V–12. Brick Diaphragm Wall Structures—Design and Application

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ABSTRACT

The diaphragm wall is simply a wide cavity wall with the two leaves braced by brick crossribs. The leaves and ribs form a series of connected box beams giving the brickwork a massive increase in vertical and lateral load resistance compared to the normal cavity wall.

The paper discusses the author's development of the technique and the design philosophy and procedure. Examples are shown of the application to tall single-storey wide-span structures where the technique has proved competitive in cost and speed of erection against the alternatives of steel or concrete framed structures clad with sheeting and insulation. Twelve structures have been built by the author in the past ten years in England and during that time have been subjected to the worst winds, hottest summer and wettest autumn on record, without distress. The technique has also been used for a mass retaining wall, where it showed cost and aesthetic advantages over reinforced concrete. Other applications are mentioned.

The paper briefly describes how the very encouraging results of the research work carried out at Liverpool University by the author and Professor Sawko has stimulated the author's interest in further applications of the method to multi-storey and semi-rigid structures.

INTRODUCTION

The brick diaphragm wall is simply a wide cavity wall with the two leaves bonded together not by the normal cavity ties but by cross-ribs of brickwork. The leaves and cross-ribs, acting integrally, form a series of connected box or I sections having a high section modulus and radius of gyration. This gives the wall an impressive resistance to lateral and vertical loading. The technique enables such walls to be used for tall single-storey structures where experience and investigation has shown them to be faster, simpler and cheaper to construct than the traditional steel frame and cladding.

The paper discusses the development of the idea, its applications, the design philosophy and future developments.

Development of the Diaphragm Wall Concept

The author 'stumbled' on the diaphragm purely by accident. He had designed a large school in load-bearing brickwork where much of the aesthetic appeal arose from the 'massing' of inter-connected plane surfaces of brick to the multi-storey and low-rise single-storey blocks. The only way to make the tall gymnasium walls a plane surface (then) was to have uneconomically thick walls. A possible solution was to have wide shallow piers to a cavity wall—this was unacceptable visually externally and equally unacceptable internally because of the inconvenience it would cause to ball games. So the piers were positioned *inside* the cavity and the brick diaphragm was born! (See Fig. 1b).

The wall was still regarded as an equivalent piered wall for effective thickness, slenderness ratio, lateral resistance etc. It only slowly dawned on the author, the blindingly obvious, that the piers could be deepened and narrowed and the wall treated not as an equivalent piered wall but as the completely different concept of the box section. (See Fig. 2 and Plate 1)

The idea of the diaphragm is so simple that many other engineers must have 'discovered' the technique and there is some evidence that the Victorian engineers may have used the technique. The author was merely lucky to have realised its potential.

Advantages of Diaphragm Walls

The application of diaphragm wall is mainly in 'shed' type structures, which accounts for a high proportion of buildings in this country. The vast majority of such structures have their roofs supported on steel columns. The columns have to be enveloped by a cladding material and on occasions, the cladding is backed up by an insulating material which in turn needs protecting by a hard lining. Frequently the cladding, insulating and lining need a subsidiary steel framework to support them. The frame, cladding, insulation and lining require maintenance (unlike brickwork) and lack brickwork's durability. The brick diaphragm forms not only the structural material but also the cladding, insulating and lining material. It has been found from experience, that brick diaphragms have always proved to be economical in capital cost (i.e. the cost to build). Accuracy in cost studies of construction is notoriously difficult to achieve and in a recent analysis of a hypothetical structure the diaphragm was found to be 4% more expensive to build[1]. Within a short time of publication of the paper a comparison was made of actual jobs and the diaphragm technique was found to be 4% cheaper! There has been no detailed cost investigation to prove the practically certain fact that they have lower maintenance costs. Since brick is a good insulator, heating costs are lower, therefore, current costs (the cost to run the building) are also low. Diaphragm structures are thus financially cheap to build and economic to run. Experience has shown that it is a faster and simpler method of construction than the traditional steel frame. The saving in site time can be of the order of 30% and an even greater saving in pre contract time can be achieved[2]. It has been found to give the architect much greater freedom, and scope in aesthetic treatment than the traditional 'tin shed'[3].

Increase in testing, knowledge, experience and confidence in the technique could lead to other applications where

lateral loading is critical and where low slenderness ratios are vital. As yet the technique is in its infancy and it is doubtful if its full potential has been appreciated.

Some application of Diaphragm walls

Over the last thirteen years, the author's practice has designed sixteen diaphragm wall projects. During that time the structures have withstood the worst gales, hottest summers and wettest autumn on record and the recent severe winter, without damage or distress. They were built to tight cost limits, there were no construction difficulties and they have required no maintenance, repair or decoration.

Some details and photographs of typical applications are given below:-

Swimming Pool—Turton, Lancashire (Plates Nos. 2 and 3)
Client, Lancashire County Council.
Architect, Charles Pearson and Sons.

The Architect decided to 'express the structure' and the diaphragm wall was made deeper at the base and top of the wall than structurally necessary. This enabled the cross ribs to be projected, over a large depth of the wall, beyond the outer leaf. This resulted in a bold modelling effect to the external elevation. Internally the inner leaf has, in places, been stepped back to form alcoves. The roof cladding is supported on precast concrete beams.

Theatre—Salford (Plate No. 4)
Client, Salford Players
Architect, Wilson and Womersley

The Architect decided to extend the roof sheeting over the parapet and down the wall to make a 'cap' effect. The roof cladding is supported on steel lattice girders. This building is situated in a run-down area yet has suffered no attack or disfiguration by vandals.

Sports Complex—Oval, Bebington (Plate No. 5)
Client, Bebington District Council
Architect, Cheshire County Council

The central core of the complex is used for offices, changing rooms, restaurants etc. On the western side are two swimming pools, with the roof supported on glu-lam timber beams. On the eastern side is the main sports hall with its roof supported by steel castellated beams.

Sports Hall, Robins Lane, St. Helens
Client, St. Helens Corporation
Architect, Ellis Williams Partnership

This is the largest project built to date. The height from ground level to top of roof is 10 m and the internal dimensions are 35 m × 37 m. Due to the large spans it was found economic to use a steel space frame for the roof (this is the largest space frame built in this country during the past few years).

Retaining Wall Freedom Gardens (Plate No. 6)
Client, Ashton-under-Lyne Council
Architect, Robert Shaw

These landscaped gardens were built on an old demolition site in the centre of the town to provide a social amenity. It was first proposed by the author to build the retaining walls in concrete with board-marked shuttering. However, the cost of shuttering, curved on plan and varying in height, was exorbitant and it was decided to use a filled diaphragm as a mass retaining wall. There was an abundance of demolished brick and stonework on the site which would have been costly to 'cart away and tip', and this was used as the filling and grouted in. The result was a very cheap and attractive retaining wall.

Sport Hall, Tomlinson School, Kearsley (Plate No. 7)
Client, Lancashire County Council
Architects, Hall and Wilson

It was decided on this project to express the roof structure, resulting in a 'battlement' elevation at roof level. The main buttresses envelope the roof beams and the minor buttresses envelope the wind bracing. This is in contrast to the Turton School project where the wall structure was expressed.

Basic Design Assumptions

The design of diaphragm walls is founded on a number of assumptions based in part on 'engineering judgment'. Probably the most important assumption is that such walls behave like *box-sections* (with integral action between the leaves and ribs)—unlike the normal cavity wall where such action cannot be considered.

Box-sections having a lower slenderness ratio and a higher section moduli than do plates of the same cross-sectional area can withstand greater axial and lateral loading. These assumptions were recently checked by research.[5]

It was decided to commence the testing work on the behaviour of the wall, as a strut, under axial loading. The load-bearing capacity of a strut is determined partly by its slenderness ratio. If the diaphragm wall behaved like a box-section then it would carry a greater axial load, because of its higher radius of gyration, than a conventional 'plate' wall (i.e. a solid wall, cavity wall or a wall stiffened by piers). It might also be more likely to 'crumple' under the load and not bow or buckle like a plate.

Further, if the wall did behave like a box-section it would give some indication that the concept of *'effective depth'* used in the Code might not be applicable to diaphragm walls and that instead consideration might be given to the use of the concept of *radius of gyration*.

If, too, the wall behaved like a box-section it would be capable of resisting a greater lateral load than a 'plate' wall of equivalent cross-sectional area, because of its greater section modulus.

The results of the research were very encouraging and are discussed elsewhere.[5]

The Code's recommendations and the basic research on which it is founded were probably made with normal cavity walling only in mind. Therefore, the recommendations may not be fully applicable to walls with very wide cavities and the leaves structurally tied.

Design Principles and Method

The following is an outline only of the design principles and method which have been used, since these are dealt with fully elsewhere.[6]

1. *The wall* is considered to act as a vertical box section.
2. *The roof* is designed to act as a horizontal plate member to prop and tie the top of the walls and to transfer the wind load to transverse, or gable walls.
3. *Capping beams* of concrete are commonly used to:
   a. transfer the wind force on the wall into the roof plate.
   b. provide an adequate factor of safety in 'tailing down' against wind uplift on the roof.
   c. act, when necessary, as boom members of the roof plate.
   d. assist in transferring the roof loads concentrically to the wall.
4. *The critical loading condition* on the wall is considered to be that due to combined dead and wind loading when the maximum uplift force on the roof and the maximum flexural tensile stress in the wall occurs. The compressive stresses due to combined dead and superimposed loading or the combined dead, superimposed and wind loading conditions are generally so low that the choise of brick and mortar is mainly determined by the tensile resistance of the brickwork. Thus the design of the wall is governed by its required resistance to lateral forces and stresses due to wind.
5. *The depth of the wall* (i.e. spacing of leaves) is governed by:
   a. the section modulus necessary to withstand the tensile stresses due to bending.
   b. the slenderness ratio required to cope with the compressive stresses.
   c. the need to provide an adequate 'stability moment'. As single-storey structures tend to have light-weight roofs and low superimposed loads the forces and moments due to wind, as mentioned above, will have far more effect on the stresses than they do in multi-storey structures. Since there is such little pre-compression on the wall its stability relies more heavily on its 'own weight' distribution and the resulting resistance or stability moment. (See Fig. 3)

      At the point of overturning the wall cracks at the dpc. level on the windward face and rotates at the dpc. level on the leaward face. The forces and moments causing the wall to crack are the wind force and the 'settlement' of the roof prop.
6. *The centres of the ribs* are governed by:
   a. the need to resist the shear stress, due to bending, between the ribs and the leaves.
   b. the ability of the windward leaf to span as a continuous beam subject to wind force and supported by and spanning between the ribs.
   c. the allowable flange length of the leaf when acting with the rib to form an I section.
   d. the need to prevent the possibility of the leaves buckling under compressive forces (The 'effective height' of a wall, in the Code, can be either the effective *vertical* height—or 'the *length* measured between adjacent intersecting walls').

Assumed Behaviour of Diaphragm Walls

It is assumed for initial design purposes (not 'academically' correctly) that the wall acts as a propped cantilever, 'propped' or supported at the top by the roof (which transfers the lateral forces to the gable or other transverse walls) and 'fixed' at the base. The fixity at the base is due to the stability moment from the wall's own weight and to the brickwork's slight resistance to tensile stress.

It is a simplification to refer to the wall as a propped cantilever since minor 'settlement' of the roof prop results in a major bending moment at the base; the moment at the fixed support of a propped cantilever with a deflection, Δ, at the free end being equal to $\frac{3EI\Delta}{L^2}$. (Similar simplifications occur when design engineers as distinct from structural theoreticians, discuss 'simply supported' beams, 'rigid' props, 'fixed ended' struts, 'pin' joints etc.)

If the bending moment induced by the settlement of the prop is greater than the stability moment then the wall will crack at the base on the windward face and rotate about the leaward face at the base. However, unless there is massive settlement of the roof prop the stability moment will remain relatively unaltered. The stability moment (as any resistance moment) is passive until activated by applied bending moments due to loading. The magnitude of the 'active' stability moment is dependent on both the magnitude of the bending moment due to loading and 'settlement' of the prop. When the wall cracks at the base on the windward face the reaction, to the wind force, at the base is provided by the shear resistance of the uncracked part of the leaward leaf. The wall thus no longer acts as a propped cantilever and, of course, no amount of further settlement of the prop or increased lateral loading can increase the moment on the cracked or pinned base—some inexperienced engineers may have difficulty in appreciating this.

The wall is then considered to act as a beam subject to a uniformly distributed load whose bending moment is partially counteracted by the stability moment.

The maximum forces, moments and stresses in the wall are determined at the two critical levels—the dpc. level at the base and at the position of maximum moment in the wall's height, which tends to occur above the mid-height position.

Design Procedure

The design procedure for the *wall* adopted in practice is as follows:

1. Calculate the positive and negative wind pressures.
2. Calculate the dead, superimposed and wind loading on the wall from the roof.
3. Select a trial section.
4. Calculate the size of ring beam, if required, at roof level and design, or check, the roof plate action.

5. Determine the free bending moment due to wind, $\frac{ph^2}{8}$, and the stability moment ($W \times \frac{D}{2}$, see Fig. 3) at the base of the wall and compare.
6. Calculate the position and magnitude of the maximum span wind moment and resistance moment, MR $= (f_c = f_t)\, Z$, of the wall and compare.
7. Check stresses at base of wall and at position of maximum span wind moment. If, at the base a tensile stress did occur it would be necessary to check that this tensile stress does not exceed the permissible stress at the dpc. If it does exceed the permissible stress, the section should be checked as a cracked section.
8. Choose brick and mortar.
9. Calculate the shear stress. If the ribs are not bonded to the leaves determine the type and spacing of metal ties.
10. Check the stability of the transverse walls for roof plate reactions.

The Structural Design Assumptions

(The detailed design principles are dealt with elsewhere)[5]

1. That the roof loading is carried by the whole box-section and not merely by the inner leaf.
2. That, in the vertical plane, the wall behaves as a simply-supported box-beam under wind loading, with some fixity at the base of the wall due to its own weight.
3. The section modulus, radius of gyration etc., is taken for the whole box section and not on an equivalent cavity wall in accordance with the previous and new Code.
4. That, in the horizontal plane, the outer leaf only acts as a continuous beam under wind lateral force and that the inner leaf deflects as a result of the cross ribs' deflection.
5. That the shear stress distribution across the wall resulting from lateral loading is the same as for an I section.
6. That the effective height is equal to the actual height.

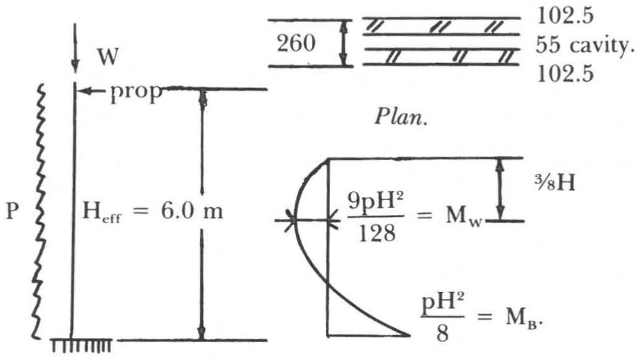
In addition to the above design assumptions there are a number of practical problems, for example:

1. *Leaf/cross rib connections:* On most contracts the cross ribs are bonded to the leaves but ties have also been used to form the connection and the outer leaf can then be built in stretcher bond. It is possible that too many (or less likely, too few) ties are being used.
2. *Roof/wall connection:* On some contracts, space frame roofs have been used, on others trusses and on other castellated beams. It is obvious that a space frame will impose a more uniform load on the wall than a truss but the magnitude of the difference is unknown. Most diaphragms have been capped with a reinforced concrete beam which helps distribute the load uniformly. Nevertheless the wall must be, in practice, loaded eccentrically and non-uniformly but by how much is unknown.
3. *Openings through walls:* There must be access to the structure—door openings are necessary. (Service engineers would like to run the services through the cavity but access to the cavity would be necessary for repairs, maintenance and alteration of the services.) Openings in walls create stress concentrations which must be considered.

Comparison Between a 260 mm Thick Cavity Wall and a 450 mm Thick Cavity Diaphragm Wall

It is interesting to compare the vertical loadbearing capacity and the resistance to lateral wind pressure of a 260 mm thick cavity wall and a 450 mm thick diaphragm wall of 6.0, effective height.

260 mm thick cavity wall



Consider axial load:

$$\text{Slenderness ratio} = \frac{6.0 \times 10^3}{2/3\,(102.5 + 102.5)} = 43.9$$

from BS 5628 - table 7 maximum slenderness ratio of 27 is exceeded therefore w = 0

i.e. 260 mm cavity wall cannot carry any axial load when the effective height is 6.0 M and therefore the building of this height of wall, is prohibited in 260 mm cavity construction.

Consider lateral load: (taking minimum axial load)

Assume wall construction comprises bricks with a compressive strength of 27.5 N/mm² and a water absorption of between 7% and 12% set in a grade (iii) mortar. (say $\gamma_m = 2.5$)

Properties of wall: (per M length of 2 leaves)

$$A = 1 \times 0.1025 \times 2 = 0.205 \text{ M}^2$$

$$Z = \frac{1 \times 0.1025^2}{6} \times 2 = 0.003502 \text{ M}^3$$

fkx from BS 5628 table 3 = 0.4 N/mm²

$$Gk = 19 \times 0.1025 \times 2 \times \frac{3}{8} \times 6 = 8.76 \text{ kN/M}$$

@ ⅜H

Design ow = Gk × f = 8.76 × 0.9 = 7.85 kN/M.

@ ⅜H
therefore:

$$\frac{fkx}{\gamma m} = \frac{\text{Design ow}}{A} - \frac{BM}{Z}$$

$$\frac{-0.4}{2.5} = \frac{7.85 \times 10^3}{0.205 \times 10^6} - \frac{BM \times 10^6}{0.003502} \times 10^9$$

$$-0.16 = 0.0383 \quad - 0.286\ BM$$

$$\underline{BM = 0.693\ kN.M}$$

also

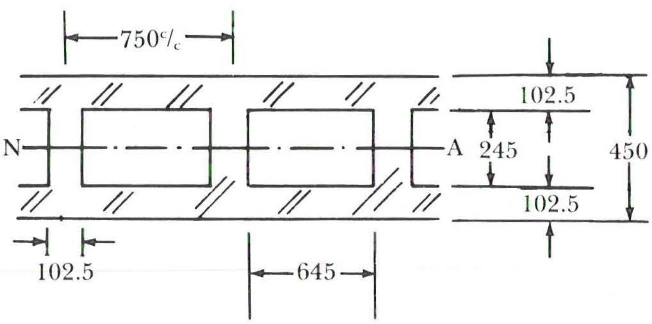
$$BM = \frac{9pH^2}{128}$$

$$0.693 = \frac{9 \times p \times 6^2}{128}$$

$$p = 0.274\ kN/M^2 = \text{Design wind pressure}$$

(and this depends upon the wall's ability to develop the base moment of $\frac{pH^2}{8}$)

450 mm thick Diaphragm Wall



Plan.

Wall properties: (per M length of wall)

$$A = [(0.1025 \times 0.245) + (2 \times 0.1025 \times 0.750)] \times \frac{1000}{750} = 0.2383\ M^2$$

$$I = \left[\left\{\frac{0.75 \times 0.45^3}{12}\right\} - \left\{\frac{0.645 \times 0.245^3}{12}\right\}\right] \times \frac{1000}{750} = 0.00655\ M^4$$

$$Z = \frac{0.00655}{0.45} = 0.0146\ M^3$$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{0.00655}{0.2383}} = 0.166\ M.$$

equivalent thickness of solid wall to give

$$r = 0.166\ M = \sqrt{\frac{\frac{bt^3}{12}}{bt}}$$

$$0.166 = \sqrt{\frac{t^2}{12}}$$

$$t = \sqrt{0.331}$$

$$\underline{t = 0.575\ M}$$

Consider axial load

$$\text{Slenderness ratio} = \frac{6.0 \times 10^3}{450} = 13.33$$

assume $e_x = 0$ to $0.05\ t$
therefore from BS 5628 Table 7

$$\beta = 0.903$$

Design Vertical load

$$W = \frac{\beta.A.\ fk}{m}$$

Note: fk from table 2a BS 5628 = 7.1 N/mm²

$$W = \frac{0.903 \times 0.2383 \times 10^6 \times 7.1}{2.5 \times 10^3}$$

Design vertical load $W = 611\ kN/M$

Consider lateral load (taking minimum axial load)

$$\text{Design own weight @ } \frac{3}{8}H = 0.2383 \times 19 \times \frac{3}{8} \times 6 \times 0.9$$

$$= 9.17\ kN/M$$

therefore:

$$\frac{fkx}{\gamma_m} = \frac{\text{Design OW}}{A} - \frac{BM}{Z}$$

$$\frac{-0.4}{2.5} = \frac{+\ 9.17 \times 10^3}{0.2383 \times 10^6} - \frac{BM \times 10^6}{0.0146 \times 10^9}$$

$$-0.16 = +0.0385 - 0.0685\ BM$$

$$BM = +2.898\ kN/M$$

$$\text{also } BM = \frac{9pH^2}{128}$$

$$2.898 = \frac{9 \times p \times 6^2}{128}$$

$$p = 1.145\ kN/M^2 = \text{Design Wind pressure}$$

Check conditions at base for $p = 0.81\ kN/M^2$

$$\text{base moment} = \frac{pH^2}{8}$$

$$MB = \frac{1.145 \times 6^2}{8}$$

$$MB = 5.153\ kN/M$$

Stability moment of resistance

Stability moment of resistance is provided the own weight of the wall acting at a leverarm (1a) produced by fully stressing the minimum width of rectangular stress block (see Figure 4).

$$\frac{fk}{\gamma_m} = \frac{7.1}{2.5} = 2.84 \text{ N/mm}^2$$

Design ow at base $= 0.2383 \times 19 \times 6 \times 0.9 = 24.45$ kN/M

Minimum width of rectangular stress block $= \dfrac{24.45 \times 10^3}{1000 \times 2.84} = 8.61$ mm

$$\text{leverarm} = \frac{450}{2} - \frac{8.61}{2} = 220.7 \text{ mm}$$

Stability moment of resistance $24.45 \times 0.2207 = 5.396$ kN/M^2

Therefore stability moment of resistance is greater than BM and the wall can resist p = 1.145 kN/M^2

SUMMARY

260 mm thick cavity wall

Maximum axial load	= zero kN/M
Maximum design lateral load (if wall could be constructed)	= 0.274 kN/m^2

450 mm thick diaphragm wall

Maximum axial load	= 611 kN/M
Maximum design lateral load	= 1.145 kN/M

The above comparison shows that, whilst the 260 mm thick cavity wall has no resistance to axial load at all, the 450 mm diaphragm has a massive resistance, far in excess of that likely to arise from wide span lightweight roofs.

Even if the 260 mm cavity wall could stand, it would blow down in a fairly light breeze, whereas the 450 mm diaphragm wall could resist greater wind pressures than those to which buildings in this country are generally subject.

SUGGESTIONS FOR FUTURE TESTING

Although the results of research and experience are very encouraging there is, as always, need for further research. Some suggestions are given below:

1. *Determination of behaviour at ultimate axial and lateral load.*
   Too few tests have been carried out to predict with complete confidence, the behaviour under ultimate load.
   It would be interesting to repeat the Redland test (6) on a wall built with lower strength bricks.
2. *Effective flange width and stress distribution in flange*
   Despite the Code's recent doubling of width to 12t, which was the author's conservative estimate, there is need for detailed information since this affects rib spacing. The design of the leaf as a 'continuous beam' supported by the ribs is a reasonable but unproven design assumption—an investigation of the stress distribution along the leaves would be valuable.
3. *Shear across bricks at interface*
   As far as the author knows, there is no research work recorded on the shear strength of bricks but only on the shear strength of brickwork. In walls the shear failure is *along* mortar joints but in diaphragms and fins[7,8] it could be *across* the bricks. Although in diaphragms there is, in the author's opinion, a more than adequate excess of strength, he is concerned about shear failure in fin wall structures.
4. *Effect of depth of rib*
   All of the author's diaphragms have been in the range of 2-3 deep brick walls. It has been shown that a 3 brick depth is the maximum efficient depth.[8] An investigation to confirm this would be valuable. This would be particularly important in tall, free-standing diaphragms.
5. *Effect of rib spacing*
   Increase of rib spacing would lead to overstressing and instability of the leaves, but would speeden and cheapen construction. At the moment, designers have to rely on experience and judgment in determining the rib spacing. It would be valuable to have experimental data.
6. *Effect of relative stiffness of wall and roof*
   The term 'prop cantilever' is an over simplication—the wall acts as a simply supported beam partially fixed at the base due to its own weight. If the support at the roof (the prop) is flimsy, it will 'settle' and deform laterally, and unexpectedly high compressive stresses might occur at the base of the wall.
7. *Stress 'spread' from capping beams*
   In many diaphragms the capping beam is carried by the ribs only and thus acts as a concentrated load on the wall. Though there has been plenty of work done on stress-spread in plane walls it is not known how rapidly it spreads in diaphragms. In practice the load on the rib could be eccentric which would complicate the stress distribution.
8. *Full size testing*
   Since the testing has been on model walls, the scale effect has to be taken into account in applying the work to actual structures. It would be reassuring to have confirmation of the predictions of scaling up.
   Testing of tall walls would confirm the theoretical analysis suggesting that the radius of gyration concept should be used and not the 'effective depth'[9].
9. *Combined loading*
   No work has been done on combined axial and lateral loading. Though, from the tests, the effect of combined loading can be predicted, it would be useful to have confirmation from testing.
10. *Eccentric loading*
    The effect of eccentric loading can only be estimated—it is not known precisely what the effect would be.
11. *Thermal Insulation*
    To meet the new requirements of the D.O.E. the

insulation will have to be improved. There are a number of simple ways this could be done but they need investigating. The B.R.E. have done some valuable calculations, but so far no experiments have been carried out.

12. *Shear Resistance at D.P.C.*
Under lateral loading shear stress develops at the D.P.C. and information on the shear resistance would be helpful.
13. *Maximum Height of the wall*
It is not *known* just how high these walls can be built. As a result of testing, the author would, with confidence design taller structures than he has in the past, but without further testing designers would be unlikely to use the technique for very tall structures such as large aircraft hangars, international exhibition centres, big theatres, sportdromes etc.
14. *Effective height of the wall*
The magnitude of the restraining effect of the capping beam or roof stiffness (and thus the effective height of the wall) is not known.
15. *Effect of Openings*
Present design is based on estimates. (All design is, of course, based on estimates and assumptions but these are founded on factual evidence—and there is none for openings in diaphragms).
16. *'In-plane' lateral loading*
The roof plate action transfers the lateral wind force to the gable walls. No tests have been done on in-plane forces on diaphragm walls, though it is likely that they have a high resistance to such forces.
17. *Comparison of shear resistance using:*
    a. brick bonding
    b. cavity wall ties
18. *Shear stress distribution across I section*
Brickwork is not a isotropic and homogenous material and it is not known if the shear stress due to bending does equal $\frac{SA\bar{y}}{Ib}$
19. *Effect of post-tensioning*
The author has proposed post-tensioning a diaphragm wall to counteract the effect of mining subsidence. The design will be based on experience—research is needed to reduce the probably conservative factor of safety.
20. *Crane gantries*
Projecting the cross-ribs through the inner leaf could provide a seating for the crane rails. Such crane loading would subject the wall to surge and sway forces and, at the moment, the wall's resistance can only be estimated.
21. *Multi-storey structures*
Any practising engineers, experienced in brick structures, will appreciate the applications of the technique to multi-storey structures. Until testing has been undertaken, such applications are likely to be ultra-cautious.
Floor slabs would be likely to be seated on the inner leaf only. Whilst this would eliminate the tiresome necessity to project the slabs over the outer leaf every third storey as in normal cavity walls it would eccentrically load the cavity wall. This creates a further need for the work mentioned in 10 above.
22. *Semi-rigid structures*
Preliminary feasibility studies show that, provided the leaves and ribs are reinforced to distribute load and stress from horizontal structural elements, semi-rigid structures are a distinct possibility.
23. *Deformation of Mortars and dpc under 'knife-edge' Loading condition.*
Rotation at base of diaphragms and fin walls will naturally produce deformation of the mortar or dpc whichever is the 'softer'. Though this is of minor interest (since only small deformation is necessary to relieve stress concentration) the work would be useful in removing an unnecessary worry from the minds of the inexperienced.

Future Developments of Diaphragm Walls

It is usually difficult to forecast the future development of new ideas but some which have occurred to the author or are on his 'drawing board' are given below:

a. *Industrial and Commercial*
The projects built so far have not been subject to by-law approval. Since the previous and new Code do not deal with this type of wall the technique may not be 'deemed to satisfy' the building regulations. When the Code is further revised there could be wide-spread applications in factories, garages, warehouses, supermarkets etc.[4]
b. *Prestressing*
Because of the high ratio of section modulus to cross-sectional area the wall is an ideal shape for prestressing. In the same way that a prestressed concrete I beam is more structurally and economically efficient than a solid rectangular concrete beam so is the diaphragm wall compared to anormal solid wall. A post tensioned diaphragm wall project will be constructed in 1979 for a structure which will suffer massive differential settlements due to mining subsidence.
c. *Multi-storey Structures*
The very high resistance shown by the walls—to vertical and lateral loading has stimulated the author to investigate their possibilities in some of his multi-storey 'frameless' structural projects.
d. *Some other Applications*
It is proposed or suggested to use a diaphragm wall for:
    (i) permanent shuttering and cladding to the 'unacceptable face' of a reinforced concrete retaining wall.
    (ii) a fire barrier in a tall, existing warehouse.
    (iii) a tall, free-standing boundary wall round an exclusive housing estate.
    (iv) a church to be built in noisy area where its acoustic insulating qualities will be a bonus.
    (v) a noise barrier on inner city motorways.
    (vi) a blast-resistant structural wall to a proposed factory adjacent to a chemical plant (after the Flix-

borough disaster it is a statutory obligation to design for blast-resistance where appropriate).

(vii) a mass-retaining wall, where the voids are filled with grouted-in rubble.

e. *Semi-Rigid Structures*
The very recent appreciation by the author of the possible new application of the stability moment concept for semi-rigid joints has led to preliminary feasibility studies—which look promising.

GUIDANCE TO ENGINEERS

It might appear imprudent to give advice to competent engineers as a result of a few buildings and some relatively unsophisticated tests but the test results give some confirmation of practical experience and the engineering assumptions and judgments. Firmer advice can be given after the proposed full size test and the other tests outlined above, have been completed.

In the end, as always, the engineer has to make his own mind up, make decisions and carry the responsibility for those decisions. The following notes have been included in this paper in the hope that they may be of some help in the decision-making process.

1. *Integral action* There is firm evidence, both in the laboratory and on site, that there is integral action between the leaves and cross ribs.
2. *Effective depth* The effective depth certainly appears to be at least equal to the overall depth of the wall.
3. *Rib Spacing* For the time being, rib spacing should not exceed twelve times the thickness of the leaves.
4. *Section Modulus* In determining the stresses under lateral loading, it is reasonable to take the section modulus of the wall as the section modulus of the 'box section' action and not merely as the sum of the section moduli of the two leaves. This is certainly applicable to one, two and possibly three brick deep cavities. Cavities of greater depth would almost certainly need stiffer (thicker) cross ribs than half brick one.
5. *Slenderness Ratio* When the wall has light lateral restraint from one side only, it would be prudent to consider the effective height as equal to the actual height. When there is light lateral restraint from both sides, a conservative choice of effective depth would be 0.9 times the actual height. This with an effective depth equal to the actual overall depth of the wall would give sufficiently low slenderness ratios to cause little concern in the overwhelming majority of present day applications of the technique.
6. *Overall depth* This would be governed by the need to provide an adequate section modulus to keep the stresses due to lateral loading within reasonable limits. The resulting depth of wall should then be checked for its capacity to resist vertical loading.
7. *Propping of the top of the wall* If the roof acts as a wind reaction, it will carry load, thus creating stresses, strain and therefore deflection of the roof. The roof must be stiff enough not to collapse (and normal lightweight roofs have yet to collapse even under unprecedented wind loading). An estimate of the roof deflection may be made and its effect on the moment at the base of the wall determined. If the moment is less than the stability moment, the stresses in the brickwork at the base should be checked in detail. If the moment is greater than the stability moment then the stresses at the base should be checked for the 'cracked section' behaviour.
8. *Specification and Supervision* Provided that the brickwork is built to the normal good specification and supervised in the normal competent manner, the author sees no need for special consideration in this technique.
9. *Rib/Leaf Connection* The author personally prefers cross bonding of the ribs and leafs and not the use of cavity ties. In view of the understandable present concern over the long term durability of cavity ties, the author could not, confidently, recommend their use—particularly for the outer leaf.

CONCLUSIONS

The success of the buildings in practice and the encouraging results of the preliminary research provide some evidence that the diaphragm wall technique is a viable, economic and attractive addition to the engineer's store of solutions in designing and building shed type structures.

Though the technique is in its infancy (and a lusty infant at that!) further experience, backed by research, suggest that there is potential for exciting developments.

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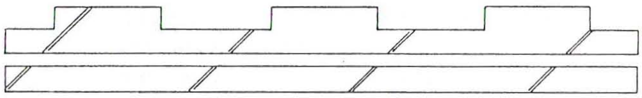
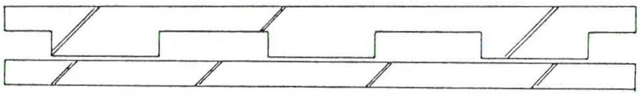
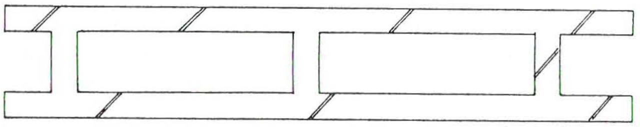


(a) Piered Cavity Wall



(b) Piers Inside Cavity



(c) Diaphragm Wall

Figure 1.

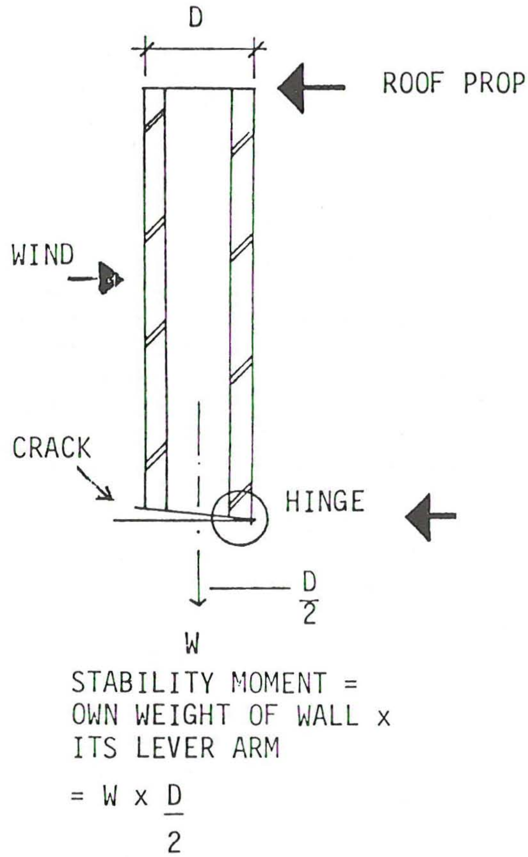


Figure 3.

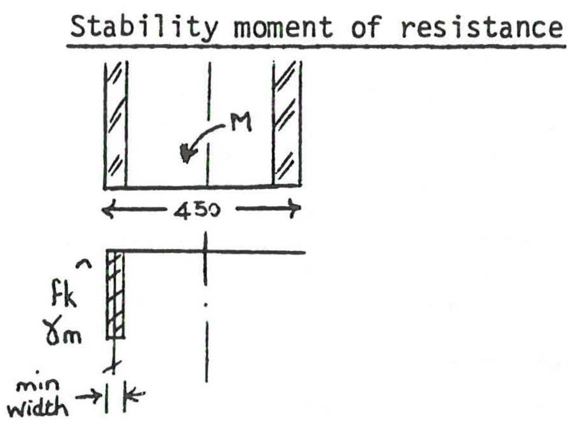


Figure 4.

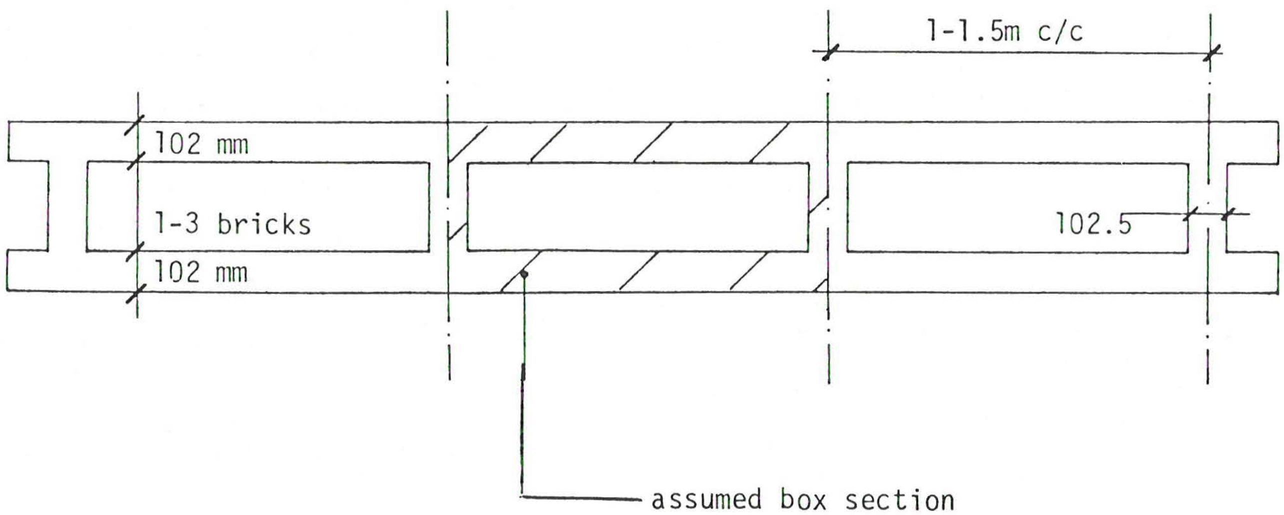


Figure 2.

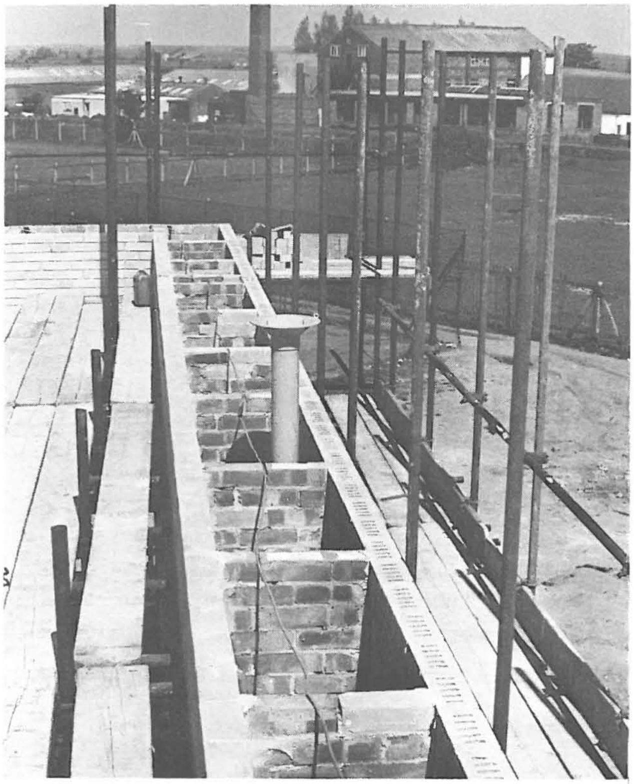


Plate 1. *Diaphragm Wall under Construction*
Clay facings external leaf, calcium silicates internal leaf. Sports hall for school at Ormskirk.

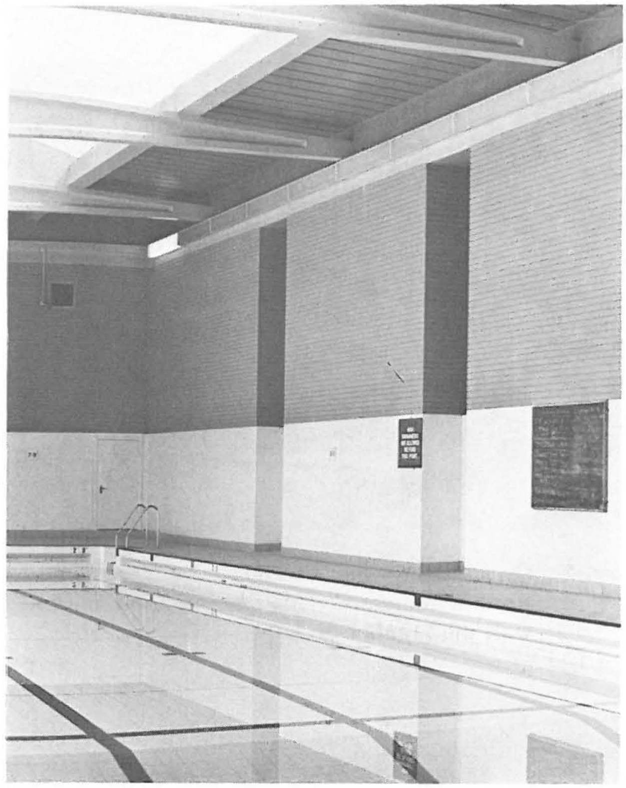


Plate 3. *Swimming Pool, Turton School, Bolton*
Roof structure is pre-cast concrete beams at 6 m centres supporting a domed roof light.

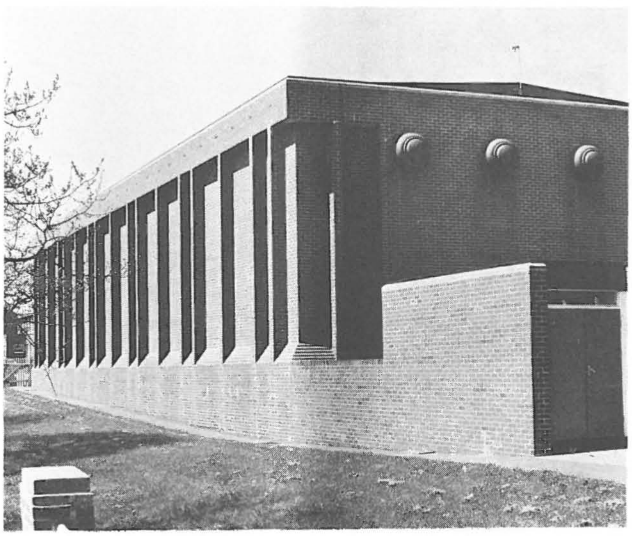


Plate 2. *Swimming Pool, Turton School, Bolton*
Cross ribs expressed externally.

Plate 4. *Salford Players Theatre*

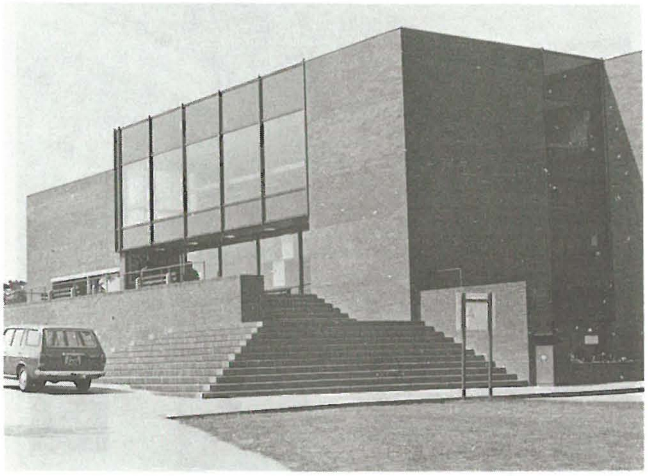


Plate 5. Oval Sports Centre, Bebington
Roof in laminated timber beams at 3.6 m centres with solid timber deck.

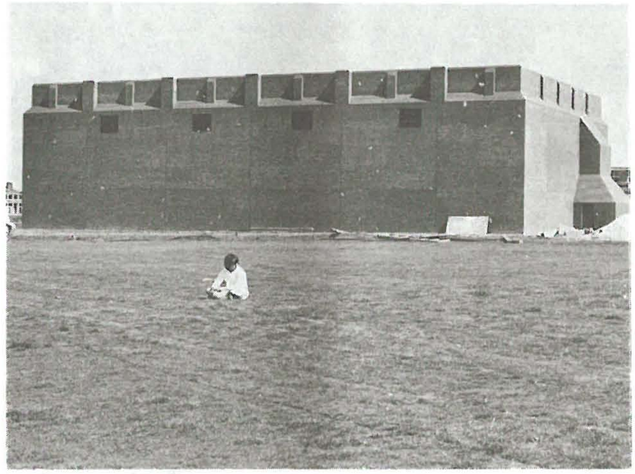


Plate 7. Sports Hall, Tomlinson School, Kearsley
Castellated effect above capping beam.

Plate 6. Mass Retaining Wall in Diaphragm Construction
Freedom Gardens, Ashton-under-Lyne.