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DESIGN OF POST-TENSIONED BRICKWORK

THE BRICK DEVELOPMENT ASSOCIATION

Cover & above: *The Orsborn Memorial Hall. Structural engineers: Curtins. Architect: Major D. Blackwell, The Salvation Army.*

As the economy of post-tensioned brickwork is becoming more widely understood, its use is developing rapidly – as are post-tensioning systems. Should galvanised bar be specified, the designer should *always* discuss its use with the manufacturer and satisfy himself that it will not suffer from hydrogen embrittlement.

Acknowledgement The authors are grateful to Thomas Telford Ltd for permission to quote from **Design of Reinforced and Prestressed Masonry.** Curtin, Shaw & Beck. Thomas Telford Ltd 1988.

Price £10.00

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CONTENTS

SYMBOLS

A horizontal cross-sectional area of brickwork
A_s cross-sectional area of steel rods
b breadth of section
B_r spacing between ribs for diaphragm wall
D overall depth of the diaphragm wall
d diameter of steel rod
e eccentricity of post-tensioning force from the neutral axis of the section
f_{bst} bursting stress
f_f characteristic compressive strength of brickwork in bending
f_p stress in steel rod/tendon due to prestress
f_k characteristic compressive strength of brickwork
f_{ki} characteristic compressive strength of brickwork at age at which post-tensioning force is applied
f_{kx} characteristic flexural strength (tension) of brickwork
f_v characteristic shear strength of brickwork
f_y characteristic tensile strength of steel
f_{uac} compressive stress in brickwork due to direct compression
f_{ubc} compressive stress in brickwork due to flexure
f_t 'theoretical' tensile stress in brickwork due to flexure
f_{pt} principal tensile stress
g_A design vertical load/unit area
g_d design vertical dead load/unit area
G_k characteristic dead load
h_{ef} effective height
M bending moment due to design load (M_w in wall height; M_b at base of wall)
I_{na} moment of inertia (second moment of area)
P axial load
P_k characteristic post-tensioning force
q lateral force applied to a prestressed element
Q_k characteristic superimposed load
r radius of gyration
T flange thickness in diaphragm wall design
S.R. slenderness ratio
t overall thickness of wall
t_{ef} effective thickness
t_r thickness of cross-rib in diaphragm wall
V shear force
W_k characteristic wind load
Z section modulus
v_h shear stress
$\bar{y}$ distance from neutral axis to centroid of area
β capacity reduction factor
γ_f partial factor of safety for loads
γ_{mm} partial factor of safety for brickwork
γ_{ms} partial factor of safety for steel
γ_{mv} partial factor of safety for brickwork in shear

1 INTRODUCTION & GENERAL CONSIDERATIONS

INTRODUCTION

Brickwork has many good qualities as a structural material, and few inadequacies when used in traditional circumstances with reasonably high compressive axial loads and relatively small lateral applied loading.

Once these circumstances are reversed, and the brickwork is asked to resist relatively high lateral forces with small, or no, applied axial compressive stress from the structure, then its low resistance to flexural stresses proves to be a highly limiting factor. The flexural (ie. bending tensile) strength of brickwork is only about $\frac{1}{20}$ of its direct compressive strength. Thus, a structural brickwork element, subject to bending alone, will fail in tension long before its compressive strength is exploited.

In order to improve the efficiency of brickwork structural elements and hence widen their application, this major weakness in resisting flexural tensile stress must be overcome. Reinforcing brickwork by introducing steel reinforcement into the bed joints, or vertical ducts, of the element, goes some way to improving the flexural tensile resistance. We have recognised, however, for many years, that further improvements could be achieved in the flexural tensile resistance of brickwork elements, by way of inducing compressive axial stresses prior to the application of the flexural tensile stresses, ie, by prestressing the brickwork.

The method of inducing this prestress can be achieved by either pre-tensioning or post-tensioning steel rods, or strands, within the element, and transferring the compressive stresses to the brickwork by means of anchorages within it, or a continuous bond throughout.

To date, the more practical of the two methods has been found, by experience, to be post-tensioning, and the following notes and details are concerned primarily with this form of construction. Designers with experience and a knowledge of the principles of prestressed concrete design, should have no problem in understanding the guidance given in this design guide.

1 & 2. An early post-tensioned project, *dating from 1967, by the authors. Several schemes were costed using steel frames and brick cladding. It was found that post-tensioned brickwork for the 10m high piers provided a more economical solution. Seeking a better solution for problems of the Flint High School Sports Hall, the diaphragm wall concept went on to develop into the fin wall. Later, the enormous potential of the post-tensioned diaphragm and the post-tensioned fin wall was recognised. Architect: Clwyd County Architects Department. Consulting engineer: Curtins*

BACKGROUND

There appears to be very little written information available concerning the post-tensioning of brickwork, unlike the prestressing of concrete, which has been widely used for many years, with both successful and very well documented results. The authors interest in and knowledge of structural brickwork applications developed over the years and it soon became apparent that there was an obvious parallel to be drawn in the advantages gained by prestressing brickwork and concrete. Both brickwork and concrete are strong in compression and relatively weak in tension, and both benefit from induced compression, to cater for flexural tensile stresses.

The authors began post-tensioning brickwork in the late 1960's when, in order to provide additional resistance to tensile stresses due to lateral loading from wind, the piers to a tall sports hall wall, in Flint, were post-tensioned. At this time, a very conservative approach using a high factor of safety was adopted for the design, little research data and even less practical evidence being available.

In the early 1970's, the practice was awarded a large contract to design some 200 primary schools. The requirements were for lightweight roof construction, 11in brick cavity walls as external cladding and few internal supports. There were, too, extremely tight cost limits and the schools had to be easily constructed by small builders.

Basing decisions on the cost of alternatives, it was decided to post-tension the 11in cavity external cladding, so that it could be used structurally, instead of the more usual mixture of steel structural frame with non-structural brick cladding. Because the contract was of a large scale, the practice was able to spend more time and money on research[15] to verify the design assumptions, instead of relying on their usual site testing. The result was an economic, durable and buildable form of construction which has proved a complete success.

Developments in fin wall design[6] also brought about further progress when, in the mid-seventies, a post-tensioned fin wall was used for a gable wall to a factory building and, later, a post-tensioned fin wall for a retaining wall.

The above developments were carried out, coincidentally, with separate advancement in the field of the diaphragm wall[5]. However, in the late 1970's a diaphragm wall sports hall was to be built in an area subject to mining subsidence. The differential settlements produced from the predicted subsidence would induce unacceptable tensile stresses in the diaphragm walls, so the problem was overcome by the, now, obvious solution of post-tensioning them. The walls in question have successfully withstood three signifcant waves of mining subsidence.

The combination of two innovations – prestressed cavity walls and the development of geometric sections – evolved separately, to solve practical problems, had produced a third development. This gives perhaps, further proof to the adage that, 'in innovation 1 + 1 = 3'. Whilst 'necessity is the mother of invention', it is the grandmother of useful research.

PRESENT AND FUTURE

The success of this, and other projects, initiated post-tensioned diaphragm research[12], which has since been applied to a number of successful projects. The authors admit that it was some time before the now obvious fact was appreciated that this was a quantum leap. Should this appear an extravagant, or immodest, claim, it is of interest to compare a plain masonry cavity wall with a post-tensioned diaphragm wall. The moment of resistance of structural elements is $M_R = f \times Z$. It can be shown[1] that a diaphragm wall can have a Z value 15 times that of a cavity wall. Plain masonry will fail in tension long before failing in compression – the compressive strength being often 20 times the tensile strength. As will be shown later, f can easily be increased by more than 10 times as, by prestressing, the governing strength limitation is compression, not tension.

Relatively, for a cavity wall $\quad f \times Z = 1 \times 1 = 1$

and,

for a post-tensioned diaphragm wall $\quad f \times Z = 10 \times 15 = 150$

It should be noted that this is not a 150% increase in strength, but a 150 times increase. It is analogous to improving a car's petrol consumption from 30 mpg to 4,500 mpg. If the prestress is applied eccentrically, at the edge of the 'middle third', f can be increased to 20, giving $f \times Z = 20 \times 15 = 300$ – a 300 times increase. We are dealing with a new material, with wide ranging applications.

Fresh development work is hampered by lack of research funding, since the scale and complexity of such research cannot be met by our usual method of simple site testing. We are confidently hopeful that, as appreciation of the potential of the technique increases, funding will be forthcoming. It is likely that innovative designers, with research back-up, will make further advances.

ADVANTAGES

(1) It considerably widens the application of structural brickwork since it greatly enhances the flexural tensile resistance.

(2) Experience has shown it to be both highly cost effective and buildable. It does not demand sophisticated 'high-tec', and many successful contracts have been carried out by small builders.

(3) It improves the flexural tensile resistance, and gives greater structural efficiency more easily than by other more convential forms of brickwork reinforcement.

(4) It improves durability due to a reduction in micro-cracking.

(5) It allows the designer to create a complete range of structural elements in a consistent material. He/she no longer has to rely on using small elements either in concrete, or more usually steelwork, in addition to brickwork, in order to make the whole structure stable.

(6) It makes the highly important change for brickwork (as for concrete) from a brittle material to a ductile one. It will bend without cracking: it will deflect and recover on removal – unlike plain brickwork, which remains cracked. This development can have far reaching consequences as designers appreciate its potential.

(7) There is increased robustness, resistance to impact, together with high improvement in accidental damage resistance.

DISADVANTAGES

(1) When used for post-tensioning horizontal brickwork elements, it is generally difficult to achieve suitable anchorages within the vertical joints. In reinforced brickwork, it is relatively easier to produce workable, buildable details for horizontal members.

(2) Post-tensioning of 'high shrinkage' units is restricted, due to the possibility of excessive losses of prestress.

It should be noted that the advantages listed are of a broad nature, whereas the disadvantages tend to be rather specific. It is suggested that this merely highlights the overall beneficial applications of post-tensioned brickwork.

APPLICATIONS

So far, the applications of post-tensioned brickwork range from spandrel panels and isolated columns to tall fin or diaphragm wall buildings and retaining walls. The use, therefore, can vary from single elements within large structures to the main structure of complete developments.

See photographs for some typical examples of post-tensioned structures. With further practical experience, innovations, good research, etc., there will doubtless be further applications.

DURABILITY

A very important consideration when adopting any structural form in any material is its durability. As with reinforced brickwork, there must be due regard given to the detailing, and the method of protection of the steelwork within the post-tensioned element. The durability of post-tensioned brickwork, and the various details which may be adopted, are discussed in section 3 and illustrate that, providing thought and care are applied to the detailing, then an adequate design life should be achieved.

The main cause of durability failure in brickwork, as in other materials, is the ingress of water. Any tension cracks in brickwork allow rain penetration. Prestressing, by reducing tension cracking, increases durability.

Many of the structures illustrated, built for public authorities, were erected more than a decade ago. They have been subject to some of the worst weather conditions of this century – none show signs of distress.

FULL PRESTRESSING

The concepts discussed so far have, in the main, been limited to partial prestress. It was appreciated, of course, that even wider application and greater potential was possible with testing of full prestressing.

In 1981, the first author carried out research into diaphragm walls, which was largely funded by BDA. Two tall narrow brick diaphragm walls were built, back to back, with an air-bag between them, and tested under varying loads of prestress to withstand increased uniformly distributed lateral loading. The walls were 7.62m long and 7.62m high, pin tied at the top, and had only one brick depth of cavity. Construction was in a relatively weak brick (characteristic compressive stress 20 N/mm^2), and the standard of brickwork, although good by common site practice, did not comply with BS 5628's 'normal' classification.

The tests fully validated his assumptions. Under a maximum prestress of 1.378 N/mm^2, the test wall withstood the high lateral pressure of 7.18 kN/m^2. This was despite the fact that in earlier tests the wall was cracked both in the span and in the base. The tests also proved that prestressing changes brickwork behaviour from brittle to ductile. Removal of lateral loading resulted in the recovery of deflection, closing of any cracks, and the wall retained its bending strength. Under increasing levels of prestress there was an increasing improvement in stiffness – for given lateral load the wall deflected less and the bending strain decreased.

Engineers' understandable criticisms of structural brickwork that it has low resistance to lateral loads, once it is cracked it remains cracked, it is brittle, etc, do not apply to prestressed brickwork.

Subsequent research was funded by the Building Research Establishment, The Royal Society, and others.

Five years later, in 1986, George Armitage & Sons plc funded further research. Two walls were built – one post-tensioned eccentrically and the other concentrically, see Figure 1.1. They were constructed back to back, with the latter (wall B) acting as a reaction force to lateral loading, and were built as free cantilevers in Class A engineering brickwork (mean strength 36.6 N/mm^2) to 'special' category of construction.

In the first test, with a prestress of 5.6 N/mm^2, the test wall (wall A) was designed to withstand a design earth pressure of 45 kN/m^2 – over 900 lb/sq ft – ie, actual pressure × 1.6 factor of safety, to a height of 4.7m. This it did totally successfully.

The wall was then tested beyond its limits, to a height of 6.0m under working load – ie, no factor of safety – when the loaded leaf cracked in tension and the cross-ribs cracked in principal tensile stress. On removal of the load, the cracks closed up, the wall was 'as good as new' and, again, successfully withstood a design earth pressure to a height of 4.7m. The deflection was only $\frac{1}{1000}$ of the span (BS 5628 permits $\frac{1}{125}$ – 8 times as much) and was practically fully recovered on removal of the load. There is no way that plain brickwork for this wall could have withstood such high loading, nor, having cracked and deflected, recovered on removal of the load.

In one of the later tests, the cracked wall was prestressed to 10.9 N/mm^2, when it again withstood a base pressure of 45 kN/m^2 to the greater height of 5.4m.

For the initial research project, the authors would like to express their gratitude for the use of UMIST laboratories for long periods of time, and for the helpful and generous co-operation of the staff. The follow-up research was conducted at British Ceramic Research Limited.

It is hoped to apply this major break-through on a contract in the near future. Design engineers will appreciate the importance of this advance, and the application this highly cost-effective, very buildable and durable technique is likely to increase the potential of structural brickwork. More research, however, is necessary if this notable advance is to be further developed.

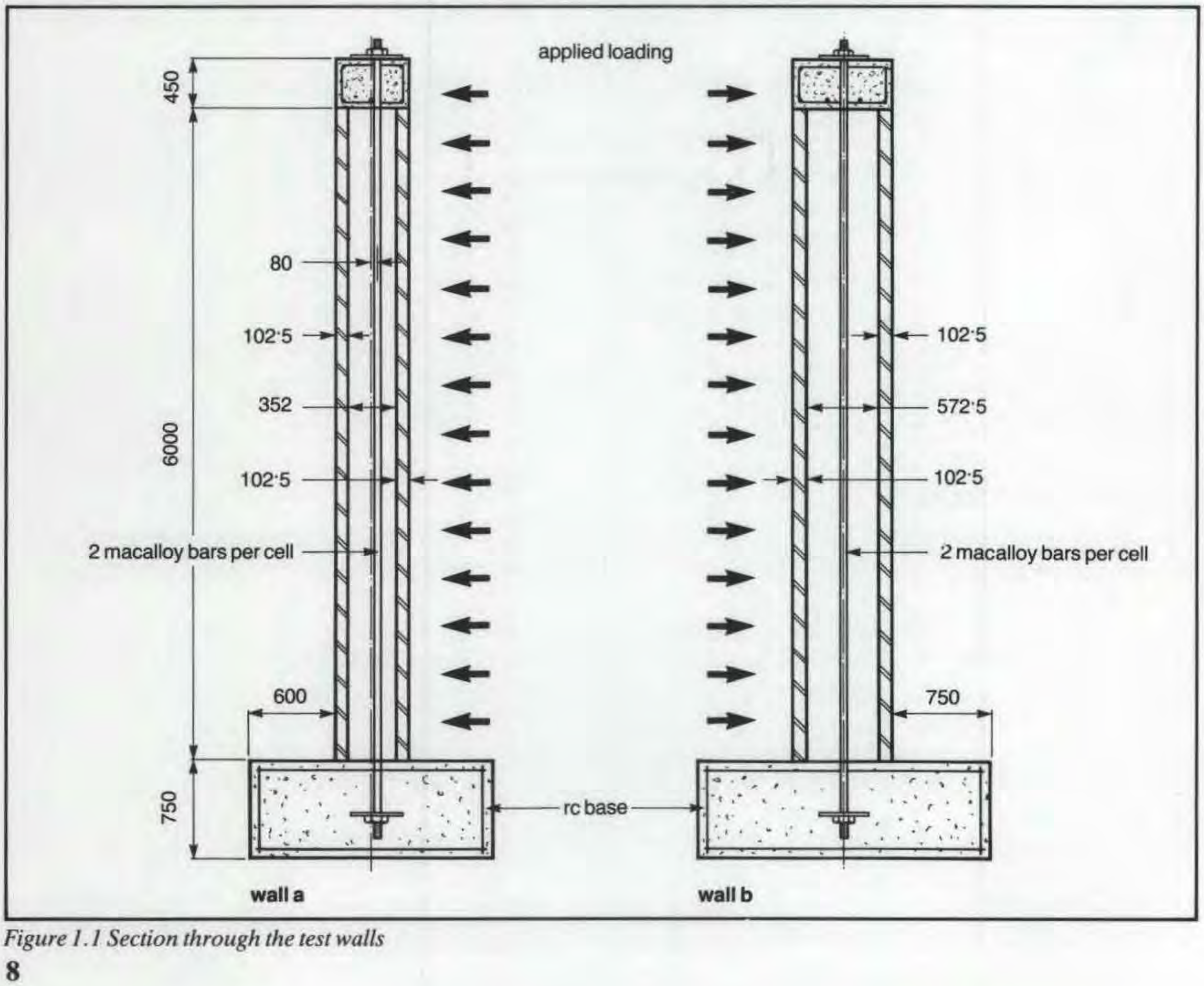

Figure 1.1 Section through the test walls

2 BASIS OF DESIGN

GENERAL THEORY OF PRINCIPLES

The improvement in resistance to applied flexural stresses, within the brickwork element is gained by applying an axial compressive stress to the element, to counteract the applied tensile stresses, see Figure 2.1.

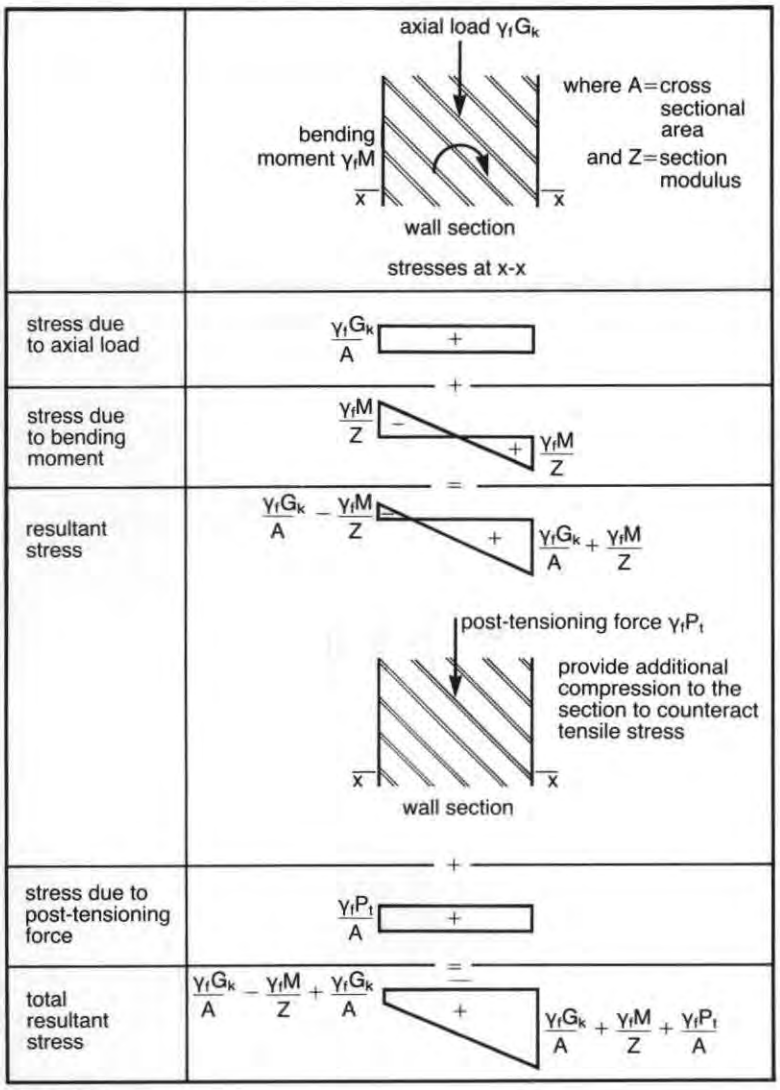

Figure 2.1

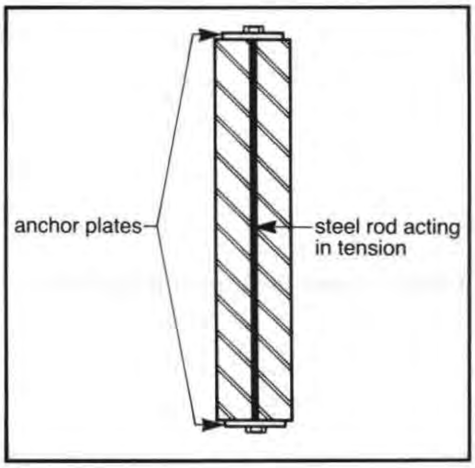

Figure 2.2

The compression may, for example, be applied by means of the reaction of steel rods, acting in tension, fixed at each end of the structural element, see Figure 2.2.

The applied flexural tensile stress may be due to lateral loading, or the result of eccentricity of axial loading – the effects, and means of overcoming them, are similar.

The structural brickwork is prestressed, ie. stressed before the flexural tensile stress is applied, with an additional compression. This is usually achieved by post-tensioning by steel rods or strands, placed within the brickwork section, and stressed *after* the construction of the element is complete.

The additional compression applied – the post-tensioning force – may be applied at the centroid of the section. Alternatively, by applying the post-tensioning force at some eccentricity to the centroid,

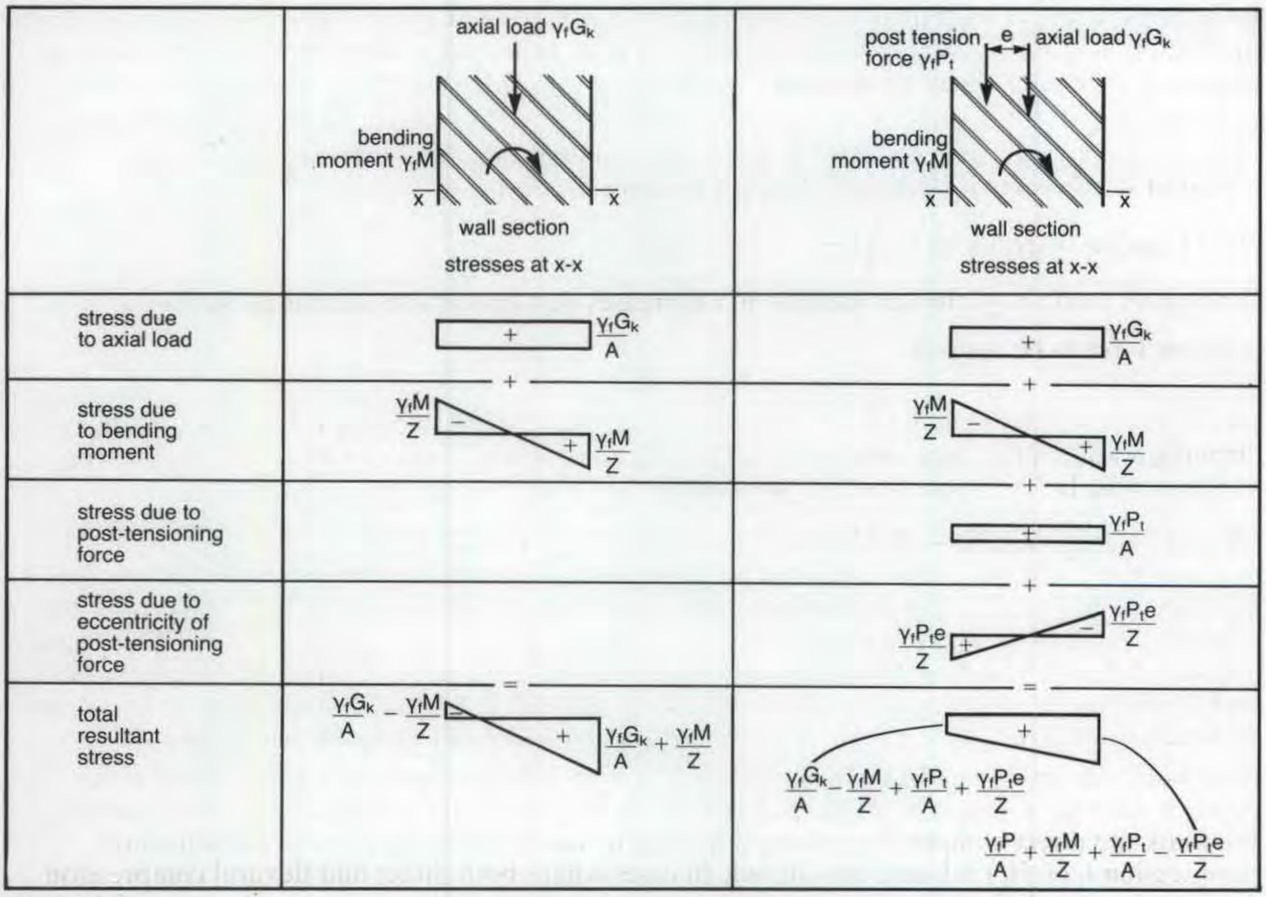

Figure 2.3

an additional counteracting moment, as well as an axial compression, may be introduced into the section. This additional moment should be applied in reverse to that due to the applied loadings, to counter-balance the effect, see Figure 2.3.

By applying a moment in the reverse direction, the compressive stress is reduced by the amount of tensile stress induced due to the reverse moment. This means that the gain in flexural tensile resistance of the section, due to post-tensioning, is not directly accompanied by a cumulative increase in compressive stress within the section, which may otherwise possibly limit the potential compressive resistance. The most suitable eccentricity for the post-tensioning force may readily be determined (as shown later) to give the optimum levels of prestress for the given loading conditions.

In general, the post-tensioning force is placed concentrically where reversal of applied moments is possible, eg. wind loading, and eccentrically where the applied moments are predominantly in one direction, eg. retaining walls. There are cases, however, where eccentrically placed post-tensioning may be used to advantage, even where there is reversal of applied moments, eg. in asymmetric sections.

IMPORTANCE OF Z/A RATIO

A high Z/A ratio is important in prestressed design. The design bending moment must not exceed an element's moment of resistance, which is equal to f × Z.

ie $$M_R = f \times Z \quad \ldots\ldots \text{(i)}$$

With eccentric prestress, $$f = \frac{P}{A} \pm \frac{Pe}{Z} \quad \ldots\ldots \text{(ii)}$$

For zero tensile stress, $$f_t = \frac{P}{A} - \frac{Pe}{Z} = 0$$

Therefore, $$\frac{P}{A} = \frac{Pe}{Z}$$

Substituting in equation (ii), $$f = 2\frac{P}{A}$$

Substituting this value of f in equation (i) gives, $$M_R = \frac{2P}{A} \times Z$$

Rearranging $$M_R = 2P \times \left(\frac{Z}{A}\right)$$

For a given P, then the moment of resistance (and thus the bending moment) is directly proportional to Z/A. For a given A, then an increased Z results in an increased moment of resistance, and thus permits an increase in bending moment.

For a given A, the Z/A ratio of a solid square, or rectangular section will be lower – and thus less structurally efficient – than a hollow box, tube, tee, or other geometric section. Similarly, an increased Z results in an increased I (second moment of area).

Since r (radius of gyration) = $\sqrt{\frac{I}{A}}$

slenderness ratio $= \frac{L}{r}$, then an increase in r decreases slenderness and permits an increased prestress force to be applied.

As normal cavity walls are 'slender', their maximum height for prestressing is less than 4 m, but a diaphragm wall, of the same cross-sectional area but increased I, can be built to twice this height. See Appendix D for a more detailed comparison.

BRICKWORK DESIGN STRENGTHS – DIRECT & FLEXURAL

In dealing with the limit state design of plain brickwork, the brickwork strengths are expressed in terms of characteristic strength (direct compression) and characteristic flexural tensile strength (tensile strength in bending). As discussed later, in general, post-tensioned brickwork is designed to give zero nett flexural tensile stress, and hence design is based on compressive strengths. In post-tensioned brickwork, as is shown in Figures 2.2 and 2.3, flexural compression occurs as well as direct compression, and both need to be considered when assessing material strengths.

Brickwork, like concrete, has a higher flexural compressive strength than direct compressive strength, and the relationship between the two values in brickwork is currently under discussion. We think the direct compressive strength f_k, may be increased by up to 50% when flexural compression f_f ($1.5f_k$) is being considered. In cases where both direct and flexural compression are combined, a sliding scale is proposed between 1 and 1.5 f_k, depending on the ratio of direct to flexural compression. Such an approach, however, needs to be supported by appropriate research. In the absence of such research, and a suitable sliding scale, we would suggest that a value for flexural compression, f_f, of $1.2f_k$ should be used for the general situation. Adjustments can then be made, up or down, at the designer's discretion and where appropriate. Values of f_k are given in Appendix A. An alternative approach, which has recently been proposed (BS 5628: Part 2[3]), is to allow no increase in flexural compression, but to reduce the partial factors of safety on material strengths from 2.5 and 2.8, for plain brickwork, to 2.0 and 2.3 respectively, for reinforced or prestressed brickwork.

We are of the opinion that there is good reason to adopt both lower values for the partial factors of safety on material strengths, and the increased value of flexural compressive strength of $1.2f_k$. But, for the purposes of this design guide, in order to retain a conservative approach and obtain results compatible with the alternative method, we would suggest that the $1.2f_k$ value should be used for flexural compressive strength, and no reduction be made to the partial safety factors for material strengths.

Both approaches produce approximately similar design strengths:

	f_k/γ_m (special)	f_k/γ_m (normal)
Code	$\frac{1.0f_k}{2.0} = 0.50f_k$	$\frac{1.0f_k}{2.3} = 0.435f_k$
Authors	$\frac{1.2f_k}{2.5} = 0.48f_k$	$\frac{1.2f_k}{2.8} = 0.428f_k$

Since brickwork, as a material, does not change whether it is reinforced, prestressed or plain, we suggest that it is perhaps inappropriate to change the material's factor of safety. However, since, as noted above, the direct compression strength differs from the bending strength, we think it is not unreasonable to alter f_k. Designers should exercise their engineering judgement as to which approach is the more logical.

DESIGN STRENGTH – INITIAL & LONG TERM

Values for the characteristic direct compressive strength, f_k, are given in BS 5628: Part 2[3], for walls tested at an age of 28 days. As it is not generally practical, from a construction point of view, to wait for 28 days to elapse before post-tensioning, any design check for the initial loadings should be based on the compressive strength at the relevant age (f_{ki}). We consider that, other than in exceptional

3 & 4. *Studies were carried out to evolve a standard form of construction for Cheshire County Architects Department. The key element in the resulting SCD system is the post-tensioned brick cantilever wall. Holywell RC Primary School (3) is a good example of the varying brick panel heights and configurations with differing post-tensioning requirements. At Richard Gwyn High School (4) the first-floor brick spandrel walls, with the long runs of glazing over, are post-tensioned to the first-floor slab. Architect: Weightman & Bullen. Consulting engineer: Curtins*

circumstances, post-tensioning should not be undertaken until 14 days have elapsed, or until the mortar has reached a specified strength.

The post-tensioning force to be applied is generally increased, to allow for the effects of losses (see p20), the initial post-tensioning force thus being larger than that applied over the long term.

The post-tensioning force, before losses occur, may be considered as a temporary load, and that remaining after losses as a permanent load. It is considered reasonable, within the context of the design approach given in this document, to modify the value of the design strength used, when considering the short-term duration of this initial loading case, ie. the post-tensioning force before losses. At present an increase of 20% is considered appropriate, giving a design *direct* compressive strength of $\frac{1.25f_{ki}}{\gamma_{mm}}$ and a design *flexural* compressive strength of $\frac{1.25\,(1.2f_{ki})}{\gamma_{mm}}$. If the loss in prestress is 20% then, to allow for such loss, the prestress must be increased by 25%, ie. 1.25 – 20% = 1.25 – 0.25 = 1.0.

DESIGN STRENGTH: CAPACITY REDUCTION FACTOR

The capacity reduction factor, β (see BS 5628: Part 1[2]), was introduced into the design of plain brickwork, as a means of allowing for the reduction in compressive load-carrying capacity due to buckling failure. In this method, the design compressive strength is reduced by a factor which takes account of the increase in the flexural compressive stresses (produced within the section) when the axial loading is applied. These flexural compressive stresses can be induced as a result of buckling, or curving, of the section, even if the applied loading is nominally concentric. There is further reduction in capacity when the axial loading is applied eccentrically, since this increases the possibility of buckling.

The use of the capacity reduction factor thus tends to obviate the need to consider flexural

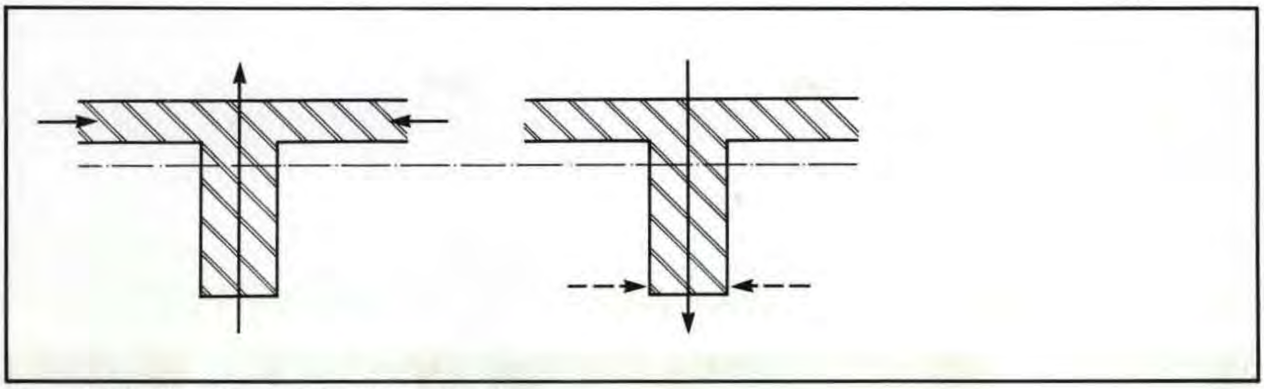

Figure 2.4 Buckling characteristics of fin wall sections

compressive stresses separately from direct compressive stress, when considering simple sections subject to basically axial loading. When, as in post-tensioned brickwork, flexural compressive stresses are specially calculated, we suggest that the relevance of the capacity reduction factor requires careful consideration.

In walls, and similar solid rectangular sections, the general possibility of buckling relates only to direct stress. The tendency of the section to buckle, when subject to flexure, is limited to some extent by the restraint offered to the compression edge by that part of the section which is in tension. Buckling is also limited along the length of the wall, by adjoining or interconnecting walls, or other stiffening elements.

However, in some geometric profiles, such as the fin, if the flexural compression occurs totally within the flange, that brickwork may not benefit from the presence of any adjacent brickwork. It is then possible for the whole flexural compression zone to buckle independently. In this condition, it is considered relevant to apply the capacity reduction factor to both the flexural compressive strength and to the direct compressive strength. Thus, when comparing applied design stresses with design strength, careful consideration must be given to the application of the capacity reduction factor, β.

Consider, for example, a rectangular wall where the sum of the compressive stresses due to flexure, f_{ubc}, and direct compression, f_{uac}, must be less than the design strength, f_f or f_k as appropriate. Compressive stress due to flexure + direct compressive stress < design compressive strength:

$$f_{ubc} + \frac{f_{uac}}{\beta} < \frac{1.2f_k}{\gamma_{mm}}$$

The design strength quoted is for other than short term loading and is the value of the flexural compressive strength. This is correct for the combined loading condition, but for the direct compression alone f_{uac} should be compared with $\frac{\beta f_{ki}}{\gamma_{mm}}$.

To summarise the design strength limitations:

SHORT-TERM LOADING – BEFORE LOSSES

$$f_{uac} < \frac{1.25f_{ki}}{\gamma_{mm}} \quad \ldots\ldots\ (1)$$

$$f_{ubc} < \frac{1.25\ (1.2f_{ki})}{\gamma_{mm}} \quad \ldots\ldots\ (2)$$

$$f_{ubc} + \frac{f_{uac}}{\beta} < \frac{1.25\ (1.2f_{ki})}{\gamma_{mm}} \quad \ldots\ldots\ (3)$$

except fin, diaphragm and special cases where β also applies to f_{ubc}

GENERAL CASE – AFTER LOSSES

$$f_{uac} < \frac{f_k\beta}{\gamma_{mm}} \quad \ldots\ldots\ (4)$$

$$f_{ubc} < \frac{1.2f_k}{\gamma_{mm}} \quad \ldots\ldots\ (5)$$

$$f_{ubc} + \frac{f_{uac}}{\beta} < \frac{1.25f_k}{\gamma_{mm}} \quad \ldots\ldots\ (6)$$

except fin, diaphragm and special cases where β also applies to f_{ubc}

Refer to section 4, Worked Example No. 2, for example of calculation of the above theory.

β, the capacity reduction factor, is based upon the slenderness ratio, S.R., of the structural element.

The Code (ref 2) bases S.R. on the

$$\frac{\text{effective height}}{\text{effective thickness}}$$

and not on the normal structural design method of

$$\frac{\text{effective height}}{\text{radius of gyration}}$$

While the concept of effective thickness is adequate for normal walls, and solid rectangular or square columns, we are of the opinion that it is not applicable to such geometric sections as the diaphragm and fin. For such sections, the radius of gyration concept should be used. This is not just another academic or mathematical quibble, but has important structural and economic consequences.

The diaphragm wall shown in Figure 2.5 has a radius of gyration of 170 mm and overall depth of 448 mm.

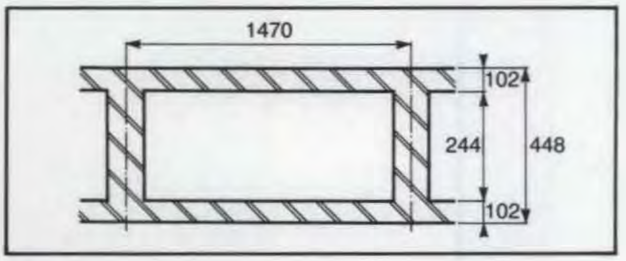

Figure 2.5

A solid wall of the same radius of gyration of 170 mm would have to be 590 mm thick, and we suggest that this 'effective' depth should be used in determining the wall's slenderness ratio – rather than its overall depth.

If the wall has an effective height of 7.62 m, then its S.R. would be:

(i) equivalent depth based on overall depth $\frac{7.62}{448} \times 10^3 = 17.00, \beta = 0.80$

(ii) equivalent depth based on radius of gyration $\frac{7.62}{590} \times 10^3 = 12.91, \beta = 0.91$

ie, there is an increase of almost 14% in design compressive strength.

Since engineers, familiar with prestressed concrete, structural steelwork and modern timber design, rarely use solid rectangular sections – because of their relatively low Z/A and r/A values – but nearly always choose the more structurally efficient I, Tee, and box sections, a similar design choice is likely to be made in prestressed brickwork.

We would suggest that designers consider the above statement, with a view to increasing the efficiency of design of prestressed brickwork geometric sections. For the purposes of this document, we have maintained a conservative approach, and used overall depth values in the example calculations for the fin and diaphragm walls in section 4.

CRACKED & UNCRACKED SECTIONS

Generally, the main purpose of post-tensioning brickwork is to overcome its relatively weak tensile resistance. Plain brickwork, subject to bending, is usually designed to limit the tensile flexural stresses, due to applied loading, to within acceptable limits. Alternatively, tensile limits may be exceeded and the elements designed on the basis of a 'cracked section' analysis (see ref. 1 and ref. 4). There appears to be no fundamental reason why a similar approach, ie. cracked section analysis, should not be used in post-tensioned brickwork design. However, it is considered that tensile stresses, in general, should be limited to zero, at least for the working load condition, thus ensuring that the design section does not 'crack'. Although in special circumstances, eg. very short-term loadings, tensile stresses within appropriate limits are acceptable, provided these can be accommodated by the section, and do not create structural or durability inadequacy.

In this publication, it is considered wise to limit the ultimate loading condition to zero tensile stress, as for the working load condition. This is a conservative approach, but for those interested in more advanced design, we would refer them to reference[16]. The application of a design, adopting the use of a cracked section at ultimate load and zero tension at working load, can produce relatively slim sections. Such a design was used on the Salvation Army Hall at Boscombe, shown on cover.
If research – for which we have been pressing for years – is carried out, then there are likely to be further advances.

DETERMINATION OF POST-TENSIONING FORCE

The first stage in the design of post-tensioned brickwork is to establish the support system, and the applied loads and moments. A trial section is adopted and then analysed, to determine the theoretical flexural tensile stress. Having established the limit to be applied to the tensile stresses – in general, zero tensile stress – the applied direct compressive stress, required to reduce the theoretical flexural tensile stress to the chosen limit, may be determined.The post-tensioning force required is the axial force necessary to produce the requisite levels of applied direct compressive stress, to eliminate the theoretical flexural tensile stress or reduce it to the chosen limit.

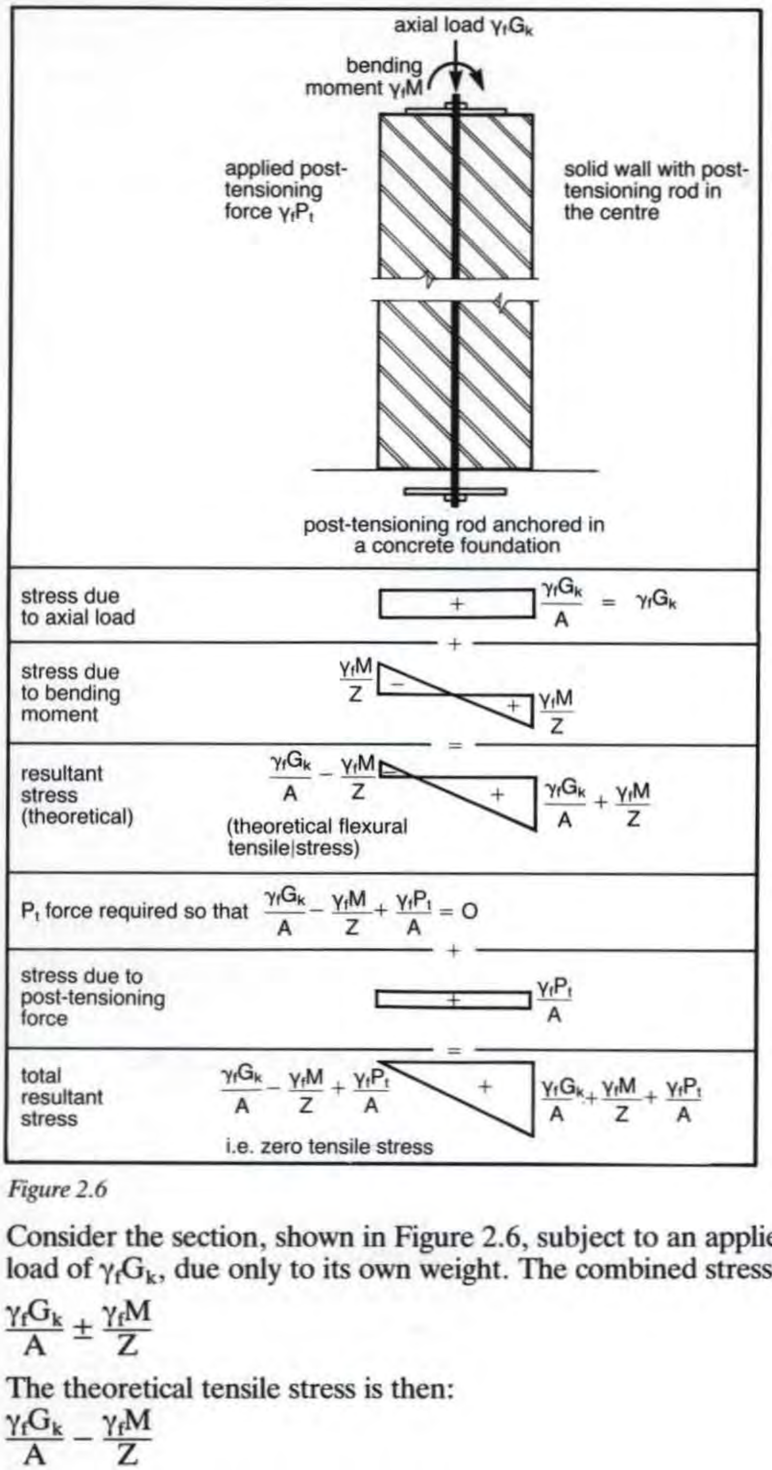

Figure 2.6

Consider the section, shown in Figure 2.6, subject to an applied design moment, $\gamma_f M$, and an axial load of $\gamma_f G_k$, due only to its own weight. The combined stresses at the base of the wall are:

$$\frac{\gamma_f G_k}{A} \pm \frac{\gamma_f M}{Z}$$

The theoretical tensile stress is then:

$$\frac{\gamma_f G_k}{A} - \frac{\gamma_f M}{Z}$$

Thus, if the net tensile stress, under this theoretical condition, is to be limited to zero, a post-tensioning stress greater or equal to the numerical value of the theoretical tensile stress is required:

$$\text{ie, } \frac{\gamma_f P_t}{A} \geqslant \left(\frac{\gamma_f G_k}{A} - \frac{\gamma_f M}{Z}\right)$$

$$\text{or } \gamma_f P_t \geqslant \gamma_f G_k - \gamma_f M . \frac{A}{Z} \qquad \ldots\ldots\ (7)$$

the effect of the post-tensioning force is thus to eliminate the tensile stress, but it should be noted it also adds directly to the total compressive stress, which now equals:

$$\frac{\gamma_f G_k}{A} + \frac{\gamma_f M}{Z} + \frac{\gamma_f P_t}{A}$$

Though this is not always a problem, there are cases where high compressive stresses may not be acceptable and the section may be uneconomic. The solution, then, is to apply the post-tensioning force eccentrically.

ECCENTRIC POST-TENSIONING

In some cases, there are advantages to be obtained by applying the post-tensioning force at some suitable eccentricity, e. For retaining walls, and similar elements with the applied loading acting permanently in one direction, it is generally most economical to provide the maximum design eccentricity to the post-tensioning force. The value of this eccentricity – and the design calculations – is then generally amended by the practicalities of the construction and detailing (see section 3).

A reversible loading, such as wind loading, generally requires an axial post-tensioning force, applied at the centroid of the section. A uniform compressive stress is applied, to counteract the maximum flexural tensile stresses which result from the bending moment exerted on either face of the section, depending on the direction of loading being considered. The magnitude of the flexural tensile stresses occurring at each face will generally be equal. With asymmetric sections, however, such as the fin wall, the respective maximum tensile and compressive stresses occurring in the outer fibres, or face, can vary considerably, under the same value of applied loading, when acting in reverse. This is due to the variation in the section modulus, Z, for each face, see Figure 2.7.

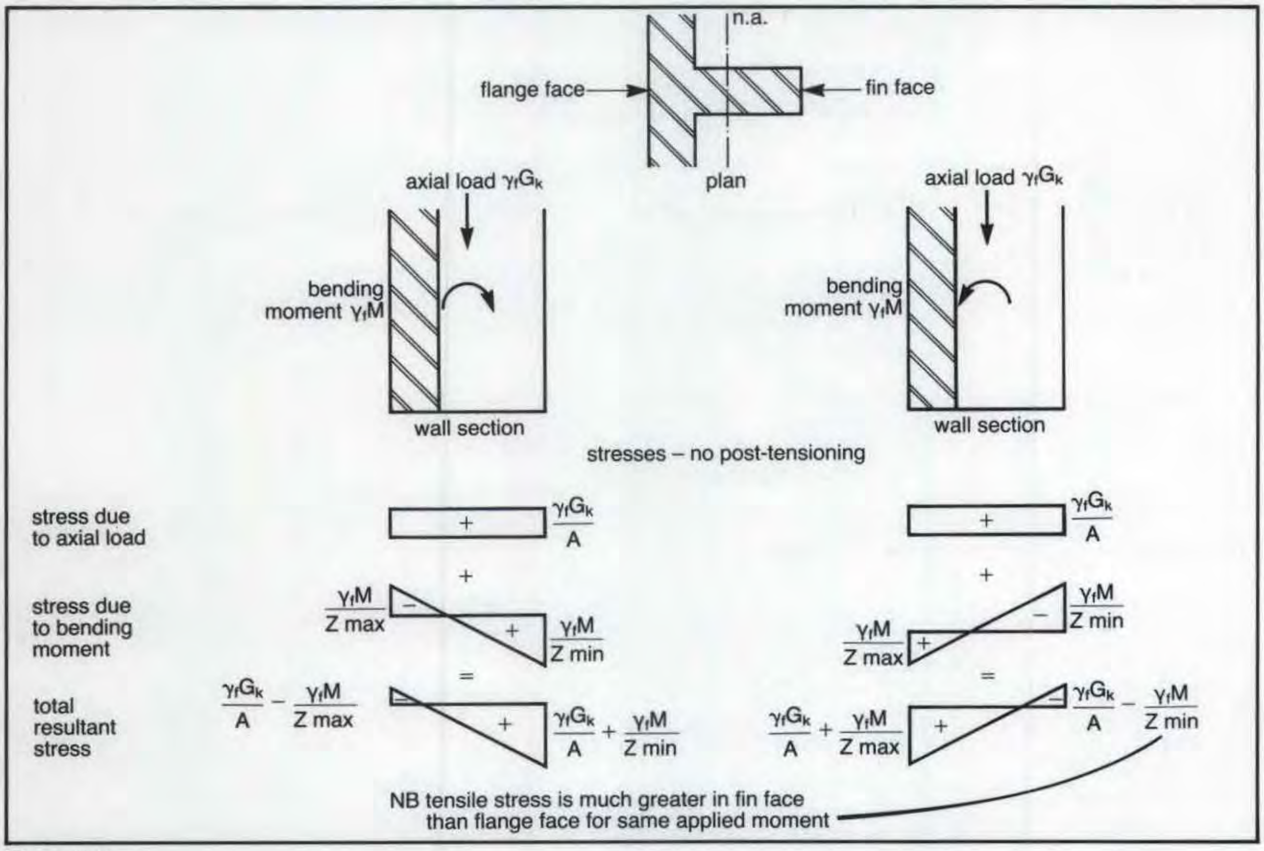

Figure 2.7

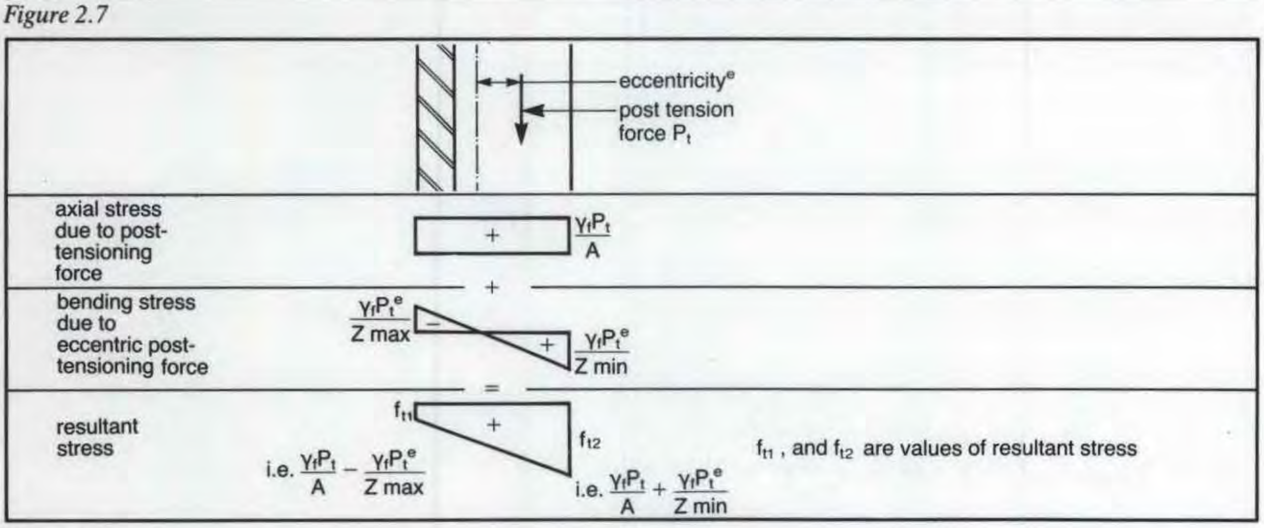

Figure 2.8

A post-tensioning force can be applied eccentrically to add, for instance, a greater compression to the fin face than the flange face, see Figure 2.8

This counteracts the two different values of tensile stress proportionately, see Figure 2.9. The post-tensioning force and the eccentricity may be calculated as follows, referring to Figure 2.8.

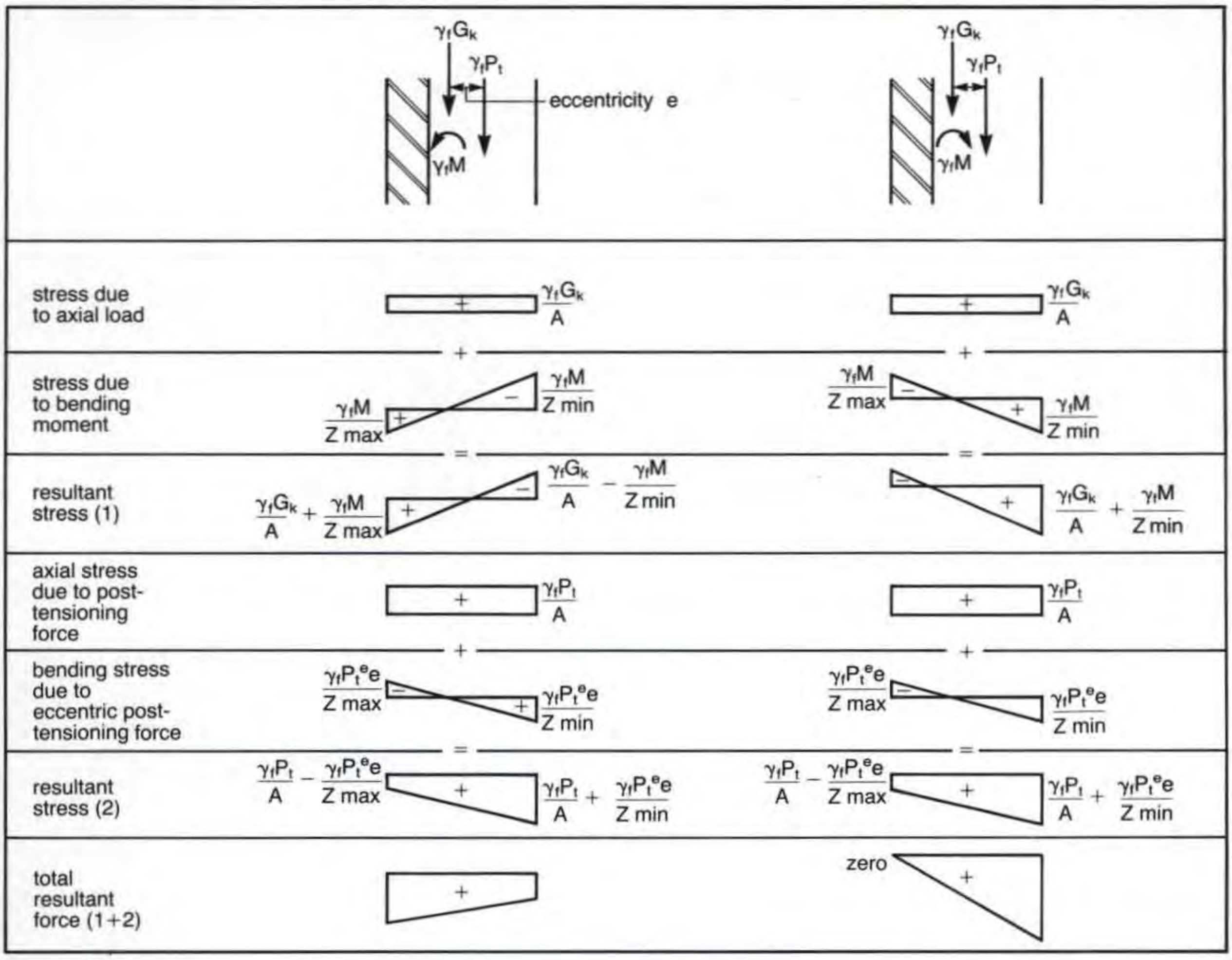

Figure 2.9

Resulting compressive stress in flange face (1):

$$f_{t1} = \frac{\gamma_f P_t}{A} - \frac{\gamma_f P_t e}{Z_{max}} \qquad \ldots\ldots\ (8)$$

Resulting compressive stress in fin face (2):

$$f_{t2} = \frac{\gamma_f P_t}{A} + \frac{\gamma_f P_t e}{Z_{min}} \qquad \ldots\ldots\ (9)$$

In order to balance out the theoretical tensile stresses, resulting from the applied loadings, the compressive stresses, f_{t1} and f_{t2}, should be of the same values, as shown in Figure 2.8. Thus, f_{t1} and f_{t2} are known, as is the cross-sectional area, A, and the section moduli, Z_{max} and Z_{min}. These two equations, 8 & 9, can therefore be solved to determine both the post-tensioning force, P_t, and the eccentricity, e.

Multiply (8) by Z_{max} and (9) by Z_{min}:

$$f_{t1}\, Z_{max} = \frac{\gamma_f P_t}{A} Z_{max} - \gamma_f P_t e \qquad \ldots\ldots\ (10)$$

$$f_{t2}\, Z_{min} = \frac{\gamma_f P_t}{A} Z_{min} + \gamma_f P_t e \qquad \ldots\ldots\ (11)$$

Adding (10) and (11):

$$f_{t1}\, Z_{max} + f_{t2}\, Z_{min} = \frac{\gamma_f P_t Z_{max}}{A} + \frac{\gamma_f P_t Z_{min}}{A}$$

Rearranging

$$\gamma_f P_t = A \left(\frac{f_{t1}\, Z_{max} + f_{t2}\, Z_{min}}{Z_{max} + Z_{min}} \right) \qquad \ldots\ldots\ldots (12)$$

The value of P_t can thus be determined for equation (12) and substituted in either (8) or (9), to find e. Rearranging equations (8) or (9) to find e:

$$e = \left(\frac{\gamma_f P_t}{A} - f_{t1} \right) \frac{Z_{max}}{\gamma_f P_t} \qquad \ldots\ldots\ldots (13)$$

or,

$$e = \left(f_{t2} - \frac{\gamma_f P_t}{A} \right) \frac{Z_{min}}{\gamma_f P_t} \qquad \ldots\ldots\ldots (14)$$

Refer to section 4, example 2, for a worked example of the above theory. Further information and discussion, relating to possible effects on eccentricity calculations due to deflection of the structure during its loaded life, is contained in reference 16. The possible insignificant effect on the applied eccentric force, due to the deflection under load of the structural element, is ignored in this design guide.

CONCENTRIC PRESTRESS

1. COMPRESSIVE STRENGTH LIMITATIONS

If a section is prestressed concentrically, up to its full design compressive strength, there would be no reserve of compressive strength to resist flexural compressive stress. Therefore,

$$\frac{\text{Actual bending stress}}{\text{Design bending stress}} + \frac{\text{Actual direct stress}}{\text{Allowable direct stress}} < 1$$

It can be shown that, for maximum efficiency for this restricted condition, $f_p = f_{ubc} = f_k/2$ (since $f_{ubc} + f_p = f_k$).

A graph T1, Figure 2.10, can be plotted of the lateral force, q, against the prestress, f_p. As q increases, so does the bending moment, M, and thus the bending stress M/Z. When the stress due to prestress is zero – and assuming no design flexural tensile stress and a weightless section – then the section cannot resist bending, and q is zero. As the stress, f_p, increases, so may q be allowed to increase. But, as shown above, when $f_p = f_k$ there is no reserve for bending stress, and again q is zero.

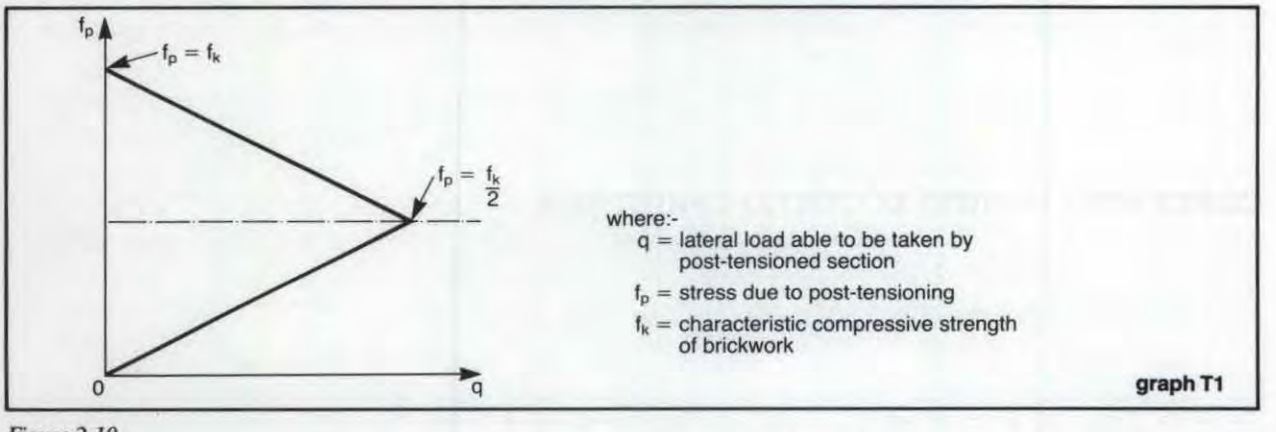

Figure 2.10

Most bricks have a compressive strength varying from 20 – 70 $N/_{mm}^2$, and f_k for a 20 $N/_{mm}^2$ brick in mortar designation (ii) = 6.4 $N/_{mm}^2$, and f_k for a 70 $N/_{mm}^2$ brick in mortar designation (i) = 19.2 $N/_{mm}^2$. Extending graph T1 for these values is shown in graph T2, see Figure 2.11.

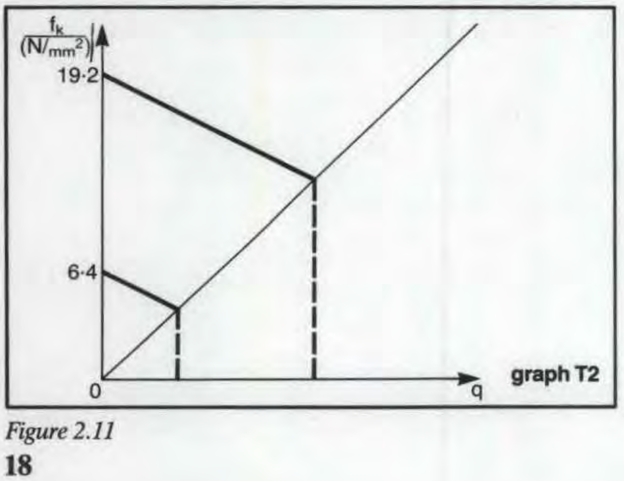

Figure 2.11

This shows that, for this condition, there is a direct proportional relationship between the compressive strength of the brickwork and the structural element's resistance to lateral loading. The graph shows that the higher strength brickwork can withstand nearly three times the lateral load of the lower strength brickwork. The lower strength brickwork must have either an increase in cross-sectional area, or an increase in section modulus, or both, to match the performance of the higher strength brickwork. The choice between the, normally, cheaper low strength brick, with greater section modulus (or area), or the relatively expensive high strength brick, depends on costing the preliminary designs with both types of brickwork.

2. EFFECT OF SLENDERNESS RATIO

(a) The greater the ratio of $\frac{\text{effective height}}{\text{effective thickness}}$ of any structural element, the lower becomes its compressive load resistance. The design compressive strength, f_k, is reduced by a factor, β, which depends upon the above ratio, known as the slenderness ratio, S.R.

When S.R. is 8 or less, $\beta = 1$, and there is no reduction in f_k, and the design compressive strength = $1.0f_k$. When S.R. is 27 (the maximum allowable), $\beta = 0.4$, and the design compressive strength is $0.4f_k$.

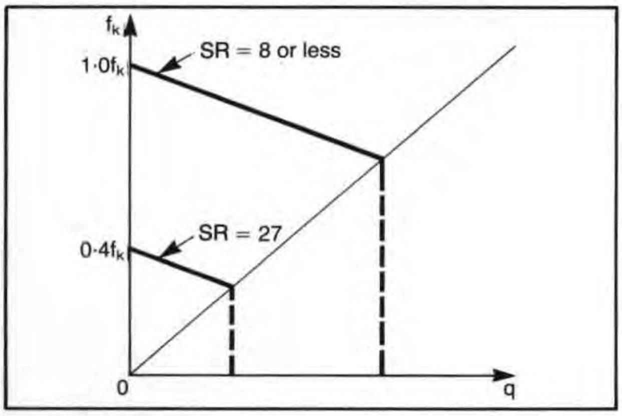

Figure 2.12

It is obvious that a tall slender wall (S.R. = 27) has less than half the resistance of a short wall (S.R. = 8). To reduce the S.R., it is necessary to increase the radius of gyration, which, in turn increases the 'effective thickness' of the section.

(b) The S.R. affects β, which, in turn, affects the design compressive strength, f_k, and that affects prestress, f_p. Cost comparisons of the alternative designs will show that it usually pays to increase the radius of gyration of the section to reduce S.R.

CONCENTRIC VERSUS ECCENTRIC PRESTRESS

For simplicity, the above discussion has been limited to concentric prestress but, as has been shown, it is often more efficient to apply the prestress eccentrically, see figure 2.13.

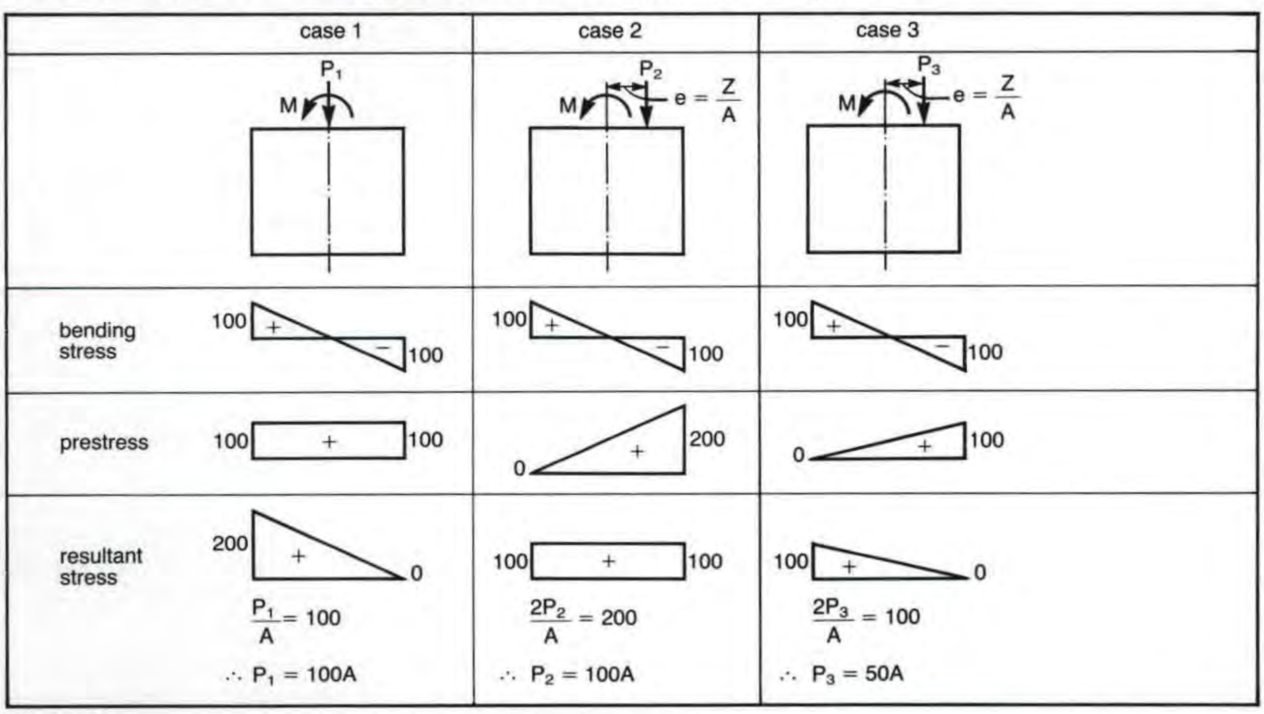

Figure 2.13

Case 1 A concentric prestress results in a non-uniform compressive stress, so that the effective eccentricity of the prestress (reduced) is *eccentric.*

Case 2 An *eccentric,* but high, prestress results in a uniform compressive stress – so that, effectively, the prestress is concentric.

Case 3 An eccentric and efficient prestress – only half the previous cases – resulting in a non-uniform compressive stress.

Case 1 Is inefficent. It not only requires a high prestressing force, but the resultant compressive stress is unnecesarily high.

Case 2 Is better, in that the resultant stress is not high, being only half case 1. But, under prestress alone, the compressive stress is high, as is the prestressing force. The prestressing, say for an earth-retaining structure, could be increased in unison with the placing of the backfill, so that at all times the resultant stress was uniform. Though this could be done, it would prove too labour intensive and introduce complications to the construction process, both of which would have cost implications.

Case 3 Is efficient. The prestress is low, as is the resultant stress.

CHANGES IN THE APPLIED POST-TENSIONING FORCES: LOSSES AND GAINS

The value of the post-tensioning force will vary over the service life of the structure. The changes in the applied post-tensioning force occur during its application, immediately afterwards and over the longer term. These changes, which are generally losses – but sometimes gains – are due to any, or all, of the following:

(a) Elastic deformation of brickwork

(b) Creep of brickwork

(c) Relaxation of the post-tensioning steel

(d) Friction losses

(e) Moisture movement of brickwork

(f) Thermal movement of brickwork, rods and anchorages

(g) Natural 'growth' of clay brickwork

The factors are dealt with, to some extent, in a number of publications, although there is little information on the actual values of each, relative to post-tensioned brickwork. The major loss of prestress is that due to elastic contraction of the brickwork. This loss can be recovered by re-stressing the rods, commonly termed 'topping-up'. Since brickwork is not a truly elastic material, it does not contract immediately on application of the prestress but, like concrete, it gradually 'beds' down or 'creeps'. It is, therefore, preferable to top-up the prestress soon after the initial prestressing.

The value of all other losses, however, relative to the applied forces for preliminary design purposes, is thought, at present, to be of the order of some 20 to 25 per cent. This is a significant amount but, given the order of factors of safety generally associated with plain brickwork design, ie. about 7, not of overwhelming importance. The accuracy required in the determination of the changes in the post-tensioning force, is only within, say, 5% for most applications. The effect of losses, or gains, however, should be considered in each case, since they can be affected by the details and nature of the construction to be adopted.

For most simple applications of post-tensioning to brickwork, as given in this document and from a limited amount of research (see ref. 12 & 17), it is considered that 20 per cent losses, due to all relevant factors, is adequate. This figure has proved satisfactory over a reasonable period of time. The changes in post-tensioning force in particularly sensitive or specialised application should be considered in more detail. In circumstances where net gains in the post-tensioning force are possible, ie. clay brickwork with an overall growth factor which, on very rare occasions, can be greater than all losses, allowance may be considered when assessing the design strengths, etc.

For calcium silicate bricks, with possible high shrinkage movement in combination with low strains in post-tensioning rods, it may not be possible to design an effective post-tensioned brickwork element. The percentage losses in the contraction of the calcium silicate brickwork may be greater than the normal range of strains induced in conventional post-tensioning rods. The experienced designer should, therefore, consider carefully both the level of percentage reductions in applied prestress force, due to shrinkage of this type of brickwork, and possibly consider using a strand type reinforcement, to gain extra strain in the post-tensioned steel. So far, we have no practical experience, nor have we carried out tests in prestressed calcium silicate brickwork.

In BS5628: Part 2: 1985[3], Clause 30.2.1, there is a final sentence regarding design where low levels of strain induced in the rods/tendons may lead to cancellation of induced prestress, by an accumulation of losses. This should not deter the designer from a useful technique which has been found, from experience on hundreds of projects, subject to the most severe weather conditions, to be completely satisfactory.

Where there is a low level of strain, designers should, as always, exercise their judgement in increasing the initial prestress, and thus the strain, to allow for the losses mentioned in the Code. To account for the losses in the post-tensioning force, an increased force is applied which will, after losses, give the value required. It is, generally, 1.25 $\gamma_f P_t$, where $\gamma_f P_t$ is the actual design force, calculated from equations (7) or (12).

PARTIAL FACTORS OF SAFETY ON POST-TENSIONING

The partial factor of safety, γ_f, is applicable to loadings, moments and post-tensioning forces, in deriving the various design equations. The numerical values for the partial factors of safety on dead, imposed and wind loadings, are all as used for plain brickwork, and are given in BS5628: Part 1[2] and Appendix A.

The partial factor of safety on loads, γ_f, is applicable to some degree to the post-tensioning force. It appears that no research has yet been carried out, to determine statistically the appropriate γ_f values, and the designer must assess the likely variables, to determine a suitable figure. In the absence of other guidance, we would suggest that the partial factors of safety applicable to dead loading, as given in BS5628: Part 1[2] and Appendix A, should be used.

DESIGN STRENGTH OF POST-TENSIONING RODS

The post-tensioning rods are generally of high yield steel to BS4449, being grade 460. The characteristic strengths, f_y, and the relevant partial factor of safety, γ_{ms}, are given in Appendix A. The steel is generally stressed to its design level, subject, of course, to slight variation due to the changes, as discussed previously, throughout its service life. (Unlike most reinforcement, which is stressed to the maximum design value only when the full dead and imposed loading is applied – a situation which, in many cases, only occurs infrequently, and for short periods, if at all.)

In view of this, and in order to limit the relaxation of the steel, the stress in the post-tensioning rods is generally limited to 70 per cent of the maximum design value. Thus, the design strength of the rods is given as follows:

$$\text{Design strength of steel} = \frac{0.7f_y}{\gamma_{ms}} \qquad \ldots\ldots\ (15)$$

DESIGN OF POST-TENSIONING RODS

Knowing the magnitude of the post-tensioning force to be applied, and the design strength of the rods, the required area of the rods can be determined as follows:
Design post-tensioning force $\leqslant$ Area of rods $\times$ design strength of rods

$$\text{ie. } 1.25\,\gamma_f P_t \leqslant A_s \times \frac{0.7f_y}{\gamma_{ms}}$$

$$A_s \text{ required} \geqslant \frac{1.25\,\gamma_f P_t\,\gamma_{ms}}{0.7f_y} \qquad \ldots\ldots\ (16)\ \text{(Assuming 20\% losses)}$$

This area of steel required is the actual area, ie. the net tensile area of the rod. As post-tensioning rods are generally threaded, the reduction in the area must be allowed for. (Refer to section 4, Example 1, for worked example of the above theory.)

It is possible to vary the number of rods, or the area of the rods, or both, in a given situation, and the most suitable, practical and economic combination must be chosen by the designer-detailer. The spacing of the rods is often governed by such practical considerations as accommodating the required bearing plates, or positioning the rods within voids in the brickwork construction. The size of rods is also governed by practical considerations (see section 3).

APPLICATION OF THE POST-TENSIONING FORCE

The compression within the brickwork, as a result of the post-tensioning, is induced by action of a rod, or strand, in tension on anchorages at each end. One end is usually fixed and the other adjustable. There are two main methods of applying the post-tensioning force to the brickwork section. Both involve the stretching of the steel rod, or strand – the difference being in the system used to apply the force:

1. By means of jacking – usually used for high levels of prestress.

2. By means of a torque wrench – usually used for relatively lower levels of prestress.

The jacking system allows a direct reading of the post-tensioning force on the dial gauge, mounted on the hydraulic pump which applies pressure to the jack. The rod, or strand, is locked into the jaws of the jack by means of wedges. When the required predetermined tensile force has been applied, cross-checked by measurement of the extension of the rod, or strand, the lock nut is tightened against the anchorage plate, fixing the rod/strand at the required extension/force. Each jack and pump is calibrated, to ensure accurate results in application of the post-tensioning force.

5

6

5 & 6. Barnton Library & Handforth Library. *These projects demonstrate another option of the SCD system. Both have a steel frame set well inside the building, with the roof cantilevered out. Using post-tensioned brickwork, one is freed from the restraints of the perimeter frame. In both these projects, the fenestration is precisely geared to the needs of the internal layout. Architect: Cheshire County Architects Department. Consulting engineer: Curtins*

The torque wrench system stretches the rod by turning a nut on the threaded adjustable end, against an anchorage plate. This extends the rod, which produces a tensile stress within it. The required tension within the rod is known, and is related to the amount of tightening on the nut, ie. the torque which is the turning moment required to produce the requisite strain in the rod. The torque value required is dependent on the amount of prestress required, and a number of other factors. Of these, the most significant are the type and pitch of the thread and the friction developed between the contact surface of nut, bolt and spreader plates, etc. The advice of the manufacturer of the equipment used to torque the rods should be sought as to the relationship between bolt tension and torque.

The method of determining torque, given in equation 17, is based on a general engineering formula derived from test research. It involves using lightly oiled metric threads, with self-finish nuts and bolts and hardened washer between the nut and the spreader plate.

$$\text{Torque required} = \frac{\text{rod tension} \times \text{rod diameter}}{5}$$

$$= \frac{1.25\ \gamma_f P_t \times d}{5} \qquad \ldots\ldots\ (17).$$

Torque values are usually expressed in $k_g f.m$ units, whereas the post-tensioning force is usually expressed in kN units –

$$1kN = \frac{10^3}{9.81},\ k_g f = 102\ k_g f.$$

Since the actual force produced in the rod is dependant on many variables, the engineer should satisfy himself, from experience or testing, that the finally achieved force is in compliance with his requirements.

High tensile steel rods are commonly used in preference to wire strands, for practical reasons. Wire strands have the advantage of being able to form a curved profile, to suit moment and shear considerations, and can make for a more structurally efficient design. Whilst such designs are perfectly feasible, experience has shown, so far, that such designs are neither cost-effective nor possess good buildability.

The design of the upper anchorage will normally require a concrete capping beam to be placed between the lock nut and steel anchorage plate and the brickwork section being prestressed. The thickness of this capping beam and the reinforcement required within it, should be in accordance with current codes of practice appertaining to prestressed concrete. The bursting stress, f_{bst}, produced by the wedging action of the rod/tendon down through the concrete beam, must be catered for in the appropriate manner, as described in the Code.

Should a detail require that the prestressing force be applied directly through a steel anchorage plate to the brickwork section, then the thickness of the plate should be checked. This will minimise the likelihood of bending within the plate, and hence inducing of additional stresses, due to curvature of the plate, to those already calculated, ie. local bearing stress and maximum combined compressive stress.

CHECKING OF FLEXURAL AND DIRECT STRESS: ALL LOAD CASES

The magnitude of the post-tensioning force is based, generally, on the condition of 'no tensile stress' in the section, under the worst loading combination and after losses have occurred. The tensile stresses are, therefore, catered for. The combined flexural compressive stresses should then be checked for the various combinations of loading after losses, against the limitations given in equations (4), (5) and (6).

The direct compressive stresses should be checked for the various combinations of dead load, imposed load and the post-tensioning force, before losses, against the limitations given in equations (1), (2) and (3).

VERTICAL AND HORIZONTAL SHEAR STRESS

Having calculated the post-tensioning force, and checked the direct and flexural stresses, the shear stresses should be considered. Generally, the design of post-tensioned brickwork in shear is treated in the same way as plain brickwork. The post-tensioning force adds to the applied axial loadings on the section, and affects the design shear strength, but not the applied shear load.

The expression for shear stress, v_h, is as follows:

$$v_h = \frac{VA\bar{y}}{Ib} \qquad \ldots\ldots\ (18)$$

v_h = shear stress
V = horizontal or vertical shear force
A = area of cross-section to one side of position where shear stress is being checked
$\bar{y}$ = distance from neutral axis to centroid of area
I = moment of inertia (second moment of Area)
b = width at point being checked

However BS5628: Part 2[3] follows the same design procedure as that for concrete sections in BS8110[4], assuming $v_h = V/bd_c$ where b = the width of the web section for rectangular sections, or the width of the web for T or I sections, and d_c = the depth of masonry in compression. This gives a lower calculated shear stress for geometric sections than that from equation 18 and, whilst it has been found appropriate for concrete design, we would recommend the higher value in brickwork design.

As always, engineers should exercise their judgement in accepting advice.

The design shear strength is equal to the characteristic shear strength, f_v, divided by the partial factor of safety for the material strength in shear, γ_{mv}. The characteristic horizontal shear strength is given a value in BS5628: Part 1[2] as:

$f_v = 0.35 + 0.6g_A$, with a maximum value of 1.75N/mm^2 .... (19)

(mortar designations (i) and (ii), and where g_A is the design vertical load in the wall). The value of $_{mv}$ given in BS5628: Part 1[2] is 2.5, but in Part 2[3] it is 2.0. Refer to section 4, Example 3, for a worked example of the above theory, for use in comparisons with maximum shear, calculated for geometric section, as against average shear values.

In post-tensioned brickwork, it is advisable that only mortar designations (i) and (ii) should be used, and we are of the opinion that a maximum value of $1.75\ \text{N/mm}^2$ may be reasonable for plain brickwork, but not necessarily so for prestressed brickwork. The post-tensioning force may be included in the design vertical load, g_A, and can thus increase the horizontal shear strength. In brickwork, sections designed for flexure, on the basis of an uncracked section, the horizontal shear resistance may be taken as occurring over the whole plan cross-sectional area. When the cracked

7 & 8. Congelton Library. *Anxious to avoid an uninteresting rectangular box structure, the designers required the ground floor brick walls to be set back, well inside the perimeter frame. As clerestorey lights were required at this level, the walls are unrestrained cantilevers and, therefore, post-tensioned to resist wind loads. The first floor brickwork is not post-tensioned, being treated as panels spanning between rc columns. Architect: Cheshire County Architects Department. Consulting engineer: Curtins* section analysis is applicable, *only* the uncracked section of brickwork will provide resistance to horizontal shear. It should be noted, however, that the vertical stress on this reduced area will be increased considerably, and thus the horizontal shear resistance will be maintained at a comparably high level.

There is no guidance on analysis of shear in cracked sections in BS 5628: Part 2[3]. Generally, only horizontal shear on solid rectangular uncracked sections is dealt with, and designers working outside this narrow restriction should assess the shear resistance, in each case, on the basis of sound engineering principles.

Vertical axial loading is not recognised, in the Code, as providing any improvement in the vertical shear strength in brickwork – nor is there any guidance on the latter given. In the more complex sections, however, such as fins and diaphragms, it seems likely that vertical loading, particularly when this is due to a post-tensioning force, will improve to some extent, and also, within a limited range, the vertical shear resistance. For the present, we recommend the adoption of the values from BS 5628: Part 1[2], for average horizontal shear resistance, as being also the maximum shear resistance values for geometric sections in any plane.

In certain instances, horizontal and vertical shear stress can govern the design, eg. heavily loaded retaining walls, when the post-tensioning force may need to be increased solely to improve the horizontal shear resistance of the section.

PRINCIPAL TENSILE STRESS

Brickwork is a combination of bricks and mortar, which are bonded together in traditional patterns. The methods of combining the elements, together with the shape of the units, results in different structural characteristics when loaded in different directions, so that the material is very anisotropic, and more so than concrete. For example, its flexural tensile strength, when bending about an axis perpendicular to the bed joints, is between two and three times that when bending about the axis parallel to the bed joints. Its direct tensile strength is lower than its flexural strength, and differs parallel and perpendicular to the bed joints; also the horizontal, vertical, and perpendicular shear strengths differ.

Analysis of sections is thus, necesarily, more complicated than when dealing with a homogeneous material such as structural steel. Although even when using structural steel in shapes such as I-beams, etc, the analysis can be more complex than using simple rectangular shapes. In addition, when considering the analysis of more complex geometric shapes, such as I-sections (e.g. in brick diaphragm walls) even in homogeneous materials, the analysis must be considered in greater detail, due to the possibility of combined stress levels greater than the individual stress values. In brickwork, due to its inherent nature and, more particularly, where geometric shapes other than the simple rectangular section are used, a more detailed analysis of stresses is often required.

Brickwork design generally requires the applied maximum tensile and compressive stresses to be compared with the relevant respective material strengths, as well as the applied maximum shear stress. Additionally, particularly with the more complex geometric shapes, the principal stresses should be calculated and compared with the relevant material strength. The derivation of principal stresses is given in various text books on the strength of materials[14].

For a section in bending, subject to a normal stress, σ_x, and a shear stress, τ_{xy}, acting together, a principal stress may be produced which is numerically larger than the stress at the extreme edges of the section. The principal stresses are given by the following expressions[14]:
principal compressive stress

$$\sigma \max = \frac{\sigma x}{2} \sqrt{\frac{\sigma x^2}{2} + \tau_{xy}{}^2} \quad \ldots\ldots. (20)$$

principal tensile stress

$$\sigma \min = \frac{\sigma x}{2} \sqrt{\frac{\sigma x^2}{2} + \tau_{xy}{}^2} \quad \ldots\ldots. (21)$$

maximum shearing stress

$$\tau \max = \frac{\sigma \max - \sigma \min}{2} = \sqrt{\frac{\sigma x^2}{2} + \tau_{xy}{}^2} \quad \ldots\ldots. (22)$$

The values of σ_x will be appropriate values, as obtained for the point being analysed, from consideration of the applied moments and axial forces with the section properties, eg.

$$\left(\frac{P}{A} \pm \frac{M}{Z} \pm \frac{Pe}{Z}\right)$$

The shear stress, τ_{xy}, will be as calculated from equation (18) for the corresponding section being considered. See Appendix E for Principal Stress Analysis by 'Mohr's Circle'.

DESIGN PRINCIPAL TENSILE STRENGTH

In many cases, it will be the principal tensile stress which will require checking. However, the determination of the design principal tensile strength, to compare with the design applied stress, is not straightforward. There is little research information available on this phenomena in brickwork, and no guidance whatsoever in the codes of practice. Designers must, therefore, assess a suitable value on the basis of their engineering judgement. We consider that – admittedly from limited research experience – the principal tensile strength is, in general, likely to be half the shear strength, f_v.

Designers are advised to check the principal tensile stress in heavily prestressed geometric sections, but, due to the high partial safety factors, there is rarely a serious design problem. Those that may arise, can be solved by thickening the section in the area of high principal stress.

SHEAR LAG

With geometric sections, such as diaphragms and fins, the tensile and compressive stresses in the flanges, arising during flexure, result in a shear transfer between the ribs and the flanges. In design, the stress is transferred from the rib across the full width of the flange, at the rib/flange interface, so that the stress in the flange is uniform for the full width. In pure theory, however, the stress appears to be at a maximum across the width of the rib, reducing to a minimum value within the section of the flange between the ribs (see Figure 2.14). This difference in stress level is termed 'shear lag' and is caused by shear deflection. It has been observed in some steel box girders and other thin walled structural elements.

The result of the shear lag in such sections, can be an increase in the flexural stresses and deflection. The magnitude of the shear lag effect appears to be related to the ratio of span to depth and span to rib spacing – the shorter the span the greater the effect of shear lag. It seems, that for relatively short spans, the increase in the flexural compressive stress could be as much as 25%.

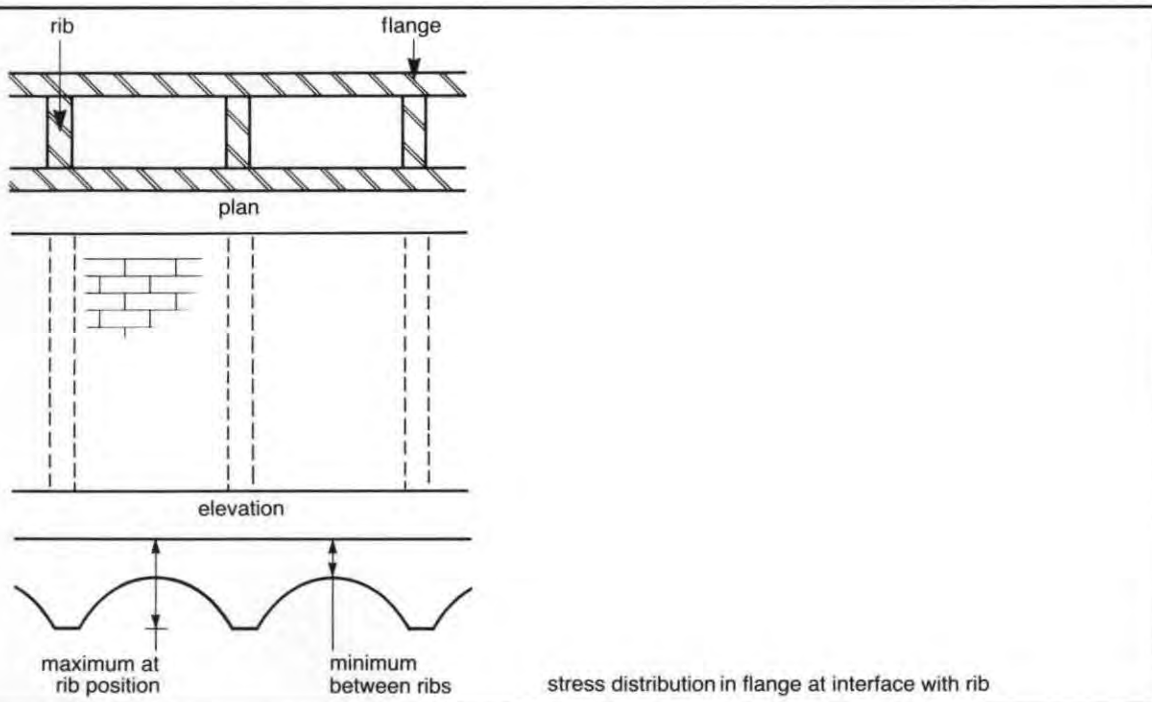

Figure 2.14

However, shear lag is a localised effect and, away from the rib/flange interface, the stresses will redistribute to more equal levels. In brickwork design, increases of up to 50% are considered appropriate for localised compressive stress effects so, in most cases, it is considered that the shear lag effect may be discounted in any analysis of stresses. (In recent research on prestressed diaphragm walls[12], no shear lag effect was detected.)

In particular instances of very high stresses, approaching ultimate stresses, and/or where the span to depth and span to rib spacing ratios are comparatively low, and/or where tensile stresses may be critically affected, then the shear lag should be considered and stress levels adjusted appropriately. It should be noted that the critical effect of shear lag is restricted by the normal considerations of maximum flange widths which are relatively 'thick', embraced within the design guides for geometric shapes, so that additional analysis will rarely be required.

ELASTIC MODULUS 'E' VALUES FOR BRICKWORK

Brickwork is not a truly elastic material. Its stress/strain relationship is similar to concrete, in that the graph is approximately parabolic and not straight. Nevertheless, within the design strength range it may be considered to be 'elastic'. The elastic modulus E_m, depends on the brick strength, mortar strength and their combination, as for the characteristic compressive strength, f_k. Tests have shown that E_m for clay brickwork varies from about 600 – 1000f_k N/mm^2, depending on the mix of brick and mortar strengths. BS 5628 recommends that a value of the short term E_m of $0.9f_k$ kN/mm^2 may be used, and half that for the long term, $E_m = 0.45f_k$ kN/mm^2.

Designers may use this value in preliminary assessment of elastic contraction, but are advised to check on site when re-stressing.

DEFLECTION

So far, no problem with excessive deflection has occurred with post-tensioned brickwork. But, designers are advised to check that the deflection does not exceed the values given in BS 5628: Part 2[3] as follows:

(i) length/125 for cantilevers
(ii) span/250 for all other elements
(iii) span/500 or 20mm, whichever is the lesser, on the rare occasions when partitions or finishes may be affected after their construction.

Calculations for deflections are based on:

$$\Delta = \frac{KWL}{EI} \text{ where,}$$

Δ = deflection — K = factor depending on load distribution
W = load — L = span
E = Elastic modulus — I = second moment of area

It is often difficult to determine W accurately. The span, L, is affected by the end conditions, and thus tends to be an estimate. There may be uncertainty about the exact value of E, and it is common to ignore the rods in determining I. I is constant about its centre of gravity which, in normal bending, coincides with the section's neutral axis. Under prestress, the neutral axis does not coincide with the centre of gravity, so that I, about the neutral axis, is a variable. As every experienced designer knows, in view of these variables, it is highly unlikely that the calculated and actual deflection will equate. Furthermore, when the post-tensioning force is applied eccentrically, there will be a reverse camber. As with other structural materials, the actual deflection is commonly less than that calculated.

9

9. Corwen Emergency Services Centre. *The architects wanted brick cladding to the slender columns between garage doors, and also wanted to avoid internal projections. Post-tensioned cavity brickwork provided a functionally superior and a much simpler and cheaper alternative to the more obvious choice of steel or rc columns. Architect: Clwyd County Architects Dept. Consulting engineer: Curtins*

3 DETAILING & WORK ON SITE

It is important that the design intentions are transmitted into constructional details which are buildable, durable and economic. The detailer should not only fully understand the principles involved, but also the effects on the design of poor detailing. Two fundamental aims must be paramount in the detailer's mind.

(i) to transfer correctly the required post-tensioning force in the appropriate position to the correct area of brickwork;

(ii) to ensure that the post-tensioning system is protected from corrosion, so that no loss of design strength or unacceptable cracking of brickwork results.

In both of these aims, the detailer must also consider the operatives who will carry out the construction of the details, and aim to make their jobs as simple as possible.

The basic procedure, generally adopted for post-tensioning brickwork, is to anchor one end of a high-tensile steel rod, apply any additional corrosion protection, and then build the brickwork section around it. The construction must ensure that a gap is maintained between the rod and the brickwork, to prevent friction losses of the post-tensioning force. At the top of the brickwork, it is usual to form the second anchorage position for the post-tensioning rod. The rod is then tensioned by means of screwing a nut onto the threaded end, and tightening by a torque wrench or jacking against the spreader plate placed on the second anchorage. See Figure 3.1.

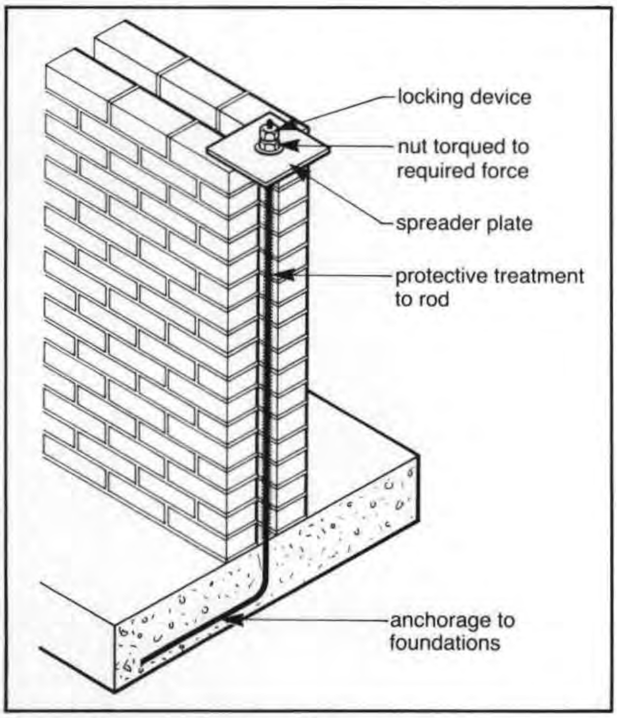

Figure 3.1 Post tensioned cantilever cavity wall

Whilst the process seems simple enough, there are some pitfalls which can be encountered. It is, therefore, intended to give some pointers in the following paragraphs to assist in avoiding, or overcoming, the more common ones.

STEEL TENDON/ROD CONSIDERATIONS

There are two types of tendon used in prestressing, wire strands and steel bars; both of which should conform to the requirements of either BS 4486 or BS 5896. The use of mild steel rods should be avoided, due to the excessive loss of prestress resulting from creep in the steel. For most brickwork elements, particularly vertical ones, it is more practical, both in terms of construction and of post-tensioning, to use high-tensile steel rods in preference to wire strands, since the rods are more rigid and need less temporary support.

The location of the post-tensioning rod and detailing of the brickwork section is governed by a number of important design requirements, the main ones being:

(a) the optimum position for the post-tensioning force to provide the largest resistance to applied loading

(b) the practical physical dimensions and bonding arrangements around the rod location

(c) the required cover to the rod, in order to achieve adequate protection against corrosion.

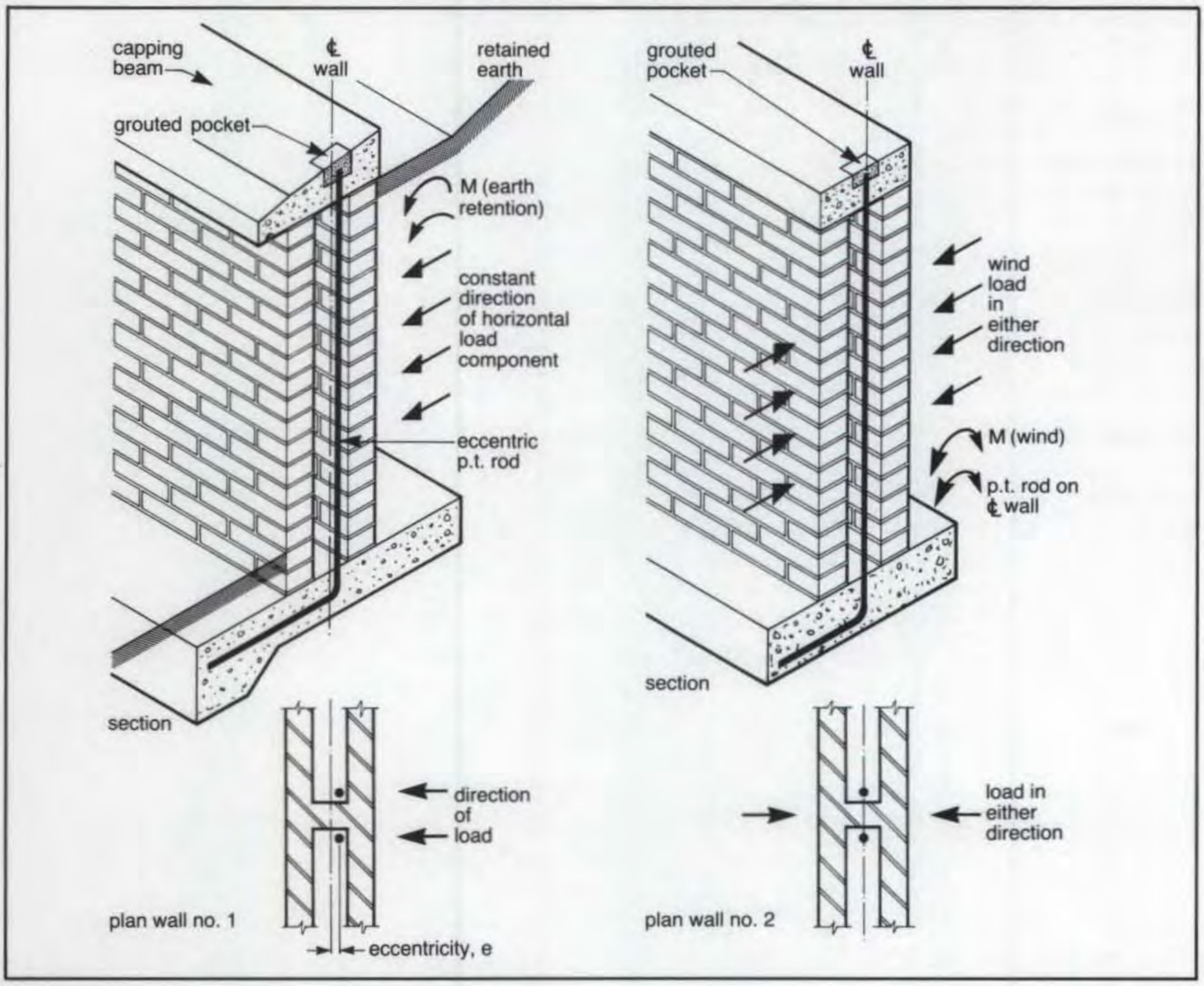

Figure 3.2

Requirement (a) is determined from the type and direction of the applied loading. In Figure 3.2, the cantilever wall No 1 is designed to resist lateral forces in one direction only, from the earth pressure and surcharge at the rear of the retaining wall. For this design, the optimum location for the post-tensioning rod is eccentric to the centroid of the wall section, so as to cancel out part of the applied moment.

The cantilever wall No 2 is subjected to wind load in both directions, ie. pressure and suction of approximately equal intensity. The optimum location of the post-tensioning rod is, therefore, central to the symmetrical wall section shown, so as to give the maximum balance of resistance under each loading.

Various locations for post-tensioning rods, to fulfil requirement (b), are available in a wide variety of cavities, formed by different types of bonding, and some examples are shown in figure 3.3. The method of protecting the steel rod will, in some cases, dictate the size of the pocket required. This is dealt with in more detail in the Durability section of this design guide.

CURTAILMENT & EXTENSION

As well as ensuring the design requirements are translated into practice, it is important to take account of the physical dimensions applicable to the detail. As with all forms of construction, detailers should put themselves in the place of the tradesmen, whose task it is to build the structure

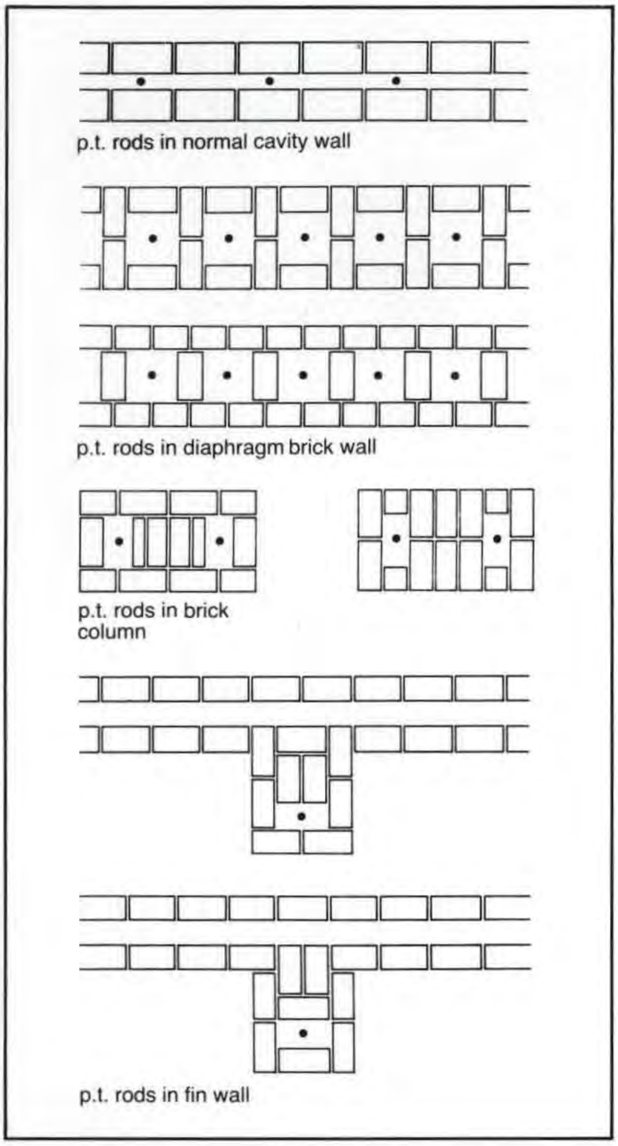

Figure 3.3

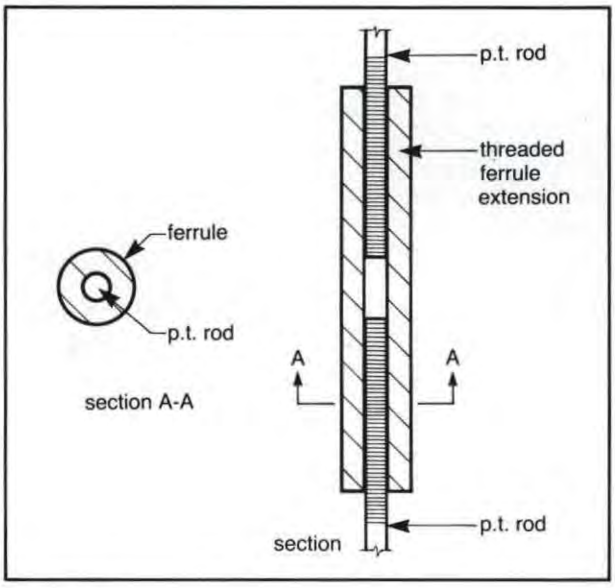

Figure 3.4

in accordance with the detail and to satisfy the design requirements. It is important that the bricklayer's job is not made more difficult by impractical details, such as numerous unwieldy steel rods, located in difficult bonding arrangements. The simpler the detail is to build, the more likely it is that the design standards will be met on site.

Table 3.1 is a guide to the detailer, to indicate typical examples of maximum practical limits for projection of post-tensioning rods in various construction conditions.

Condition	**Rod location**	**Diameter of Rod (mm)**	**Recommended maximum projection length (m)**
Vertical wall or column, normal access	Foundation starter bars	12 16 20 25	1.5 1.5 2.0 2.0
Vertical wall or column, normal access	Upper lift of main bars	12 16 20 25	2.0 2.0 3.0 3.0
Vertical wall or column, good access and temporary support to rods	Foundation starter bars	12 16 20 25	2.5 2.5 3.5 3.5
Vertical wall or column, good access and temporary support to rods	Upper lift of main bars	12 16 20 25	4.0 4.0 4.5 4.5

Should these lengths, or those thought to be practical by the detailer, need to be exceeded, then the extension of the rod can be achieved using a threaded ferrule, as shown in Figure 3.4.

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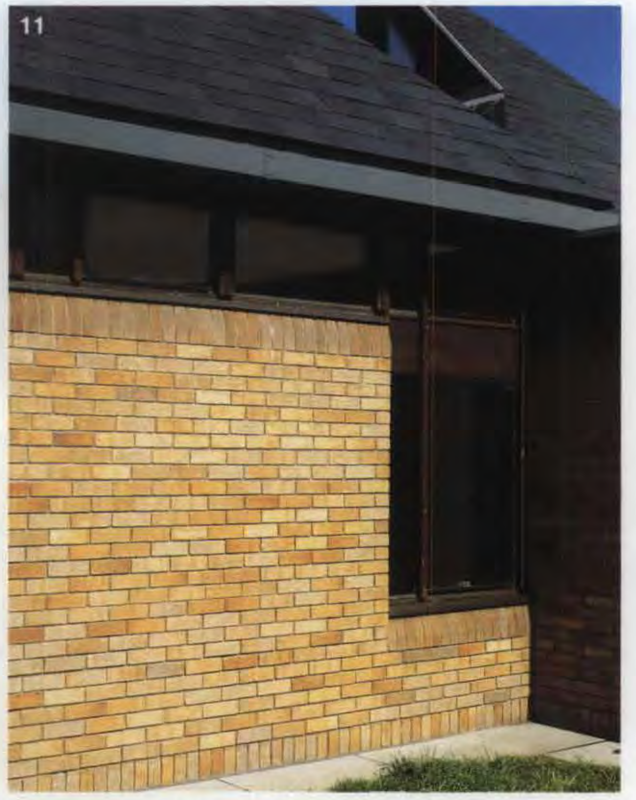


12

10 & 11. Rhyl Parish Centre. *The roof is supported by a common steel column, at the central point of intersection, and by steel columns located at the corners of the four units. Bearing in mind its use as a church, clerestorey lighting was felt to be a functionally appropriate solution to the problem of providing natural light. Which meant, of course, that the perimeter walls would become unrestrained cantilevers and vulnerable to wind loads. Post-tensioning provided the answer. Architect: Weightman & Bullen. Consulting engineer: Curtins*

12. Oak Tree Lane Community Centre, Mansfield. *A project, that recognised the enormous potential of post-tensioned diaphragm wall buildings. It was built in an area subject to mining subsidence. Post-tensioned rods were introduced into the walls and anchored into the raft foundation (see diagram) – the effect of the torqued walls was to cause the raft foundation and the diaphragm walls to act compositely, thus providing greater stiffness and much increased resistance to the induced tensile stresses. Architect: F. R. Walters, District Architect, Mansfield District Council. Job architect: Jenny Bishop. Consulting engineer: Curtins*

APPLYING THE FORCE

As a general rule, the post-tensioning force must not be applied until the mortar has reached the design strength. Table 3.2 gives guidance on the time to be allowed for different conditions.

Mortar designation	Curing conditions	Curing period	Post-tensioning
(i) (ii)	Winter	14 days + number of days below specified minimum temperature	P.T. after curing re-stress (top up) 14 days later
	Summer	14 days	ditto

The more common method of applying relatively light prestressing force is by tightening a nut at the top of the rod, using a torque wrench and multiplier, against a steel plate anchored into, or on to, a concrete pad or capping beam. See Figure 3.5.

In order to ensure that the design post-tensioning force is actually applied in practice, using this detail, a number of points are critical. These are, the cleanliness and condition of the threads, the type of thread, the degree of lubrication and the freedom of the nut to slip. All effect the final tension produced in the rod from the application of the specified torque. In addition, corrosion could affect the long term stresses in the rod.

The following should be specified:

(a) the thread to be used;

(b) cleaning and lubrication of the threads;

(c) method of application of the force;

(d) method of locking the anchorage nut into position;

(e) any periodic checks or tightening, to adjust for creep or other losses;

(f) the need to keep mortar, brick chippings and other debris away from the rod and its void;

(g) protection treatment against corrosion.

Based on past experience, it is important to provide protective covering to the threaded rod, prior to applying the torque.

To ensure that the nut passes freely over the threads, it should be specified that, prior to applying the force, the nut should be run up and down, at least once, to a level below which it will finally rest. Lubrication of nut and threads should then follow. This will need to be done before construction of the upper anchorage, since the lower threads would be below the level of the anchor plate, and hence inaccessible.

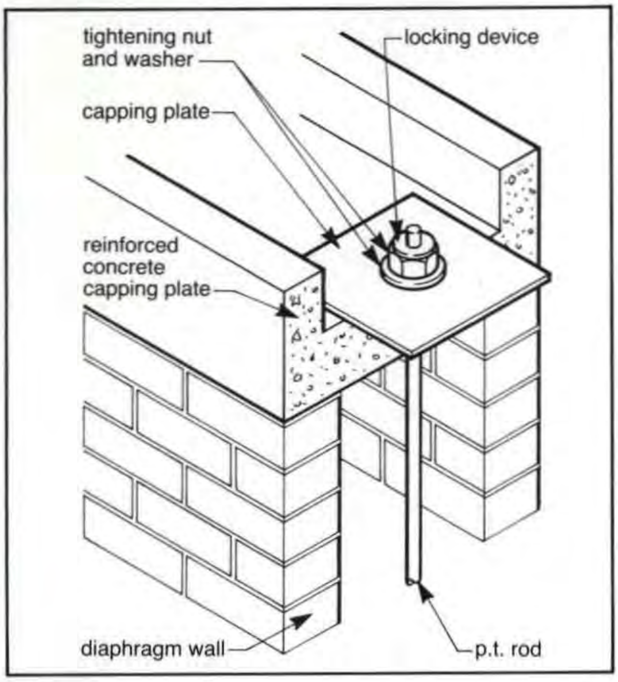


Figure 3.5

The detail at the head anchorage is critical, to ensure efficient transfer or torque to the post-tensioning force in the rod. In addition, a number of other points within the total system must be carefully detailed, to avoid excessive losses. Figure 3.6 illustrates several key areas where losses may occur, if not properly detailed.

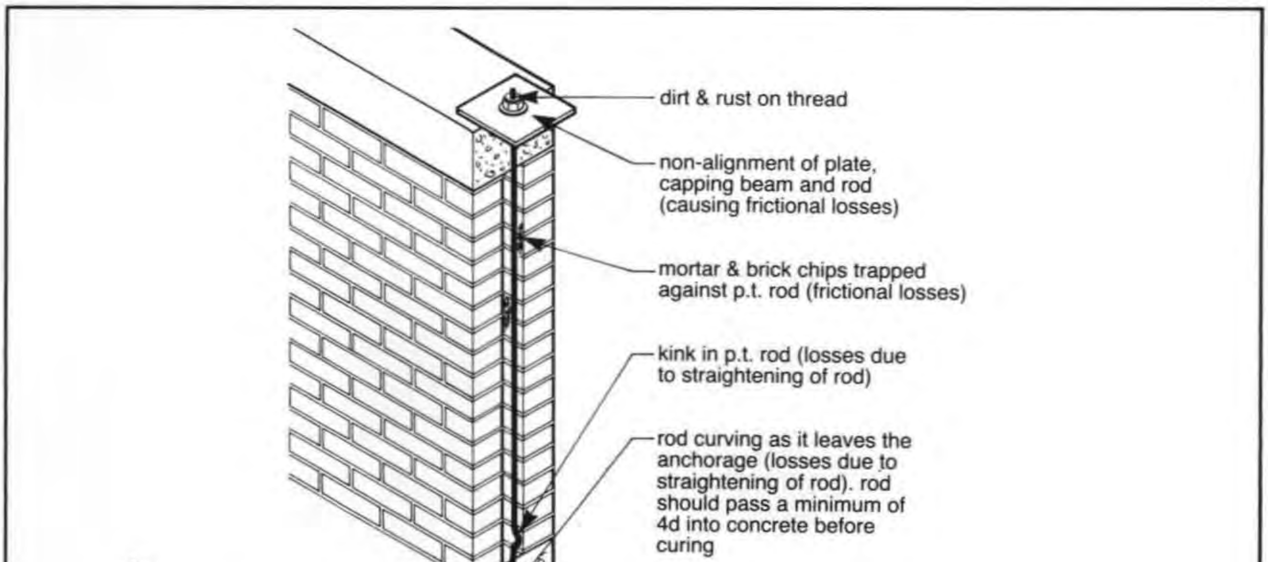


Figure 3.6

DURABILITY

When it is intended to adopt post-tensioning brickwork as a structural element, prevention of corrosion of the steel rods is one of the main considerations. Poor detailing can lead to inadequate life of the structural element, due to corrosion of the rod, and possible cracking of the surrounding concrete/brickwork, and ultimate loss of prestress.

The following are possible protection treatments for high tensile steelwork within the brickwork element:

(a) use waterproof paint and wrapping around the rod;

(b) use galvanised or stainless steel for all steelwork elements;

(c) grout the voids/ducts around the rod and anchorage detail, with an adequate cover of dense, compacted grout or concrete.
(Grouting must only be carried out after application of the prestress.)

As with reinforced brickwork detailing, it is essential to adopt the most suitable form of corrosion protection for the conditions and location of the element in question. Each of the above methods, or a combination of them, could provide adequate protection to high tensile steel, given that the detailing is compatible with the method chosen. The following series of Figures (3.7 – 3.9) give examples of protection and outline the details which are required to make them work. It is obviously not possible to cover every situation and, of course, the economics of adopting certain details may appeal to some designers but may not be desirable to others.

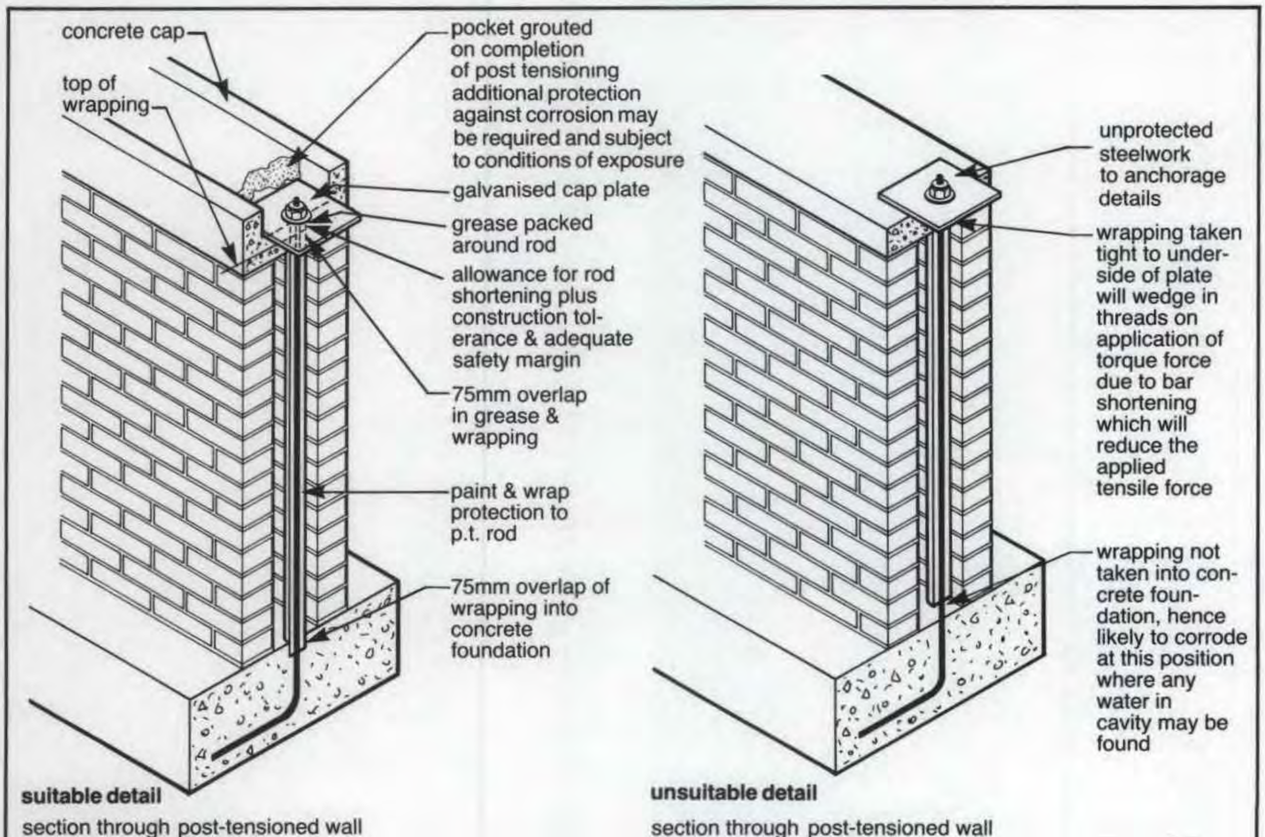
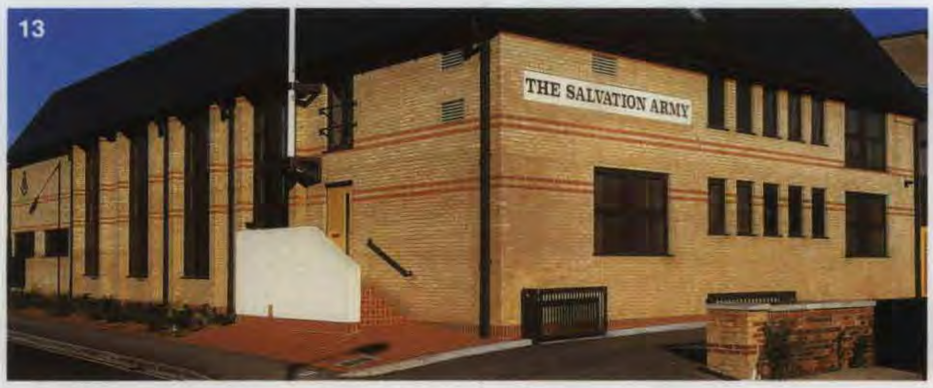


Figure 3.7



13 & 14. The Orsborn Memorial Hall, Boscombe, Bournemouth. *In order to maximise the effect of natural lighting, the architect required minimal brick panels between full height glazing. It was also desirable to break up the large flank elevation, fronting the road, with shallow brick piers which would reflect the domestic scale of the majority of the surrounding properties. Post-tensioned U-shaped brick panels designed between full height windows to span a height of 5.5m as a propped cantilever. These post-tensioned brick panels also provided support for the cranked steel roof beams, which were designed as 'simply supported' between the tops of the piers. Lateral loading was resisted at the roof level by incorporating bracing, which transferred the wind loads back to the gable shear walls. Architect: Major D. Blackwell, The Salvation Army. Consulting engineer: Curtins*

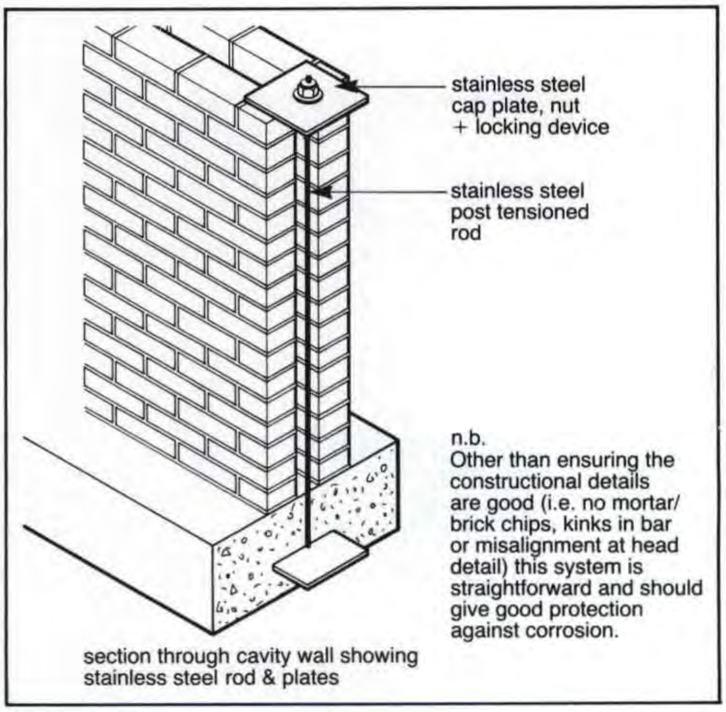


Figure 3.8

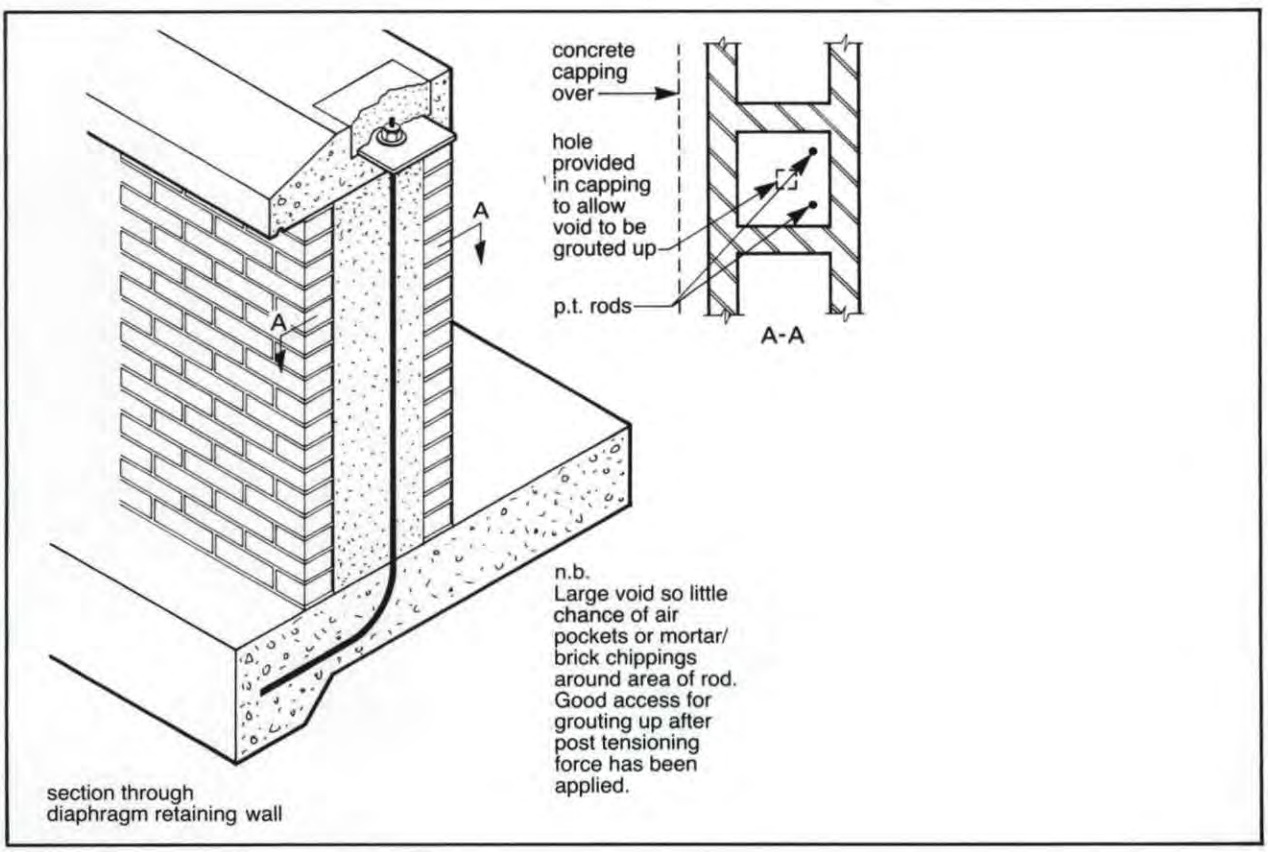


Figure 3.9

As a general rule, we would prefer a protective treatment which is inherent in the materials used in the stressing system, such as stainless steel. Certainly, stainless steel is more expensive, but the 'extra-over' cost for the total project is usually insignificant, as a percentage of the total cost. If galvanised steel is used, we would recommend a secondary protective coating of a bitumen, or water resistant paint, in order to combat micro-cracking of the zinc coating of the stressed steel. Should the above protective treatment prove ecomonically acceptable, then it is important to ensure that compatibility of materials in contact, or close proximity, is adhered to, in order to prevent electrolytic reactions and hence the risk of corrosion.

When considering a high tensile steel system, painting with bitumen, or other flexible and waterproof coating, and wrapping the painted rod with a flexible impervious membrane (Denso tape or similar) can be very effective, as it can be monitored and inspected prior to the brickwork being constructed around it. The anchor plate, nut and locking device would require protection and one method of achieving this is to encase the anchorage in grout, within the preformed pocket in the capping beam.

The use of grout as the sole method of protection, we think to be the most hazardous and, therefore, the least likely to succeed. If adopted, great care must be taken to ensure that both air voids, and voids caused by trapped mortar/brick chippings, are totally eliminated. See figure 3.10. This entails particularly good practical details and close site supervision of the construction, since the grout protection is not applied until the brickwork has been constructed, and this can restrict the quality of workmanship and its inspection. A final inspection of the quality of treatment is, in most cases, difficult if not impossible, without destroying the element.

When adopting the grouting method of protection, we recommend that the grout cover provided should be at the very least equal to that for concrete cover in reinforced concrete, and no reliance should be given to the additional protection provided by the brickwork.

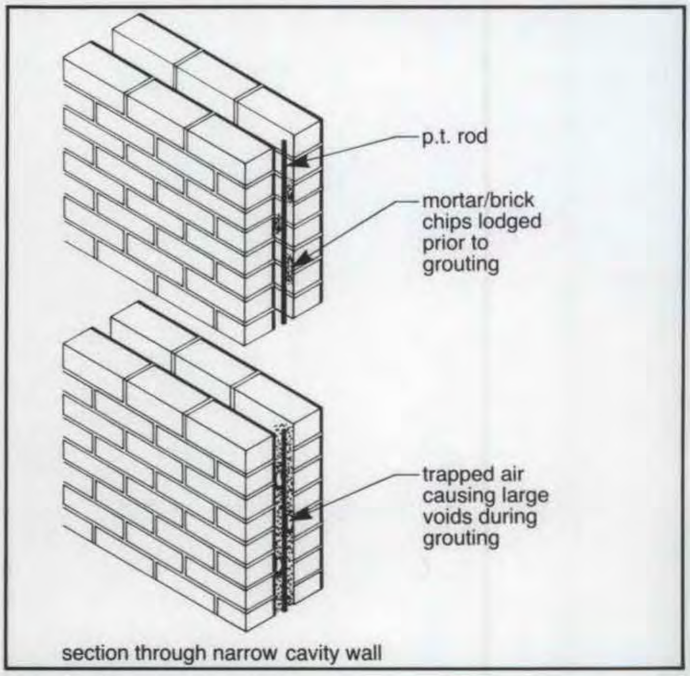


Figure 3.10

15 & 16. Ambulance Canopy, St Michael's Hospital, Braintree. *The ambulance port forms the main entrance into the Rehabilitation Department, for both ambulances and the general public. Consequently, it was important to achieve an aesthetically pleasing appearance, whilst maintaining the overall purpose of providing shelter to incoming patients, during transfer between the ambulance and the hospital. The architect elected to use a tiled roof canopy on brick piers, without interconnecting side walls, to give an open structure on all four sides. Structurally, therefore, the problems were of overall long-term stability, and short-term stability after accidental damage due to impact. The supporting brick piers are a hollow-box, 440mm square, with a central void of 235mm through which two high yield 16mm diameter bars pass to post-tension each pier. The moment of resistance of each pier was thus increased by a factor of well over 8 times compared to a 440mm square solid brick pier, and its impact resistance was massively increased. Should a pier be accidentally removed, the post-tensioning rods also act as ties in the opposite corner pier, to prevent total roof collapse. Regional Architect: North East Thames Health Authority. Consulting engineer: Curtins*

CONSTRUCTION CONSIDERATIONS

LOWER ANCHORAGE

There are several ways in which the lower anchorage of the post-tensioning rod can be achieved. Whether the anchorage mass in which the detail occurs is concrete, steel or masonry, the anchorage detail must be durable and fulfil the main structural requirement of resisting the post-tensioning force. It must resist the applied load, within the allowable stresses of bond, shear, bearing, bursting, etc, for the materials used. Typical details of lower anchorages in various materials and locations are shown in Figure 3.11.

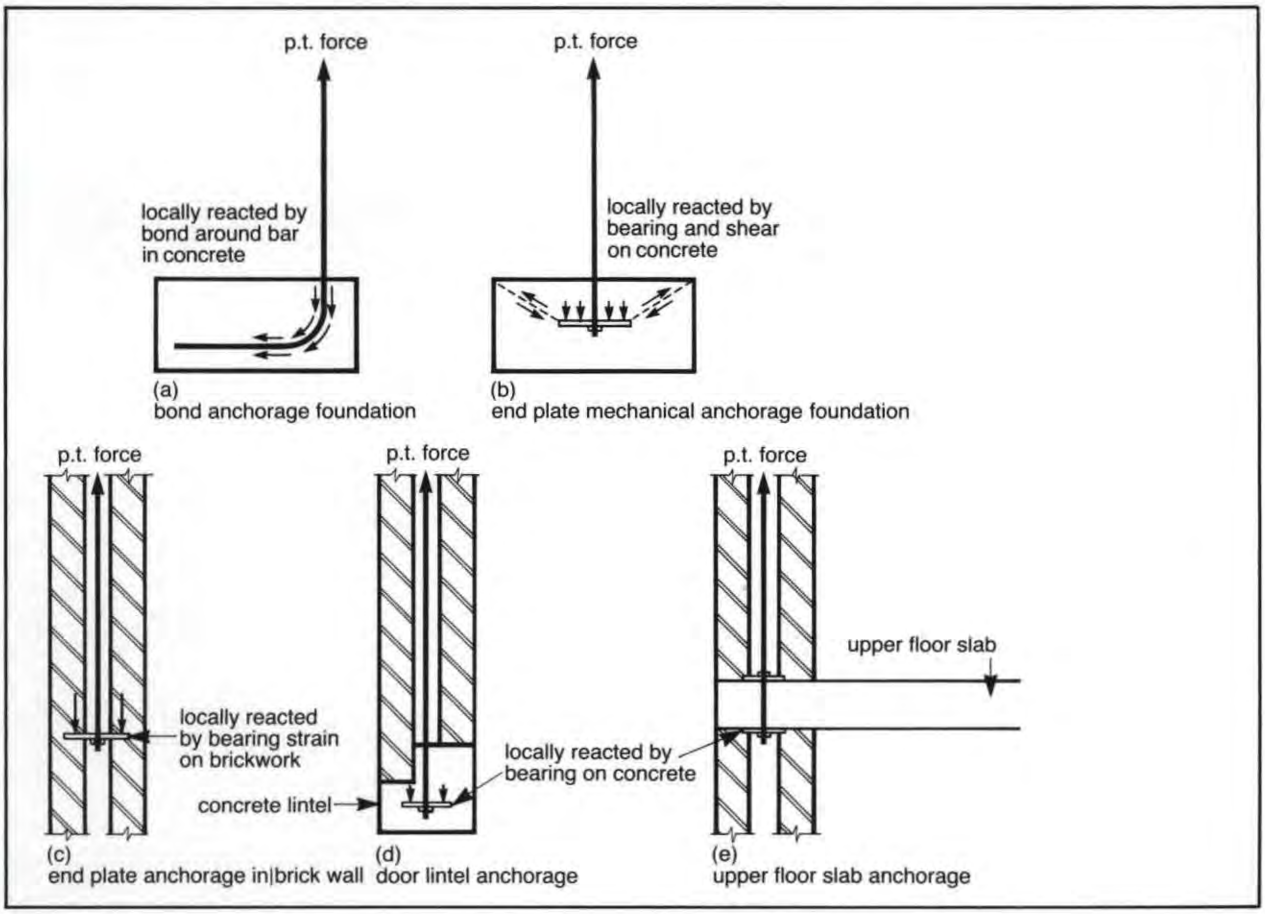


Figure 3.11

UPPER ANCHORAGE

The post-tensioning force is applied at the top of the rod and secured by means of an upper anchorage. Three main criteria must be considered at this position:

(i) the transfer of the force into the brickwork;

(ii) the application of the forces;

(iii) durability.

Typical details of upper anchorage systems are shown in Figure 3.12.

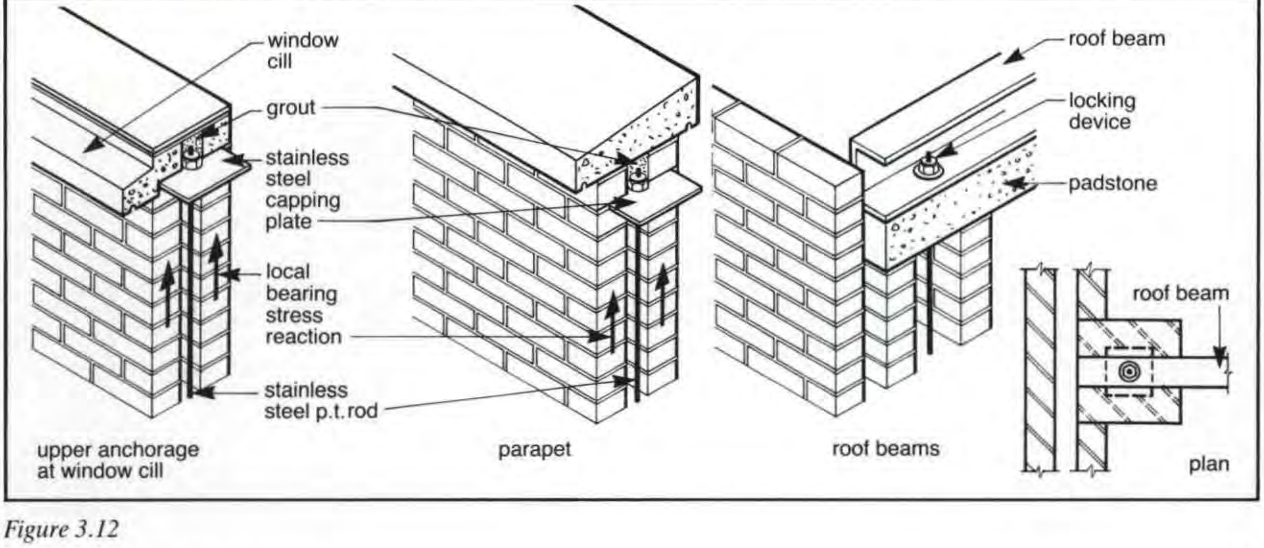


Figure 3.12

DPC'S (HORIZONTAL)

The horizontal dpc in a vertically post-tensioned brick wall is critical to the structural performance of the wall. Since compressible dpc membranes would cause a loss of prestress, they should neither squeeze out, nor be affected by the expected temperature changes. In many cases, the post-tensioned element will be subjected to quite large horizontal loads, so the dpc must be able to resist horizontal shear without excessive movement, and also without damage to the damp proof resistance of the membrane.

The most suitable dpc's are engineering brick or slate. Some modern damp proof membranes are also suitable, but must be carefully checked and researched before use, since the data may not be available from the manufacturer.

DPC'S (VERTICAL)

Vertical dpc's between vertically post-tensioned elements do not normally require any special structural consideration, except that they create discontinuity and, in effect, separate the elements. However, when vertical shear is critical to the design at a joint location, it will be necessary to ensure that the vertical shear is transferred across the dpc membrane, by means of galvanised metal or stainless steel, or other suitable ties. For example, in the diaphragm wall section, shown in figure 3.13, where it is necessary to transfer the vertical shear force into the ribs from the external walls, ties are inserted across the dpc membrane (bitumen painted joint), in order to prevent the wall from shear failure.

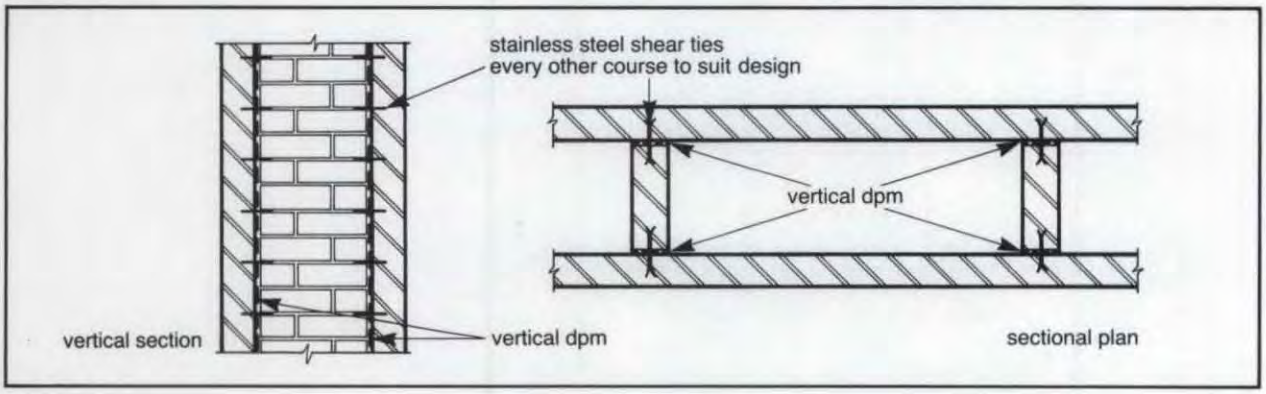


Figure 3.13

MOVEMENT JOINTS

No ties should be used across movement joints, since this would obviously negate their function. Such joints effectively separate the structure into elements, and each should be designed to function independently.

SUMMARY

There are a wide variety of specific applications of post-tensioning brickwork elements and varying details could be applied to all of them. It is outside the scope of this design guide to discuss every possible detail. All we have tried to do is to list some typical examples of details used by us in some of our structural designs.

Typical construction details for post-tensioned brick walls.

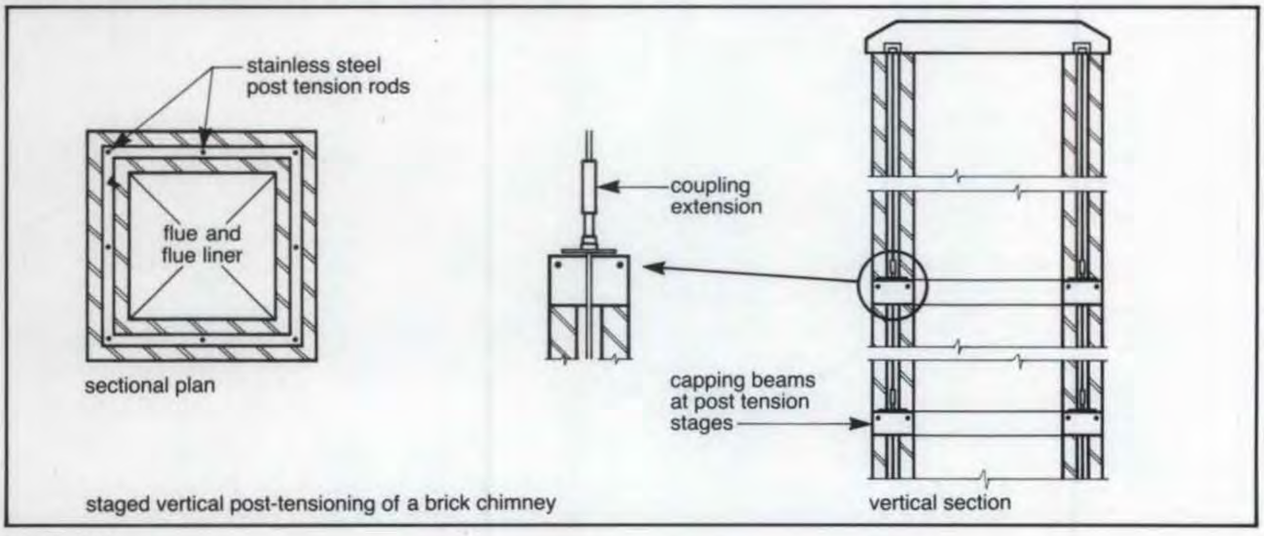


Figure 3.14

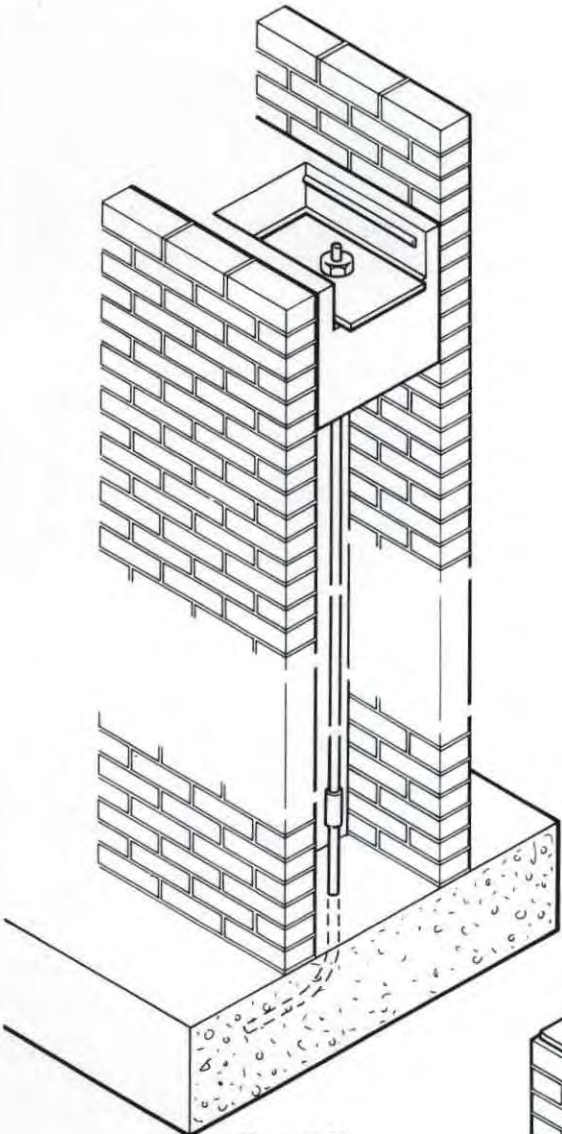


Figure 3.15

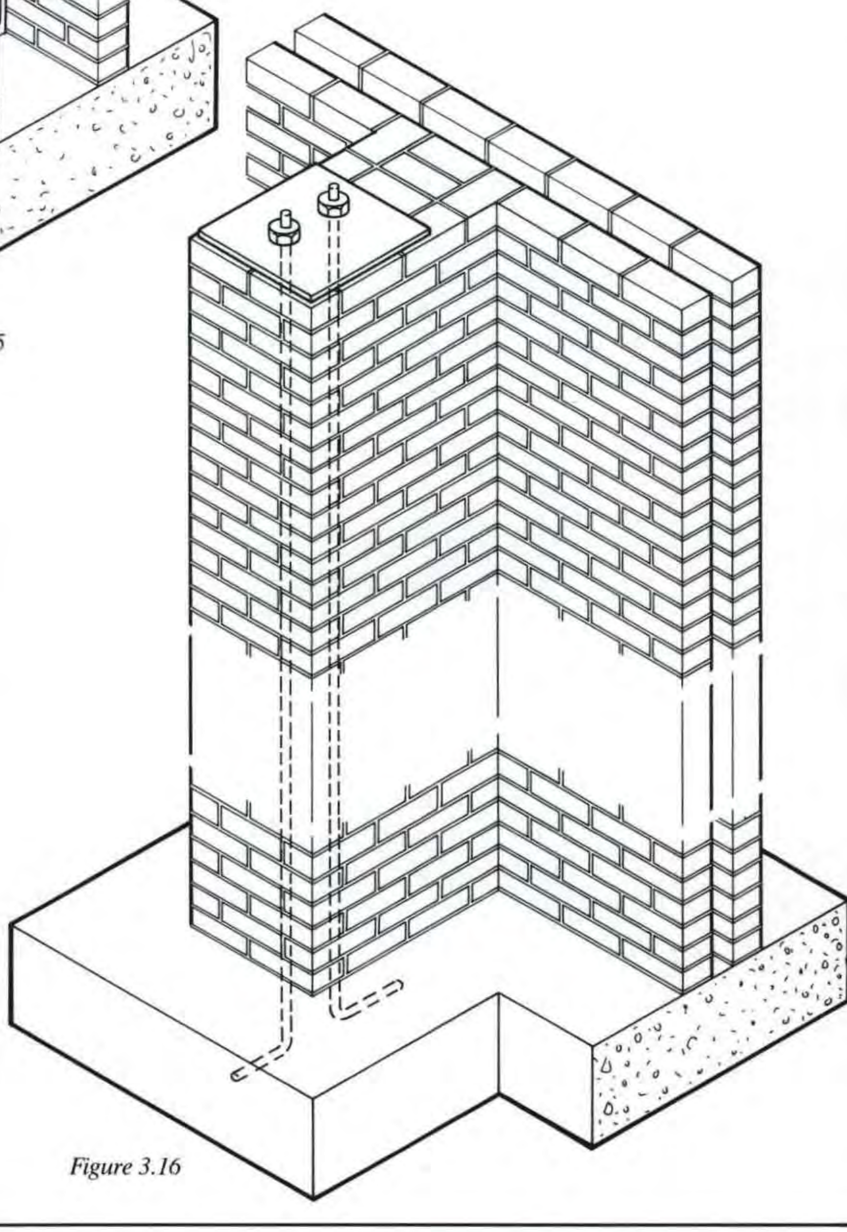


Figure 3.16

Typical construction details for post-tensioned brick columns.

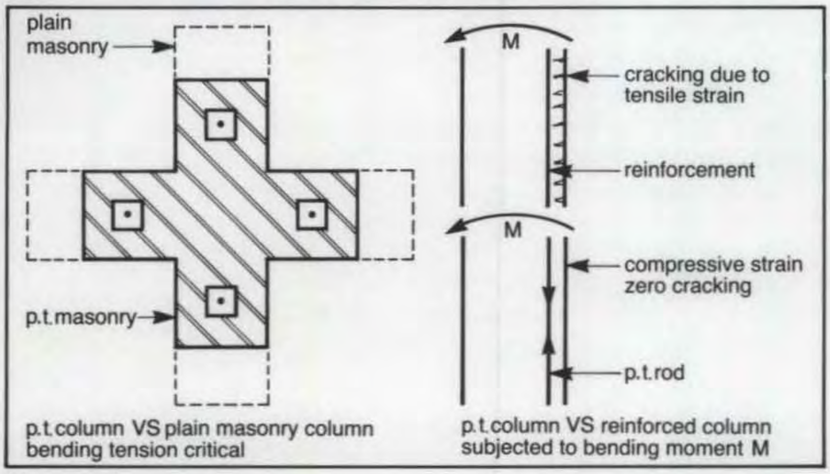


Figure 3.17

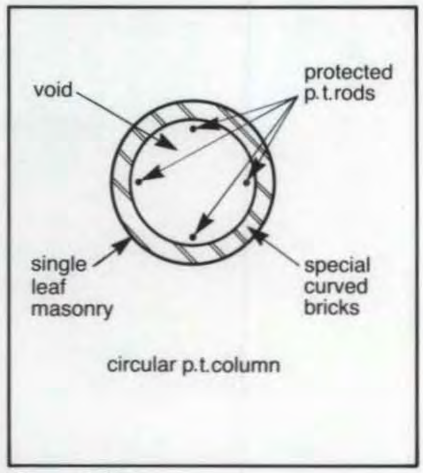


Figure 3.18

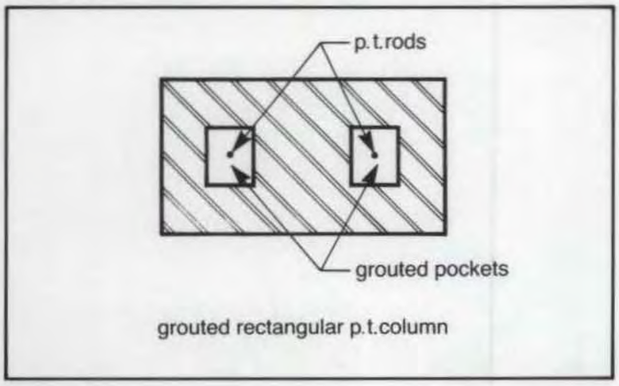


Figure 3.19

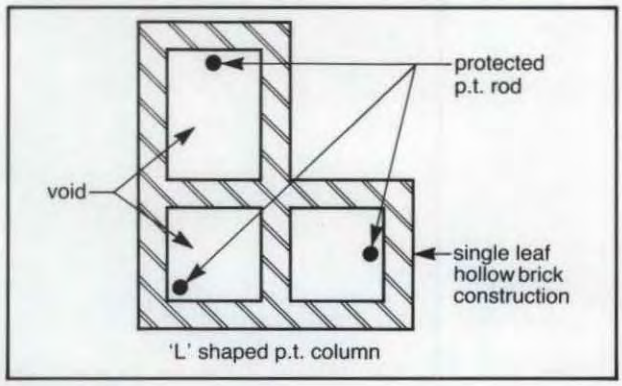


Figure 3.20

17 & 18. BUPA Hospital, Havant. *The first floor link area between the wards and the operating theatres was required by the architects to be given ample natural light. To achieve this, continuous windows for the full length of the link were needed, resulting in an unrestrained cantilevering brick wall. To resist the lateral loads on this cavity wall post-tensioning was utilised, with the post-tensioning rods cast into the first floor slab and taken up the cavity of the two leaf brick wall. Architect: Lambert Scott & Innes. Consulting engineer: Curtins*

19 & 20. Goodwill Centre, Milton Keynes. *The aims of the design were three-fold:*

1. *To allow plenty of sunshine into the building and to maximise the pleasant views, particularly to the north.*

2. *To open up the building to the Community, and so dispel any mystique surrounding The Salvation Army.*

3. *To provide a strong roof shape, to complement the corner site and distinguish the building from the surrounding flat-roofed housing.*

The resultant design makes extensive use of corner windows, and hipped roofs with large overhangs. Cill heights are kept low, to allow views of the ample landscaping that surrounds the building. In order to achieve these requirements, particular note had to be taken regarding the tall storey-height walls of the main hall, coupled with the large glazed areas. The cavity wall panels between windows were designed to span vertically under direct wind loads. The dwarf walls below windows were post-tensioned, to act as vertical cantilevers so as not to transfer additional loads onto the panel walls. This facility kept the thickness of the panel walls down to acceptable proportions. Architect: Major D. Blackwell, The Salvation Army. Consulting engineer: Curtins

CAVITY WALL RESISTING LATERAL WIND LOAD

The side walls of the link corridor between a sports hall and swimming pool, shown in Figure 4.1, are required to have high level horizontal strip glazing, in order to provide natural lighting while maintaining a discrete link between the changing facilities and the pool.

The two main buildings are of loadbearing brick construction, and the link corridor is to be built in brickwork, without protrusions either internally or externally.

The bricks to be used are clay bricks, having a minimum compressive strength of 30 N/mm^2, and a water absorption of 9%, set in a designation (ii) mortar. The partial safety factor for the material is taken as 2.5, and the characteristic wind load is 0.7 kN/m^2.

The density of the masonry will be taken as 19 kN/m^3. No vertical load is imparted from the link corridor roof, which spans between the two main buildings.

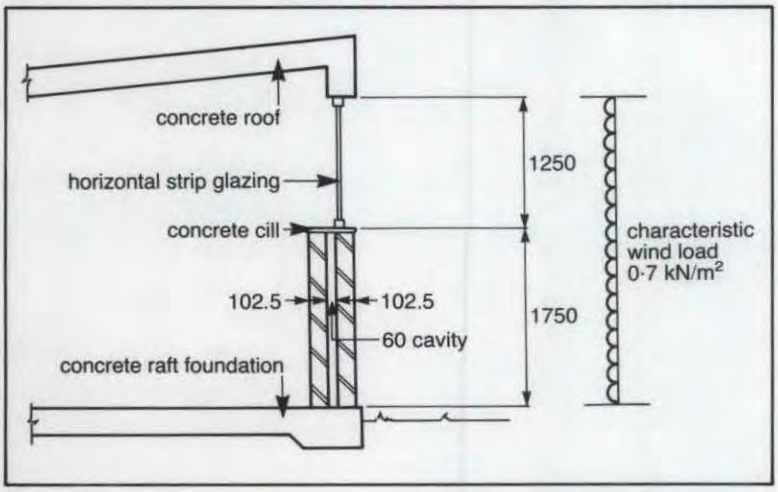


Figure 4.1

Basic steps involved in calculation

(1) Determine the characteristic loads.

(2) Calculate design loads.

(3) Assume wall thickness of section and check slenderness ratio and, hence, capacity reduction factor.

(4) Calculate design strengths of brickwork

(5) Calculate section modulus of the wall.

(6) Calculate maximum bending moment, due to applied loading.

(7) Calculate maximum theoretical flexural tensile stress, due to applied load.

(8) Calculate design post-tensioning stress required.

(9) Consider compressive stresses – after losses.

(10) Check compressive stresses – before losses.

(11) Calculate post-tensioning rod area, and No. required.

(12) Calculate required torque to develop tension in rod.

(13) Design anchorage and spreader plate.

(1) DETERMINE THE CHARACTERISTIC LOADS

Wind = 0.7 kN/m^2 = 0.7 kN/m, for 1m wide strip.

No load imposed from roof.

Self weight of wall = $1 \times 0.102 \times 19 \times 1.75$ = 3.4 kN/m, on each leaf.

Weight of strip glazing = 0.05 kN/m, on each leaf.

Total characteristic dead load, G_k = 3.45 kN/m, on each leaf.

We would point out that significant horizontal applied loading may result from handrail loading, pushing crowds or other circumstances. While, in this example, no such loading has been assumed, designers should consider the possibility of horizontal applied loading for the particular structure being designed.

(2) CALCULATE THE DESIGN LOADS

From BS 5628: Part 1: cl 22:

For the loading conditions dead plus wind only, the applicable partial safety factors are 0.9 or 1.4 and 1.2 respectively.

Design dead loads = $0.9 \times 3.45 = 3.105$ kN/m length, on each leaf

= $1.4 \times 3.45 = 4.83$ kN/m length, on each leaf

Design wind loads = $1.2 \times 0.7 = 0.84$ kN/m height.

(3) SLENDERNESS RATIO

Effective height, h_{ef} = 2L, assumed as a free cantilever,

= 2×1750

= 3500 mm

Effective thickness, t_{ef} = $\frac{2}{3}(102.5 + 102.5)$, cavity wall construction, effectively tied together.

= 136.66.

Slenderness ratio $= \frac{\text{effective height}}{\text{effective thickness}} = \frac{3500}{136.66}$

= 25.6, say 26.

Since loading is own weight only, and post-tensioning is applied concentrically, eccentricity = zero.

From Table 7 (BS 5628: Part 1), capacity reduction factor, $\beta = 0.45$.

(4) CALCULATE DESIGN STRENGTH OF BRICKWORK

From Table 2 (BS 5628: Part 1)

Characteristic compressive strength, f_k = 8.4 N/mm^2

Characteristic flexural compressive strength, f_f, = 1.2×8.4 = 10.08 N/mm^2

Characteristic flexural tensile strength is limited to zero for design purposes.

Steel

Characteristic tensile strength, f_y = 460 N/mm^2

Design strength – after losses

Design compressive strength, at base (BS 5628: cl.23.1.2 for narrow wall factor 1.15)

$$= \frac{1.15 f_k}{\gamma_{mm}} = \frac{1.15 \times 8.4}{2.5}$$

= 3.86 N/mm^2

Design compressive strength, in wall height

$$= \frac{1.15 \beta f_k}{\gamma_{mm}}$$

$$= \frac{1.15 \times 0.45 \times 8.4}{2.5}$$

= 1.74 N/mm^2.

Design flexural compressive strength

$$= \frac{1.15 f_f}{\gamma_{mm}} = \frac{1.15 \times 10.08}{2.5}$$

= 4.64 N/mm^2

Design flexural tensile strength = limited to zero.

(5) CALCULATE THE SECTION MODULUS OF THE WALL

For a cavity wall, the section modulus for each individual leaf will be calculated, and the applied stresses will be distributed accordingly, each leaf being checked independently. In this example, both leaves are of the same thickness:

Hence, $Z = \frac{1 \times 1000 \times 102^2}{6} = 1.734 \times 10^6$ mm^3, for each leaf.

Design method

The theoretical flexural tensile stress developed in the wall due to lateral loading, in the absence of any post-tensioning force and in combination with minimum compressive load, will be calculated. A compressive stress, required to eliminate this theoretical flexural tensile stress, will then be applied by means of a post-tensioning force. The section will be checked for stability for the compressive stresses induced by the post-tensioning force, and other load combinations, both before and after losses.

The post-tensioning force is used to determine the required diameter and spacing of the rods, and the torque required to induce the tensile force in the rods is also calculated.

Finally, the upper and lower anchorages are designed to ensure that the compressive stress is applied to the brickwork without causing local compressive failure of the brickwork or concrete footing.

(6) CALCULATE BENDING MOMENTS AT THE BASE OF THE WALL

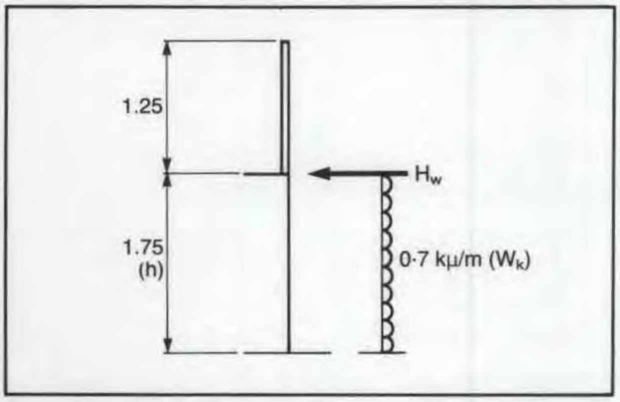


Figure 4.2

Therefore, maximum bending moment at base of each leaf is:

$$\gamma_f Hw_1 h + \gamma_f W_{k1}\frac{h^2}{2} \text{ and } \gamma_f Hw_2 h + \gamma_f W_{k2}\frac{h^2}{2}$$

Therefore, design moments at base are:

1. Wind on glazing,

$$\begin{aligned}
M_b &= \gamma_f Hw_1 h \text{ and } \gamma_f W_2 h.\\
\text{As } Z_1 &= Z_2,\ Hw_1 = Hw_2,\\
&= \frac{0.7 \times 1.25}{2 \times 2}\\
&= 0.219 \text{ kN.}\\
\text{Therefore, } M_{b1} &= 1.2 \times 0.219 \times 1.75\\
&= 0.46 \text{ kN.m, for each leaf.}
\end{aligned}$$

2. Wind on wall,

$$\begin{aligned}
M_b &= \gamma_f W_{k1}\frac{h^2}{2} \text{ and } \gamma_f W_{k2}\frac{h^2}{2}.\\
\text{As } Z_1 &= Z_2,\ W_{k1} = W_{k2},\\
\text{Therefore, } M_{b2} &= 1.2 \times \frac{0.7}{2} \times \frac{1.75^2}{2}\\
&= 0.643 \text{ kN.m. for each leaf}
\end{aligned}$$

Combining for total bending moment at base:

$$\begin{aligned}
M_{b1} + M_{b2} &= 0.460 + 0.643\\
&= 1.103 \text{ kN.m, for each leaf.}
\end{aligned}$$

Therefore, stress at base due to applied lateral loading is:

$$\begin{aligned}
\pm\frac{M}{Z} &= \frac{1.103 \times 10^6}{1.734 \times 10^6}\\
&= \pm 0.636 \text{ N/mm}^2\text{, for each leaf.}
\end{aligned}$$

(7) CALCULATE THE MAXIMUM TENSILE STRESSES AT THE BASE

Design vertical dead load on each leaf, $g_d = \dfrac{3.105 \times 10^3}{102 \times 1000}$

$= 0.030 \text{ N/mm}^2$.

Hence, theoretical tensile stress without post-tensioning force:

$$f_t = g_d - \frac{M}{Z}$$

Where, M = design bending moment at the base
Z = section modulus of the wall
g_d = axial compressive stress from dead load

Theoretical flexural tensile stress = 0.030 – 0.636
= – 0.606 N/mm^2

(8) CALCULATE DESIGN POST-TENSIONING STRESS REQUIRED

In order to eliminate this theoretical flexural tensile stress the design post-tensioning stress required = + 0.606 N/mm^2

Apply the partial safety factor for loads,
$\sigma_f = 0.9$ (in which the post-tensioning force is treated as a dead load)

Therefore, characteristic post-tensioning stress,

$$f_p = +\frac{0.606}{0.9}$$
$$= +0.673 \text{ N/mm}^2$$

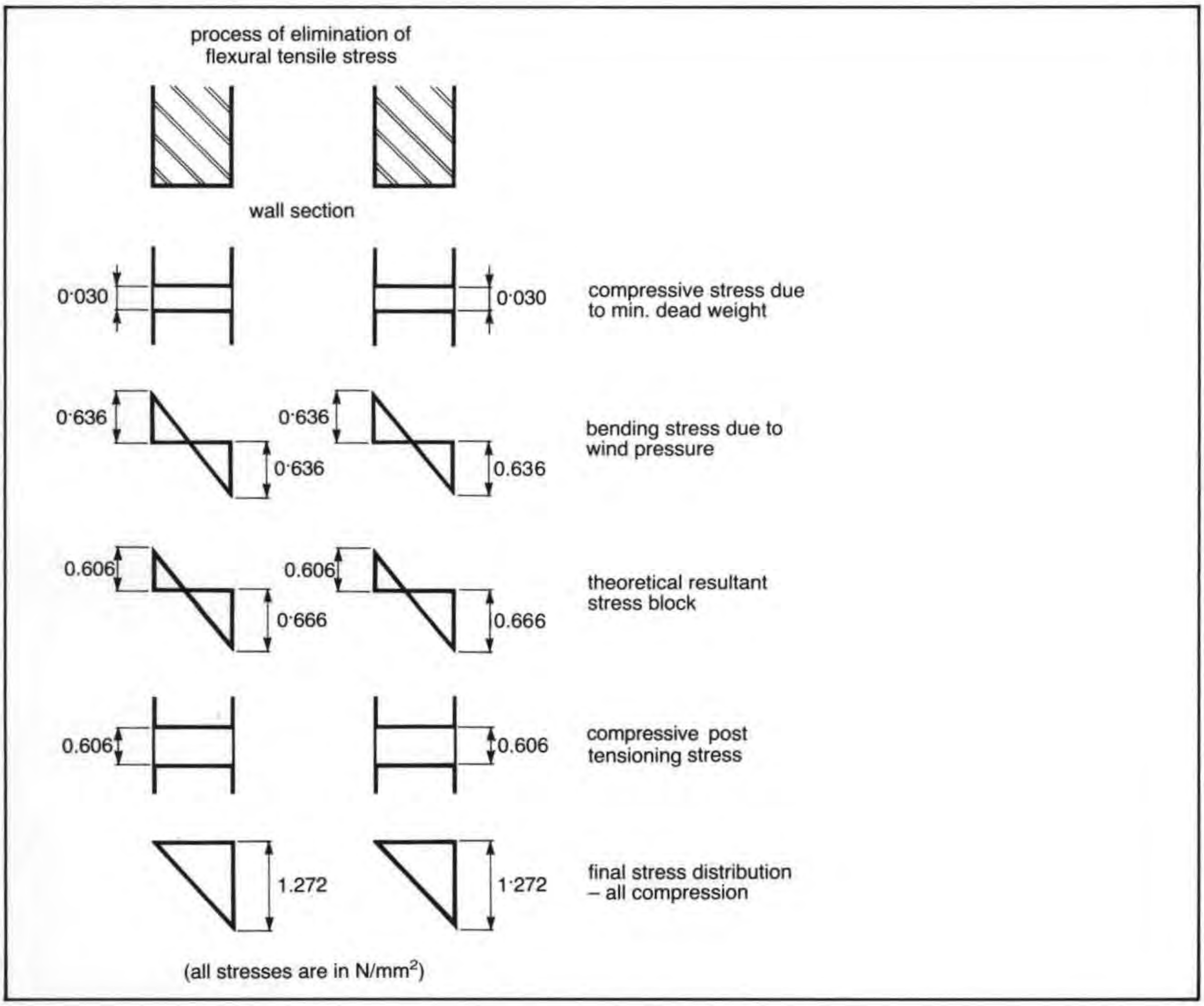


Figure 4.3

(9) CONSIDER COMPRESSIVE STRESSES – AFTER LOSSES

In this example, dead load plus wind load will be the most onerous case for checking, using the appropriate partial safety for loads. Where superimposed loading is present (from roof) then dead + super and dead + super + wind must also be checked (see example 2).

Design axial stress f_{uac} $= \frac{(\gamma_f G_k)}{A}$ + post-tensioning stress
where G_k = characteristic dead load.

Hence, f_{uac} for each leaf $= \frac{(1.4 \times 3.45 \times 10^3)}{(1 \times 102 \times 1000)} + (1.4 \times 0.673)$
= 0.990 N/mm^2

Design compressive strength of wall is 1.74 N/mm^2 (taken at mid-height as the majority of axial load is from P_t force), which > 0.99 N/mm^2 as calculated above; therefore wall is adequate for this loading condition.

Stress from wind loading	$= \pm 0.636$ N/mm^2
Combined stresses	$= 0.99 \pm 0.636$ N/mm^2
	$= 1.63$ or $+ 0.35$ N/mm^2
Design flexural compressive strength of wall	$= 4.64$ N/mm^2, which exceeds 1.63 N/mm^2; the wall is therefore satisfactory for this loading condition.

(10) CHECK COMPRESSIVE STRESSES – BEFORE LOSSES

Increased post-tensioning stresses by 20%, to cover allowance for losses.
Design axial stress before losses (for each leaf),

$$f_{uac} = \frac{(\gamma_f G_k)}{A} + \frac{\gamma_f \times \text{characteristic post-tensioning stress}}{0.8}$$

$$= \frac{(1.4 \times 3.45) \times 10^3}{102 \times 1000} + \frac{1.4 \times 0.673}{0.8}$$

$$= 1.225 \text{ N/mm}^2.$$

Design strength of the wall before losses:

$$= \frac{1.25\beta\,(1.15 f_{ki})}{\gamma_{mm}}$$ where f_{ki} is the characteristic compressive strength of the brickwork when the post-tensioning force is applied ($f_{ki} = f_k$ for this example).

For brickwork of 30 N/mm^2 in designation (ii) mortar, design strength

$$= 1.25 \times 0.45 \times 1.15 \times \frac{8.4}{2.5}$$

$$= 2.17 \text{ N/mm}^2$$

This exceeds the stress before losses and is therefore satisfactory.

(11) POST-TENSIONING RODS

Characteristic post-tensioning stress required to be transmitted into the brickwork = 0.673 N/mm^2.

Which, per metre run of wall,

$$= \frac{0.673 \times 2 \times 102 \times 10^3}{10^3} = 137.3 \text{ kN/m, on both leaves.}$$

Allowance for 20% losses

$$= \frac{137.3}{0.8} \qquad = 171.6 \text{ kN}$$

Limit stress to

$$= \frac{0.7 f_y}{\gamma_{ms}} = \frac{0.7 \times 460}{1.15} \qquad = 280 \text{ N/mm}^2.$$

Therefore, area of steel required

$$= \frac{171.6 \times 10^3}{280} \qquad = 613 \text{ mm}^2 \text{ per m.}$$

Use T25 at 725 mm c/c (675 mm^2 per m).

(12) CALCULATE TORQUE TO PROVIDE TENSION

$$= \frac{\text{Bolt tension} \times \text{bolt diameter}}{5}$$

$$= \frac{171.6 \times 10^3 \times 0.725 \times 0.025}{9.81 \times 5}$$

$$= 63.4 \text{ kgf.m}$$

(13) DESIGN UPPER AND LOWER ANCHORAGES

(a) Design upper spreader plate anchorage

Rod force $= 171.6 \times 0.725$	$= 124.41$ kN.
Maximum design force per rod	$= 124.41 \times \gamma_f = 121.41 \times 1.4$
	$= 174.2$ kN
Design compressive strength of wall	$= \dfrac{1.5 \times 1.15 \times 8.4}{2.5}$
	$= 5.80$ N/mm^2.

In which a 1.5 strength factor has been incorporated, to take account of the local bearing condition of the spreader plate on the brickwork.

Area of spreader required $= \dfrac{174.2 \times 10^3}{5.8}$

$= 30034 \text{ mm}^2$.

Therefore, length of spread along two leaves (see Figure 4.4)

$= \dfrac{30034}{2 \times 102}$

$= 147\text{mm}$.

With a 50mm concrete cill, and allowing for a 45° load spreading through the concrete, length of plate required: $= 147 - 2\,(37.5)$

$= 72$ mm.

Therefore, use a 125mm long by 210mm wide × 20mm thick spreader plate.

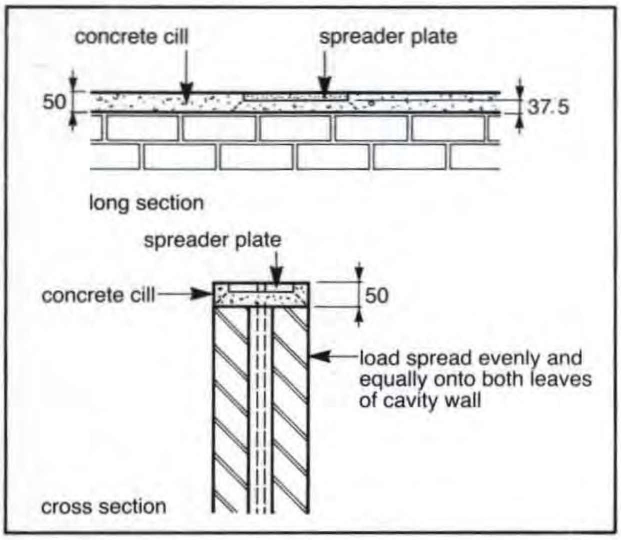


Figure 4.4

(b) Design lower anchorage

The lower anchorage for this example will be achieved by the bond developed between the T25 reinforcing bar and the concrete foundation into which it is cast.

Care should be taken to see that the bar required to develop the necessary bond is detailed with a suitable radius, to ensure no local bond failure within the concrete foundation. The bar should have sufficient length, cast into the concrete foundation, to allow for possible wrapping in protective material at the top of the foundation. It is important to ensure that the bar is straight and parallel to the direction of the applied post-tensioning force. (See details in Figure 3.7 and 3.11).

N.B. When designing spreader plates to bear directly onto two leaves of a cavity wall, or to span across a large void in a brickwork element, care should be taken to avoid bending curvature in the spreader plate, which could cause horizontal thrusts at the top of the wall/element. As well as ensuring that the steel plate is of sufficient thickness, consideration should be given to placing extra horizontal brick ties near the top of the wall, to cater for any additional horizontal thrust from the anchorage detail. For high levels of prestress a concrete spreader beam, or pad, should be used, and the bursting stress, F_{bst}, should be checked in accordance with the relevant codes for prestressed concrete.

FIN WALL RESISTING LATERAL WIND LOAD

The 265mm thick external cavity brick wall of a library building is required to support vertical full height glazing panels, 1m wide at 3.0m centres (see Figure 4.5). The planning authority have placed a restriction on the size of any protrusion extending externally, and the client desires that there be no protrusion inside the building.

The clay bricks to be used for both external and internal leaves are facing bricks, with a minimum crushing strength of 30 N/mm^2, and having water absorption of 8% set in a mortar designation (ii). Partial safety factor for materials is assumed as 2.5. The characteristic wind pressure applicable in this location, and for the height and size of the building, will be assumed to be 0.6 kN/m^2.

The roof is of lightweight construction, and will be assumed capable of providing an adequate prop to the head of the wall. There is no uplift resulting during maximum wind pressure, the load of the roof and the wind uplift force being equal and opposite.

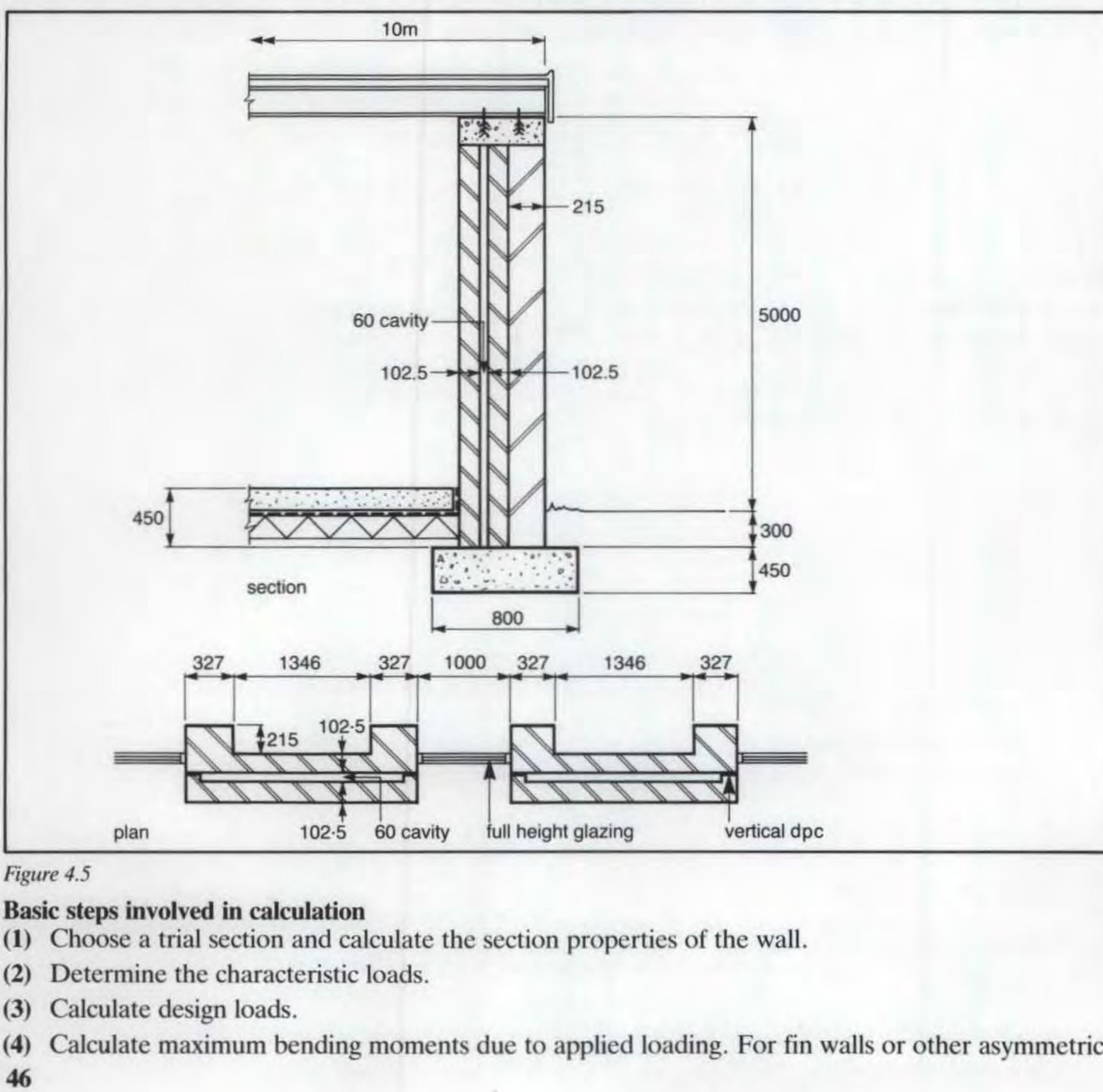


Figure 4.5

Basic steps involved in calculation

(1) Choose a trial section and calculate the section properties of the wall.

(2) Determine the characteristic loads.

(3) Calculate design loads.

(4) Calculate maximum bending moments due to applied loading. For fin walls or other asymmetric walls, check for both wind pressure and suction and compute the most onerous case.

(5) Estimate deflection of wall.

(6) Check slenderness ratio and determine capacity reduction factors.

(7) Calculate design strengths of brickwork.

(8) Calculate maximum tensile stress due to the worst case of loading.

(9) Calculate design post-tensioning stress required.

(10) Consider combined compressive stresses – local stability of the flange and fin.

(11) Check design stress in wall, both after and before losses.

(12) Design of post-tensioning rods.

(13) Design of anchorages and spreader plate.

(1) CALCULATE THE SECTION PROPERTIES OF THE WALL

The internal leaf of the external wall is ignored for the purpose of this design, except for consideration of its stiffening effect when assessing slenderness ratios and effective flange width.

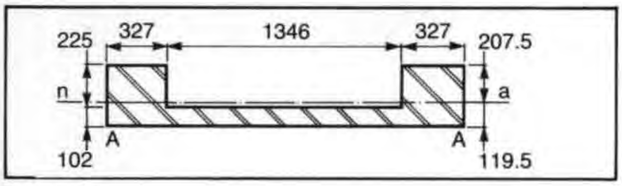


Figure 4.6

For the section shown in Figure 4.6, the effective flange to the two protrusions must be checked, to ensure that the section may be considered as acting as one. The maximum effective flange which can be considered to act with the protrusions, in accordance with BS 5628: Part 1:cl.36.4.3(b), is $6 \times t_{ef}$ (where t_{ef} is the effective thickness of the flange wall).

$$= 6 \times \frac{2}{3} \times 204 = 816\text{mm, which exceeds } \frac{\text{distance between fins}}{2}$$

Therefore, section is considered as one.

$$\text{Area} = (2000 \times 102) + (327 \times 225 \times 2)$$
$$= 0.351 \times 10^6\ \text{mm}^2.$$

To find N.A. take moment of area above A-A.

$$\left(2\,(327 \times 327) \times \frac{327}{2}\right) + \left(102 \times 1346 \times \frac{102}{2}\right) = 351150 \times \bar{y}.$$

Therefore,

$$\bar{y} = 119.5\ \text{mm}.$$

$$I_{na} = \left[327 \times 327 \times \left(\frac{327}{2} - 119.5\right)^2 + \frac{327 \times 327^3}{12}\right] \times 2 + \left[1346 \times 102 \times \left(\frac{102}{2} + 17.5\right)^2 + \frac{1346 \times 102^3}{12}\right]$$
$$= [(0.207 + 0.953)\,2 + 0.644 + 0.119] \times 10^9$$
$$= 3.083 \times 10^9\ \text{mm}^4$$

Hence,

$$Z_1 = \frac{3.083 \times 10^9}{207.5}$$
$$= 0.0149 \times 10^9\ \text{mm}^3,$$

$$Z_2 = \frac{3.083 \times 10^9}{119.5}$$
$$= 0.0258 \times 10^9\ \text{mm}^3.$$

For design purposes wall section is assumed to have section properties:

Area $= 0.351 \times 10^6\ \text{mm}^2$

$I_{na} = 3.083 \times 10^9\ \text{mm}^4$

$Z_1 = 0.0149 \times 10^9\ \text{mm}^3$

$Z_2 = 0.0258 \times 10^9\ \text{mm}^3$

(2) CHARACTERISTIC LOADINGS

Wind

Suction on leeward face, W_{k1} $= 0.45\ \text{kN/m}^2$.

Pressure on windward face, W_{k2} $= 0.72\ \text{kN/m}^2$.

Up-lift due to wind + own weight of roof = 0.00

Dead load

Roof load, assuming 0.5 kN/m^2 = 2.5 kN/m run of wall.

Assuming density of 19 kN/m^3 = 0.351×19

= 6.67 kN/m height of section.

(3) CALCULATE DESIGN LOADS

For the loading combination dead + wind, the partial safety factors for loads may be taken for dead and wind respectively as 0.9 or 1.4 and 1.4 using the most severe conditions.

Characteristic superimposed load, Q_k = 0.75 kN/m^2.

Design wind loads

Suction = $3.00 \times 0.45 \times 1.4$ = 1.89 kN/m height of section.

Pressure = $3.00 \times 0.72 \times 1.4$ = 3.02 kN/m height of section.

(4) CALCULATE BENDING MOMENTS DUE TO APPLIED LOADING

The roof is assumed to act as a plate providing a prop to the head of the wall section. It is also assumed, at this stage, that the wall section and roof beam support will be detailed to suit. Hence, the wall is designed as a propped cantilever (see Figure 4.7).

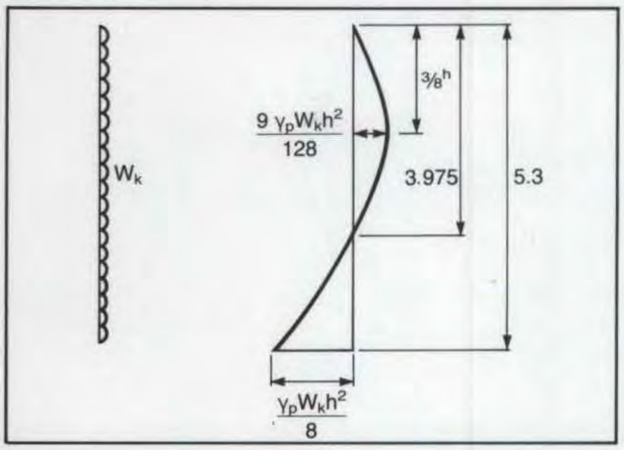


Figure 4.7

Design bending moments are therefore:

Case (a) suction

(i) at $\frac{3}{8}$ h level,

$$M_w = \frac{9 \times 1.89 \times 5.3^2}{128} = 3.73 \text{ kN.m.}$$

(ii) at base level,

$$M_b = \frac{1.89 \times 5.3^2}{8} = 6.64 \text{ kN.m.}$$

Case (b) pressure

(i) at $\frac{3}{8}$ h level,

$$M_w = \frac{9 \times 3.02 \times 5.3^2}{128} = 5.96 \text{ kN.m.}$$

(ii) at base level,

$$M_b = \frac{3.02 \times 5.3^2}{8} = 10.6 \text{ kN.m.}$$

(5) ESTIMATE DEFLECTION OF WALL

Conditions of structure and applied loading assumed are 'propped cantilever with a uniformly distributed applied loading'.

$\Delta \max = \frac{W L^3}{185 E_m I}$ where E_m will be taken as the short term, $E_m = 0.9 f_k$ kN/mm^2, due to the short term nature of the applied wind loading.

$$\Delta \max = \frac{3.02 \times (5.3)^4 \times 10^{12}}{185 \times 8.4 \times 0.9 \times 3.083 \times 10^{12}}$$

$= 0.55$ mm, this is acceptable since $< \frac{L}{250}$ ie, 21 mm.

(6) CALCULATION OF SLENDERNESS RATIOS AND CAPACITY REDUCTION FACTORS FOR LOCAL AND OVERALL STABILITY CHECKS

Local stability of fin wall (see ref 5 for more detailed explanation of stability checks in plain fin walls).

Capacity reduction factors

(a) Suction

Slenderness ratio $= \dfrac{0.75\ 1}{t_{ef}}$ where 1 > is the distance between the centres of the projecting fins, and where $1 > 2 \times t_{ef}$

$$= \frac{0.75 \times 1673}{\frac{2}{3}(102 + 102)}$$

$$= 9$$

Eccentricity of compressive stress in the flange of the fin wall may be taken to be (0 – 0.05t), hence $\beta = 0.98$.

(b) Pressure (maximum combined compressive stress at end of fin.)

$$\text{Slenderness ratio} = \frac{\text{distance between points of contraflexure}}{\text{actual thickness of fin}}$$

$$= \frac{3.975 \times 10^3}{327}$$

$$= 12$$

Again, eccentricity of compressive stress (0 – 0.05t), $\beta = 0.93$. Overall stability under dead load + superimposed + post-tensioning force. For the section in this example the slenderness ratio will be calculated as for a wall stiffened by piers, i.e. effective thickness, $t_{ef} = t \times K$, where t = thickness of wall, and K (from Table 5, BS 5628 Part 1) is a stiffness coefficient dependent on the ratios of

$\dfrac{\text{pier spacing}}{\text{pier width}}$ and $\dfrac{\text{pier thickness}}{\text{wall thickness}}$ For this example, K = 2.0.

$t_{ef} = 102 \times 2.0$
$= 204$ mm.

This > ⅔ (102.5 + 102.5) t_{ef}, based on K factor taken.

$\text{S.R.} = \dfrac{0.85 \times 5.3 \times 10^3}{204} = 22.1$ Effective height taken as 0.85 1, due to full restraint at base, partial restraint at roof and restraint of fins.

As e = (0 – 0.05t),
therefore $\beta = 0.62$

(7) CALCULATE DESIGN STRENGTHS OF BRICKWORK

Characteristic strengths for materials in this example may be obtained from BS 5628: Part 1 as follows:

Brickwork:

Characteristic compressive strength, f_k	= 8.4 N/mm²
Characteristic flexural tensile strength (failure plane parallel to bed joints)	= 0.4 N/mm²
Characteristic flexural tensile strength (failure plane perpendicular to bed joints)	= 1.1 N/mm²
Characteristic flexural compressive strength, 1.2 f_k	= 10.1 N/mm²

Steel:

Tensile strength, $f_y = 460$ N/mm².

Local stability of flange and fin wall

Design flexural compressive strength – after losses. At base level, where by inspection $\beta = 1.0$, design flexural compressive strength

$$= \frac{1.2 \times f_k}{\gamma\,\text{mm}} = \frac{1.2 \times 8.4}{2.5}$$

$$= 4.03 \text{ N/mm}^2$$

At $\frac{3}{8}$h level, where $\beta = 0.98$, by calculation, design flexural compressive strength

$$= \frac{1.2 \times \gamma f_k}{\gamma_{mm}} = \frac{1.2 \times 0.98 \times 8.4}{2.5}$$

$$= 3.95 \text{ N/mm}^2.$$

Overall stability

Buckling of section as a whole under dead load + superimposed + post-tensioning force – after losses.

Design strength of wall

$= \frac{\beta f_k}{\gamma_{mm}} = \frac{0.62 \times 8.4}{2.5}$

$= 2.083 \text{ N/mm}^2$

Design strength of wall – before losses

$= \frac{1.25\beta\, f_{ki}}{\gamma_{mm}}$

$= 2.604 \text{ N/mm}^2$, where $f_{ki} = f_k$ for this example.

(8) THEORETICAL FLEXURAL TENSILE STRESS

Case (a) suction

(i) At $\frac{3}{8}$h level,

compressive stress due to self-weight $= \frac{0.9 \times 19 \times 1.988}{1000}$ $= + 0.034 \text{ N/mm}^2$

theoretical flexural stresses $\pm \frac{M_w}{Z_1} = \frac{3.73 \times 10^6}{0.0149 \times 10^9}$ $= \pm 0.250 \text{ N/mm}^2$

Theoretical flexural tensile stress, f_{t1} $= \underline{-\ 0.216 \text{ N/mm}^2}$

(ii) At base level, axial compressive stress

due to self-weight $= \frac{0.9 \times 19 \times 5.3}{1000}$ $= + 0.091 \text{ N/mm}^2$

theoretical flexural stresses $\pm \frac{M_b}{Z_2} = \frac{6.64 \times 10^6}{0.0258 \times 10^9}$ $= \pm 0.257 \text{ N/mm}^2$

Theoretical flexural tensile stress, f_{t2} $= \underline{-\ 0.166 \text{ N/mm}^2}$

Case (b) pressure

(i) At $\frac{3}{8}$h level,

axial compressive stress due to self-weight, as case (a) $= + 0.034 \text{ N/mm}^2$

theoretical flexural stresses $\pm \frac{M_w}{Z_2} = \frac{5.96 \times 10^6}{0.0258 \times 10^9}$ $= \underline{\pm 0.231 \text{ N/mm}^2}$

Theoretical flexural tensile stress, f_{t2} $= \underline{-\ 0.197 \text{ N/mm}^2}$

(ii) At base level,

axial compressive stress due to self-weight, as case (a) $= + 0.091 \text{ N/mm}^2$

theoretical flexural stresses $\pm \frac{M_b}{Z_1} = \frac{10.6 \times 10^6}{0.0149 \times 10^9}$ $= \underline{\pm 0.711 \text{ N/mm}^2}$

Theoretical flexural tensile stress, f_{t1} $= \underline{-\ 0.620 \text{ N/mm}^2}$

From the above calculations the most onerous values (largest) of f_{t1} and f_{t2} are given by Case (b) (ii) and (i),

$f_{t1} = -\ 0.620 \text{ N/mm}^2$ and

$f_{t2} = -\ 0.197 \text{ N/mm}^2$

These values will be used to calculate the post-tensioning force required and its eccentricity.

(9) CALCULATE THE POST-TENSIONING FORCE AND ECCENTRICITY

The post-tensioning force will be positioned eccentrically within the wall section, in order to balance the different flexural tensile stresses induced in the asymmetric section shape by lateral wind pressure and suction. The basic theory of this process is indicated in Appendix C.

From Appendix C, calculation of P and e:

$$P = \frac{(f_{t1}Z_1) + (f_{t2}Z_2)}{Z_1 + Z_2} A$$

$$e = \left(\frac{1}{A} - \frac{f_{t2}}{P}\right) Z_2$$

Substituting the values appertaining to this example in the above equation:

$$P = \frac{(0.620 \times 0.0149 + 0.197 \times 0.0258) \times 0.351 \times 10^6}{(0.0149 + 0.0258) \times 10^3}$$

$= 123.5$ kN.

Therefore

$$e = \left(\frac{1}{0.351 \times 10^6} - \frac{0.197}{123.5 \times 10^3}\right) \times 0.0258 \times 10^9$$

$= 32$ mm.

Characteristic post-tensioning force, P_k

$$= \frac{P}{\gamma_f} = \frac{123.5}{0.9}$$

$= 137.2$ kN.

This force will be used later, in the check on compressive stresses in the wall and to establish the size of the post-tensioning rods. The post-tensioning force will be shared equally between 2 No. rods, one in each fin of the cross-section, as shown in Figure 4.9. Losses will be taken into account at the appropriate checks and design stages.

(10) CHECK COMBINED COMPRESSIVE STRESSES – AFTER LOSSES

Check local stability of fin and flange in the design of flexural compressive stresses.

The critical loading conditions are:

(i) dead + wind (partial safety factors for loads are $1.4G_k$ and $1.4W_k$), (ii) dead + superimposed + wind (partial safety factors for loads are $1.2G_k$, $1.2Q_k$ and $1.2W_k$).

Case (i) dead + wind

As stated previously, wind uplift cancels roof dead load.

Therefore, dead load = weight of brickwork only.

G_k at $\frac{3}{8}$h level $= 1.988 \times 6.67 = 13.26$ kN

G_k at base level $= 5.3 \times 6.67 = 35.35$ kN.

Case (ii) dead + superimposed + wind

Q_k per m (for roof span of 10 m)	$= 0.75 \times \frac{10}{2}$	$= 3.75$ kN/m run.
Therefore, Q_k on wall section	$= 3.75 \times 3$	$= 11.25$ kN
G_k at $\frac{3}{8}$h level	= as case (i)	$= 13.26$ kN
G_k at base level	= as case (i)	$= 35.35$ kN
Characteristic wind loads per double fin section:		
Case (a) suction	$= 0.45 \times 3$	$= 1.35$ kN/m
Case (a) pressure	$= 0.72 \times 3$	$= 2.16$ kN/m

DESIGN BENDING MOMENTS

Case (i) dead + wind

Case (a) suction

at $\frac{3}{8}$h level, M_w $= 3.73$ kN.m

at base level, M_b $= 6.64$ kN.m

Case (b) pressure

at $\frac{3}{8}$h level, M_w $= 5.96$ kN.m

at base level, M_b $= 10.6$ kN.m

Case (ii) dead + superimposed + wind

Case (a) suction

at $\frac{3}{8}$h level, M_w $= \frac{1.2 \times 3.73}{1.4} = 3.20$ kN.m

at base level, M_b $= \frac{1.2 \times 6.64}{1.4} = 5.69$ kN.m

Case (b) pressure

at $\frac{3}{8}$h level, M_w $= \frac{1.2 \times 5.96}{1.4} = 5.11$ kN.m

at base level, M_b $= \frac{1.2 \times 10.6}{1.4} = 9.09$ kN.m

COMBINED COMPRESSIVE STRESSES FOR CASE (A) SUCTION.

Case (i) dead + wind – at base level

Design axial stress due to self-weight, G_k

$$= + \frac{\gamma_f G_k}{A} = \frac{1.4 \times 35.35 \times 10^3}{0.351 \times 10^6} \qquad = + 0.141 \text{ N/mm}^2$$

Design axial stress due to post-tensioning force, P_k

$$= + \frac{\gamma_f P_k}{A} = \frac{1.4 \times 137.2 \times 10^3}{0.351 \times 10^6} \qquad = + 0.547 \text{ N/mm}^2$$

Design flexural stress due to post-tensioning force, P_k

$$= + \frac{\gamma_f P_k e}{Z_1} = \frac{1.4 \times 137.2 \times 32 \times 10^3}{0.0149 \times 10^9} \qquad = + 0.413 \text{ N/mm}^2$$

or,

$$= - \frac{\gamma_f P_k e}{Z_2} = - \frac{1.4 \times 137.2 \times 32 \times 10^3}{0.0258 \times 10^9} \qquad = - 0.238 \text{ N/mm}^2$$

Design flexural stress due to base wind moment,

$$= + \frac{M_b}{Z_1} = \frac{6.64 \times 10^6}{0.0149 \times 10^9} \qquad = + 0.446 \text{ N/mm}^2$$

or,

$$= - \frac{M_b}{Z_2} = - \frac{6.64 \times 10^6}{0.0258 \times 10^9} \qquad = - 0.257 \text{ N/mm}^2$$

Hence,

maximum combined compressive stress, $0.141 + 0.547 + 0.413 + 0.446 = + 1.547 \text{ N/mm}^2$

minimum combined compressive stress, $0.141 + 0.547 - 0.238 - 0.257 = + 0.193 \text{ N/mm}^2$

Case (ii) dead + superimposed + wind – at base level

Design axial stress due to $G_k + Q_k$

$$= + \frac{\gamma_f G_k + \gamma_f Q_k}{A} = \frac{(1.2 \times 35.35) + (1.2 \times 11.25)}{0.351 \times 10^3} \qquad = + 0.159 \text{ N/mm}^2$$

Design axial stress due to post-tensioning force, P_k

$$= + \frac{\gamma_f P_k}{A} = \frac{1.2 \times 137.2 \times 10^3}{0.351 \times 10^6} \qquad = + 0.469 \text{ N/mm}^2$$

Design flexural stress due to post-tensioning force, P_k

$$= + \frac{\gamma_f P_k e}{Z_1} = \frac{1.2 \times 137.2 \times 32 \times 10^3}{0.0149 \times 10^9} \qquad = + 0.354 \text{ N/mm}^2$$

or,

$$= - \frac{\gamma_f P_k e}{Z_2} = - \frac{1.2 \times 137.2 \times 32 \times 10^3}{0.0258 \times 10^9} \qquad = - 0.204 \text{ N/mm}^2$$

Design flexural stress due to base wind moment, M_b

$$= + \frac{M_b}{Z_1} = \frac{5.69 \times 10^6}{0.0149 \times 10^9} \qquad = + 0.382 \text{ N/mm}^2$$

or,

$$= - \frac{M_b}{Z_2} = - \frac{5.69 \times 10^6}{0.0258 \times 10^9} \qquad = - 0.221 \text{ N/mm}^2$$

Hence,

maximum combined compressive stress, $0.159 + 0.469 + 0.354 + 0.382 = + 1.364 \text{ N/mm}^2$

minimum combined compressive stress, $0.159 + 0.469 - 0.204 - 0.221 = + 0.203 \text{ N/mm}^2$

Case (i) dead + wind – at $\frac{3}{8}$ h level

Design axial stress due to G_k

$$= + \frac{\gamma_f G_k}{A} = \frac{1.4 \times 6.67 \times 1.988 \times 10^3}{0.351 \times 10^6} \qquad = + 0.053 \text{ N/mm}^2$$

Design axial stress due to P_k

$$= + \frac{\gamma_f P_k}{A} = \frac{1.4 \times 137.2 \times 10^3}{0.351 \times 10^6} \qquad = + 0.547 \text{ N/mm}^2$$

Design flexural stress due to P_k

$$= + \frac{\gamma_f P_k e}{Z_1} = \frac{1.4 \times 137.2 \times 32 \times 10^3}{0.0149 \times 10^9} \qquad = + 0.413 \text{ N/mm}^2$$

or,

$$= - \frac{\gamma_f P_k e}{Z_2} = - \frac{1.4 \times 137.2 \times 32 \times 10^3}{0.0258 \times 10^9} \qquad = - 0.238 \text{ N/mm}^2$$

Design flexural stress due to wall wind moment,

$$= -\frac{M_w}{Z_1} = -\frac{3.73 \times 10^6}{0.0149 \times 10^9} \qquad = -\ 0.250\ \text{N/mm}^2$$

or,

$$= +\frac{M_w}{Z_2} = \frac{3.73 \times 10^6}{0.0258 \times 10^9} \qquad = +\ 0.145\ \text{N/mm}^2$$

Hence,

maximum combined compressive stress, $0.053 + 0.547 + 0.413 - 0.250 = +\ 0.763\ \text{N/mm}^2$

minimum combined compressive stress, $0.053 + 0.547 - 0.238 + 0.145 = +\ 0.507\ \text{N/mm}^2$

Case (ii) dead + superimposed + wind – at $\frac{3}{8}$ h level

Design axial stress due to G_k and Q_k

$$= +\frac{\gamma_f G_k + \gamma_f Q_k}{A} = \frac{(1.2 \times 13.26) + (1.2 \times 11.25) \times 10^3}{0.351 \times 10^6} \qquad = +\ 0.084\ \text{N/mm}^2$$

Design axial stress due to P_k

$$= +\frac{\gamma_f P_k}{A} = \frac{1.2 \times 137.2 \times 10^3}{0.351 \times 10^6} \qquad = +\ 0.469\ \text{N/mm}^2$$

Design flexural stress due to P_k

$$= +\frac{\gamma_f P_k e}{Z_1} = \frac{1.2 \times 137.2 \times 32 \times 10^3}{0.0149 \times 10^9} \qquad = +\ 0.354\ \text{N/mm}^2$$

or,

$$= -\frac{\gamma_f P_k e}{Z_2} = -\frac{1.2 \times 137.2 \times 32 \times 10^3}{0.0258 \times 10^9} \qquad = -\ 0.204\ \text{N/mm}^2$$

Design flexural stress due to wall wind moment, M_w

$$= -\frac{M_w}{Z_1} = -\frac{3.20 \times 10^6}{0.0149 \times 10^9} \qquad = -\ 0.215\ \text{N/mm}^2$$

or,

$$= +\frac{M_w}{Z_2} = \frac{3.20 \times 10^6}{0.0258 \times 10^9} \qquad = +\ 0.124\ \text{N/mm}^2$$

Hence,

maximum combined compressive stress $= +\ 0.692\ \text{N/mm}^2$

minimum combined compressive stress $= +\ 0.473\ \text{N/mm}^2$

COMBINED COMPRESSIVE STRESS FOR CASE (B) PRESSURE

Case (i) dead + wind – at base level

Design axial stress due to G_k = as Case (a) $= +\ 0.141\ \text{N/mm}^2$

Design axial stress to P_k = as Case (a) $= +\ 0.547\ \text{N/mm}^2$

Design flexural stress due to P_k = as Case (a) $= +\ 0.413\ \text{N/mm}^2$

or $= -\ 0.238\ \text{N/mm}^2$

Design flexural stresses due to base wind moment, M_b

$$= -\frac{M_b}{Z_1} = -\frac{10.6 \times 10^6}{0.0149 \times 10^9} \qquad = -\ 0.711\ \text{N/mm}^2$$

or,

$$= +\frac{M_b}{Z_2} = \frac{10.6 \times 10^6}{0.0258 \times 10^9} \qquad = +\ 0.411\ \text{N/mm}^2$$

Hence,

maximum combined compressive stress $= +\ 0.861\ \text{N/mm}^2$

minimum combined compressive stress $= +\ 0.390\ \text{N/mm}^2$

Case (ii) dead + superimposed + wind at base level

Design axial stress due to G_k and Q_k = as Case (a) $= +\ 0.159\ \text{N/mm}^2$

Design axial stress to P_k = as Case (a) $= +\ 0.469\ \text{N/mm}^2$

Design flexural stresses due to P_k = as Case (a) $= +\ 0.354\ \text{N/mm}^2$

or $= -\ 0.204\ \text{N/mm}^2$

Design flexural stresses due to base wind moment, M_b

$$= -\frac{M_b}{Z_1} = -\frac{9.09 \times 10^6}{0.0149 \times 10^9} \qquad = -\ 0.610\ \text{N/mm}^2$$

or,

$$= +\frac{M_b}{Z_2} = \frac{9.09 \times 10^6}{0.0258 \times 10^9} \qquad = +\ 0.352\ \text{N/mm}^2$$

Hence,

maximum combined compressive stress $= +\ 0.776\ \text{N/mm}^2$

minimum combined compressive stress $= +\ 0.372\ \text{N/mm}^2$

Case (i) dead + wind at $\frac{3}{8}$ h level

Design axial stress due to G_k = as Case (a)	$= + 0.053$ N/mm^2
Design axial stress to P_k = as Case (a)	$= + 0.547$ N/mm^2
Design flexural stresses due to P_k = as Case (a)	$= + 0.413$ N/mm^2
	or $= - 0.238$ N/mm^2

Design flexural stresses due to wall wind moment, M_w

$$= + \frac{M_w}{Z_1} = + \frac{5.96 \times 10^6}{0.0149 \times 10^9} = + 0.400 \text{ N/mm}^2$$

or,

$$= - \frac{M_w}{Z_2} = - \frac{5.96 \times 10^6}{0.0258 \times 10^9} = - 0.231 \text{ N/mm}^2$$

Hence,

maximum combined compressive stress	$= + 1.413$ N/mm^2
minimum combined compressive stress	$= + 0.131$ N/mm^2

Case (ii) dead + superimposed + wind at $\frac{3}{8}$ h level

Design axial stress due to G_k = as Case (a)	$= + 0.084$ N/mm^2
Design axial stress to P_k = as Case (a)	$= + 0.469$ N/mm^2
Design flexural stresses due to P_k = as Case (a)	$= + 0.354$ N/mm^2
	or $= - 0.204$ N/mm^2

Design flexural stresses due to wall wind moment, M_w

$$= + \frac{M_w}{Z_1} = \frac{5.11 \times 10^6}{0.0149 \times 10^9} = + 0.343 \text{ N/mm}^2$$

or,

$$= - \frac{M_w}{Z_2} = - \frac{5.11 \times 10^6}{0.0258 \times 10^9} = - 0.198 \text{ N/mm}^2$$

Hence,

maximum combined compressive stress	$= + 1.250$ N/mm^2
minimum combined compressive stress	$= + 0.151$ N/mm^2

The two most critical cases for design, for wind suction and for wind pressure, are shown below.

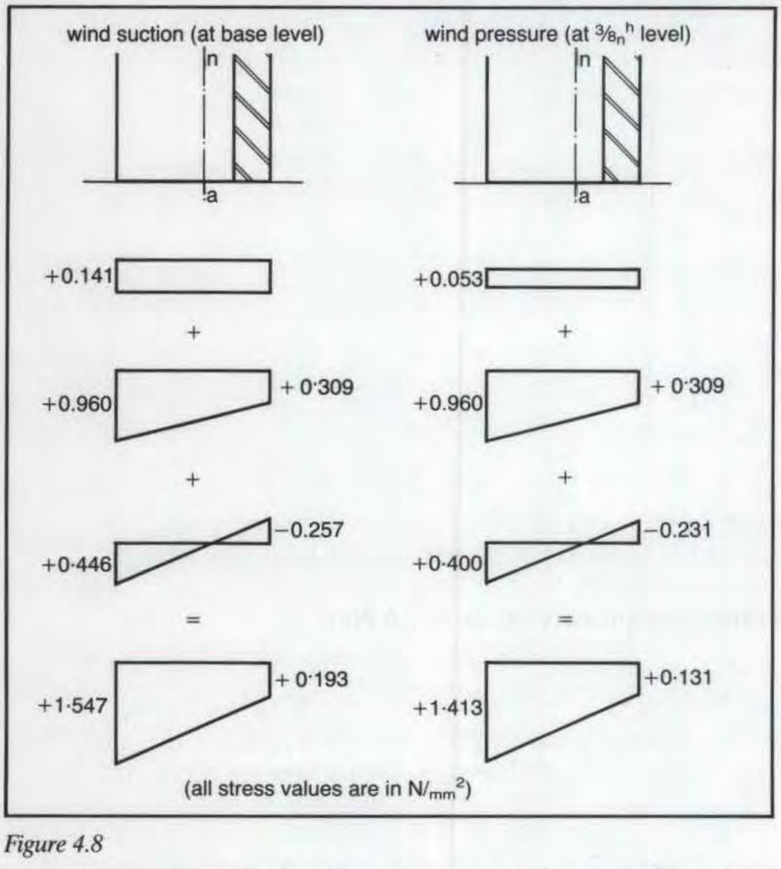


Figure 4.8

Comparison of the design flexural compressive strengths of the wall, based on local stability of fin and flange with the previously calculated combined compressive stresses, shows that the wall is acceptable for all loading cases thus far. The overall stability of the wall will be checked, both before and after losses in the post-tensioning force, for the loading condition of dead + superimposed + post-tensioning force.

The capacity reduction factor applicable in this case will be as calculated in (5), ie, $\beta = 0.62$.
Design strength of wall, after losses = 2.083 N/mm^2
Design strength of wall, before losses = 2.604 N/mm^2

(11) CHECK DESIGN STRESS IN WALL – BEFORE AND AFTER LOSSES
Design stress in wall, before losses

$$= \frac{1.4G_k + 1.6Q_k + \frac{1.4P_k}{0.8}}{A}$$

P_k is increased by 25% in anticipation of losses. G_k is the sum of the dead loads, roof dead load taken as 0.5 kN/m^2 in this example.

$$= \frac{1.4\,[(35.35) + (0.5 \times 3 \times 5)] + (1.6 \times 11.25) + (1.4 \times 171.5)\,10^3}{0.351 \times 10^6}$$

$= +\ 0.906$ N/mm^2, compared to design strength = 2.604 N/mm^2

Design stress in wall after losses

$$= \frac{1.4G_k + 1.6Q_k + 1.4P_k}{A}$$

$$= \frac{[(1.4 \times 42.85) + (1.6 \times 11.25) + (1.4 \times 137.2)]\,10^3}{0.351 \times 10^6}$$

$= +\ 0.769$ N/mm^2, compared to design strength = + 2.083 N/mm^2

(12) DESIGN OF POST-TENSIONING RODS
The design stress in the high yield steel rods will be limited to:

$$\frac{0.7f_y}{\sigma ms} = \frac{0.7f_y \times 460}{1.15} = 280 \text{ N/mm}^2.$$

The post-tensioning force required before losses

$$= \frac{137.2}{0.8} = 171.5 \text{ kN.}$$

Steel area required

$$= \frac{171.5 \times 10^3}{280} = 613 \text{ mm}^2$$

This will be provided by 2 No. 25 dia. rods, see figure 4.9.

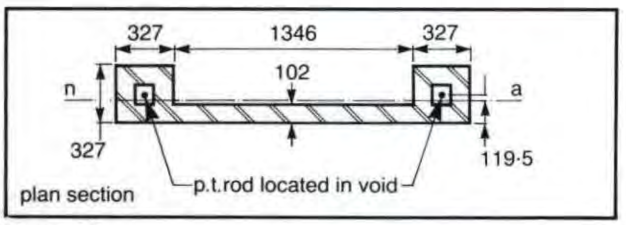


Figure 4.9

$$\text{Torque required} = \frac{\text{bolt tension} \times \text{bolt diameter}}{5}$$

$$= \frac{171.5 \times 25}{2 \times 5 \times 9.81} = 44 \text{ kgf/m}$$

(13) DESIGN OF ANCHORAGE AND SPREADER PLATE
Area of spreader plate required

$$= \frac{171.5 \times 10^3}{2 \times 7.6}$$

$= 11283$ mm^2

(rods will be taken up through precast concrete capping beam, which forms both the seating for the roof beams and the post-tensioning spreader plates.
Allowable direct compressive stress = 7.6 N/mm^2)

Therefore use, say, 150 × 150 minimum plate size, by 10mm thick.

Depth of the precast concrete capping beam will be governed by anchor bolt lengths of the roof beams bearing onto it. The lower anchorage could be formed using a similar size spreader plate cast into the concrete footing.

EXAMPLE 3

EARTH RETAINING DIAPHRAGM WALL

The external works surrounding a new office development include a split-level car parking area, and the landscape architect wishes to separate these areas by means of planting boxes. It is proposed to use post-tensioned diaphragm walls to form the retaining wall between the two levels, and at the same time provide the facility of brick planting boxes for the landscaping. The maximum difference in car park levels is 1.4m, the retained earth height being 1.7m (see Figure 4.10).

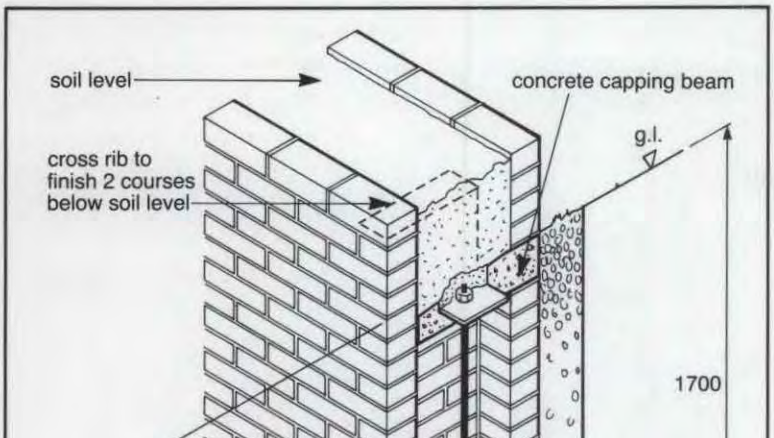


Figure 4.10

The wall is to be constructed with engineering facings Class A, with a crushing strength of 50 N/mm^2, and a water absorption of less than 7% set in a designation (i) mortar. A density of 19 kN/m^3 will be assumed. The surcharge from the cars at higher level will be taken as 10 kN/m^2. Density of granular backfill is assumed as 18 kN/m^3, the angle of internal friction, $\Phi = 30°$, The retained fill material will be adequately drained, and no build up of water pressure is expected.

As the applied moment from the retained earth occurs in one direction only, the post-tensioning force will be positioned eccentrically to counter it, and hence produce the most economic design.

Basic steps involved in calculation

(1) Determine characteristic applied loadings.
(2) Calculate design loads.
(3) Choose trial diaphragm wall section and calculate properties.
(4) Estimate of deflection.
(5) Calculate theoretical flexural tensile stresses.
(6) Determine the post-tensioning force and the eccentricity required.
(7) Determine capacity reduction factors.
(8) Determine design strengths of wall.
(9) Check combined compressive stresses.
(10) Check shear between cross-rib and abutting brickwork leaf (flange).
(11) Check principal tensile stress.
(12) Design of post-tensioning rods.
(13) Anchorage and spreader plate design.

(1) DETERMINE CHARACTERISTIC APPLIED LOADING

Earth pressure, using Rankine's formula, earth pressure at any level = $K_1 \times$ density $\times$ height

where K_1 $= \dfrac{1 - \sin \sigma}{1 + \sin \sigma}$

$\sigma = 30°$ in this example.

Therefore K_1 $= 0.333$, refer to Figure 4.11.

Maximum earth pressure at base of wall
$= 0.333 \times 18 \times 1.7$ $= 10.19$ kN/m^2.
Surcharge from vehicles $= 10.00$ kN/m^2.
Both of the above loadings will be classed as superimposed loadings, a load factor of 1.6 being applied.

Self-weight of brickwork $= \dfrac{19 \times 1.7}{10^3}$
$= 0.0323$ N/mm^2

Temporary case, with no applied dead load from soil in planter, will be considered.

Design earth pressure at base of wall $= 1.6 \times 10.19$
$= 16.30$ kN/m^2.
Surcharge $= 1.6 \times 10$
$= 16.0$ kN/m^2.
Design dead load stress at base wall of $= 0.9 \times 0.0323$
$= 0.029$ N/mm^2.

Design bending moment

$$= \left(16 \times 1.7 \times \frac{1.7}{2}\right) + \left(16.3 \times \frac{1.7}{2} \times \frac{1.7}{3}\right) = 30.97 \text{ kN m.}$$

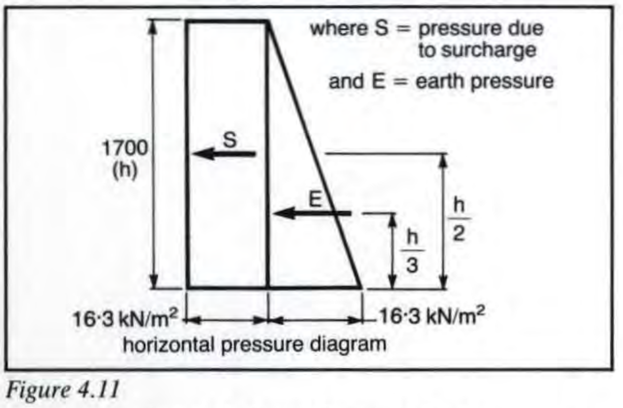


Figure 4.11

Design horizontal shear force at base,
surcharge + earth pressure $= (16 \times 1.7) + \left(16.3 \times \dfrac{1.7}{2}\right)$

$= 41.06$ kN/m.

(3) CHOOSE TRIAL DIAPHRAGM WALL SECTION AND CALCULATE PROPERTIES

Three variables of dimensions require consideration in order to arrive at a trial section for further analysis:

(a) the overall depth of the wall, D;

(b) the thickness of the wall flanges, T; and the spacing of the cross-ribs, B_r;

(c) the thickness of the cross-ribs, t_r.

(a) Overall depth, D

There is not a simple rule-of-thumb method available for reasonable assessment of the overall depth. The designer must balance the stability benefits of a deeper wall section against the space requirements, the quantity of walling materials, the effect on the size of the post-tensioning rods, and the magnitude of the post-tensioning force. As with many areas of design, experience in the use of a design process will lead to reasonable assessment of the overall depth, D, for the trial section.

As an appropriate guide to a first estimate of suitable overall depth, the section sizes obtained from Figures 32 & 33 and Table 1 (ref. 5), as for a non-post-tensioned diaphragm wall, reduced by approximately 25% (to the nearest brick dimension) should be reasonable. For the example under consideration, we have chosen an overall depth of 665mm, which will allow sufficient space between front and rear walls to be utilised as a planting box, whilst producing an economic structure.

(b) Flange thickness, T, and cross-ribs spacing, B_r

The wall flange, on the earth face, is required to span horizontally between cross-ribs, to withstand earth pressures and transfer them to the cross-ribs. Due to the pressure distribution on the earth face of the wall, and the tendency for the wall flange to act as a cantilever for a certain height, rather than span horizontally, we believe it unreasonable to use the maximum pressure at the base of the wall, when determining the cross-rib centres and wall flange thickness.

In figure 4.12, the assumed basis for designing the thickness of the wall flange and spacing of the cross-ribs is shown.

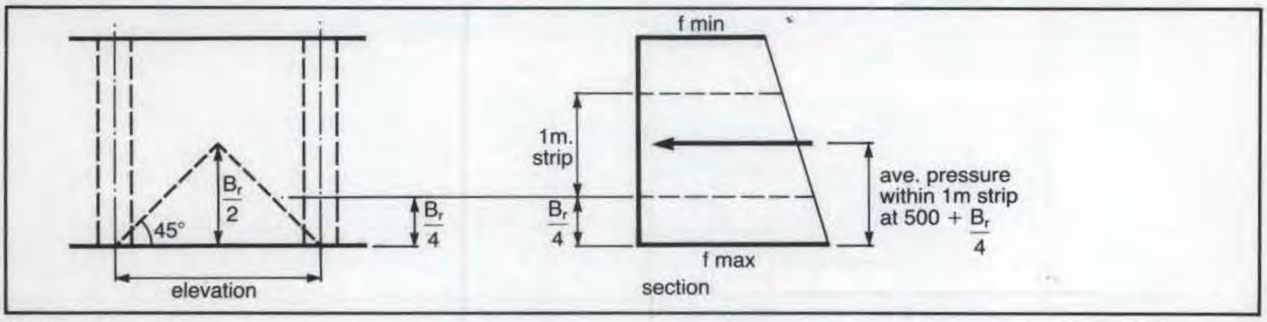


Figure 4.12

For this example, assume minimum B_r = 600 mm. Therefore, height to the average pressure from base of wall on 1m strip of the wall flange = 150 + 500 = 650mm.

For pressure distribution, that at 650mm above foundation, is:

$$16 + \frac{[16.3 \times (1.7 - 0.65)]}{1.70}$$

$= 26.07$ kN/m^2.

The design of the wall flange is shown below.

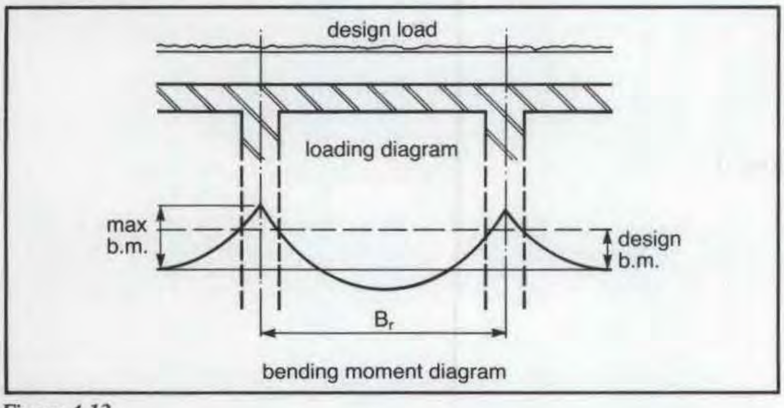


Figure 4.13

Maximum B.M. $= \dfrac{\gamma_f Q_k B_r^{\,2}}{-15}$ Assuming full fixity. At end bays and movement joints may be less.

Design B.M. (estimated) $= \dfrac{\gamma_f Q_k B_r^2}{15} = \dfrac{26.06 \times B_r^2}{15}$

$= 1.74\, B_r^2.$

Design MR $= \dfrac{f_{kx} Z}{\gamma_{mm}} = \dfrac{2 \times 1.0 \times 0.102^2 \times 10^3}{2.5 \times 6}$

$= 1.39$ kNm/m.

Therefore, B_r $= \sqrt{\dfrac{1.39}{1.74}}$

$= 894$ mm, ie, maximum span of flange.

The nearest suitable bond arrangement for this type of construction is 675mm. Depending on the shear check for the cross-rib size and centres, 675mm rib spacing will be assumed.

(c) Cross-rib thickness, t_r

The resistance to horizontal shear tends to be the governing factor for calculating the cross-rib thickness.

Shear force per cross-rib,

V $= 41.06 \times 0.675$ (calculated at base of wall)

$= 27.72$ kN.

The maximum horizontal shear stress occurs on the centroid of the overall section, and may be derived from the formula:

$$v_h = \frac{V A_1 \bar{y}}{I_{na} t_r}$$

Where,

v_h = horizontal shear stress

V = applied horizontal shear force

A_1 = half the cross-sectional area (shown hatched in Figure 4.14)

$\bar{y}$ = distance from N.A. to centroid of area A_1 (see Figure 4.14)

I_{na} = second moment of area about N.A.

t_r = thickness of cross-ribs.

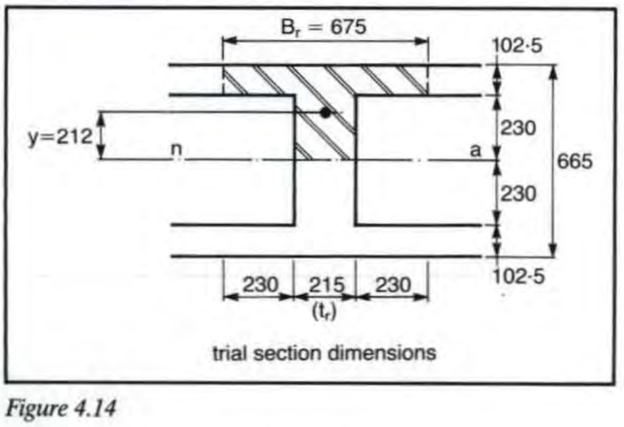


Figure 4.14

For this example,

V $= 27.72$ kN per cross-rib

$A_1 = 0.119\ m^2$

$$\bar{y} = \frac{(0.1025 \times 0.675 \times 0.28125) + (0.215 \times 0.230 \times 0.115)}{0.119}$$

$= 0.211$ m

$$I_{na} = \frac{0.675 \times (0.665)^3}{12} - \frac{0.460 \times (0.460)^3}{12}$$

$= 12.8 \times 10^{-3}\ m^4.$

$t_r = 0.215$ m (assumed for trial purposes).

Hence,

$$v_h = \frac{27.72 \times 0.119 \times 0.212}{12.8 \times 0.215}$$

$= 0.254$ N/mm^2.

N.B. Alternative method as described in BS 5628: Part 2: cl.28.3:

$$v_h = \frac{V}{b\ dc}$$

where V is the shear force due to the design loads; b is the width of the section for rectangular sections, or width of web, for T and I sections; dc is the depth of masonry in compression.

Therefore,

$$v_h = \frac{27.72 \times 10^3}{215 \times 665}$$

$= 0.194$ N/mm^2 (ie. some 25% lower than the authors' preferred method as above).

From BS 5628: Part 1:c1.25 shear resistance $= \frac{f_v}{\gamma_{mv}}$

where $f_v = 0.35 + 0.6g_A$ N/mm^2 for walls built with mortar designations (i) and (ii), and where g_A is the summation of the self-weight of brickwork plus imposed loads.

In this example, the imposed load – the post-tensioning force – is not yet known. However, a minimum post-tensioning force may be calculated, to provide the required horizontal shear resistance, and later checked against the minimum post-tensioning force required to eliminate the flexural tensile stress due to bending. The larger of the two forces should be used in the design.

$$v_h = \frac{0.35}{\gamma_{mv}} + \frac{0.6g_A}{\gamma_{mv}}$$

(max. permissible value = 1.75 N/mm^2)
(for class (i) & (ii) for $\gamma_{mv} = 2.5$)

Therefore, $0.254 = \frac{0.35}{2.5} + \frac{0.6g_A}{2.5}$

and hence, $g_A = 0.475$ N/mm^2.

But $g_A = g_d$ + design post-tensioning stress,

therefore, $g_d = \frac{0.9G_k}{A}$

where $G_k = 2 \times 0.119 \times 1.7 \times 19 = 7.69$ kN/cell (weight of soil in plant box ignored) ie,

$$G_k/m = \frac{7.69}{0.675} = 11.39 \text{ kN/m}$$

And $A = 2 \times 0.11g = 0.238$ m^2 [for the trial section.]

$$= \frac{0.9 \times 7.69 \times 10^3}{0.238 \times 10^6}$$

$= 0.029$ N/mm^2.

Hence, $g_A = 0.029$ + design post-tensioning stress.

Therefore, design post-tensioning stress $= 0.475 - 0.029 = 0.446$ N/mm^2.

Whilst the design post-tensioning stress varies across the section, depending on its eccentricity, it is assumed at this stage that the maximum value of the horizontal shear occurs where the design post-tensioning stress has its average value. (This can be checked later, when the post-tensioning force and eccentricity are known.)

Hence,

design post-tensioning force $= 0.446 \times A$
$= 0.446 \times 0.238 \times 10^3$
$= 106$ kN per cross-rib
$= 157$ kN/m.

The trial section, as shown in figure 4.14, will be adopted to check brickwork stresses and to design the post-tensioning rods.

(4) ESTIMATE OF DEFLECTION OF WALL SECTION ADOPTED

Conditions of structure and loading assumed are a free cantilever with triangular distributed applied loading.

$\triangle\text{max.} = \frac{W L^3}{15E_m I}$ where E_m will be taken as the long term, $E_m = 0.45f_k$ kN/mm^2. due to the long term nature of the applied loading.

$$\triangle\text{max.} = \frac{41.06 \times 1.7^3 \times 10^{12}}{15 \times 0.45 \times 15 \times 12.08 \times 10^{12}}$$

$= 0.16$ mm, this is acceptable because $\frac{L}{125}$, ie, 14 mm.

(5) CALCULATE THEORETICAL FLEXURAL TENSILE STRESSES

For one metre width:

$$f_t = \frac{0.9G_k}{A} - \frac{M_b}{Z}$$

$$= \frac{0.9 \times 11.39 \times 10^3}{0.353 \times 10^6} - \frac{30.97 \times 10^6}{57.03 \times 10^6}$$

$= 0.029 - 0.543.$

Where $Z = \dfrac{\frac{675 \times 665^3}{12} - \frac{460 \times 460^3}{12}}{332.5}$

$= 38.5 \times 10^6$mm^3 (per cell)

For Z/metre width,

$$Z = \frac{1000}{675} \times 38.5 \times 10^6$$

$= 57.03 \times 10^6$mm^3

Therefore,

$f_t = -0.514$ N/mm^2, and

$f_c = +0.029$ N/mm^2 due to swt only. Maximum compressive stress given by $\frac{0.9G_k}{A} + \frac{M_b}{Z}$

An eccentric post-tensioning force is therefore required to produce a maximum tensile stress = f_c

= 0·029 N/m^2

and a maximum compressive stress = $- f_t = f_o$ .514 N/mm^2.

(6) CALCULATION OF POST-TENSIONING FORCE AND ECCENTRICITY – AFTER LOSSES

Minimum required post-tensioning force based on bending stress:

$P = -(f_t + f_c)\dfrac{A}{2}$, (for derivation of formulae see Appendix B),

where $f_t = -0.514$ N/mm^2,

$f_c = +0.029$ N/mm^2,

$A = 0.353 \times 10^6$ mm^2 $\left(\text{ie, } \dfrac{1000}{675} \times 0.238 \times 10^6 \text{ mm}^2\right)$.

Hence $P = -(-0.514 + 0.029)\dfrac{0.353}{2} \times 10^6 = 85.6$ kN/m.

This force is less than that required to ensure adequate shear resistance (157 kN/m as calculated previously.) The higher value is adopted for subsequent calculations.

Determination of required eccentricity of applied post-tensioning force:

$$e = \left(\frac{-f_t}{P} - \frac{1}{A}\right) Z$$

$$= \left(\frac{-(0.514)}{157 \times 10^3} - \frac{1}{0.353 \times 10^6}\right) \times 57.03 \times 10^6$$

= 25mm

This is well inside the maximum practical eccentricity of the trial section, ie,:

$$= \frac{(665 - 2 \times 102)}{2} - \frac{\text{rod diameter}}{2} - \text{minimum cover to pt rod}$$

$$= 231 - \frac{\text{rod diameter}}{2} - 50 \text{ say}$$

= 169mm

Characteristic post-tensioning force, P_k,

$= \dfrac{P_t}{\gamma_f}$ where γ_f is the partial safety factor for the applied load. In the absence of further research, we suggest that for post-tensioning forces a similar value of γ_f as for dead load should be applicable, ie, $\gamma_f = 0.9$.

Hence, $P_k = \dfrac{157}{0.9}$

= 174 kN/m.

This force will now be used to check the design compressive stresses in the wall, and to establish the size of the post-tensioning rods. (An increase of 25% will be applied to the post-tensioning force, P_k, when considering stresses, before losses in prestress have occurred.)

If calculation required an eccentricity > than that practical for the section, the post-tensioning force must be increased so as to bring the eccentricity calculated withing practical limits. The new force required,

$P = \dfrac{-f_t}{\dfrac{1}{A} + \dfrac{e}{Z}}$ where e is the maximum practical eccentricity of the section.

(7) CAPACITY REDUCTION FACTORS

The overall stability of the wall section and the local stability of the flanges (leaves) will be checked under combined axial and flexural loading.

(a) Overall stability

Effective height, h_{ef} = 2h = 2 × 1700 = 3400mm (free cantilever)

Effective thickness, t_{ef} = actual thickness = 665mm.

Slenderness ratio, S.R. $= \dfrac{3400}{665} = 5.1$

Effective eccentricity $= \dfrac{P_k e}{P_k + \text{design dead load}}$

$= \dfrac{174 \times 0.025}{174 + 10.25}$

= 0.024m

Effective eccentricity $= \frac{24}{665}$

ie. $< 0.05t$.

Hence, for S.R. = 5.1, $e \leqslant 0.05t$, from BS 5628: Part 1, Table 7, $\beta = 1.0$.

(b) Local stability

The flange is restrained against buckling by the cross-ribs, which may be taken to constitute enhanced resistance to lateral movement. Hence, slenderness ratio:

$$= \frac{h_{ef}}{t_{ef}} = \frac{0.75 \times 675}{102}$$
$$= 4.9$$

It is assumed that combined axial and flexural stresses are applied to the flange with zero eccentricity for the purposes of the calculation.

Hence, $\beta = 1.0$.

(8) DETERMINE DESIGN STRENGTH OF THE WALL

Under dead load + post-tensioning force, before losses.

Design strength before losses:

$$= \frac{1.2\,(1.25\beta f_{ki})}{\gamma_{mm}}$$

(where factor 1.2 is for flexural compressive strength, and 1.25 for 20% increase, before loss, in prestress force)

$$= \frac{1.2\,(1.25 \times 1.0 \times 15)}{2.5}$$
$$= 9.0 \text{ N/mm}^2$$

The wall is assumed to have achieved its full characteristic strength at the time of post-tensioning, $f_{ki} = f_k$ (see Table 2, BS 5628; Part 1). (N.B. for this check, the value of β is that of the flange – local stability.)

Design strength after losses:

$= \frac{1.2\beta f_k}{\gamma_{mm}}$ (where factor 1.2 is for flexural compressive strength)

$$= \frac{1.2 \times 1.0 \times 15}{2.5}$$
$$= 7.2 \text{ N/mm}^2.$$

Alternatively, if eccentricity of prestress and dead load are catered for in selection of β factor, and combined axial stress only is compared to design strength:

$$= \frac{\beta f_k}{\gamma_{mm}} = \frac{1.0 \times 15}{2.5}$$
$$= 6.0 \text{ N/mm}^2.$$

The value of β, in this case and the one below, is that for the whole wall, ie, overall stability.

Design strength of wall, after losses, under dead + superimposed + post-tensioning force:

$$= \frac{1.2\beta f_k}{\gamma_{mm}} = \frac{1.2 \times 1.0 \times 15}{2.5}$$
$$= 7.2 \text{ N/mm}^2.$$

(9) CHECK COMBINED COMPRESSIVE STRESS

Consider the wall subject to dead loading + post-tensioning force only, before losses. In anticipation of 20% loss of post-tensioning force, the characteristic post-tensioning force, P_k, should initially be increased by 25%.

Hence,

Post-tensioning before losses $= \frac{174}{0.8}$

= 218 kN/m.

Design post-tensioning force before losses = $218 \times \gamma_f$

= 305 kN/m ($\gamma_f = 1.4$)

Maximum design dead load = G_k = 1.4 (11.39 + 4.16)

= 21.77 kN/m (includes soil for planting box).

Dead load = $(450 \times 353 \times 19) + (18 \times 0.300 \times 0.460 \times 0.460)$

= 4.16 kN/m.

Maximum flexural compressive stress due to post-tensioning force, P_k, before losses:

$$= \frac{P}{A} + \frac{Pe}{Z} = \frac{305 \times 10^3}{0.353 \times 10^6} + \frac{305 \times 10^3 \times 25}{57.03 \times 10^6} = 0.864 + 0.134$$
$$= 0.998 \text{ N/mm}^2.$$

Minimum flexural compressive stress due to post-tensioning force, P_k, before losses:

$$= \frac{P}{A} - \frac{Pe}{Z}$$
$= 0.730\ N/mm^2$.

Axial stress due to maximum dead load $= \dfrac{21.77 \times 10^3}{0.353 \times 10^6}$
$= 0.062\ N/mm^2$

Combined stresses due to G_k and post-tensioning force, before losses.
Maximum combined stress $= 0.062 + 0.998 = 1.060\ N/mm^2$,
minimum combined stress $= 0.062 + 0.730 = 0.792\ N/mm^2$,
compared to design strength of $9.0\ N/mm^2$.

Consider stability of overall wall section under dead load + post-tensioning force, after losses.
Design axial load $= (\gamma_f G_k) + (\gamma_f P_k) = (1.4 \times 15.55) + (1.4 \times 174)$
$= 21.77 + 243.6$
$= 265.4\ kN/m$,

design axial stress $= \dfrac{265.4 \times 10^3}{0.353 \times 10^6}$
$= 0.752\ N/mm^2$,
compared to design strength of $6.0\ N/mm^2$.

The design strength exceeds the design stress, and clearly the wall is acceptable for this loading condition.

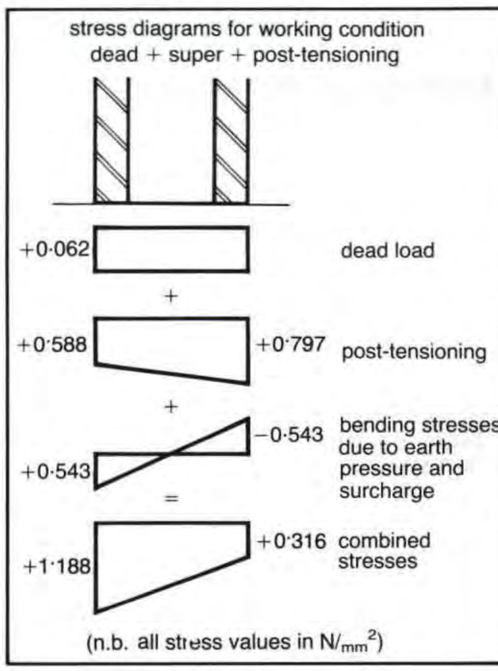
Consider wall, subject to dead + superimposed loading + post-tensioning force, after losses.

Axial stress due to G_k
$$= \frac{\gamma_f G_k}{A} = \frac{1.4 \times 15.55 \times 10^3}{0.353 \times 10^6}$$
$= +\ 0.062\ N/mm^2$.

Maximum flexural stress due to post-tensioning force, P_k,
$$= \frac{\gamma_f P_k}{A} + \frac{\gamma_f P_k e}{Z}$$
$$= \frac{1.4 \times 174 \times 10^3}{0.353 \times 10^6} + \frac{1.4 \times 174 \times 25 \times 10^3}{57.03 \times 10^6} = 0.690 + 0.107$$
$= 0.797\ N/mm^2$.

Minimum flexural stress due to post-tensioning force, P_k,
$$= \frac{\gamma_f P_k}{A} - \frac{\gamma_f P_k e}{Z} = 0.690 - 0.107$$
$= +\ 0.583\ N/mm^2$.

Flexural stress due to applied moment
$$= \pm \frac{M_b}{Z} = \frac{30.97 \times 10^6}{57.03 \times 10^6}$$
$= \pm\ 0.543\ N/mm^2$



Hence:
maximum combined compressive stress $= +\ 1.188\ N/mm^2$,
minimum combined compressive stress $= +\ 0.316\ N/mm^2$,
compared to design strength of $7.2\ N/mm^2$. See figure 4.15 for stress diagrams.

Check minimum combined stress in which γ_f for dead and post-tensioning force, after losses $= 0.9$.

Axial stress due to $G_k = \dfrac{0.9}{1.4} \times 0.062$
$= +\ 0.040\ N/mm^2$.

Maximum flexural stress due to post-tensioning force $= \dfrac{0.9}{1.4} \times 0.797$
$= +\ 0.512\ N/mm^2$.

Minimum flexural stress due to post-tensioning force $= \dfrac{0.9}{1.4} \times 0.583$
$= +\ 0.375\ N/mm^2$.

Flexural stress due to applied moment $= \pm \dfrac{M_b}{Z}$
$= \pm\ 0.543\ N/mm^2$.

Minimum combined compressive stress $= +\ 0.009\ N/mm^2$.

ie. NO TENSILE STRESS DEVELOPED

(10) CHECK SHEAR BETWEEN LEAF AND CROSS-RIB

Another critical section for checking shear stresses could be in the vertical plane at the junction of the cross-ribs and leaves, for other than fully bonded walls, see Figure 4.16.

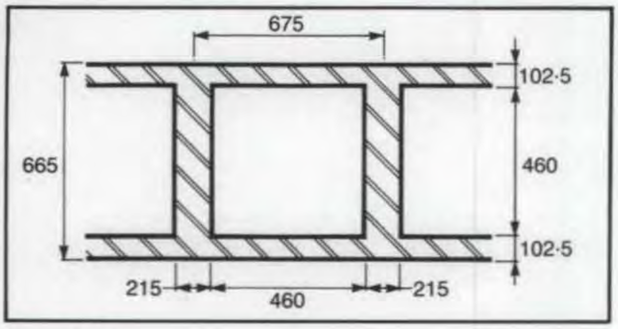


Figure 4.16

Area per cell = 0.238 m^2
Area per m = 0.353 m^2

Shear stress, $vh = \dfrac{V\,A_2\,\bar{y}}{I_{na}\,t_r}$

where V = design shear force = 27.72 kN/per cross-rib,
A_2 = area of leaf = 102.5 × 675 = 0.069 × 10^6 mm^2,
$\bar{y} = 230 + \dfrac{102.5}{2}$ = 281 mm,
I_{na} = Moment of Inertia = 12.8 × 10^{-3}m^4,
t_r = thickness of cross-rib = 215 mm.

Hence, $vh = \dfrac{27.72 \times 10^3 \times 0.069 \times 10^6 \times 281}{12.8 \times 10^9 \times 215}$
$= 0.195$ N/mm^2.

This stress, when multiplied by the stressed areas of brickwork between the chosen spacing of the shear connectors, can be used to design the required connectors. It should be noted that since horizontal and vertical shear stresses are the same value at any point, and since the vertical shear resistance of bonded masonry with a crushing strength of > 7N/mm^2 is greater than its horizontal resistance, for fully bonded masonry, only the horizontal shear stress need be checked.

(11) CHECK PRINCIPAL TENSILE STRESS

For maximum value of principal tensile stress in a diaphragm wall acting as a retaining wall, as in this example, the combined compressive stress under permanent loading and the associated shearing stress, will be used to calculate the principal tensile stress at the base of the wall.

Therefore, $\delta \min = \dfrac{\delta x}{2} - \sqrt{\left(\dfrac{\delta x}{2}\right)^2 + \tau_{xy}{}^2}$

where δx = 1.188 N/mm^2 (from page 63)
and τ_{xy} = 0.254 N/mm^2 (from page 59)

Therefore principal tensile stress $= \dfrac{1.188}{2} - \sqrt{\left(\dfrac{1.188}{2}\right)^2 + 0.254^2}$
$= -\,0.052$ N/mm^2, this is less than $\dfrac{fv}{2}$,
therefore is acceptable.

(12) DESIGN OF POST-TENSIONING RODS

Characteristic post-tensioning force required $= \dfrac{157}{0.9 \times 0.8}$
= 218 kN/m.
Characteristic post-tensioning force per cell = 218 × 0.675
= 147.15 kN.

In order to limit relaxation of the steel and hence minimise losses,
the stress in the rod is limited to $\dfrac{0.7f_y,}{\gamma_{ms}}$
where f_y = 460 N/mm^2, Grade steel strength,
γ_{ms} = partial safety factor for materials taken as 1.15 (ultimate state).

Therefore stress in rod is limited to $\dfrac{0.7 \times 460}{1.15} = 280$ N/mm^2.

$$\text{Area of rod required per cell} = \frac{147.15 \times 10^3}{280} = 526\ \text{mm}^2.$$

Therefore, either 2 No. T20 s, 628 mm^2
or 1 No. T32 s, 804 mm^2

Try 1 No. T32 diameter high tensile rod per cell.
Torque required to provide rod tension:

$$= \frac{\text{bolt tension} \times \text{bolt diameter},}{5}$$

$$\text{bolt tension (Kgf)} = \frac{147.15 \times 10^3}{9.81} \text{(per cell/bolt)},$$

$$= 15000\ \text{Kgf/rod}$$

$$\text{torque required} = \frac{15000 \times 0.032}{5}$$

$$= 96\ \text{Kgf/m}.$$

(13) ANCHORAGE/SPREADER PLATE DESIGN

Spreader plate design

Maximum design force per rod $= 147.15 \times \gamma_f = 147.15 \times 1.4$
$= 206$ kN.

Design compressive strength of wall $= 1.5 \times 6.0$
$= 9.0\ N/mm^2$.

Area of spreader plate required $= \frac{206 \times 10^3}{9.0} = 22889\ mm^2$
$= 151 \times 151$ mm minimum required.

Use, say, 200×200 mm plate.

N.B. Concrete capping beam should be designed to ensure that load is transferred to brickwork axially, and that no outward thrust is exerted by excessive deflection of concrete beam.

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21, 22 & 23. Bolton District General Hospital. *The link corridor was designed to have maximum natural lighting with full height windows, and to be constructed in brickwork, to match the predominant cladding/structural material throughout the hospital. The brick piers between the windows of the link corridor were post-tensioned, in order to carry the lateral window-loading which is very prominent in this location. The post-tensioned piers act as free cantilevers, as the pitched tiled roof structure is not capable of transferring load to any suitable shear walls or the like. Architect: Gilling Dodd Partnership. Consulting engineer: Curtins*

5 APPENDICES

APPENDIX A: DESIGN DATA

Table 5.1 Design loads: Partial safety factors, γ_f, (extract from BS 5628: Part 1).

(a) Dead and imposed load Design dead load = 0.9 G_k or 1.4 G_k Design imposed load = 1.6 Q_k
(b) Dead and wind load Design dead load = 0.9 G_k or 1.4 G_k Design wind load = 1.4 W_k or 0.015 G_k, whichever is the larger. *N.B. For freestanding walls or laterally loaded panels, whose removal would not affect the stability of the structure, γ_f applied to wind load may be taken as 1.2.*
(c) Dead, imposed and wind load Design dead load = 1.2 G_k Design imposed load = 1.2 Q_k Design wind load = 1.2 W_k or 0.015 G_k, whichever is the larger.
Where, G_k is the characteristic dead load, Q_k is the characteristic imposed load, W_k is the characteristic wind load, and the numerical values are the appropriate γ_f factor.

Table 5.2: Characteristic compressive strength of brickwork, f_k, in N/mm^2.

Mortar designation	Compressive strength of bricks, N/mm^2								
	7	10	15	20	27.5	35	50	70	100
(i)	3.4	4.4	6.0	7.4	9.2	11.4	15.0	19.2	24.0
(ii)	3.2	4.2	5.3	6.4	7.9	9.4	12.2	15.1	18.2

Table 5.3: Partial safety factors for materials, γ_{mm} and γ_{ms}

Brickwork strength: special category manufacturing control, γ_{mm}	= 2.5 (2.0)	The more experienced engineer may choose to adopt the lower figures (in brackets) in accordance with BS 5628: Part 2
normal manufacturing control, γ_{mm}	= 2.8 (2.3)	
Shear strength of brickwork, γ_{mm}	= 2.5 (2.0)	
Steel strength, γ_{ms}	= 1.15	

Table 5.4: Characteristic tensile strength of steel

Designation	Characteristic strength, f_y N/mm^2
Hot rolled steel and stainless steel, Grade 460 (BS 4449)	460
Cold worked steel Grade 460 (BS 4461)	460
Should wire strand or macalloy bars be adopted the designer should consult the manufacturer for details of strand/bar properties for varying diameters and types.	

Table 5.5: Capacity reduction factors, β

Capacity reduction factors, β for axial loading, ie. e = 0 – 0.05t	
Slenderness ratio h_{ef} t_{ef}	**Capacity reduction factor**
0	1.00
6	1.00
8	1.00
10	0.97
12	0.93
14	0.89
16	0.83
18	0.77
20	0.70
22	0.62
24	0.53
26	0.45
27	0.40
Maximum slenderness ratio 27	

The above table only includes values of β for eccentricity of applied loading of e = 0 – 0.05t. If the eccentricity is 0.05t, we would suggest checking the actual effect of this, by comparing the maximum combined compressive flexural stress against an allowable strength of 1.2 f_k. This seems preferable to using an approximation of the effect of the eccentricity, by adopting a lower factor from the table in BS 5628: Part 1, and checking against an allowable direct compressive strength of f_k.

Should the reader choose to adopt the latter approach, the values of β for a range of eccentrically applied loads are given in BS 5628: Part 1, Table 7.

APPENDIX B

Derivation of formulae used for determining post-tensioning force and eccentricity in a diaphragm wall

The design process is outlined in stress diagram form in Figure 5.1.

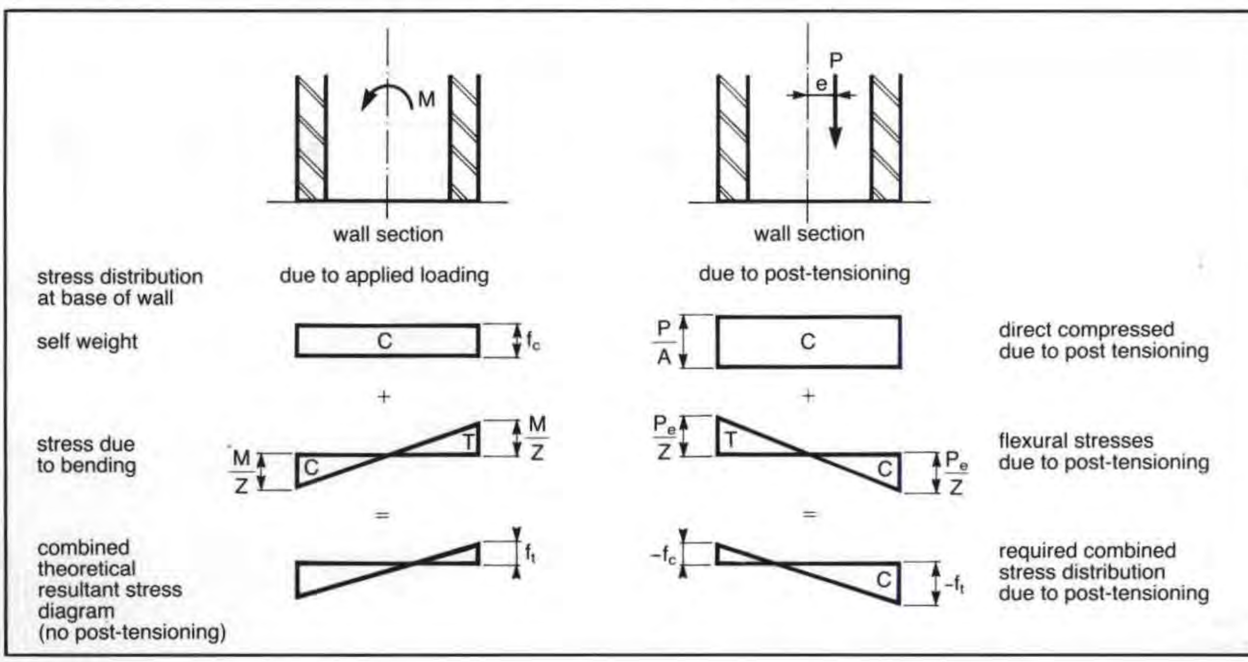


Figure 5.1

The design equations for the required post-tensioning force and its eccentricity may then be derived as follows:

$$\frac{P}{A} + \frac{Pe}{Z_1} = -f_t$$

$$\frac{P}{A} - \frac{Pe}{Z_2} = -f_c$$

but $Z_1 = Z_2$ for a diaphragm wall

$$\therefore \quad \frac{PZ}{A} + Pe = -f_tZ$$

and $$\frac{PZ}{A} - Pe = -f_cZ$$

Adding $$\frac{2PZ}{A} = -(f_t + f_c)Z$$

Rearranging $$P = -(f_t + f_c)\frac{A}{2}$$

This is the minimum value of P necessary to produce the required post-tensioning stress. The corresponding value of its eccentricity, e, may now be calculated by substituting P into one of the original equations, thus:

$$\frac{P}{A} + \frac{Pe}{Z} = -f_t$$

$$\frac{Pe}{Z} = -f_t - \frac{P}{A}$$

$$e = \left(-f_t - \frac{P}{A}\right)\frac{Z}{P}$$

$$e = \left(\frac{-f_t}{P} - \frac{1}{A}\right)Z$$

This equation provides the eccentricity corresponding to the minimum value of P already calculated. Such eccentricity may be found to be larger than can be accommodated within the trial section selected. In such a case, the maximum value of e which can be accommodated should be inserted into the latter equation, and a revised value of P obtained. This revised value of P will be larger than that originally calculated.

APPENDIX C

Derivation of forumulae used for determining post-tensioning force and eccentricity in a fin wall

In Figure 5.2:

P = design post-tensioning force,
A = area of effective fin section,
e = eccentricity of P about neutral axis,
Z_1 = minimum section modulus.
Z_2 = maximum section modulus.

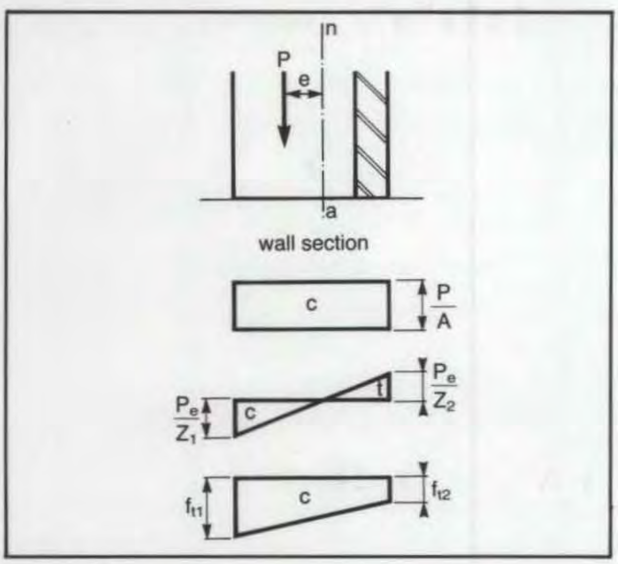


Figure 5.2

The stresses at each of the extreme edges of the section may be expressed as:

$$f_{t1} = \frac{P}{A} + \frac{Pe}{Z_1} \qquad \text{-----}(1)$$

$$f_{t2} = \frac{P}{A} - \frac{Pe}{Z_2} \qquad \text{-----}(2)$$

A pair of simultaneous equations may be written from equations (1) and (2) to solve for P and e. Multiply (1) by Z_1 and (2) by Z_2 giving:

$$f_{t1}Z_1 = \frac{PZ_1}{A} + Pe \qquad ----- (3)$$

$$f_{t2}Z_2 = \frac{PZ_2}{A} - Pe \qquad ----- (4)$$

Adding (3) and (4):

$$(f_{t1}Z_1) + (f_{t2}Z_2) = \frac{P}{A}(Z_1 + Z_2)$$

thus:

$$P = \frac{[(f_{t1}Z_1) + (f_{t2}Z_2)]A}{Z_1 + Z_2} \qquad ----- (5)$$

The value of P, calculated in equation (5), can now be substituted into equation (2) to find the value of e. Equation (2) transposed gives:

$$e = \left(\frac{1}{A} - \frac{f_{t2}}{P}\right)Z_2 \qquad ----- (6)$$

APPENDIX D

Importance of $\frac{Z}{A}$ and $\frac{r}{A}$

Comparison of sections of same area

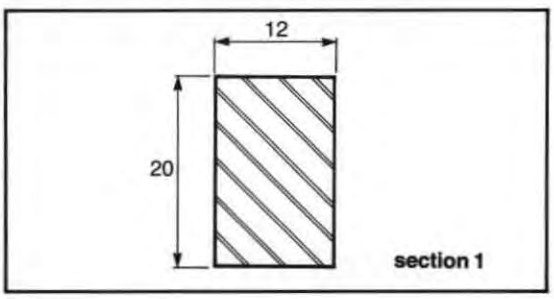


Figure 5.3

$$A = b \times d = 12 \times 20 = \underline{240}$$

$$Z_{xx} = \frac{b \times d^2}{6} = \frac{12 \times 20^2}{6} = 800$$

$$I_{xx} = \frac{b \times d^3}{12} = \frac{20 \times 12^3}{12} = 8000$$

$$I_{yy} = \frac{d \times b^3}{12} = \frac{12 \times 20^3}{12} = 2880$$

$$r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{2880}{240}} = 3.464$$

$$\frac{r_{yy}}{A} = \frac{3.464}{240} = 0.014$$

$$\frac{Z_{xx}}{A} = \frac{800}{240} = 3.33$$

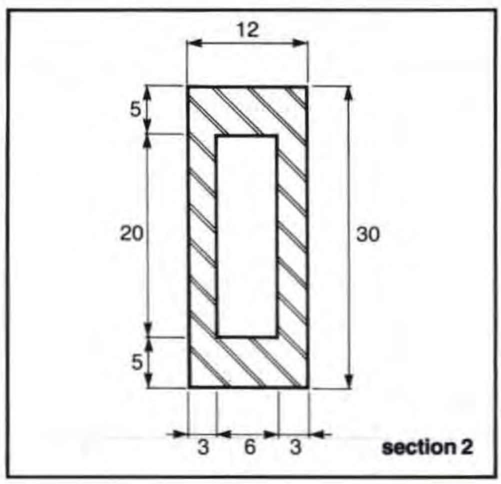


Figure 5.4

$$A = BD - bd = (12 \times 30) - (6 \times 20) = \underline{240}$$

$$Z_{xx} = \frac{I_{xx}}{D/_2} = \frac{23000}{15} = 1533.33$$

$$I_{xx} = \frac{BD^3}{12} - \frac{bd^3}{12} = \frac{12 \times 30^3}{12} - \frac{6 \times 20^3}{12} = 23000$$

$$I_{yy} = \frac{DB^3}{12} - \frac{db^3}{12} = \frac{30 \times 12^3}{12} - \frac{20 \times 6^3}{12} = 3960$$

$$r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{3960}{240}} = 4.06$$

$$\frac{r_{yy}}{A} = \frac{4.06}{240} = 0.017$$

$$\frac{Z_{xx}}{A} = \frac{1533.33}{240} = 6.39$$

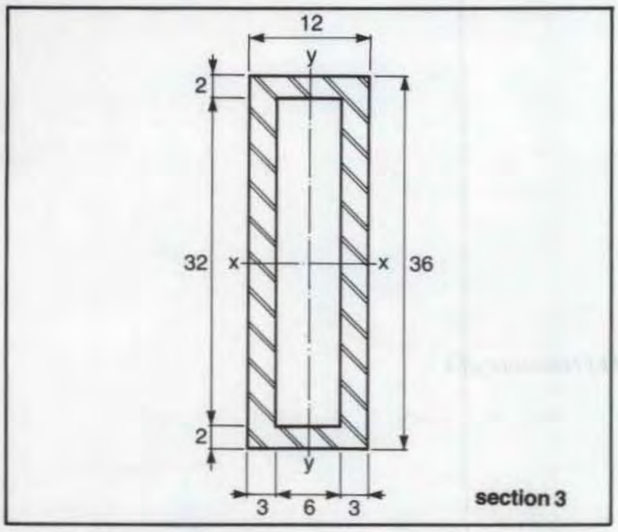


Figure 5.5

$$A = BD - bd = (12 \times 36) - (6 \times 32) = \underline{240}$$

$$I_{xx} = \frac{BD^3}{12} - \frac{bd^3}{12} = \frac{12 \times 36^3}{12} - \frac{6 \times 32^3}{12} = 30272$$

$$I_{yy} = \frac{DB^3}{12} - \frac{db^3}{12} = \frac{36 \times 12^3}{12} - \frac{32 \times 6^3}{12} = 4608$$

$$r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{4608}{240}} = 4.382$$

$$Z_{xx} = \frac{I_{xx}}{D/_2} = \frac{30272}{18} = 1681.78$$

$$\frac{r_{yy}}{A} = \frac{4.38}{240} = 0.018$$

$$\frac{Z_{xx}}{A} = \frac{1681.78}{240} = 7.01$$

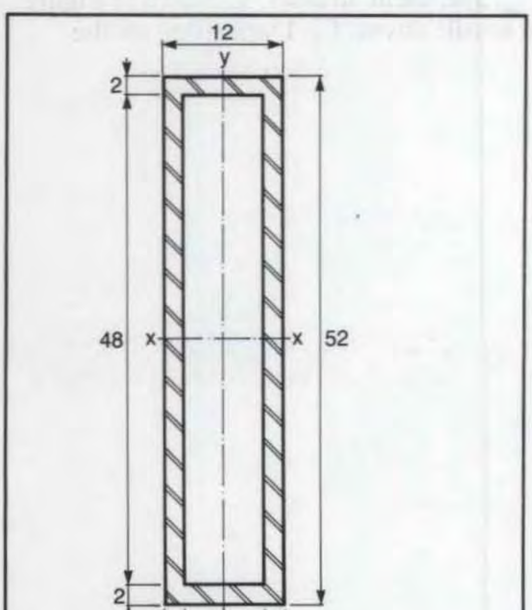


Figure 5.6

$$A = BD - bd = (12 \times 52) - (8 \times 48) = \underline{240}$$

$$I_{xx} = \frac{BD^3}{12} - \frac{bd^3}{12} = \frac{12 \times 52^3}{12} - \frac{8 \times 48^3}{12} = 66880$$

$$I_{yy} = \frac{DB^3}{12} - \frac{db^3}{12} = \frac{52 \times 12^3}{12} - \frac{48 \times 8^3}{12} = 5440$$

$$r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{5440}{240}} = 4.76$$

$$Z_{xx} = \frac{I_{xx}}{D/_2} = \frac{66880}{26} = 2572.31$$

$$\frac{r_{yy}}{A} = \frac{4.76}{240} = 0.02$$

$$\frac{Z_{xx}}{A} = \frac{2572.31}{240} = 10.72$$

Table 5.6: Summary of comparison

Section	$\frac{Z}{A}$	γ	$\frac{r_{yy}}{A}$	γ
1	3.33	1.0	0.014	1.00
2	6.39	1.92	0.017	1.21
3	7.01	2.11	0.018	1.29
4	10.72	3.22	0.020	1.43

Notes

A $M\gamma \frac{Z}{A}$ *section 4 for the same area has three times the moment of resistance of 1.*

B $P\gamma \frac{r}{A}$ *section 4 has 43% more compressive force resistance than 1 (assuming 1/A with f is a straight line graph).*

C *Sections 2, 3 and 4 give scope for easy placing of rods. Section 1 requires special bonding, is difficult for grouting and other practical considerations.*

D *Despite the seemingly obvious benefits of adopting geometric sections which maximise* $\frac{Z}{A}$ *and* $\frac{r}{A}$ *ratios, the present codes have been compiled with little or no reference to them.*

It is hoped that the engineers concerned with producing efficient cost-effective and practical designs will heed the above comments and use their common sense.

APPENDIX E

Principal stress

Some designers may need reminding of principal stress, and it is hoped that the following brief notes will prove helpful.

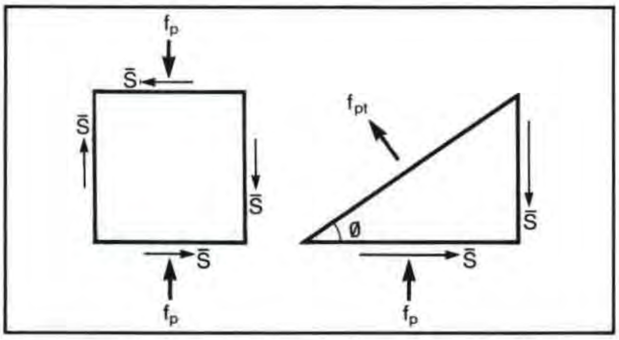


Figure 5.7

Consider the plane element subject to compressive stresses, f_p, and shear stresses, $\bar{s}$, shown in Figure 5.7. Across an inclined plane, equilibrium is maintained by a tensile stress, f_{pt}. Depending on the angle Φ, f_{pt} will vary, and it can be shown to be maximum – known as 'principal tensile stress' – where:

$$f_{pt} = \frac{f_p}{2} - \sqrt{\bar{s}^2 + \frac{f_p^{\,2}}{2}}$$

similarly, across another plane there will be a maximum principal compressive stress,

$$f_{pc} = \frac{f_p}{2} + \sqrt{\bar{s}^2 + \frac{f_p^{\,2}}{2}}$$

Mohr appreciated that the equations for principal stress were those of a circle, and many engineers (being more visually and graphically orientated) prefer to use 'Mohr's Circle' in analysis, rather than calculating equations.

The method is simple.

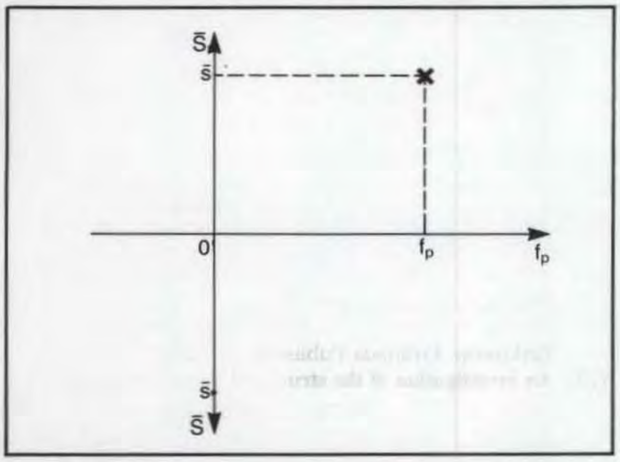
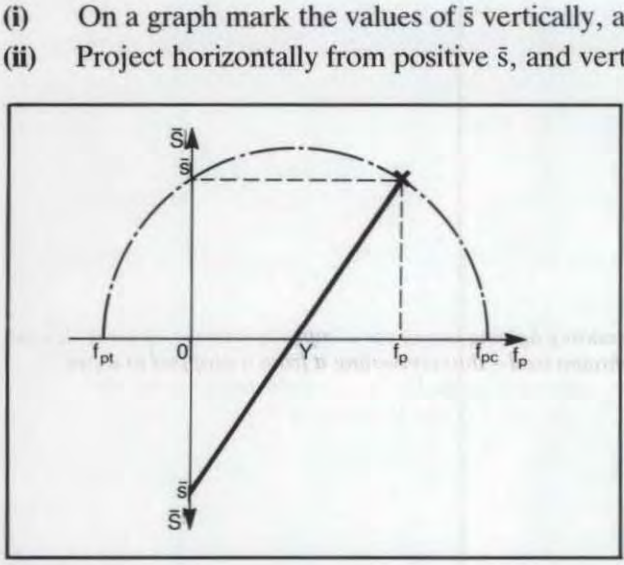


Figure 5.8

(i) On a graph mark the values of $\bar{s}$ vertically, above and below the origin, and f_p horizontally.

(ii) Project horizontally from positive $\bar{s}$, and vertically from f_p, and make intersection X.



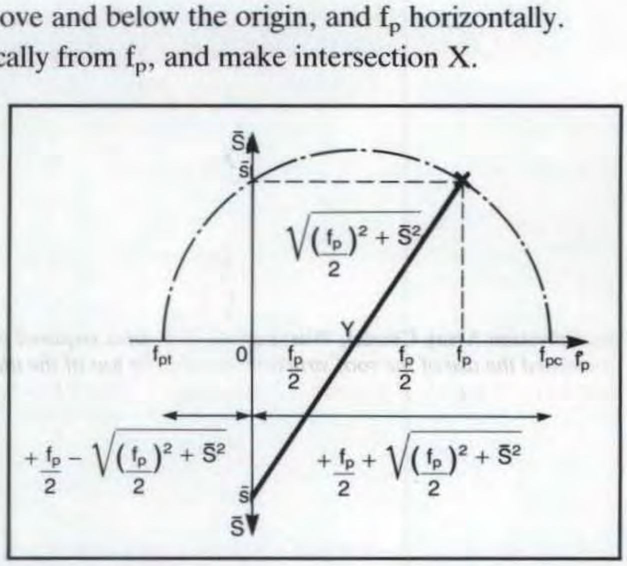


Figure 5.9

Figure 5.10

(iii) In Figure 5.9, from the intersection of line $\bar{s}$ to X with the horizontal axis, at point Y, strike an arc of radius YX to intercept the horizontal axis on both positive and negative sides. The negative intercept is f_{pt}, and positive f_{pc}.

Proof

(a) The two triangles OY$\bar{s}$ and Yf_pX, are identical, therefore the base, $OY = Yf_p = \frac{f_p}{2}$. Since the perpendicular is $Xf_p = \bar{s}$, the hypotenuse

$$YX = \sqrt{\bar{s}^2 + \frac{f_p^{\,2}}{2}}$$

AND THIS IS THE RADIUS OF THE ARC.

(b) $$- Of_{pt} + OY = \sqrt{\bar{s}^2 + \frac{f_p^{\,2}}{2}}$$

$$\text{but } OY = \frac{f_p}{2}$$

$$- Of_{pt} = \sqrt{\bar{s}^2 + \frac{f_p^{\,2}}{2}} - \frac{f_p}{2}$$

This is normally written (for sake of convenience):

$$f_{pt} = \frac{f_p}{2} - \sqrt{\bar{s}^2 + \frac{f_p^{\,2}}{2}}$$

(c) Similarly:

$$f_{pc} = \frac{f_p}{2} + \sqrt{\bar{s}^2 + \frac{f_p^{\,2}}{2}}$$

The principal compressive stress is usually well within the design compressive strength, but nevertheless it should be checked.

The principal tensile stress is often an important design criteria and must be verified. It is common to find in heavily loaded geometric sections that, whilst the section is adequate in compression, shear, etc, it is inadequate in principal tensile stress.

The common solution is to increase the cross-sectional area of the section at positions of high principal tensile stress. This can be achieved by increasing the width of the web, or closing up the centres of webs in the diaphragm wall[16].

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24. Salvation Army Citadel, Warrington. *The client required clerestorey lighting round the complete perimeter of the hall. This prevented the use of the roof structure to prop the top of the diaphragm wall – thus converting it from a propped to a free cantilever. A four-fold increase in bending moment and bending tensile stress resulted so, to eliminate tensile stress, the diaphragm wall was prestressed. Architect: Major D. Blackwell, The Salvation Army. Consulting engineer: Curtins*

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