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STRESS ANALYSIS OF MASONRY WALLS
SUBJECTED TO CONCENTRATED AMOUNTS OF PRESTRESSING

by

© Esmail Mohamed Fathy Naguib, B.Sc.

A thesis submitted to the
Faculty of Graduate Studies and Research
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for the degree of
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ABSTRACT

Loadbearing masonry walls have been used extensively in the last few decades as structural members not only for carrying gravity loads, but also for resisting lateral forces.

Modern practice for masonry require that reinforced masonry loadbearing and shear walls shall be reinforced horizontally and vertically with steel having a specified minimum area. It is also required that the reinforcing steel shall be distributed in each direction according to the given spacing limits.

Distributing the vertical reinforcing steel requires a considerable amount of labour and consequently constitutes a considerable share in the construction cost.

Using concentrated amounts of vertical reinforcing steel located at a few selected positions may be a practical way of reducing the required amount of work.

Moreover, if the concentrated vertical steel is prestressed, it would improve the behaviour of masonry shear walls in resisting lateral forces. Vertical prestressing will counteract the tensile stresses caused by lateral forces especially in the case of walls with light gravity loads. Furthermore, since the allowable horizontal shear stress in a masonry shear wall depends partly on the degree of precompression normal to the bed joints, vertical prestressing will increase the allowable shear stress.

To investigate the effect of using concentrated prestressed vertical steel on masonry walls, a two-dimensional finite element analysis was carried out. Two cases of steel locations were considered: Case 1 has prestressed bars located only at the ends of the wall, while Case 2 involves prestressed bars located at the ends as well as at the third points. The walls considered have height to width ratios ranging from 0.5 to 5.0, thus representing a range of squat to fairly slender walls.

Aside from a region in the vicinity of points of load application, a favorable precompression could be developed in the wall due to concentrated amounts of prestressing. In this way a wall can resist higher lateral loads with the same cross section or else resist the same load with a decrease of section.

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LIST OF SYMBOLS

E_m	Modulus of elasticity of masonry
f'_m	Ultimate compressive strength of masonry
H	Height of wall
t	Thickness of wall
W	Width of wall
ν	Poisson's ratio
δ	Displacement

CHAPTER 1

INTRODUCTION

1.1 General

Masonry is one of the oldest building materials. Evidence existing in many parts of the world shows that masonry has been used for over 5,000 years. However, only during the last few decades have scientific principles been applied to masonry, thus leading to a new concept of so-called engineered masonry design and construction [17].

Modern masonry construction employs masonry units, mortar, grout and steel reinforcement.

Masonry units may be made of clay or shale, concrete, sand-lime, stone, glass and other materials. Clay or shale bricks and concrete blocks are the most commonly used masonry units in structural masonry.

Bricks are available in many sizes based on a 100x100 mm module. The 'standard' brick size of 90x57x190 mm provides three vertical courses of bricks every 200 mm when laid in mortar joints of 10 mm thickness. Other larger clay brick units, available with nominal 100 mm height, 400 mm length and 100, 200, or 300 mm width, provide core spaces for placement of reinforcing steel and grout. Typical compressive strengths of North American clay bricks are in excess of 55 MPa.

Concrete blocks are produced for various strengths, sizes and shapes. Typical unit compressive strengths based on net area are about 14 to 28 MPa. The 'standard' block has a nominal height of 200 mm, length of 400 mm and widths increasing in 50 mm increments from 150 to

300 mm. To allow for mortar joints, actual dimensions are normally 10 mm less than the nominal dimensions. Most commonly, these blocks have two or more voids which can be used for placement of steel and grout.

Mortar and grout are similar products to the extent that both consist of cementitious material and aggregates. They differ in that they are used for different purposes, are of different consistency and usually require a different size of aggregate.

Mortar is used for a number of purposes, but its principal function is to join masonry units together into an integral structure which will withstand the imposed conditions of loads and weather. Mortar consists of portland cement, lime, and sand, or of masonry cement and sand. Basically, portland cement provides strength, lime contributes to workability, and sand provides a cheap filler.

Grout can be considered a high slump (about 200 to 250 mm) concrete. But in contrast to concrete, a high water-cement ratio is essential to grout. The primary function of grout is to bond masonry units and steel together in reinforced masonry walls so that they act together to resist imposed loads. It is usual to place grout in only those cells containing steel reinforcement, but in some loadbearing reinforced masonry walls, all cells are grouted. Sometimes the cells of non-reinforced masonry walls are grouted to provide added strength, and grout may also be used to bond two wythes together to constitute one integral wall.

Masonry is used for a variety of structural members like beams, columns, pilasters and walls, or for non-structural purposes such as partition walls.

Because masonry cannot withstand significant tensile stresses without cracking, major flexural members generally are reinforced with steel in much the same way as reinforced concrete members. Typical reinforced masonry flexural members are lintels, bond beams, walls subjected to out-of-plane bending and slender shear walls subjected to in-plane bending. However, the main use of masonry is in wall construction.

Structural masonry walls must take into account the variety of functions they may be required to fulfill. A prime objective is to transmit gravity loads, including the weight of the wall, to the foundation. The lateral stability of the structure is often achieved by using the in-plane strength and stiffness of the wall to resist the horizontal forces. The lateral forces acting in the plane of the wall produce in-plane bending moments and shear. Also loads applied normal to the wall surface tend to cause out-of-plane bending and shear.

A large amount of masonry wall construction is plain, i.e. unreinforced. However, in many cases of large lateral forces and slender walls, it is economic and structurally beneficial to use reinforcing steel and design masonry as reinforced masonry [8].

A number of methods or systems are used in the construction of masonry walls. Solid masonry walls are built up of solid units most of which are brick or in some cases stone. Hollow masonry walls are built

with hollow masonry units, which may be brick or concrete block.

Cavity walls consist of two wythes of masonry, separated typically by an air space of 25 to 75 mm and joined together by horizontal metal ties for structural strength. The outside or face wythe and the backup wythe may be of similar materials; or, as in many cases, the face wythe may be brick or facing tile and the back-up wythe is concrete block or structural tile. While masonry walls may be loadbearing or non-loadbearing walls, a rapid increase has occurred in the use of the mid- to high-rise loadbearing masonry structures in which lateral stability is provided by relatively slender shear walls [26].

1.2 Use of a Concentrated Amount of Prestressed Reinforcement

When designing a low-rise masonry wall, which is the case in typical commercial and industrial buildings, the gravity loads alone may not be sufficient to prevent the occurrence of tensile stresses due to wind or earthquake actions especially in the lower parts of the wall.

This situation may also be encountered in the case of relatively slender walls in mid- and high-rise buildings. Steel reinforcement is required to carry these tensile stresses.

The amount of steel required to reinforce the wall must be distributed along the horizontal and vertical directions according to spacing limits specified by CSA Standard S304 [22]. Normal spacing may be 1.2 m, but in some cases, such as near edges of a wall, may have to be as close as 0.2 m to carry the excessive tensile stresses. The latter situation requires the labour-intensive placement of one reinforcing bar in each cell of the standard 400 mm long concrete block.

Concentrating the amount of steel required in the vertical direction appears to be a potential way of saving labour, time, and hence, construction costs. Instead of distributing reinforcing steel over the whole width of the wall, it may be advantageous to put the required amount of steel in a few selected locations. Moreover, if the steel bars are prestressed we may gain an additional improvement in the performance of masonry walls subjected to lateral forces.

The lateral load resisting capacity of a masonry shear wall is highly dependent on the shear strength of masonry. Shear strength of masonry is not a constant value but depends on various factors [4, 7, 11, 12, 15, 18, 24, 27]. The variable that most significantly affects the shear strength is the level of normal compressive stress or so-called precompression [8]. Researchers have found that increased shear strengths are obtained with increase in precompression. This fact has been recognized in the Code [22] by specifying the allowable shear stress as a function of the compressive stress due to dead loads as follows:

$$v_{sw} = (v \text{ or } v_m) + 0.3 f_{cs}$$

where

v_{sw} = shear stress in a shear wall

v = shear stress in masonry having shear reinforcement

v_m = shear stress in masonry having no shear reinforcement

f_{cs} = compressive stress due to dead loads

The lateral load carrying capacity also depends on the tensile strength of masonry, especially for slender shear walls having height

to width ratios greater than 2.0 [1]. Such walls behave substantially like flexural members.

While the apparent tensile strength of masonry is dependent on the rather low tensile bond strength at the interface between masonry units and mortar [17], it can be increased by higher gravity loads or by other means of inducing compressive stresses in the wall.

As a way of inducing precompression, vertical prestressing of masonry walls may be used. Such precompression in turn will lead to increased shear resistance of the walls.

Prestressing is usually associated with concrete but there are a number of examples where it has been successfully applied to masonry. Prestressed masonry was used as early as 140 years ago by Brunel [17] in the construction of brickwork caissons for the Thames tunnel project [14]. While developments in prestressed masonry have been rather slow, perhaps due to the extensive use of prestressed concrete, there are signs that the potential of prestressed masonry is being recognized by more designers and researchers.

The use of prestressed masonry has been increasing in Britain over the last 15 years [29]. Several systems of prestressed masonry load-bearing and non-loadbearing walls are now available and others are in the process of development [32]. A prestressed free cantilever brick diaphragm wall is under construction which, unless prestressed, would have to be built in structural steelwork [6]. Also, by using prestressing, it has been possible to construct 7.34 m high walls in 280 mm cavity brickwork which would otherwise have to be of the order of 457 mm solid

construction [14]. Moreover, a water tank with a capacity of 545.3 m^3 was constructed in post-tensioned brickwork. The tank is 10.6 m in diameter, 4.8 m high with 228 mm solid brick walls. The walls were prestressed vertically and also circumferentially to counter the hoop forces caused by the hydraulic pressure [17]. Furthermore, in prefabricated construction, prestressing with cables crossing the joints between prefabricated elements has ensured structural continuity [16]. Besides these examples, prestressing has also been employed in masonry beams [19].

According to St. Venant's principle, the use of concentrated prestressed bars will create a region at the top of the wall in which the stresses are localized. The areas not close to this region will have stresses of relatively uniform magnitude. At the top region, horizontal tensile stresses will develop which may split the wall vertically while prestressing the bars. The analysis presented here will investigate the stress distribution in the top region of a wall subjected to concentrated prestressing forces.

1.3 Previous Work On Prestressed Masonry Walls

- Although several prestressed masonry examples have been mentioned in the previous section, to the best of the author's knowledge, little research has been carried out on prestressed masonry shear walls.

Rosenhaupt and Mueller [25] investigated the effect of vertical prestressing on the behaviour of masonry walls with openings. They concluded that vertical prestressing is effective in increasing the rigidity of the wall. They also noted that prestressing is efficient

in repairing cracked walls and can be applied to the sides of the openings in place of reinforced concrete ties.

Sinha and Hendry [27] in their experimental work on a single storey shear wall concluded that precompression increased the rigidity of the structure.

Curtin [6] tested two full sized post-tensioned prestressed brick diaphragm walls 7.62 m high x 7.62 m wide x 0.45 m thick. They were subjected to various levels of axial prestress and, at each level of prestress, they were tested under uniform lateral load. There were 10-40 mm diameter prestressing bars per wall equally distributed along the width. Curtin concluded that the working load capacity of walls under lateral loads can be increased by vertically prestressing the walls.

The most important work related to the use of a concentrated amount of prestressing in masonry is Hinkley's experimental work [14] on one storey prestressed brickwork shear walls. Hinkley tested brickwork wall panels by loading them horizontally in the plane of the wall through a loading beam. The base of the wall was supported by another beam. There were no columns at the ends of the wall. Two sets of walls were tested: square walls were 1.575 by 1.575 m, and rectangular walls were 1.372 m high and 2.083 m wide. Two tests were carried out on two non-prestressed square walls, and five on vertically prestressed walls. Two of the prestressed walls were square and were furnished with seven equally spaced wires which in turn resulted in a uniform vertical prestress. Another square wall was furnished with a single wire at each

end of the wall. Two rectangular walls were built, one with nine equally spaced wires, and the other with two wires at each end. UngROUTED 7 mm (0.276 in.) diameter high-tensile steel wires were used throughout.

Hinkley found that a wall, even when badly cracked, would not necessarily cease to be a structural member. The wall would continue to function structurally as long as the compressive forces normal to the mortar joints prevented shearing failure.

He also analysed the walls for uncracked behaviour using the lattice analogy described by McCormick [20] and McHenry [21]. This approach considers the walls as two-dimensional elastic bodies and treats masonry as a homogeneous, isotropic, and elastic material.

Hinkley concluded that the strength and reliability of brickwork shear walls can be increased by vertical prestressing and also that prestressing at the ends of a wall is more efficient than prestressing uniformly along its width. However, he reported that although stressing sequences were chosen to limit tensile stresses, cracking sounds were heard during the stressing operation of the two walls which contained wires at the ends only.

1.4 Scope of Investigation

There is a rapid increase in the application of reinforced masonry walls to building construction. Reinforcing masonry walls with steel bars distributed along its width required a considerable amount of work. Concentrating reinforcing bars in a few locations may save labour and

time in the construction process. Moreover, prestressing the steel vertically will improve the performance of masonry walls when subjected to lateral forces.

The analysis presented in this thesis investigates the stress distribution resulting from prestressing steel bars located in masonry walls. Two cases have been considered with regard to the locations of the prestressed bars:

Case 1: Two bars were used, one at each end of the wall.

Case 2: Four bars were used, two bars were placed as in case 1, the other two bars were located at the third points.

The walls considered had aspect ratios, i.e. height to width ratios, of 0.5, 1.0, 2.5 and 5.0 representing a fairly good range of low- to high-rise walls.

A two-dimensional finite element analysis was carried out and a computer program was developed to perform the analysis.

Masonry was assumed to be an isotropic, homogeneous and linearly elastic material in compression and tension.

The following chapter describes the finite element model along with the assumptions. The results of the finite element analysis depicting the stress distribution in prestressed masonry walls are presented in Chapter 3. Conclusions and recommendations are given in Chapter 4.

CHAPTER 2

FINITE ELEMENT ANALYSIS2.1 Introduction

The finite element method is a numerical technique for obtaining approximate solutions to complex problems. The method has been developed extensively and is widely used in structural and continuum mechanics as well as in dynamics. Depending on the degree of the required accuracy, body forces, non-linear behaviour, and anisotropic material characteristics can all be taken into account.

Since the finite element method is widely known, only the necessary steps involved in this investigation will be mentioned. A detailed description of the method is available in Ref.33 and in numerous other text books and references.

2.2 Assumptions

The assumptions made for this analysis are as follows:

1. Properties of masonry:

- (a) Masonry is assumed as an isotropic, homogeneous and linearly elastic material.
- (b) Modulus of elasticity, $E_m = 20000$ MPa.
- (c) Poisson's ratio, $\nu = 0.15$.

2. Wall dimensions:

- (a) The width of the walls, W , has been limited to 6.0 m.
- (b) Four different heights of the walls have been considered: 3.0, 6.0, 15.0 and 30.0 m.
- (c) The thickness of the wall, t , is assumed to be 190 mm.

3. Size and location of prestressed bars:

- (a) 25 M bars are used throughout.
- (b) A bar located at the end is at a distance of 100 mm from the wall edge.
- (c) No bond exists between the bars and the grouted masonry.
- (d) The bars are connected to the foundation at the lower end; at the upper end they are connected to the wall by means of anchorage plates resting on top of the wall.
- (e) There is full contact between the anchorage plates and the wall. No slip or shearing is allowed at the plate/wall interface.

4. Boundary conditions:

- (a) The wall is resting on a rigid foundation and there exists complete bond between wall and foundation.
- (b) Since there is symmetry about the vertical centreline of the wall, only one half of the wall is analyzed.
- (c) The effect of prestressing only is considered. The self-weight of the wall and time dependent deformations are neglected.

The following subsections will deal with a discussion of the various assumptions.

2.2.1 Properties of Masonry

Masonry is a complex material. It is composed of bricks or concrete blocks embedded in a matrix of mortar. Its physical characteristics may vary along and perpendicular to the bed joint direction. Despite this complexity, masonry can be idealized as an isotropic, homogeneous

and linearly elastic material provided the stresses are well below the ultimate stresses [30].

Modulus of elasticity of masonry, E_m , in tension and compression is taken as 20000 MPa. The value of E_m depends on the properties of the units and the mortar. It is related to the compressive strength of masonry, f'_m . The value of f'_m may range from about 6 to 30 MPa for brick masonry, and from about 5 to 16 MPa for concrete block masonry. Among the factors that affect f'_m are the masonry unit compressive strength and the type of mortar. The Code [22] specifies the value of E_m as:

$$E_m = 1000 f'_m \leq 20000 \text{ MPa}$$

This relationship seems to be fairly realistic for concrete block masonry but appears to over-estimate E_m for brick masonry [8]. An assumed value of 20000 MPa is the maximum limit according to the code. However, it may be on the higher side for concrete block masonry and on the lower side for brick masonry according to accumulated experimental data [8].

Poisson's ratio is assumed as 0.15, a typical value for masonry. On the basis of previous finite element investigations [23, 31] it was concluded that the effect of a slight variation in Poisson's ratio on the magnitude and distribution of stresses is not significant enough to warrant consideration in this study.

2.2.2 Wall Dimensions

Masonry walls are used in buildings with dimensions varying from long wall segments separated by control joints, to allow for thermal

and moisture deformations in low-rise commercial buildings, to slender shear walls in mid- to high-rise residential buildings. Wall widths in residential buildings typically may vary between 1 and 8 m depending on apartment layouts, openings and floor systems.

The analysis has been carried out on fairly substantial walls having a constant width of 6.0 m. However, the analysis is not limited to one wall width since various ratios of width to height are taken into account. Four different heights of 3.0, 6.0, 15.0 and 30.0 m were considered. These heights represent one, two, five and ten storey walls, respectively. The walls considered hence have aspect ratios, height to width, of 0.5, 1.0, 2.5 and 5.0.

The wall thickness was taken as 190 mm, the actual thickness of a 'standard' concrete block. Such a wall thickness is typical for a loadbearing masonry structure up to about 10 storeys in height.

2.2.3 Location of Prestressed Bars

The Code [22] provisions for reinforced masonry loadbearing and shear walls require that the reinforcing steel shall be spaced uniformly according to specified limits. The basic idea for carrying out the analysis presented in this thesis is to determine whether it is possible to use some concentrated amount of prestress rather than distributing it over the wall width.

The 25M prestressing bars used in the analysis provide a substantial cross sectional area of 548mm^2 concentrated at one location.

With regard to bar locations, two cases were considered:

Case 1: Two bars were used representing 0.19% of the gross cross sectional area of the wall. Each bar was located at a distance of 100 mm from the edge, simulating the position of the first cell in a hollow 'standard' concrete block.

Case 2: Four bars were used representing 0.38 % of the gross sectional area of the wall. Two bars were located as in case 1. The other two bars were located at the third points.

It is assumed that there is no bond between the prestressed bars and the grouted masonry. The only contacts between a bar and the wall area at the lower and upper ends of the bar. At the lower end the bar is fastened to the rigid foundation; at the upper end the prestressing force is transferred to the wall by means of an anchorage plate. This situation simulates a condition where the prestressed bar is in a duct or sheath.

The anchorage plate dimensions were assumed as 190x250x32 mm. Typical Dywidag dimensions [10] vary between 51x127x25 mm and 127x241x44 mm. These dimensions were modified to match the finite element mesh used in the analysis.

It was assumed that the anchorage plate is in full contact with masonry, i.e. no separation or slip was allowed. This assumption simplifies the complexity at the interface between two materials which otherwise might require special treatment [2].

2.2.4 Boundary Conditions

The wall was assumed to be resting on a rigid foundation. No separation or slip was allowed to occur at the interface between the foundation and the wall.

Since there is symmetry about the centerline of the wall in geometry and loading, only one half of the wall was analysed; the other half of the wall was replaced, by roller supports preventing horizontal movements as illustrated in Figure 2.1.

Preliminary analysis work showed that the wall self-weight did not affect the stress pattern in the upper portion of the wall where the analysis was focused, thus the effect of self-weight has been ignored.

2.3 The Finite Element Model

Three types of elements were used in the model namely:

1. Constant strain triangular (CST) elements to represent masonry.
2. Bar elements to represent the prestressed bars.
3. Frame elements to represent the anchorage plates.

A detailed description of the elements' theories may be found in Ref.3.

2.3.1 The Choice of A Wall Element

The cost of a finite element analysis depends on many factors, among which the type of element chosen for the analysis is one factor. The type of element implies the number of elements, the number of degrees of freedom and the bandwidth to be used.

The constant strain triangular element (CST) is a simple element with three corner nodes and with two degrees of freedom at each node. The two degrees of freedom are two displacements perpendicular to each other as shown in Figure 2.2. Because of its simplicity, a CST element does not give good results unless its size is reduced in areas where rapid change of stress is expected. This is a point in favour of the analysis. Reducing the size of the elements, results in a larger number of data points to be plotted and hence in better curve fitting.

The triangular shape of CST elements enables one to break the mesh easily into smaller elements. Moreover, the CST elements do not require a large bandwidth in the stiffness matrix as compared to, for example, linear strain triangular (LST), elements. The usage of LST elements may roughly double the bandwidth which would mean doubling the space required to store the stiffness matrix in the computer memory.

2.3.2 Special Anchorage Plate Element

During exploratory analyses, a prestressed bar was connected to the CST elements at a node. It was observed that the prestressing force created singularity in the vicinity of that node. This singularity appeared in the form of an appreciable change in stresses. It was observed that it was not realistic to apply a concentrated force to masonry at one point. Also in practice, a prestressed bar transfers its force to masonry by means of a bearing or an anchorage plate. Thus, an anchorage plate has been introduced at the connection between the bar and the wall.

The anchorage plate acts as a bearing plate on top of the wall at

each bar. It resists the force applied to it by the bar mainly by flexure, and its function is to distribute the pressure resulting from the prestressing of the bars over a certain area. Therefore, frame elements are introduced in the model to represent the anchorage plates.

The frame element used in the analysis is shown in Figure 2.3. It has two nodes, one at each end, with three degrees of freedom at each node, two translations and one rotation.

To combine the flexural elements having three degrees of freedom at each node, with the CST elements having two degrees of freedom at each node, an elimination operation was performed on the anchorage plate stiffness matrices to eliminate the rotational degrees of freedom. Then the anchorage plate condensed global stiffness matrix was added to the overall stiffness matrix of the wall.

2.4 Accuracy of the Results

A computer program was developed to perform the finite element analysis. Most of the subroutines were taken from Ref.5. The flow chart of the program is shown in Figure 2.4. The program was checked for the CST elements by simple tension and simple bending of a beam. For the case of simple tension, stresses and deformations were exact; for the case of simple bending, deformations were sufficiently close to those calculated by beam theory. Increasing the number of elements representing the beam would have brought the deformations closer to those determined by beam theory.

Also, the program was checked for the frame elements by analysing a simple beam. The deformations were exact.

The finite element mesh used for the analysis is shown in Figure 2.5. The size of the elements varied from $W/12$ (where W is the width of the wall) to $W/48$ at the top and close to the locations of the anchorage plates. The number of elements and the number of degrees of freedom corresponding to each aspect ratio are given in Table 2.1.

To examine the accuracy of the results, the mesh shown in Figure 2.5 was first used to analyse the whole wall. Then the same mesh was scaled down to represent the top left corner of the wall. The boundary deformations were determined by inducing displacements at the boundary nodes. These displacements were taken from the first analysis of the whole wall. The same procedure was performed for the top right corner of one half portion of the wall for the case of prestressing at ends and at third points.

The increase in the maximum vertical displacement under the anchorage plate did not exceed 2.6% of the displacement determined in the earlier analysis.

Unfortunately, the increase in stresses was much higher than that for displacements. The increase in stresses ranged from 50% to 90% of the stresses obtained from the analysis of the whole wall. But these increases occurred due to a bearing problem only in a small region under the applied load. Away from that region, increase in the stresses is negligible.

The analysis results are discussed in the following chapter.

Figure 2.1 Boundary Conditions

Figure 2.2 Constant Strain Triangular Element

Figure 2.3 Frame Element

Figure 2.4 Flow Chart of the Computer Program

Figure 2.4 (contd.) Flow Chart of the Computer Program

Figure 2.5 Finite Element Mesh

Table 2.1

Aspect Ratio	Number of Elements	Number of Degrees of Freedom
0.5	409	462
1.0	481	546
2.5	697	798
5.0	1057	1218

CHAPTER 3

ANALYSIS OF PRESTRESSED MASONRY WALLS3.1 General

The walls under consideration had aspect ratios, H/W , equal to 0.5, 1.0, 2.5 and 5.0.

The locations of the prestressing bars giving rise to two cases of analysis have been described in Section 2.2. The two cases have been compared on the basis of the unit value of precompressive stress. Hence, the results are presented in a non-dimensional form.

The normalized stresses were obtained by dividing the stresses calculated from the analysis by the effective prestress. The effective prestress was calculated by dividing the total force in the prestressed bars, after elastic deformations in masonry have taken place, by the cross-sectional area of the wall. The reduction of steel stress due to the elastic shortening of masonry varied between 4 and 6% of the initial prestress. This reduction was taken into account in the analysis.

The analysis results for the upper wall regions in the vicinity of the points of load application are of particular interest since in these regions stress changes occur most rapidly.

The results are discussed for walls of H/W equal to 0.5 followed by the other aspect ratios.

5

3.2 Analysis for H/W Equal to 0.5

3.2.1 Principal Stresses

Figure 3.1 illustrates the principal stresses for case 1. It was observed that the stresses in the region of the applied force were very high. That region is shown in Figure 3.2 to an enlarged scale. Away from this region, stresses start to diminish. It was also seen that the top fibers of the wall are highly stressed horizontally in tension. Towards the bottom, the principal stresses tend to be vertical, but still not uniform. The fixed supports at the base have only a small effect on the stress pattern.

Figure 3.3 illustrates the principal stresses for case 2. The stresses differ from those produced in case 1, especially the central region due to the intermediate bar. Regions under the applied forces have a similar configuration to the region seen in case 1 except for the stress magnitudes. The magnitudes in case 2 are about one half due to the normalization process. Away from these regions, case 2 stresses seem to be more uniform than for case 1.

3.2.2 Horizontal Stresses

The diagrams shown in Figures 3.4 a,b,c and d illustrate the horizontal stress distributions for case 1 and case 2 at the vertical sections A, B, C and D at distances of $W/8$, $W/4$, $3W/8$ and $W/2$ from the edge, respectively. One may observe that there is a limited region at the top of the wall in which the horizontal stresses are tensile. This region has a depth which varies gradually from $W/12$, or theoretically zero, at the edges to $W/4$ at the centerline of the wall. The tensile stresses have maximum values at the edges and decrease gradually towards the centerline of the wall.

Below that region of tensile stresses, the horizontal stresses become compressive stresses having roughly a uniform value. The effect of fixity at the base is judged to be small. The only difference between the two cases is that, as expected, the magnitudes of stress in case 2 are approximately one half of those in case 1.

Figure 3.4(b) and (d) show the effect of the intermediate bars on the stress pattern. A more rapid stress change occurs at those sections because of their proximity to that bar. Figure 3.4(c) shows no major difference between the two cases although this section is closer to the intermediate bar. However, it is important to notice that the two data points closest to the top of the wall are unusually far apart from each other. This is so because these data points are very close to the point of load application. Apart from that and apart from the stress magnitudes commented on previously, no significant difference is seen between the two cases of prestressing.

3.2.3 Vertical Stresses

The diagrams shown in Figures 3.5(a), (b), (c) and (d) illustrate the vertical stress distribution at the horizontal sections E, F, G and H at distances of $W/8$, $W/4$, $3W/8$ and $W/2$ from the top, respectively. For both cases, the stresses at section E in Figure 3.5(a), are highly disturbed. The stresses are a maximum at the edges and decrease rapidly towards the centerline of the wall. While they reach zero at the centerline for case 1, the influence of the intermediate bar for case 2 can be clearly observed. At section F, Figure 3.5(b), stresses are less disturbed than those at section E, but the same general trend is valid.

Sections G and H, Figures 3.5 (c) and (d), show the gradual relaxation that takes place at sections further removed from the points of load application. Particularly, for case 2, considerable uniformity of stress can be observed.

3.3 Effect of the Aspect Ratio

The analysis was carried out for aspect ratios H/W of 0.5, 1.0, 2.5 and 5.0. The results for $H/W=0.5$ have been discussed in the previous section.

Generally speaking, the aspect ratio had no effect on the stresses in the top region of the wall, i.e. the observations made for $H/W=0.5$ apply also for H/W greater than 0.5.

The principal stresses for the upper part of the wall for case 1 are shown in Figures 3.6(a), (b) and (c) for H/W of 1.0, 2.5 and 5.0, respectively. Figures 3.7(a), (b) and (c) show the corresponding principal stresses for the lower part of the walls.

For case 2, Figures 3.8(a), (b) and (c) and Figures 3.9(a), (b) and (c) show the principal stresses for the upper and the lower parts of the walls, respectively.

Comparing these results with the corresponding evidence for H/W equal to 0.5, one may observe no significant difference except for the influence of the base fixity: the stress patterns are similar and the observations made for H/W equal to 0.5 are valid for aspect ratios greater than 0.5 as well.

To support this conclusion, the stress distributions at various sections are presented in Figures 3.10(a), (b), (c) and (d) to Figures 3.15(a), (b), (c) and (d). It can be seen that these stress distributions are almost identical to the corresponding distributions for H/W equal to 0.5.

Also, the aspect ratio did not influence the depth where the vertical stresses became uniform. For case 1, the vertical stresses varied between 90 and 110% of the average prestress at a depth equal to 0.875 times the wall width downwards from the applied load level. The stresses became uniform at a depth of 2.46 times the wall width. For case 2, the vertical stresses varied between 90 and 110% at a depth equal to 0.675 times the wall width. The stresses became uniform at a depth equal to 1.96 times the wall width. Summarizing, for case 1 the vertical stresses are virtually uniform along any section below a distance from the top equal to twice the wall width for case 1, and equal to the wall width for case 2.

3.4 Discussion

Concentrating an amount of steel reinforcement at a few locations may save some labour and other costs in the construction process. Moreover, prestressing the steel may improve the behaviour of masonry walls while resisting lateral loads.

As expected, tensile stresses develop in the top region of the wall. This region is limited at the top with a depth varying from zero at the edges to $W/4$ at the middle of the wall. Maximum tensile stresses occur at the edges and then decrease as the depth of the prestressed region

increases. These tensile stresses are high enough to crack the masonry and hence must be accommodated in a special structural detail which may entail additional costs. The special structural detail may consist of a reinforced or even prestressed bond beam similar to reinforced bond beams commonly employed today. Alternatively, the floor slab may be reinforced along the edges to carry the additional tensile stresses induced by prestressing the masonry walls.

Vertical prestressing may be applied to masonry walls subjected to low gravity loads to help in reducing tensile stresses created by lateral forces. Furthermore, precompression will increase the shear carrying capacity of masonry walls.

From one point of view, one may argue that case 1 is more effective in developing compressive stresses at the edges which reduce or eliminates the tensile stresses induced by wind or earthquake effects.

On the other hand, one can also argue that case 2 develops lower horizontal tensile stresses in the top portion of the wall. Moreover, case 2 is more efficient in obtaining a uniform precompressive stress at less distance from the top to increase the shear capacity of the wall. Case 2 certainly produces more uniform compressive stress than case 1. However, the areas under the two curves in Figure 3.5(d) are the same, i.e. the total increase in the shear capacity along the width of the wall is the same for both cases.

Figure 3.1 Principal Stresses For $H/W=0.5$, Case 1

Figure 3.2 Principal Stresses for $H/W=0.5$, Case 1,
Critical Area Under the Applied Load

Figure 3.3 Principal Stresses for $H/W=0.5$, Case 2

(a) At Section A, W/8 From the Edge

(b) At Section B, W/4 From the Edge

Figure 3.4 Horizontal Stress Distribution for $H/W=0.5$

(c) At Section C, $3W/8$, From the Edge

(d) At Section D, at the Centerline

Figure 3.4 (contd.) Horizontal Stress Distribution for $H/W=0.5$

Figure 3.5 Vertical Stress Distribution for $H/W=0.5$

(c) At Section G, 3W/8 From the Top

(d) At Section H, At the Base

Figure 3.5 (contd.) Vertical Stress Distribution for H/W=0.5

Figure 3.6(a) Principal Stresses for the Upper Part for $H/W=1.0$, Case 1

Figure 3.6(b) Principal Stresses for the Upper Part for $H/W=2.5$, Case 1

Figure 3.6(c) Principal Stresses for the Upper Part for $H/W=5.0$, Case 1

Figure 3.7(a) Principal Stresses for the Lower Part
for $H/W=1.0$, Case 1

Figure 3.7(b) Principal Stresses for the Lower Part for $H/W=2.5$, Case 1

Figure 3.7(c) Principal Stresses for the Lower Part for $H/W=5.0$, Case 1

Figure 3.8(a) Principal Stresses for the Upper Part for $H/W=1.0$, Case 2

Figure 3.8(b) Principal Stresses for the Upper Part for $H/W=2.5$, Case 2

Figure 3.8(c) Principal Stresses for the Upper Part for $H/W=5.0$, Case 2

Figure 3.9(a) Principal Stresses for the Lower Part for $H/W=1.0$, Case 2

Figure 3.9(b) Principal Stresses for the Lower Part for $H/W=2.5$, Case 2

Figure 3.9(c) Principal Stresses for the Lower Part for $H/W=5.0$, Case 2

(a) At Section A, W/8 From the Edge

(b) At Section B, W/4 From the Edge

Figure 3.10 Horizontal Stress Distribution for $H/W=1.0$

(c) At Section C, 3W/8 From the Edge

(d) At Section D, at the Centerline

Figure 3.10 (contd.) Horizontal Stress Distribution
for $H/W=1.0$

(a) At Section E, W/8 From the Top

(b) At Section F, W/4 From the Top

Figure 3.11 Vertical Stress Distribution For $H/W=1.0$

(c) At Section G, $3W/8$ From The Top (d) At Section H, $W/2$ From the TopFigure 3.11 (contd.) Vertical Stress Distribution for $H/W=1.0$

(a) At Section A, W/8 From the Edge

(b) At Section B, W/4 From the Edge

Figure 3.12 Horizontal Stress Distribution for $H/W=2.5$

(c) At Section C, 3W/8 From the Edge

(d) At Section D, at the Centerline

Figure 3.12 (contd.) Horizontal Stress Distribution for $H/W=2.5$

Figure 3.13 Vertical Stress Distribution for $H/W=2.5$

Figure 3.13 (contd.) Vertical Stress Distribution for $H/W=2.5$

(a) At Section A, $W/8$ From the Edge (b) At Section B, $W/4$ From the EdgeFigure 3.14 Horizontal Stress Distribution for $H/W=5.0$

Figure 3.14 (contd.) Horizontal Stress Distribution for $H/W=5.0$

(a) At Section E, W/8 From the Top

(b) At Section F, W/4 From the Top

Figure 3.15 Vertical Stress Distribution for H/W=5.0

(c) At Section G, $3W/8$ From the Top (d) At Section H, $W/2$ From the TopFigure 3.15 (contd.) Vertical Stress Distribution for $H/W=5.0$

CHAPTER 4

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK4.1 Summary and Conclusions

An investigation was carried out on masonry walls to study the effect of concentrated amounts of prestressing on the wall's stress distribution. Two cases were considered. Case 1 used bars located at the wall edges only. Case 2 used bars located at the edges as well as at the third points. The walls investigated had aspect (height to width) ratios between 0.5 and 5.0. A two-dimensional finite element analysis was employed to perform the analysis assuming masonry as a homogeneous elastic material.

1. Conclusions for case 1:

(a) Regardless of the aspect ratio, horizontal tensile stresses developed in a limited region at the top of the wall. This region has a very small depth at the edges and then increases towards the wall's centerline where it reaches a depth of one-quarter of the wall width. These tensile stresses may limit the use of full prestressing unless special structural measures are taken to resist them. An axially stiff member resting on top of the wall may be useful in resisting the tensile stresses.

(b) High vertical compressive stresses were found to occur in the vicinity of the applied load. Aside from this localized region, vertical compressive stresses were virtually uniform at a depth equal to twice the wall width.

2. Conclusions for case 2:

(a) The region of horizontal tensile stresses had the same depth as in case 1.

(b) Again, localized vertical compressive stresses were found to occur in the vicinity of the applied loads. At a depth equal to the wall width, the stresses became uniform.

3. Aside from the problems associated with the horizontal stresses within a limited region at the top of the wall, the use of a concentrated amount of prestressed bars shows promise towards increasing the shear carrying capacity of masonry walls.

4.2 Recommendations for Future Work

The following topics are suggested for future study:

1) In practice, masonry displays a more complex behaviour than that idealized in this analysis. Recently, Page [23] introduced a finite element model for in-plane deformations and failure of masonry. It would be beneficial to extend this thesis' analysis by using such a model to take into account various factors affecting masonry's more complex behaviour.

2) Long term deformations for instance due to temperature, moisture, and creep were not considered. It is recommended to extend this analysis to take into account long term deformations.

3) The present analysis has not investigated the behaviour of prestressed masonry walls subjected to out-of-plane, in-plane, or cyclic shear forces. It would be desirable to perform theoretical and experimental studies to determine the effect of prestressing on masonry walls under the action of these shear forces.

4) The tensile stresses at the top of the wall may result in limiting the prestressing force. Further studies should be carried out to propose practice-oriented structural details to overcome the horizontal tensile stress problem.

5) The present study assumed that the anchorage plate is bonded perfectly to the wall such that no slip can occur at the interface between plate and masonry. Using Arya's finite element model for interface problems [2], it is recommended to incorporate such a model to investigate the stress distribution under bearing plates.

REFERENCES

- [1] Amrhein, J.E., "Reinforced Masonry Engineering Handbook, Brick and Other Structural Clay Units", Published by Masonry Institute of America, 1972.
- [2] Arya, S.K., "Finite Element Method For Interface Problems", Journal of the Structural Division, ASCE, Vol.108, No.ST2, February 1982, pp.327-342.
- [3] Bathe, K.J. and Wilson, E.L., "Numerical Methods in Finite Element Analysis", Prentice-Hall, Inc., Englewood Cliffs, N.J., 1976.
- [4] Blume, J.A. and Proulx, J., "Shear In Grouted Brick Masonry Wall Elements", Western States Clay Products Association, San Francisco.
- [5] Bowes, W.H. and Russell, L.T., "Stress Analysis By the Finite Element Method for Practicing Engineers", Lexington Books, Massachusetts.
- [6] Curtin, W.G. and Phipps, M.E., "Prestressed Masonry Diaphragm Walls", the Sixth International Brick Masonry Conference, Roma, Maggino, 1982, pp.971-980.
- [7] Drysdale, R.G., Vanderkeyl, R., and Hamid, A.A., "Shear Strength of Brick Masonry Joints", Preprints of the Fifth International Brick Masonry Conference, Washington, D.C., October 1977.
- [8] Drysdale, R.G. and Suter, G.T., "Modern Engineered Masonry, A 2-Day Practice Oriented Course", sponsored by Canadian Society for Civil Engineering, 1981.
- [9] Dunder, V. and Ridlon, S., "Practical Applications of Finite Element Method", Journal of Structural Division, Vol.104, No.ST1, January 1978, pp.9-21.
- [10] DYWIDAG, Threadbar Postensioning System, Dywidag Canada Limited, Dywidag Prestressed Concrete.
- [11] Hamid, A.A. and Drysdale, R.G., "Behaviour of Brick Masonry Under Combined Shear and Compression Loading", Paper presented to the Second Canadian Masonry Symposium, Ottawa, June 1980.
- [12] Hamid, A.A., Drysdale, R.G., and Heidebrecht, A.C., "Shear Strength of Concrete Masonry Joints", Journal of Structural Division, ASCE, Vol.105, No.ST7, July 1979, pp.1227-1240.
- [13] Haseltine, B.A., "A Design Guide for Reinforced and Prestressed Brickwork", Proc. of the North American Masonry Conf., August 1978, University of Colorado, Boulder, Colorado.

- [14] Hinkley, A.T., "Tests of One-Story Prestressed Brickwork Shear Walls", New Zealand Engineering, Vol.21, June 1966, pp.245-252.
- [15] Jolly, R., "Shear Strength: A Predictive Technique for Masonry Walls", Thesis presented to Brigham University, Provo, Utah, 1976, in partial fulfillment of the requirements for the degree of Doctor of Philosophy.
- [16] Kavyrchine, M., "Reinforced, Precast and Prestressed Concrete", Technical Committee No.3, Structural Systems, State of Art Report No.1, Planning and Design of Tall Buildings, Vol.1a, pp.415-436.
- [17] Lenczer, D., "Elements of Loadbearing Brickwork", International Series of Monographs in Civil Engineering, Vol.5, General Editor: Silverleaf, D.J., Executive Editor: Raikes, J.L., Pergamon Press, 1972.
- [18] Mayes, R.L. and Clough, R.W., "A Literature Survey: Compressive, Tensile, Bond and Shear Strength of Masonry", Report No.EERC75-15, College of Engineering, University of California, Berkeley, California, June 1975.
- [19] Mehta, K.C. and Fincher, D., "Structural Behaviour of Pretensioned Prestressed Masonry Beams", Proc. of the Second International Brick Masonry Conference (SIBMAC), held in Stoke-on-Trent, England, April 1970.
- [20] McCormick, C.W., "Plane Stress Analysis", Journal of Structural Division, ASCE, Vol.89, August 1963, pp.37-54.
- [21] McHenry, D., "A Lattice Analysis for the Solution of Stress Problems", Journal of the Institution of Civil Engineers, London, December 1943, pp.59-82.
- [22] National Standard of Canada, CAN3-S304-M78, Masonry Design and Construction for Buildings, Canadian Standards Association, Rexdale, Ontario.
- [23] Page, A.W., "A Model for the In-Plane Deformations and Failure of Brickwork", Engineering Bulletin CE8, The University of New Castle, New South Wales, Australia, March 1978.
- [24] Pieper, K. and Trautsch, W., "Shear Tests on Walls", Proc. of the Second International Brick Masonry Conference, (SIBMAC), held in Stoke-on-Trent, England, 1970, pp.140-143.
- [25] Rosenhaupt, S. and Mueller, G., "Openings in Masonry Walls on Settling Supports", Journal of the Structural Division, ASCE, Vol.89, No.ST3, June 1963, part 1, p.107.

- [26] Sinha, B.P., Maurenbrecher, A.H.P., and Hendry, A.W., "Model And Full-Scale Tests on a Five-Storey Cross-Wall Structure Under Lateral Loading", Proc. of the Second International Brick Masonry Conf. (SIBMAC), held in Stoke-on-Trent, England, 1972, pp.201-208.
- [27] Sinha, B.P., and Hendry, A.W., "Racking Tests on Storey Height Shear-Wall Structures With Openings Subjected to Precompression", Design, Engineering and Construction With Masonry Products, Proc. of the International Conf. on Masonry Structural Systems, Austin, Texas, Gulf Publishing Co., Houston, Texas, 1969.
- [28] Smith, B.S. and Carter, C., "Distribution of Stresses in Masonry Walls Subjected to Vertical Loading", Proc. of the Second International Brick Masonry Conf. (SIBMAC), held in Stoke-on-Trent, England, 1970, pp.119-124.
- [29] Sutherland, R.J.M., "Reinforced and Prestressed Masonry, What Do You Want In A Code?", The Structural Engineer, Vol.57A, No.4, April 1979, p.119.
- [30] Yokel, F.Y. and Dikkers, R.D., "Strength of Load Bearing Masonry Walls", Journal of the Structural Division, ASCE, Vol.97, 1971, pp.1593-1609.
- [31] Yokel, F.Y. and Fattal, S.G., "Failure Hypothesis For Masonry Shear Walls", Journal of the Structural Division, ASCE, Vol.102, No.ST3, March 1976, pp.515-532.
- [32] Yorkdale, A.H., "Masonry Building Systems", Technical Committee No.3, Structural Systems, State of the Art Report No.9, Proc. of the International Conf. on Planning and Design of Tall Buildings, Vol.1a, pp.567-589.
- [33] Zienkiewicz, O.C., "The Finite Element Method", Third Edition, McGraw-Hill Book Company (UK) Limited.

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