
6	305.1	31.3	209.4	179.4
7	386.7	32.0	264.0	220.8
8	380.1	26.2	280.2	235.2

Table 4
Showing the cracking and ultimate moment

Beam no.	Cracking moment kNm		Ultimate moment kNm		Ratio
	Exptl.	Theor.	Exptl.	Predicted	Exp/Pred.
1	7.15	7.0	48.9	55.20	0.89
2	7.15	7.2	48.9	55.35	0.89
3	14.40	12.59	54.50	58.85	0.93
4	12.80	12.38	52.34	58.71	0.89
5	19.00	16.39	65.88	62.27	1.06
6	18.26	18.89	65.59	63.76	1.03
7	22.70	20.97	69.14	66.08	1.05
8	24.00	21.93	71.22	66.79	1.07

thick mild steel plates were fixed to either end of the beam with rich mortar. The plates were bedded in such a way that the tendons could be placed at the required depth. The walls were prestressed at the age of 21d. During the prestressing, the applied prestressing force was monitored exactly by the electrical strain gauge attached to each tendon. The "lock-off" loss was also recorded by the strain gauges. After prestressing, the pocket was grouted and covered with the polythene sheet for 7d. The walls were tested at 28d. Beams 1 and 2 were not prestressed. The "lock-off" loss and effective prestress of the remainder are given in Table 3. During the prestressing the loss was mainly caused by the slip of wedges into the barrel. These losses were subject to large amount of variation due to the random nature of the slippage. In the case of the first two walls, the prestressing was only applied to straighten the strands.

2.3 Arrangement for testing

Each pocket-type specimen was tested as a simply supported beam with a two line loading approximately at a third span in a special frame which provided pin and roller supports (Figure 1). Before the beam was set on the loading frame, the dead weight of each beam was obtained from the load cells of the lifting crane. The applied load, monitored at the jacking points with load-cells, was increased gradually until failure occurred. The strains in the tendons were measured, as before, using the electrical strain gauges and the brickwork strains at various depths were measured using a Demec gauge. The mid-span deflection and support settlements, if any, were measured with dial gauges.

Table 5
Shear strength of brickwork, prestressed normal to bed-joint

No	Prestress force kN	Prestress in the centre N/mm²	Characteristic shear strength N/mm² $f_v=0.35+0.6g_B$	Design shear stress N/mm² f_v/γ_m (1)	Ultimate shear strength N/mm² $v=V/_{bd}$ (2)	Ultimate shear stress/design stress (2)/(1)
1	12.0	0.10	0.41	0.205	0.91	4.44
2	15.0	0.13	0.43	0.215	0.91	4.23
3	83.4	0.70	0.77	0.385	0.99	2.57
4	78.0	0.66	0.75	0.370	0.95	2.57
5	142.8	1.20	1.07	0.540	1.20	2.22
6	179.4	1.51	1.26	0.630	1.20	1.90
7	220.8	1.86	1.47	0.730	1.25	1.71
8	235.2	1.98	1.54	0.770	1.29	1.68

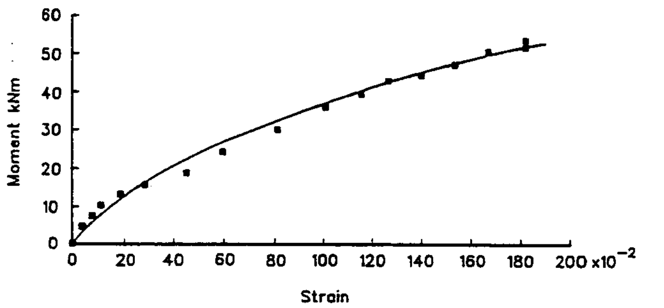

Figure 2- Relationship between moment and brickwork strain at extreme fibre (beam 4)

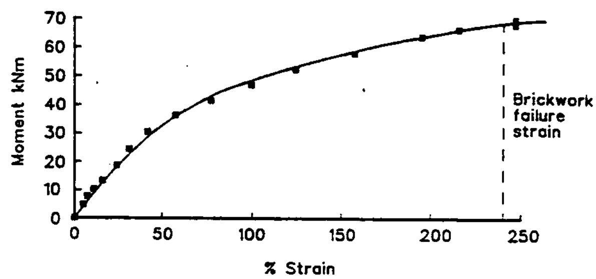

Figure 4- Relationship between moment and brickwork strain at extreme fibre (beam 8)

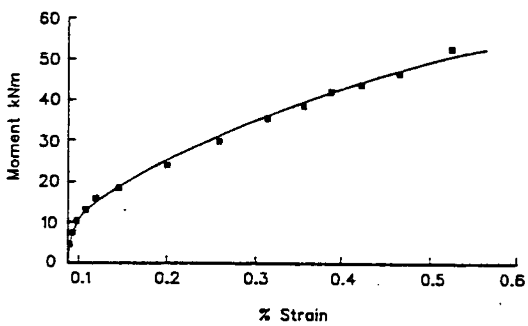

Figure 3- Relationship between moment and additional strain in steel (beam 4)

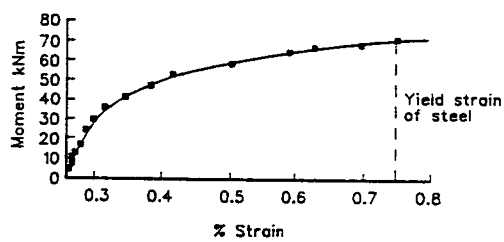

Figure 5- Relationship between moment and additional strain in steel (beam 8)

3. RESULTS AND DISCUSSION

3.1 Results

The results of the tests are given in Tables 4 and 5. The theoretical cracking and ultimate moments were predicted by a computer program which takes into account the non-linear behaviour of materials, both steel and brickwork. The brickwork strength and ultimate strain were obtained from the tests on 6-course prisms. The method used is fully described elsewhere[3,4,5,6].

It can be seen from Table 4 that there is very good correlation between the theoretical and experimental cracking moment of the beams. The theoretical cracking moment is, in effect, the sum of two moments; that required to nullify the prestress at the soffit plus that required to produce a tensile stress equal to the modulus of rupture of the brickwork. The average modulus of rupture of the brickwork was 1.5N/mm². The experimental cracking moment is taken when the cracks become visible to the naked eye. As expected, beams 1-4 failed in shear before their predicted ultimate flexural capacity was realised. In the case of beams 5-8, the flexural and shear failure happened at the same time, and there was no degradation of moment. Therefore, the predicted ultimate moment agrees well with the experimental results.

3.2 Compressive and Tensile Strains

The observations made above are confirmed from the results of the strains measured in brickwork and steel during the experiments. The compressive strain in the extreme fibre of the beams and steel strain in the bottom layer were both measured. Some typical relationships between the bending moment and compressive and tensile strains are shown in Figures 2 to 5.

Initially, there is a linear relationship up to the occurrence of cracking after which both compressive and tensile strains increase more rapidly to failure. From Figures 2 and 3, it can be seen that neither the brickwork nor the steel reached their ultimate or yield strains; the failure was solely due to shear. Both steel and brickwork strains reached their yield or ultimate values, Figures 4 and 5, where failure was due to combined shear and flexure.

Figures 6 to 8 show the typical failures of the prestressed beams. Initially, the crack appeared in the maximum bending moment zone where the modulus of rupture strength was reached after the prestress had been neutralised. Figure 6 shows the propagation of cracks. Beams 1 and 2 with no prestress failed in shear with no or little crushing at the loading point. Three types of failure were noticed. Beam 4, having low prestress, failed

Figure 6- Showing the crack propagation and failure with crushing of brickwork at load point

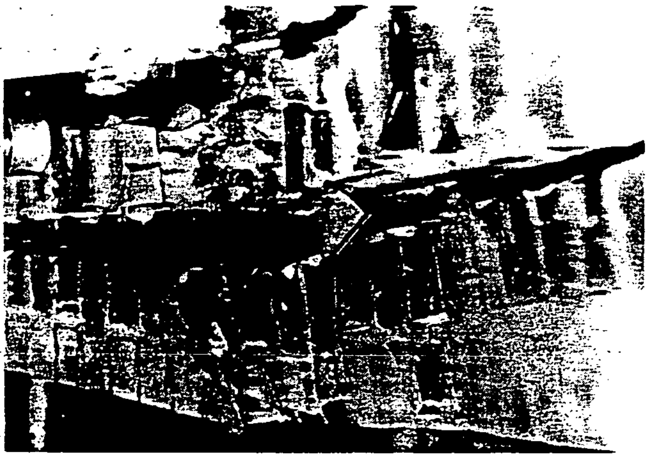

Figure 7- Failure of the Beam 5

Figure 8- Showing longitudinal splitting with crushing of brickwork

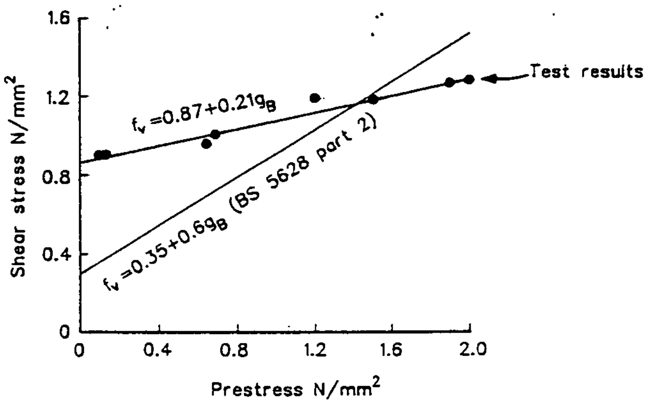

Figure 9- Showing the shear strength of prestressed brickwork prestressed normal to bed-joint

finally in shear with some crushing of the brickwork at the loading point. The shear failure of Beam 5, with moderate prestress, was accompanied by vertical splitting of the beam into three sections with crushing in the centre (Figure 7). Beam 8 with high prestress failed finally with longitudinal splitting and crushing (Figure 8). This longitudinal splitting happened in similar reinforced walls[7]. The longitudinal splitting along the line of action of compressive force was observed in the 6-course prism loaded eccentrically.

4. SHEAR STRENGTH

The ultimate shear stress is given in Table 5 and Figure 9. The characteristic shear has been calculated in Table 5 from the equation given in clause 19.1.3.3 of BS5628:Part 2[2]. The design shear stress was obtained by dividing the characteristic shear strength by the material partial safety factor of 2. The ultimate shear strength from the test results was calculated from the equation:

$$v = V/_{bd}$$

where b is the width of the section and d is the effective depth. It is the author's belief that the prestressed section will behave as an ordinary reinforced section once the prestress is fully neutralised.

From Table 5, it can be seen that the factor

$$\frac{\text{ultimate shear stress}}{\text{design shear}}$$

varied from 4.44 to 1.68 which ranges from conservative to unsafe. It appears that the formula for calculating the shear strength given in the code (clause 19.1.33) is not applicable to the prestressed section tested in this study. The value of the initial shear strength of 0.35N/mm² is too low for this type of the wall. However, the effect of the prestress is not as pronounced as is suggested by the code equation. It is suggested that the shear strength of prestressed pocket-type wall may be obtained from the formula:

$$f_v = 0.87 + 0.21g_B$$

where g_B is prestress/unit area

The correlation coefficient is equal to 97% for this relationship between shear strength and initial prestress. The a/d ratio for the beams was approximately equal to 5; some adjustment needs to be made for higher and lower a/d (shear arm/effective ratio).

No revision is required for beams where prestress is parallel to the bed-joint. It must be pointed out, here, that the equation given above is not valid for prestressed beams with unbonded tendons[8,9] these have lower shear capacity than a prestressed beam with bonded tendons[10].

In the test beams, flexural cracks also appeared in the shear span. The shear cracks developed as the extension of flexural cracks. It would seem that the principal stress theory[11] is not applicable to cracked[12] prestressed brickwork beams.

5. CONCLUSIONS

From the tests carried out, the following conclusions are made:

1. For brickwork flexural members, loaded so that the direct stresses are normal to the bed planes, the shear strength is enhanced by prestressing.
2. The characteristic shear strength for prestressed brickwork beams with bonded tendons is given by:

$$f_v = 0.87 + 0.21\ g_B$$

and may be limited to a maximum prescribed by the British Code of Practice.
3. When the beams have sufficient prestress, the ultimate moment is very close to the theoretically predicted value.

ACKNOWLEDGEMENT

The author wishes to acknowledge the financial support given by the Brick Development Association for this work.

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The Shear Strength of Prestressed Brickwork Beams

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ABSTRACT

The effect of a/d ratio (shear span/effective depth) and percentage of steel appears to have no significant effect on the shear strength within the range considered. As a result of shear failure, reductions in the collapse moment ranging from 19% to 36% occurred in the various beams.

1. INTRODUCTION

The assessment of the shear strength of reinforced and prestressed concrete beams has caused researchers much concern. Shear failures are generally sudden and devastating and, under certain conditions, occur before the flexural capacity of the beam is realized.

It is well known that the shear strength of concrete beams increases with shear span/effective depth ratio and increasing steel area. A similar situation occurs in brickwork beams. It has been shown experimentally that the shear strength of reinforced brickwork, like concrete, increases with decreasing shear span, effective depth ratio and increasing steel[1,2,3]. More recent work on the contribution of dowel action, compression zone transmission and aggregate interlock to the shear strength of reinforced grouted cavity brickwork beams has shown that the greatest proportion of the shear was carried by compression zone transmission.[4] Previous work on post-tensioned brickwork beams[5] has shown that beams with a lower percentage of steel tended to fail in flexure. However it was felt that if the percentage of steel was increased the mode of failure might shift from flexure to shear. In the design of prestressed brickwork beams it is very important to know when a shear failure will predominate. To investigate this a series of tests were carried out to study the influence of the following variables on the ultimate shear strength of prestressed brickwork beams:

(i) shear span/effective depth ratio (a/d), 2, 4, 7, 11·2.
(ii) percentage of steel, 0·411% and 0·548%.

2. EXPERIMENTAL AND CONSTRUCTION DETAILS

2.1 Materials

All materials were tested according to the relevant British Standards. Three hole, 82 N/mm² bricks were used. Compressive strength tests were carried out in three orthogonal directions (Table 1). The 24 h water absorption was 4·17%.

Table 1
Compressive strength of bricks tested in different directions

	On bed	*On edge*	*On end*
Mean compressive strength N/mm²	82·03	53·17	40·23
Range N/mm²	63·80–96·70	33·54–68·03	30·00–50·76
Standard Deviation N/mm²	5·85	9·43	6·94
C.V%	7·13	17·93	17·25

A 1:¼:3 (cement:lime:sand) mortar mix by volume was used throughout. 100 mm cubes were taken and tested at 28 days. The average compressive strengths of the mortar are given in Table 2.

A 1:2¼ (cement:sand) grout mix by volume was used, with the addition of a plasticizer 'Conbex', to help to reduce shrinkage and shorten the setting time. Grout was poured through the perforations of the bricks. 100 mm cubes were taken and tested at 7 days. The compressive strengths of the grout are given in Table 2.

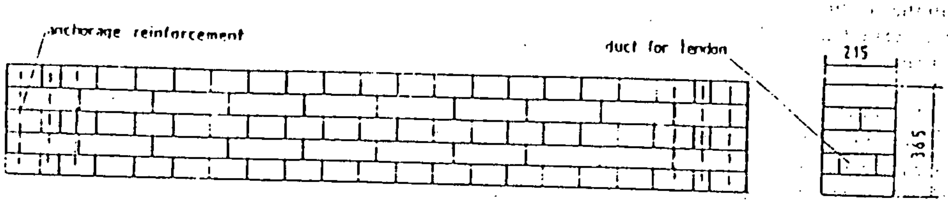

FIGURE 1. Bonding pattern and beam section.

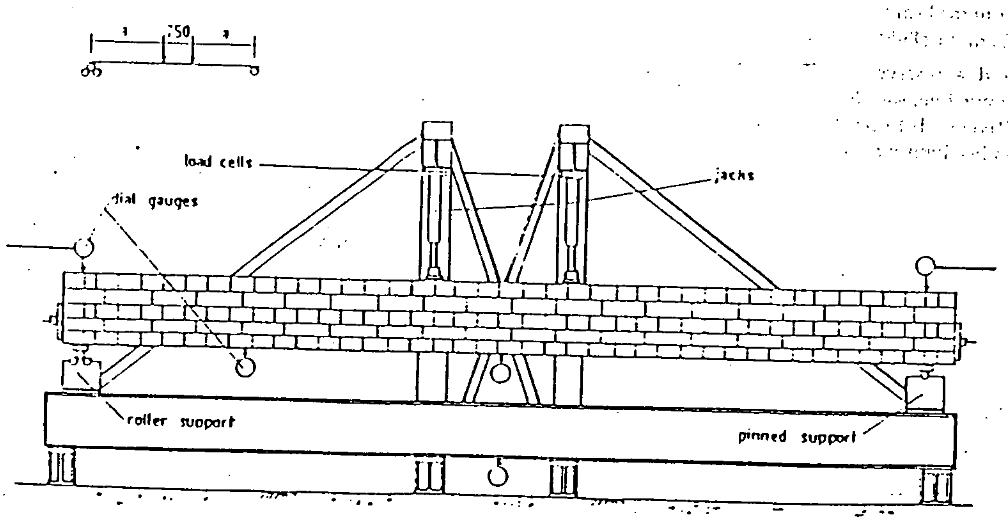

FIGURE 2. Layout of test rig.

Table 2

Effect of $\frac{\text{Shear span}}{\text{Effective depth}}$ ratio and percentage area of steel on shear strength of prestressed brickwork beams

Beam no.	Mortar Strength N/mm²	Grout Strength N/mm²	span m	Area of steel %	Effective Prestress kN	Shear span / Effective depth	Shear strength N/mm²	Mean	Failure moment kNm (M_{exp})	Predicted moment kNm (M_{pre})	$(M_{exp})/(M_{pre})$
1	19·5	14·0	1·75		180	2	2·1	2·4	59·1	92·3	0·64
2	16·4				176		2·7		71·6	91·8	0·78
3	19·5	14·0	2·75		180	4	1·3	1·3	69·3	92·4	0·75
4	19·5			0·548	194		1·3		65·7	92·4	0·70
5	16·4	15·0	4·5		190	7	0·75	0·73	73·8	92·3	0·80
6	16·4				191		0·70		68·9	91·9	0·75
7	15·2	13·6	6·2		213		0·55	0·56	72·5	93·0	0·78
8	18·6	11·3			212		0·55		71·5	93·0	0·77
9	18·6	11·3			199	11·2	0·57		75·2	92·8	0·81
10			6·2	0·411	221		0·55	0·56	72·6	75·6	0·96
11					221		0·60		79·9	75·6	1·06
12					196		0·54		74·5	75·2	0·94

10·9 mm diameter, stabilized prestressing strand was used with an ultimate stress of 1736 N/mm², a 0·2% proof stress of 1597 N/mm² and Young's modulus of 214 kN/mm².

2 Method of Construction and test

All the beams were built on the floor of the testing laboratory by an experienced bricklayer. The section of the beam is shown in Figure 1. The cavity forming the duct for the tendon was made by splitting the bricks in the second course lengthwise and placing them flush with the face of the beam.

The beams were allowed to cure for 21 days after which steel anchor plates were attached. The beams were prestressed and then grouted immediately afterwards. The beams were cured for a further 7 days before testing. In beams 1 to 9 the tendon consisted of four strands, two of these were stressed up to the allowable 70% of the ultimate strength[4] and the remaining two stressed to half this value. In beams 10–12, the tendon consisted of three strands, all stressed to 70%. By doing this the prestressing force for both groups remained constant, although the steel area was reduced in the latter.

The beams were tested in a four point loading rig, Figure 2, capable of testing beams with spans up to 6·5m. The supports for the beams consisted of one roller and one pin. The load was applied in increments, using hydraulic jacks attached to load cells and a pen chart recorder to enable the failure load to be accurately determined.

Strains in the strand were measured using electrical resistance strain gauges. Strains in the brickwork were measured using Demec gauges. Deflections were measured using dial gauges reading to 0·002 mm. Measurements of strain and deflection were taken at each increment of load. As the beam apoproached failure the load increment was reduced, to enable readings to be taken as close as possible to failure.

3. RESULTS AND DISCUSSION

Table 2 summarizes the experimental results. Two basic forms of shear failure were observed. Beams with a/d ratio 2 failed by diagonal cracking running from load point to support (Figure 3). The remainder tended to fail by splitting of the top bedjoint, running from the constant moment zone along the shear span, into the support, (Figure 4).

In all cases considerable flexural cracking in the constant moment zone took place prior to failure. In the case of beams with a/d ratios 2 and 4 very little flexural cracking in the shear span occurred. In beams with higher a/d ratios flexural cracking extended well into the shear span usually progressing upwards through the section at an angle of approximately 45°. The diagonal cracks normally moved in a stepwise manner through the mortar joints.

3.1 Influence of shear span/effective depth ratio on shear strength

Beams 1 to 9 had the same area of steel and were initially prestressed to the same degree. However, the effective prestress recorded on the day of the testing of these beams was not exactly the same due to lock off and other losses. The effective prestress force was 17% higher for the beams with a/d ratio 11·2 compared to beams with a/d ratio 2. This may not have caused any increase in the shear strength of the

FIGURE 3. Typical diagonal shear failure.

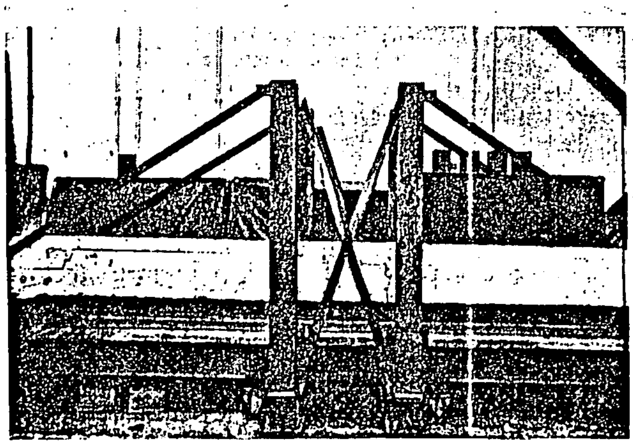

FIGURE 4. Typical splitting shear failure.

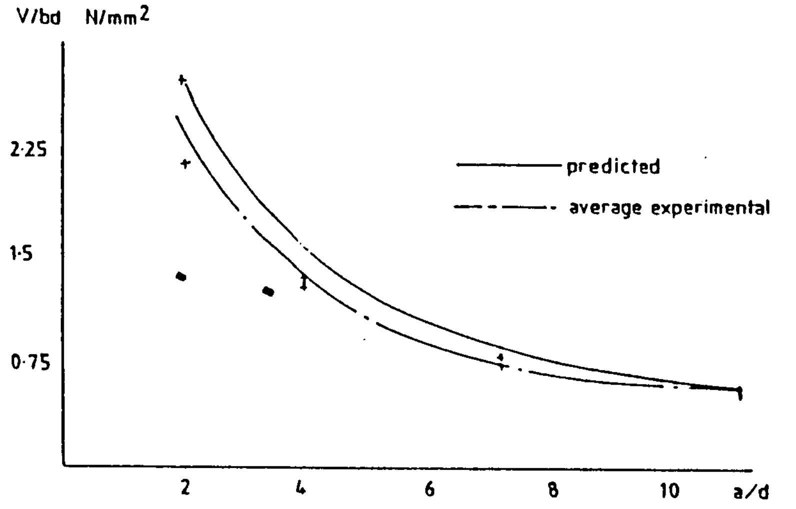

FIGURE 5. Influence of shear span/effective depth ratio on the shear strength of prestressed brickwork beams.

beams with a/d ratio 11·2, because the prestressing force does not significantly influence the shear strength. Thus, the only variable in these beams was shear span/effective depth ratio (a/d). From Table 2 and Figure 5 it can be seen that the shear span/effective depth ratio has a marked influence on the shear strength which decreased with increasing a/d ratio from an average of 2·4 N/mm² for a/d = 2 to 0·56 N/mm² for a/d = 11·2. The shear strength of these beams can be predicted by plastic theory[8] recently developed for concrete beams. Figure 5 shows very good agreement between theoretical and experimental results.

As the shear strength of beams depends on several factors[3], it is difficult to compare the results of the prestressed beams directly with reinforced brickwork beams unless they are identical. Recently the second author has carried out a limited investigation on the shear strength of reinforced brickwork beams of identical cross-section to those shown in Figure 1. The average shear strength of brickwork beams of a/d ratio 6 with 0·8% and 1·0% areas of steel was 0·37 N/mm² and 0·48 N/mm² respectively, both of which are lower than 0·73 N/mm² obtained for prestressed beams of a/d ratio 7 having 0·548% area of steel. It appears that the shear capacity of a prestressed brick beam is better than reinforced brick beams but further work needs to be done to clarify this.

The reduction in ultimate flexural moment due to premature shear failure is given in the last column of Table 2. For beams 1–9, the experimental ultimate moments were 64% to 81% of the predicted theoretical flexural moment calculated from the mechanical properties obtained from axial compression tests on single-course brickwork prisms. The detailed derivation of the theoretical method is given elsewhere[7].

3.2 Influence of steel area on shear strength

The average shear strengths of both groups of beams built with different percentages of steel was the same (Table 2). From these results it appears that this small difference in steel area did not significantly affect the shear strength. However, the beams with lower steel areas, did not show the same degree of reduction in moment. In fact, the failure moments were very close to the predicted.

It has previously been shown[2,3] that an increase in steel area increases the shear strength of reinforced brickwork beams. From the limited number of tests this is not apparent in prestressed brickwork beams. In reinforced brickwork beams increasing the steel area may increase the contribution of dowel action to the shear strength. The strand used in the prestressed brickwork beams is very much more flexible than ordinary reinforcement and thus may not be very effective in transmitting shear due to dowel action. The effect of dowel action in prestressed concrete is generally considered less than in reinforced concrete[9] and this may apply to brickwork also.

4. CONCLUSIONS

1. The shear strength of prestressed brickwork beams decreases with increasing shear span/effective depth ratio.
2. From the limited test results it appears that the shear strength of prestressed brickwork beams is not affected by the percentage of steel.
3. The shear strength of prestressed beams can be predicted by the plastic theory originally developed for concrete.

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