BEHAVIOUR OF SHEAR CONNECTED CAVITY WALLS

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ABSTRACT

Cavity wall construction involves two wythes separated by a 25 to 100 mm air space. The cavity walls that were investigated had an external brick veneer wythe and an inner wythe of concrete blocks or· metal studs and plywoods. In order to transfer the out-of-plane pressure from the facing to the backup wythe, a connector with shear resisting capability was developed.

Furthermore, the relative deformations between the two wythes due to environmental factors, such as humidity and temperature change, and due to material properties, induce significant internal forces to the masonry components and connectors:

Experimental and analytical studies were conducted to evaluate the performance of cavity walls under positive lateral pressure and opposite movements of the two wythes.

The testing program consisted of twenty three concrete block wall segments with a shear connector, twelve full scale cavity walls subjected to lateral pressure and a full size masonry cavity wall exposed to the climatic conditions.

The analytical study included an elastic finite element model that was developed to calculate the internal forces and deformations of masonry cavity walls subjected to in-plane loads, out-of-plane pressure and vertical movements. After being confirmed with the test results the finite element model was used for parametric studies. Finally, a simplified method was proposed to calculate the maximum internal forces and deformations of a masonry cavity wall subjected to differential vertical movements.

ACKNOWLEDGEMENTS

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CHAPTER 1

INTRODUCTION

1.1 Cavity Walls

Cavity wall construction involves two wythes separated by 25 to 100 mm cavity. The cavity consists of a continuous air space and an insulation. The two wythes are tied together by one of the different available types of connectors. This type of wall assembly has become very popular in North America due to its excellent thermal resistance and its ability to control moisture penetration and sound transmission.

The interior or backup wythe can be a load bearing or a non-load bearing wall. Load bearing backup wythes, in most cases, are made of concrete masonry units and can therefore be reinforced or unreinforced. Non-load bearing backup wythes are made of either concrete block units or metal studs and gypsum boards. The backup wythe is a structural element and as such it requires to be designed to carry all applied loads and to accommodate movements and deflections. The cavity walls having a concrete block wythe as a backup system will be referred hereafter as masonry cavity walls while the ones having a metal stud backup will be referred as brick - metal stud cavity walls.

The exterior, or facing wythe is usually constructed with burned clay, or concrete units and it is referred to as masonry veneer. Traditionally, the veneer has served to provide for architectural appearance, weathering and moisture control. Current design codes such as CAN3-S304-M84 "Masonry Design for Buildings" (Ref. 1) do not consider or utilize the facing wythe as a structural component. All external loads applied to the veneer are assumed to be transferred to the backup system using arbitrary spaced devices referred to as connectors.

Most of the research in cavity walls has its focus in developing connectors capable of transferring the lateral pressure from the facing wythe to the backup wythe. This study investigated the performance of a cavity wall usmg connectors with shear resisting capability. Such a shear connector was developed recently at the University of Alberta (Ref. 3) and was investigated and modified with the present research.

1.2 Origin of Internal Forces of a Cavity Wall

Knowledge of the applied loads and deformations that can occur in a cavity wall is essential to the understanding of its structural behavior.

The significant deformations that can occur to a structural element during its lifetime are due to loading and to environmental factors such as humidity and temperature· changes. In the case of a masonry wall both external loads and imposed deformations due to environmental factors may generate large internal stresses for which the wall must be designed.

The origin of internal stresses for a cavity wall are in-plane loads, out-of-plane pressure and differential deformations between the two wythes due to material properties and environmental factors.

1.2.1 Vertical Loads

Vertical loads result from the weight of roof systems passing into the wall assembly or from upper floors. These loads are usually directly applied to the backup wythe. A conservative design of the concrete block backup wythe, for this type of load condition, can be carried out using the provisions of CAN3-S304-M84 "Masonry Design for Buildings" (Ref. 1).

1.2.2 Out - of - Plane Loads

Out-of-plane load is primarily due to wind pressure or earthquake load. The wind pressure is applied to the facing wythe. By providing a "capable connection" between the two wythes the veneer is tied to the backup • system and the lateral pressure is at present assumed to be entirely transferred to the backup structural element.

A large portion of this study is dedicated to obtain the optimum location and spacing of shear resisting connectors in order to improve the performance of a cavity wall subjected to positive lateral pressure.

1.2.3 Internal Forces Due to Climatic Conditions and Material Properties

Concrete block units, mortar and grout, having water as an essential ingredient in the mix and being moist cured, undergo a period of dying after construction. This loss of moisture results in shrinkage, the amount of which depends upon the composition of the component material and upon environmental humidity. Concrete block masonry can be expected to undergo, after construction, a linear shrinkage of the order of 0.02% to 0.03%. This value will be greater for lightweight concrete blocks made with expanded shale aggregates, and less for concrete blocks that have been autoclaved or steam cured at high pressure. Subsequent wetting and drying results in lesser amounts of expansion and contraction.

Bricks, on the other hand, having been fired in kilns, experience expansion over an extended. period of time. This expansion is the result of hydration of amorphous materials that are formed during the burning of the bricks. While there 1s some shrinkage taking place in the mortar at the same time, there is generally a resultant expansion. A typical value of brick masonry linear expansion after two years would be of the order of 0.015% to 0.025%.

It is important to account for these deformations, especially, when the two masonry materials (brickwork and blockwork) are used in close proximity. In the case where the two wythes are bound together using shear connectors the opposite vertical movements between the two wythes induce significant forces at the connectors.

On the other hand, the connectors restrain these movements and generate internal forces in the masonry components.

In addition, as is common for other materials, masonry will expand when heated and contract when cooled. This will lead to differential deformations between the interior walls in a building, which are at a relative constant temperature, and the exterior walls exposed to the weather temperatures. Again, in the case of a shear connected cavity wall these deformations due to temperature effect will generate internal forces at the connectors.

In order to investigate the behavior of shear connected cavity walls under imposed deformations due to environmental factors (such as humidity and temperatures changes), an experimental and theoretical study were conducted at the University of Alberta.

1.3 Objectives

The main objectives of this research are:

1. To investigate the performance of a cavity wall with shear connectors subjected to out - of - plane loads.

2. To estimate the internal forces induced m a masonry cavity wall due to environmental factors and material properties.

3. To estimate the distribution of the connectors' internal forces along the wall, to establish the location of the critical shear connectors and to recommend the optimum location for connector spacing with due consideration given to ease of construction and placing of both the connector and accessories.

4. To develop a computer model to help predict the internal forces and deformations of a shear connected cavity wall subjected to any kind of loads.
5. To establish a simplified method for hand calculation of forces and deformations of a shear connected cavity wall due to climatic conditions and material properties.
6. To experimentally and analytically determine the effect that certain geometric parameters (ie. concrete block width, cavity width, shear connector arrangement, reinforcement etc.) and material properties have on a cavity wall.
7. To compare the performance of masonry cavity wall to walls constructed with metal studs and gypsum boards.
8. To develop a practical shear connector.

CHAPTER 2

EXISTING CONNECTORS AND DEVELOPMENT OF SHEAR CONNECTOR

2.1 Introduction

The current design procedures as they relate to cavity wall connectors are presented in the first section of this chapter. Also commercially available connector types are briefly discussed. Furthermore, this chapter introduces the concept of "shear connector", reviews the previous investigation and presents the connector prototype that will be used throughout this study.

2.2 Current Design Procedures

The increasing use of a large variety of connectors led the Canadian Standard Association to develop a code exclusive to the design and specification of masonry connectors. This code which is entitled as CAN3 - A370 - M84 (Reference 2) subdivides the masonry connectors into two categories, the standard and non-standard connectors.

The standard connectors are those which have been used traditionally and which are described in the code. For each of these type of connectors a set of guidelines governing their use is presented in the code. These recommendations relate to the configuration of the connector, fabrication, strength requirements, corrosion resistance, construction procedure and connector spacing.

Non-standard connectors are those not conforming to the requirements of Clause 9 of Reference 2. The code recommends that these connectors should be tested with one of the methods described in Clause 11, and if these connectors meet standard connector requirements for strength or corrosion they can be. safely used. However, if the strength resistance is found to be weaker or stronger with respect to their standard counterparts, the connector spacing may be decreased or increased, respectively,

2.3 Commercially Available Connector Types

For the completeness of this report the commonly used connector types will briefly be presented. It is not within the scope of this research to study and compare the existing connector types. An extensive review of the available connector types can be found in Reference 3.

All the connectors that will be presented perform the same basic function they tie the facing wythe to the backup wythe and serve to transfer the lateral load acting on the exterior veneer to the backup system.

The commercially available connectors can be classified into three categories :

- continuous welded connectors,
- individual placed connectors,
- special design connectors.

Specially designed connectors are those manufactured for a specific project. Due to the large variety of these connectors, they will not be considered in this report.

2.3.1 Continuous Welded Connectors

This type of connector is also commonly referred as joint reinforcement. It consists of two or more parallel longitudinal steel rods embedded in the mortar bed joints of both wythes. Cross rods· are welded to the longitudinal rods. Depending on the particular pattern that the cross rods form with the longitudinal bars the continuous welded connectors may be subdivided into the following categories

(a) ladder: the cross rods are butt-welded perpendicular to the longitudinal rods. Longitudinal rods are placed in both wythes. See Figure 2.1

(b) truss: the cross rods are welded diagonally to parallel longitudinal deformed rods forming a truss design. Again. the longitudinal rods placed in both wythes. See Figure 2.2

(c) ladder or truss with rectangular ties: same as the previously described patterns with the difference that the longitudinal rods exist only within the concrete block wythe. Rectangular box-shape ties are then welded to the longitudinal rods in the block wythe. These box shape ties extend across the cavity and sit in the bed joint of the brick veneer, thereby connecting the two wythes. See Figure 2.3

The first two patterns can have one or more longitudinal rods m each mortar bed. The third pattern must have two longitudinal rods in the concrete block wythe. The continuous welded connector can be found in a non-adjustable and adjustable form. The adjustable connectors are identical to their non-adjustable counterparts with the exception that they allow movement of the rectangular ties with respect to the rest of the connector. Adjustable continuous connectors are shown in Figure 2.4. These systems are especially useful where :

- adjacent wythes do not course,
- the backup wythe is erected ahead of the insulation and veneer wythe,
- it is desired to hold rigid insulation in place in the cavity with mechanical attachment.

Continuous welded connectors are used to transfer lateral load from the facing to the backup wythe, to. provide lateral reinforcement for beam action of the wall system, and finally to control shrinkage cracking and cracking caused by foundation movements.

2.3.2 Individually Placed Connectors

These connectors are placed individually at an appropriate spacing specified in CAN3 - A370 - M84 (Reference 2).

As with the continuous welded connectors of Section 2.2.1 these connectors may also be found in a non-adjustable and adjustable form. As seen in Figure 2.5 the non-adjustable individual connectors are simple in design and can be found in three basic shapes

- corrugated strip
- Z-shape rod
- rectangular (box).

The non-adjustable individual connectors are used where the courses of the two wythes are expected to line up.

When the adjacent wythes do not line up, adjustable individual connectors are commonly used. The most common types of adjustable individual connectors can be found in four different types:

(a) corrugated strip with twisting Z-type
(b) adjustable Z-type
(c) rectangular with rectangular rod
(d) plate type with rod (Figure 2.6).

2.4 Connectors with Shear Resisting Capability

2.4.1 The Concept of Shear Resisting Connectors

As it was stated in Section 1.2.2 the performance of a cavity wall subjected to out-of-plane load (ie. wind load) depends on the ability of the connection to transfer the applied lateral pressure from .the veneer to the backup system. In addition. under a positive lateral pressure and assuming a good connection between the two wythes, the brick veneer would be forced into compression and the backup wythe into tension.

From the above it can be concluded that the major ultimate limit states for a cavity wall under positive lateral pressure are :

1. Tensile failure of the backup system. This potential mode of failure can be improved by providing adequate reinforcement if the backup system consists of concrete blocks or by placing thicker metal studs in closer proximity for the case of a metal stud backup wythe.

2. Failure of the connectors by buckling or punch-out or pullout from the mortar beds in which they are placed. That potential mode of failure can be improved by providing a stiffer connector with more end restraints.

3. Failure of the connectors in shear.

4. Compressive failure of the brick veneer.

The serviceability limit state of such a cavity wall has to do with excessive deflections. In order to minimize cracking width and water penetration deflections should not exceed the value of L/720. In order to improve the serviceability limit state a connector which. will provide larger rotational restraint between the two wythes was needed.

All the above led to the development of a stiffer connector with increased shear and rotational restraints. This new connector with shear resisting capability is referred to in this study as a shear connector.

By increasing the restraints between connections and wythes the internal stresses due· to material properties and climatic conditions may increase. The ideal type of connector for a cavity wall will, therefore, be one that is stiff enough to transfer the load to the backup wythe and enough flexible to accommodate the vertical movements of the, two wythes. The main objective of this research was to develop a shear connector which will partially restraint the vertical movements between the two wythes without inducing large stresses due to material properties and temperature effects.

2A.2 Mullins and O'Connor's Shear Connector

The first attempt to develop a connector with improved end restraint was made by Mullins and O'Connor at the University of Queensland, Brisbane, Australia. Reference 4 reports the preliminary results of their test program. The connector that they developed consists of a length of sheet metal which is placed continuously within the height of the cavity, perpendicular the two wythes. The wythes are then connected to the sheet metal by tabs extending into the head joints at every other course. Figure 2.7 shows this shear connector.

A comparison of Mullins and O'Connor's shear connector with the traditional continuous welded connectors (Section 2.3.1) showed that the shear connector resulted in an improved performance of the cavity wall under out-of-plane load.

The drawback of this connector was that both wythes must be identical in both unit size and course height. This limited the practical application of Mullins and O'Connor's shear connector. Another disadvantage is that this connector totally restraints the vertical movements between the two wythes. As it was stated in Section 2.4.1 the above may lead to large internal stresses which can cause failure of the masonry components.

2.4.3 Shear Connector used in this Study

The connector that was developed and used throughout this study can be classified as an adjustable individual non-standard connector. As it is stated in CAN3-A370-M84 (Reference 2), testing is required in order to develop design guidelines for the locations and spacing of such connectors within the cavity.

Figures 2.8 and Plate 2.1 show the initial connector prototype that was investigated in Reference 3. Figure 2.9 and Figure 2.10 show the connector used in this study. The basic components of both connectors are the same. They consist of a plate, cross legs, bent rod and a device to hold the insulation.

The cross legs are inserted into holes a and b and are embedded into the center of face shells of the block as in Figure 2.10 These cross legs provide a rotational restraint between the plate and the block wythe. They have a diameter of 4.76 mm and a length of 100 mm. The distance between holes a and b is :

$$L_1 = (\text{Block Width}) - (\text{Face Shell Thickness})$$

The main plate consists of a galvanized steel plate of gage 14 to 16 depending on the cavity width and the design value for the lateral pressure. The plate length, L, can be altered to accommodate different widths of the backup wall, the insulation and the air space. A tolerance of 7 mm must exist between the end of the connector within the cavity and the inner face of the veneer as it is specified in Clause 3.14 of Reference 1. As is shown in Figure 2.10, the plate length is :

$$L = (\text{Block Width}) + (\text{Cavity}) + (\text{Brick Width}) - 7 - 5$$

The height, H, of the plate must be between 60 and 100 mm in order for the tie (bent rod) to be always horizontal when it is embedded in the mortar bed of the veneer.

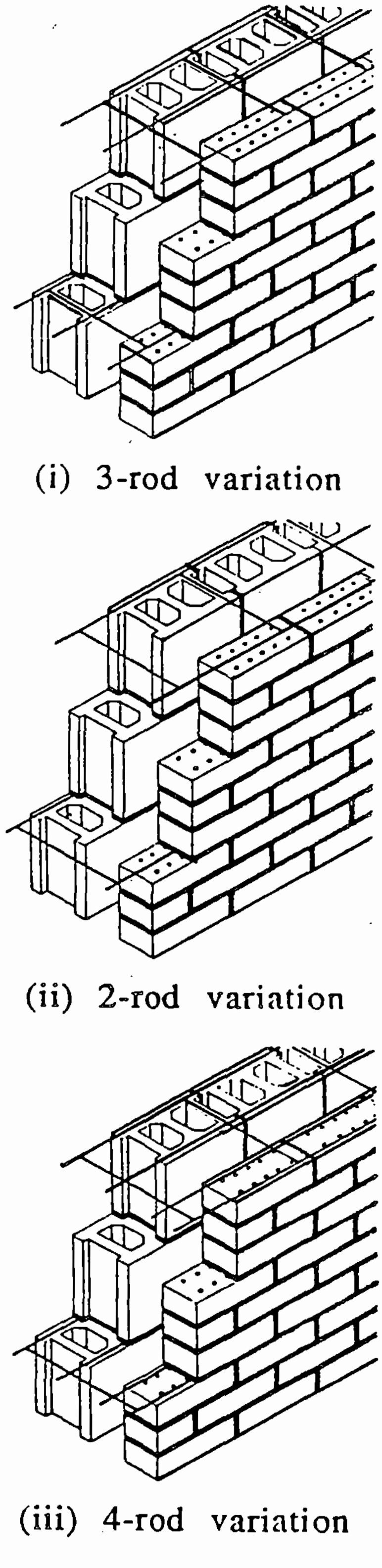
The connection of the brick veneer is achieved by inserting a rod (bent rod, see Figure 2.9) into one of the several holes in the plate enabling horizontal placement of the tie m the brick mortar joint as shown in Figure 2.10. Rotation is, therefore, allowed between the plate and the brick wythe. This connector therefore partially restraints lateral movements and rotation between the two wythes. The bent rod has a diameter of 4.76 mm. One of the differences between the connector of Figure 2.8 and Figure 2.9 1s that the tie used in this study has open legs. This improves its pull or push out resistance. The dimensions of the tie are given in Figure 2.9.

A special device is also used to hold the insulation against the backup system. For the connector of Figure 2.8 a slot was utilized to accept a wedge which holds the rigid insulation against the masonry block backup wythe. A more convenient device was developed which consists of a plastic insert with a slot that fits in front of the connector plate as it is shown in Figure 2.9 and Figure 2.10.

In order to minimize the thermo-bridge between the two wythes, five to eight large holes, depending on the width of the cavity, are drilled in the part of the metal plate within the cavity. That was also one of differences between the connectors of Figure 2.8 and Figure 2.9.

The connector presented in Figure 2.9 can also be used when the backup wythe consists of metal studs. In this case the connection of the shear connector with the metal stud can be achieved by using four screws placed at the holes a, b, c and d shown in Figure 2.9. Again, rotational restraint is provided between backup wythe and connector. The larger the spacing between the screws the greater the resisting moment of the connection. Plate 2.2 shows the placement of a shear connector in a "brick - metal stud" backup system.

After asking the opinion of masons about the ease of placement and the probability of misplacing or forgetting one of the parts of the connector, a further simplification was suggested to the shear connector without losing any of its advantages. The two cross rods were neglected and the rotational ·restraint between block wythe and connector was achieved by bending the part of the connector embedded in the concrete blocks 90°. The shear connector is therefore embedded in the horizontal and vertical joints of the concrete block wythe. The part of the connector plate embedded in the concrete block is corrugated and has holes that will interlock with the mortar and will mcrease its pullout/punch-out capacity. The simplified shear connector is shown in Figure 2.11 and Plate 2.3.

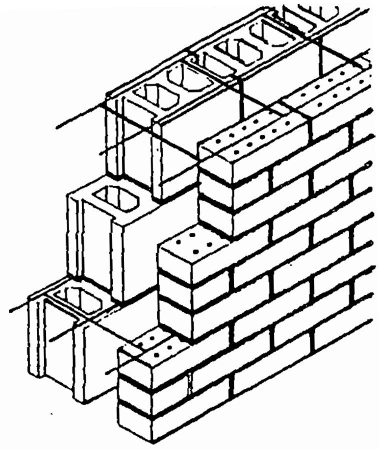


(i) 3-rod variation

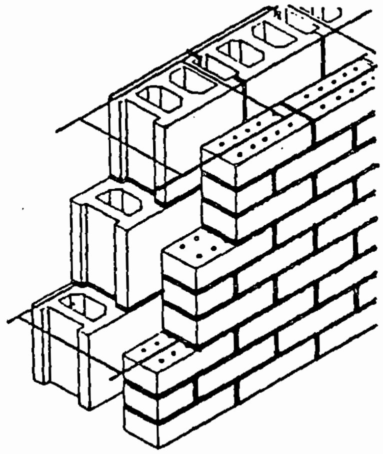
(ii) 2-rod variation

(iii) 4-rod variation

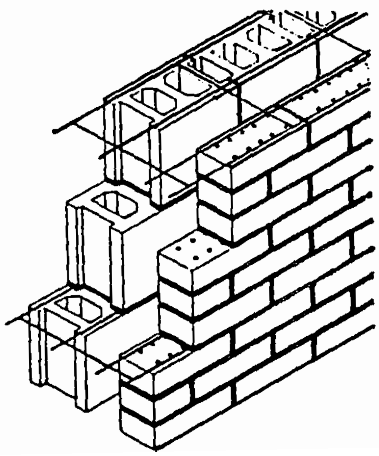
Figure 2.1 Non-Adjustable Ladder Pattern Connectors (Courtesy Reference 3)



(i) 3-rod variation

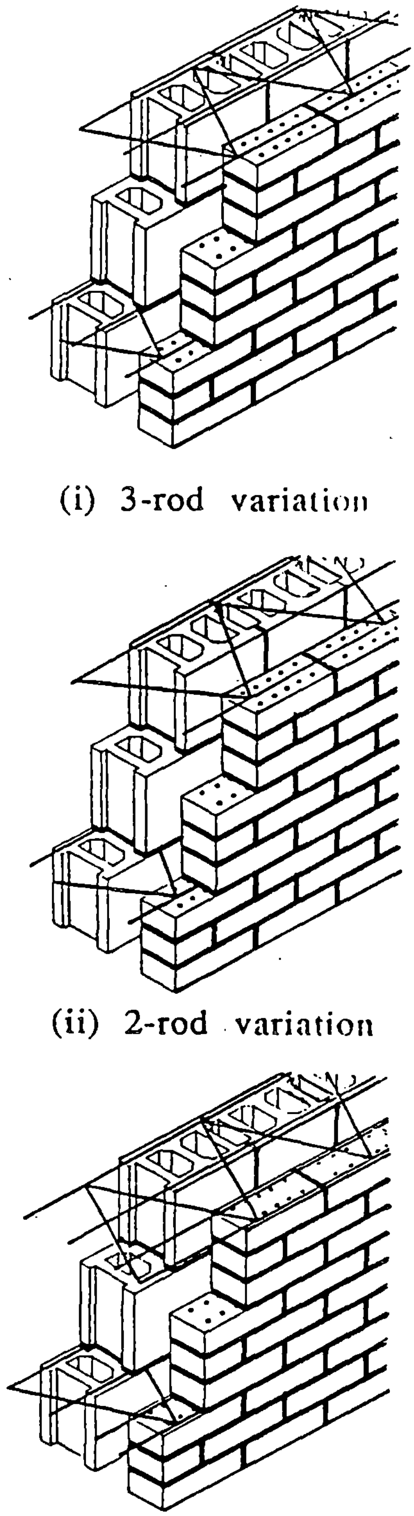


(ii) 2-rod variation



(iii) 4-rod variation

Figure 2.2 Non-Adjustable Ladder Pattern Connectors

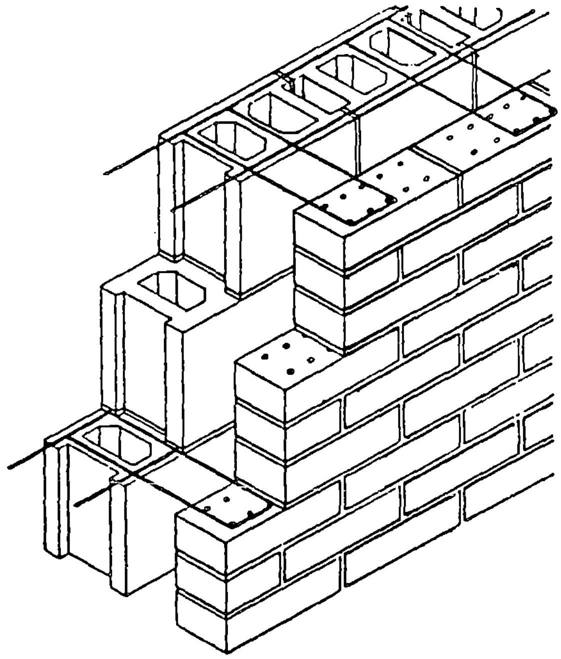


(i) 3-rod variation

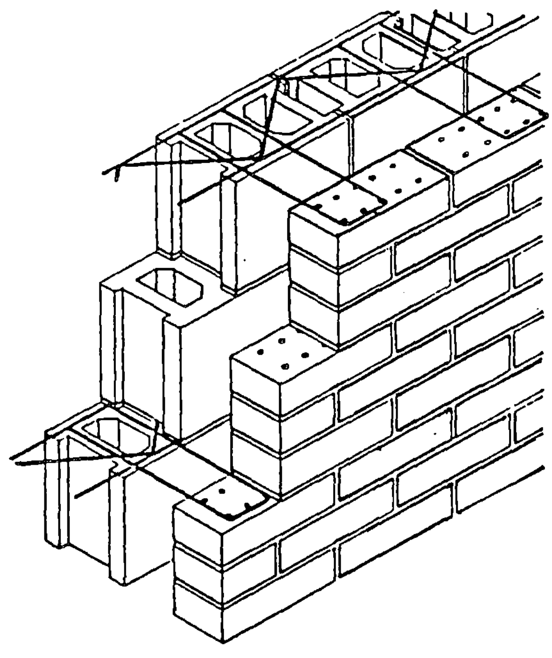
(ii) 2-rod variation

(iii) 4-rod variation

Figure 2.3 Non-Adjustable Truss Pattern Connectors

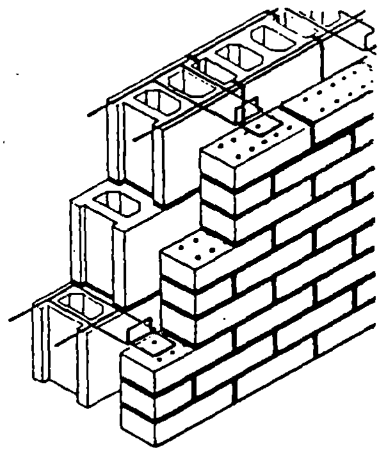


(i) ladder-type variation

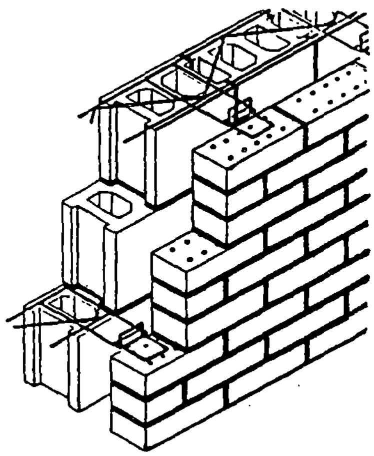


(ii) truss-type variation

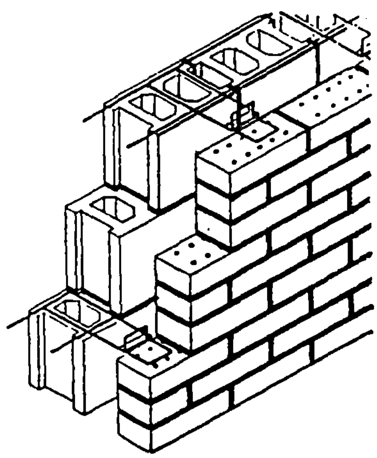
Figure 2.4 Non-Adjustable Ladder/Truss Pattern With Rectangular Ties



(i) ladder-type variation A



(ii) ladder-type variation B



(iii) truss-type variation

Figure 2.5 Adjustable Ladder/Truss Pattern With Rectangular Ties

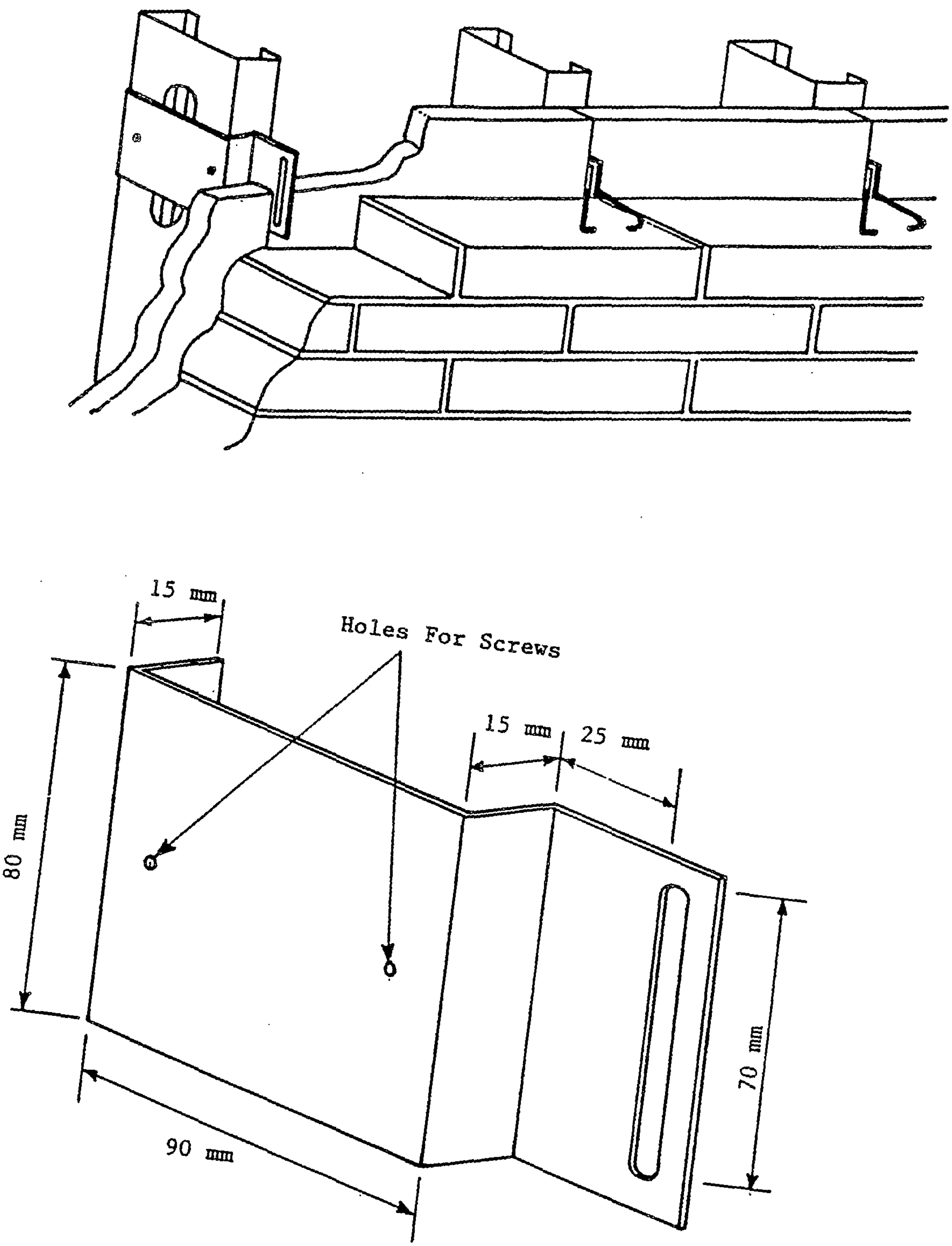


Figure 2.6 Adjustable Individually Placed Connectors
(Courtesy Ref. 3)

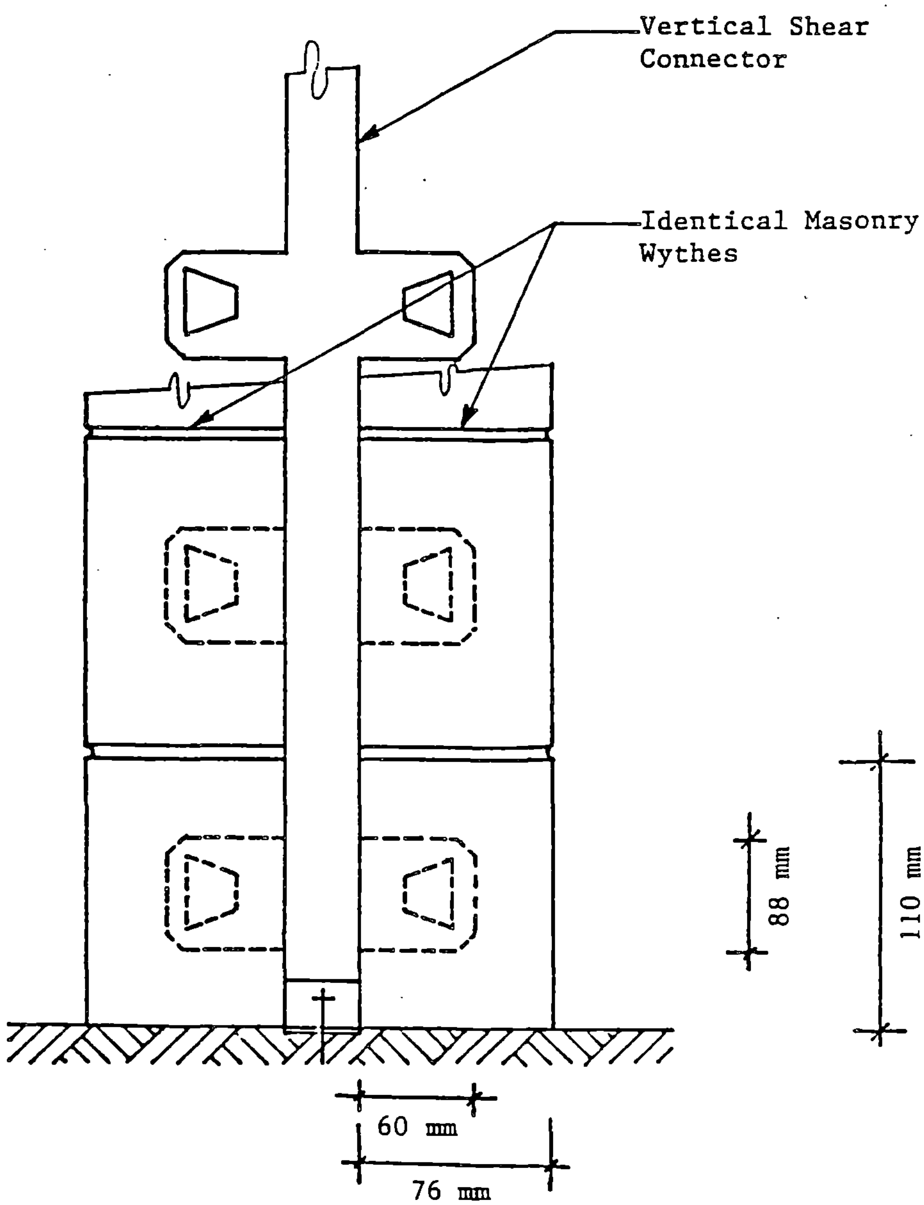


Figure 2.7 Mullins and O' Connor 's Shear Connector (Courtesy Ref. 3)

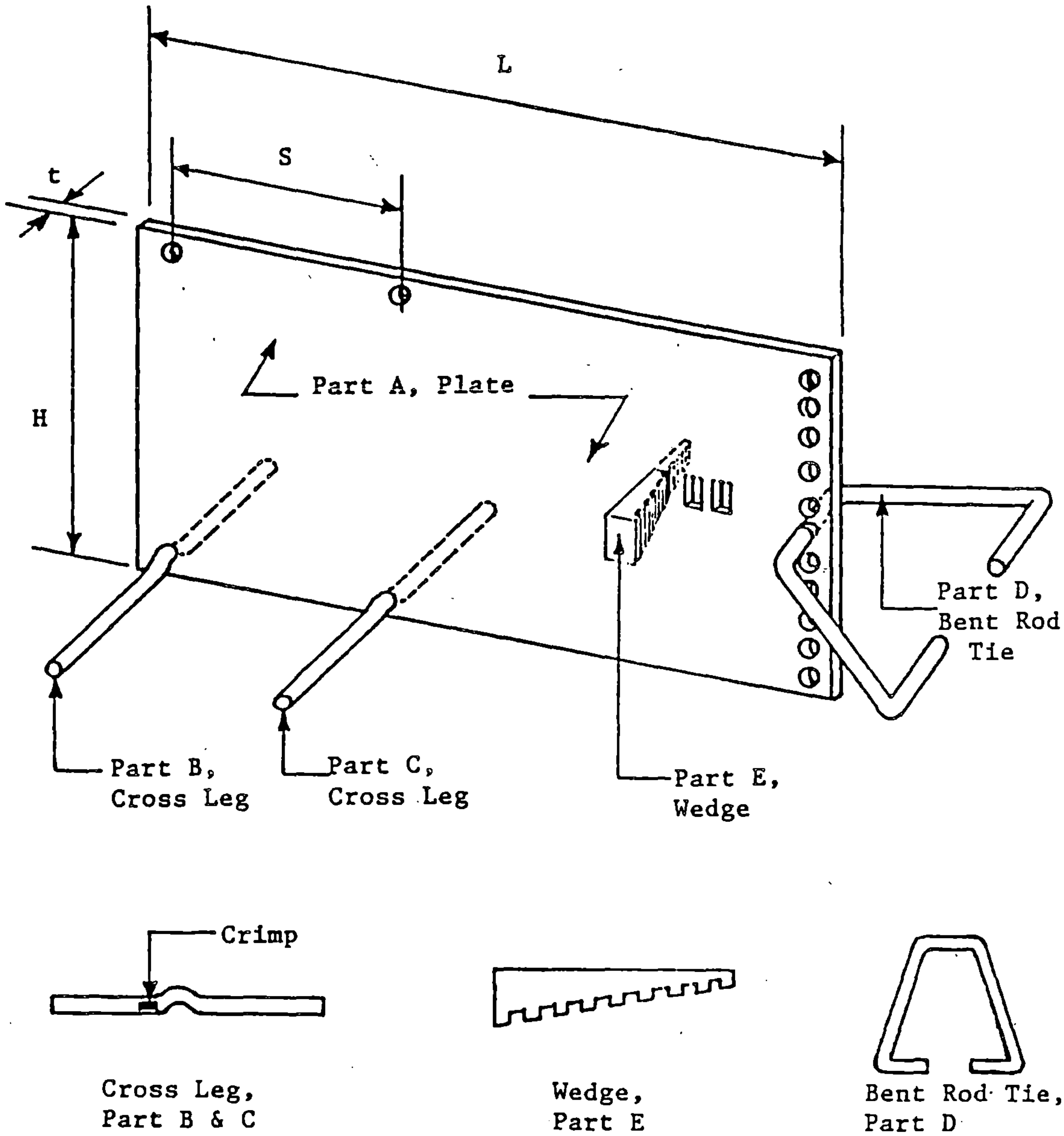


Figure 2.8 Shear Connector Developed at University of Alberta
(Courtesy Ref. 3)

Shear Connector Prototype

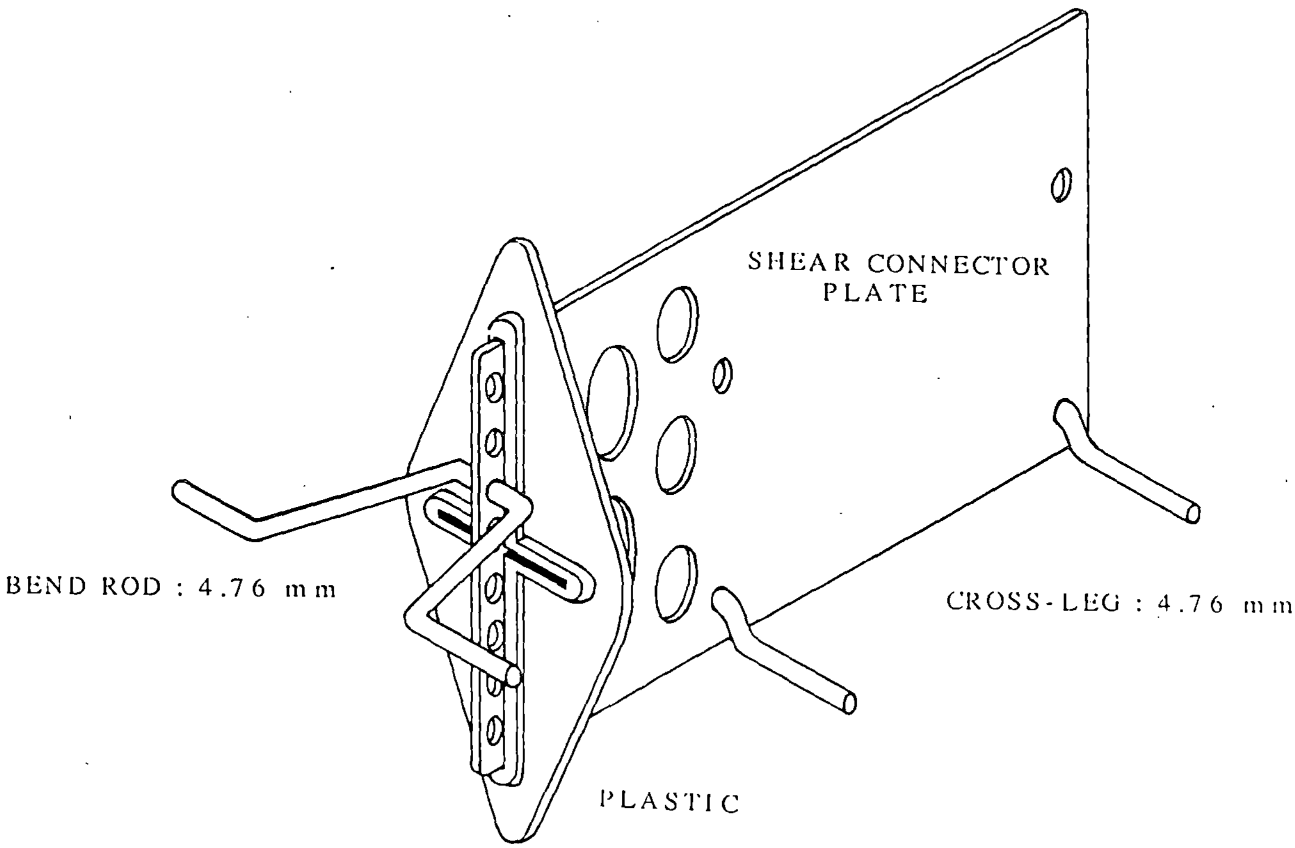


Figure 2.9 Shear Connector used in this Research

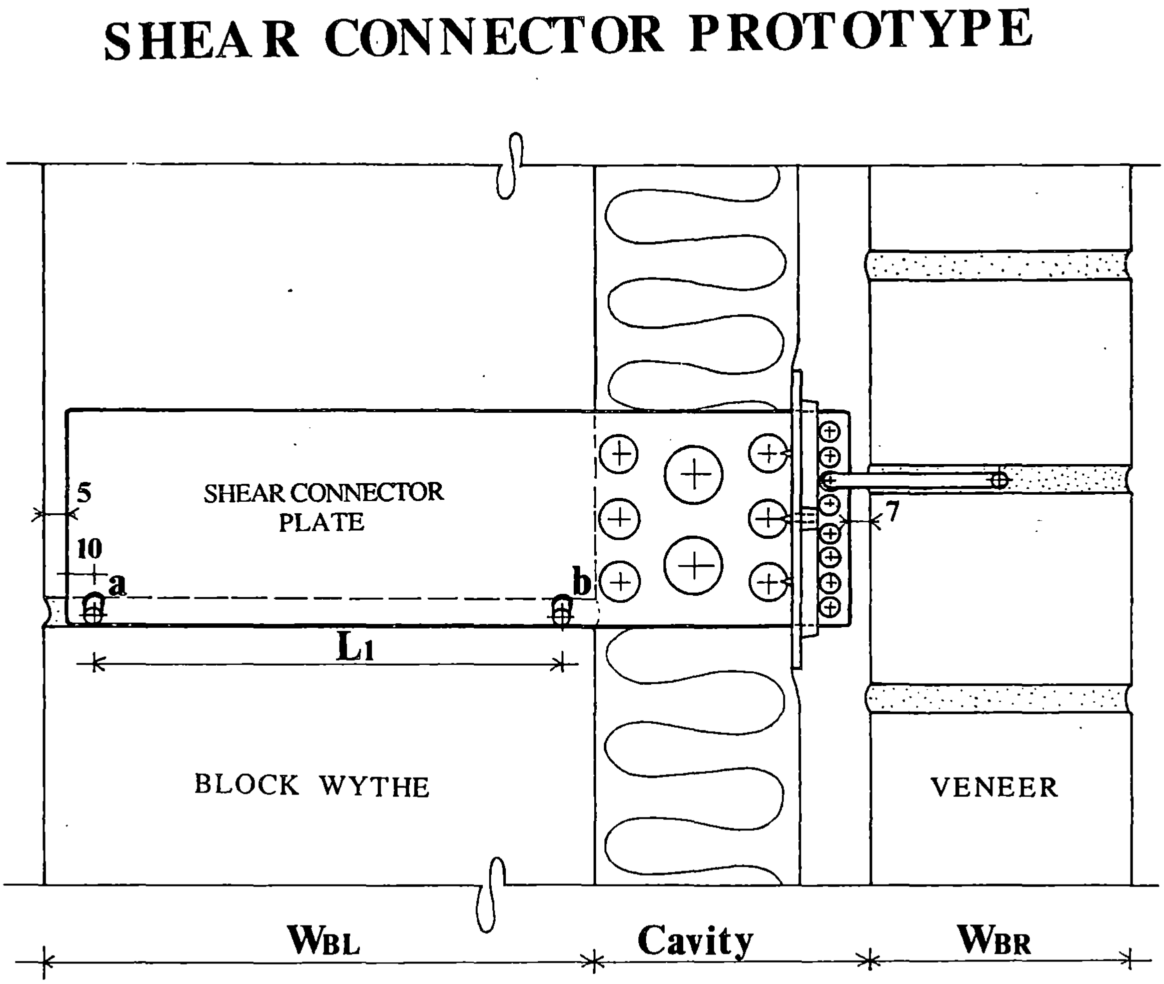
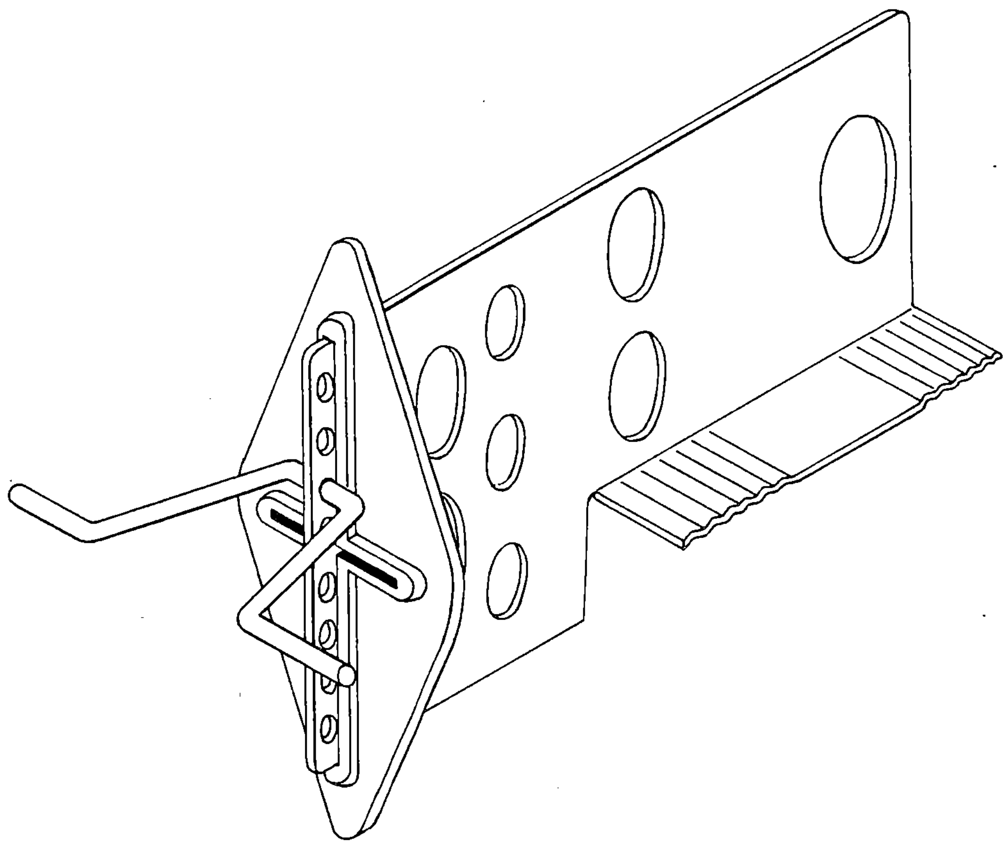


Figure 2.10 Shear Connector Placement in a Cavity Wall



SHEAR CONNECTOR LENGTH

Block Width (mm)	Cavity Including Insulation (mm) 25	50	75	100
140	243	268	293	318
190	293	318	343	368
240	343	368	393	418
290	393	418	443	468

Figure 2.11 Simplified Shear Connector

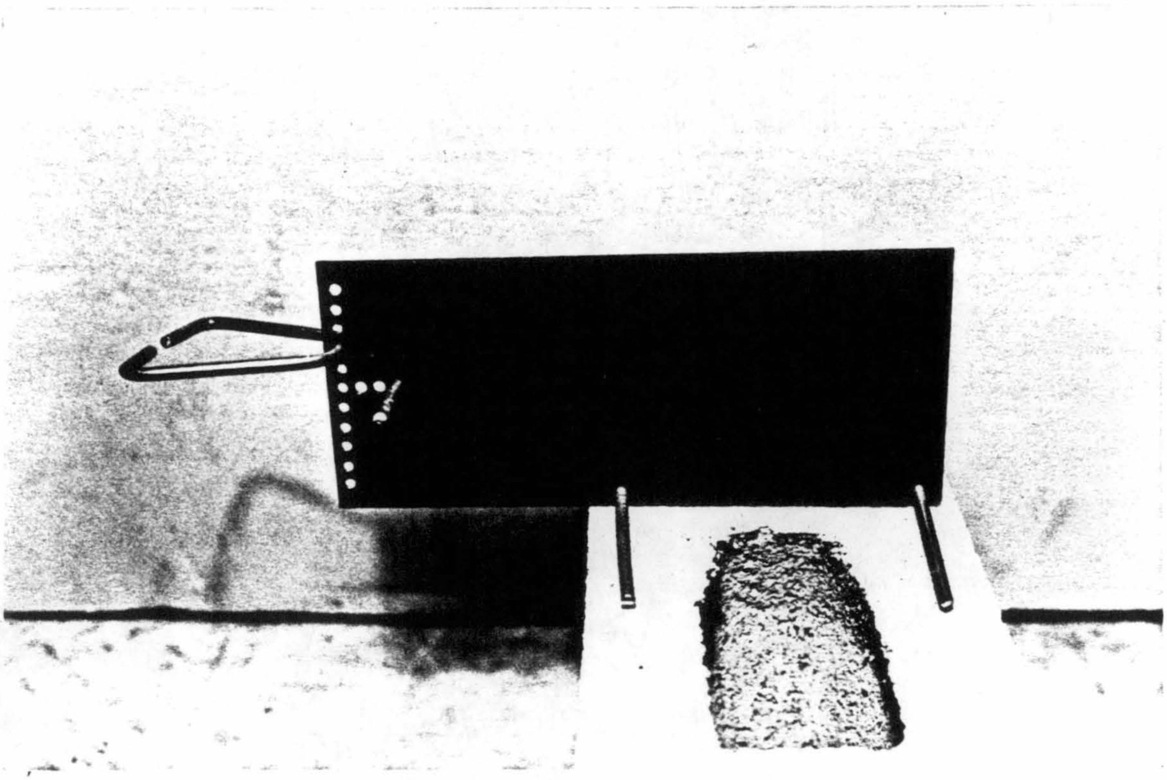


Plate 2.1 Shear Connector Prototype (Courtesy ref. 3)

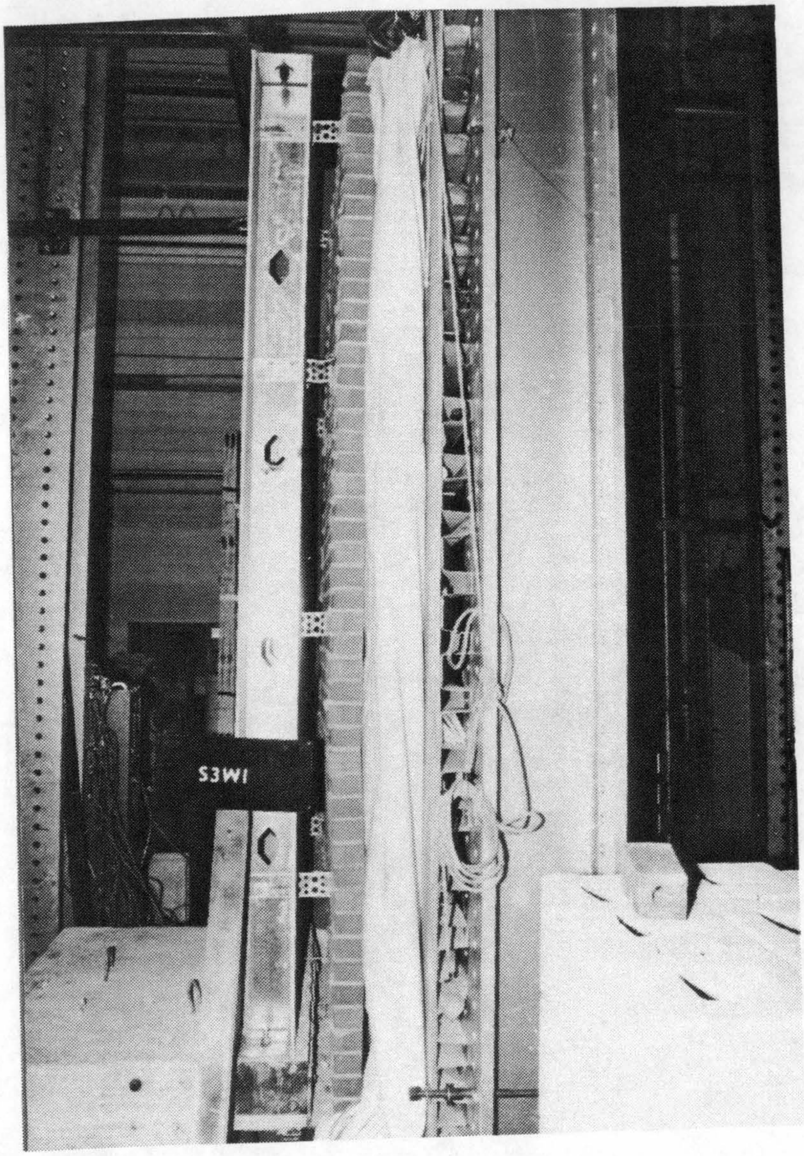


Plate 2.2 Shear Connector Placement in Metal Stud Backup System

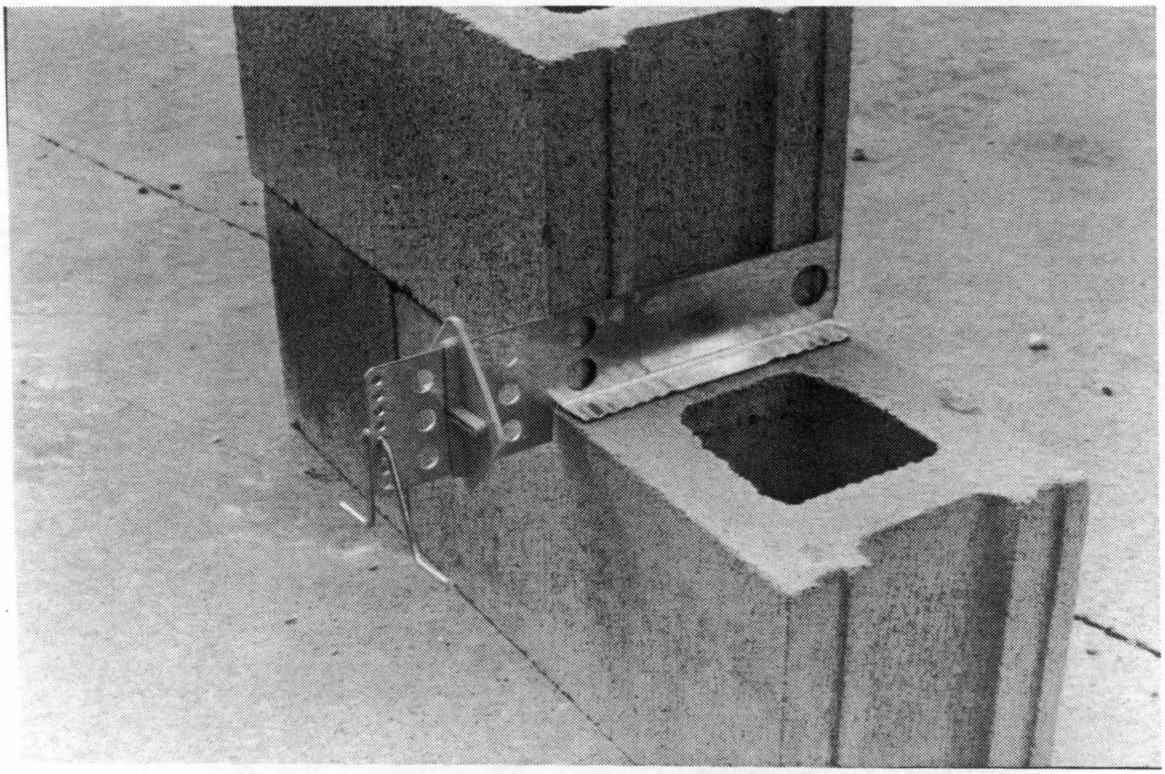


Plate 2.3 Simplified Shear Connector

CHAPTER 3

THEORETICAL ANALYSIS

3.1 Introduction

In this chapter a two-dimensional model is presented which can be used to predict the elastic behavior of masonry cavity walls subjected to external loads and imposed deformations. The external loads that can be applied to a cavity wall were described in Section 1.2. This frame model can be used by any two-dimensional frame analysis computer program. Furthermore. an approximate method · is presented for hand calculations of the maximum internal forces, induced in the masonry components and critical shear connectors, due to imposed deformations related to material properties and temperature changes. After confirming the validity of the computer model using the experimental results (Chapter 6) it is used in Chapter 7 for further parametric analysis.

3.2 Computer Modeling of the Assembly

The two-dimensional model was developed by assuming constant behavior of the wall assembly along its length and analyzing only a portion having a width of 1.0 m. The plane frame model for a cavity wall system is shown in Figure 3.1. The· model is represented by the solid lines and the simulated cavity wall by the dotted lines. Beam elements were used to model all components. As is known, beam elements are exact elements and can therefore span between points of geometric or/ and material property changes.

Particular attention was given to modelling the shear connector and the support conditions. Detail A in Figure 3.1 shows that the shear connector was modelled using three elements. The first element modelled the portion of the connector plate which is embedded in the vertical mortar joint of the block wythe. A large stiffness was assigned to that element. The second element consists of the part of the connector in the cavity. This element, equals to the length of the cavity. was assigned material and geometric - properties typical of the steel plate. The third element which modelled the portion of the rod tie embedded into the mortar bed of the brick veneer was also assigned a large stiffness. For the parts of the connector embedded in the mortar joints it is suggested that the stiffness and area are chosen equal to those of the masonry components or ten times the area and a thousand times the moment of inertia of the connector, which value is greater.

All connections between elements, except the connection of the bent-rod tie with the connector, were fixed. As it was stated in Section 2.4.3, rotation is allowed between the shear connector plate and the tie. The junction, therefore, between these two parts was simulated by a hinge. The boundary conditions which describe the experimental setup and the typical support conditions of an actual masonry cavity wall are also shown in Figure 3.1. Both wythes are simple supported at the bottom. The top reaction of the concrete block wall is simulated by a roller which allows vertical movement of the wythe. Finally, the brick wythe is not supported at the top.

The types of loads for which a cavtty must be designed were described in Section 1.2. In this study the S-FRAME, which 1s a general purpose finite element program was used. This program could be used to analyse the following load cases for a cavity wall system :

1. Vertical concentrated load and moment at the top joint of the block wall. Such loading pattern can result from eccentric vertical dead and live loads acting on a load bearing cavity wall assembly.
2. Uniformly distributed lateral load along the height of the facing wythe in order to simulate the lateral wind pressure.
3. Self weight of the masonry components.
4. Imposed deformations can also be applied to the element to account for the expansion and shrinkage of the masonry components due to differing material properties.
5. Thermal deformations can be incorporated in the analysis just by specifying the appropriate values for the linear thermal coefficients and the temperature changes.

Any combination of the above load cases can also be included in the computer analysis. Note that the analysis is elastic and superposition of internal stresses is permitted.

3.3 Approximate Method to Calculate Internal Forces due to Material Properties and Environmental Humidity

An approximate method was developed to estimate the maximum internal forces in the masonry components and in the critical shear connector due to imposed deformations. As was stated in Section 1.2.3, these deformations can result from shrinkage of concrete block wythe, expansion of clay brick and thermal deformations of both.

For the above type of load condition the critical shear connector 1s the one at the top and a conservative approach will be to neglect all the other connectors and assume that the restraint to the imposed deformations is provided by- the- top connector. In addition, the modelling of the connector was simplified by neglecting the part of the connector embedded in the masonry components. It is assumed that there is no moment capacity at the junction of the connector and the brick wythe. These · conditions are shown schematically in Figure 3.2. Bending deformations of the two wythes were also neglected in this analysis. With these assumptions the problem becomes statically determinate, and by using a constitutive law, compatibility and equilibrium requirements, a closed form solution for the final deformations ΔBL and ΔBR and the corresponding internal forces Q and M (Figure 3.2) is derived as follows:

Let BBL and BBR be the imposed deformations at nodes 1 and 2 (see Figure 3.2) due to material properties and temperature changes. These deformations correspond to unrestrained movements of the wythes and could be expressed as follows:

$$\delta_{BL} = (\varepsilon_{SH.BL} + \alpha_{BL} \bullet \Delta T_{BL}) \bullet L_{BL}$$
$$\delta_{BR} = (\varepsilon_{EXP,BR} + \alpha_{BR} \bullet \Delta T_{BR}) \bullet L_{BR} \qquad (1)$$

where :

- $\varepsilon_{SH,BL}$ and $\varepsilon_{EXP,BR}$: linear shrinkage and expansion, strain of the concrete block wythe and brick veneer, respectively.
- α_{BL} and α_{BR} are : coefficients of linear thermal expansion for block and brick walls.
- ΔT_{BL} and ΔT_{BR} : relative temperatures of the wythes.
- L_{BL} and L_{BR} : distance of uppermost connector from the bottom of block and brick wythes (see Figure 3.2).

Let Δ_{BL} and Δ_{BR} be the unknown restraint deformations at the junction of the top connector with the block wall and brick veneer The slope - deflection equations for the deformed configuration of the top connector give (see Figure 3.2 b)

$$Q = \frac{3(EI)_C}{C^3} \bullet (\Delta_{BR} - \Delta_{BL}) \qquad (2\text{-a})$$

$$M = \frac{3(EI)_C}{C^2} \bullet (\Delta_{BR} - \Delta_{BL}) \qquad (2\text{-b})$$

Due to load Q the masonry wythes will undergo axial deformations :

$$\delta_1 = +\frac{Q \cdot L_{BL}}{(AE)_{BL}} \quad \text{and} \quad \delta_2 = -\frac{Q \cdot L_{BR}}{(AE)_{BR}} \tag{3}$$

where :

$(EI)_C$: flexural stiffness of the connector.

$(EA)_{BL}$ and $(EA)_{BR}$: axial stiffnesses of masonry components.

C : width of the cavity.

For compatibility :

$$\Delta_{BL} = \delta_{BL} + \delta_1 \tag{4-a}$$

$$\Delta_{BR} = \delta_{BR} + \delta_2 \tag{4-b}$$

Substitute equation (2-a) into (3) and then (3) into (4):

$$\Delta_{BL} = \delta_{BL} + \frac{3(EI)_C}{C^3} \cdot \frac{L_{BL}}{(AE)_{BL}} (\Delta_{BR} - \Delta_{BL})$$

$$\Delta_{BR} = \delta_{BR} - \frac{3(EI)_C}{C^3} \cdot \frac{L_{BR}}{(AE)_{BR}} (\Delta_{BR} - \Delta_{BL})$$

The solution of the above system of equations with unknowns Δ_{BL} and Δ_{BR} can be expressed in a matrix form as follows:

$$\begin{Bmatrix} \Delta_{BL} \\ \Delta_{BR} \end{Bmatrix} = \frac{1}{\alpha + \beta + 1} \cdot \begin{bmatrix} (\beta + 1) & \alpha \\ \beta & (\alpha + 1) \end{bmatrix} \cdot \begin{Bmatrix} \delta_{BL} \\ \delta_{BR} \end{Bmatrix} \tag{5}$$

where :

$$\alpha = \frac{3(EI)_C}{C^3} \cdot \frac{L_{BL}}{(AE)_{BL}} \tag{6-a}$$

$$\beta = \frac{3(EI)_C}{C^3} \cdot \frac{L_{BR}}{(AE)_{BR}} \tag{6-b}$$

Substitute the values of Δ_{BL} and Δ_{BR} back into equations (2) the maximum internal forces can be founded.

Summary:

INPUT :

$$\delta_{BL} = (\varepsilon_{SH.BL} + \alpha_{BL} \cdot \Delta T_{BL}) \cdot L_{BL} \tag{1}$$

$$\delta_{BR} = (\varepsilon_{EXP,BR} + \alpha_{BR} \cdot \Delta T_{BR}) \cdot L_{BR}$$

$$\alpha = \frac{3(EI)_C}{C^3} \cdot \frac{L_{BL}}{(AE)_{BL}} \tag{6-a}$$

$$\beta = \frac{3(EI)_C}{C^3} \cdot \frac{L_{BR}}{(AE)_{BR}} \tag{6-b}$$

SOLUTION :

$$\begin{Bmatrix} \Delta_{BL} \\ \Delta_{BR} \end{Bmatrix} = \frac{1}{\alpha + \beta + 1} \cdot \begin{bmatrix} (\beta + 1) & \alpha \\ \beta & (\alpha + 1) \end{bmatrix} \cdot \begin{Bmatrix} \delta_{BL} \\ \delta_{BR} \end{Bmatrix} \tag{5}$$

$$Q = \frac{3(EI)_C}{C^3} \cdot (\Delta_{BR} - \Delta_{BL}) \tag{2-a}$$

$$M = \frac{3(EI)_C}{C^2} \cdot (\Delta_{BR} - \Delta_{BL}) \tag{2-b}$$

3.3.1 Example using the Approximate Method

A cavity wall 3000 mm high consists of 200 mm standard unreinforced concrete block wythe and 90 mm wide brick veneer. The two wythes are separated by 100 mm cavity and are connected using shear connectors spaced at 800 mm in both directions. The geometric and material properties for a strip of a wall section 800 mm wide are:

Block Wall	Brick Wall	Shear connector
E_{BL} = 10 GPa	E_{BR} = 10 GPa	E_{SC} = 200 GPa
A_{BL} = 60.3E3 mm^2	A_{BR} = 72E3 mm^2	A_{SC} = 90 mm^2
I_{BL} = 353E6 mm^4	I_{BR} = 48.6E6 mm^4	I_{SC} = 27E3 mm^4
L_{BL} = 2800 mm	L_{BR} = 3000 mm	C = 100 mm
$\varepsilon_{SH,BL}$ = -0.02%	$\varepsilon_{EXP,BR}$ = 0.04%	
α_{BL} = 5E-6	α_{BR} = 3.6E-6	
ΔT_{BL} = 18 'C	ΔT_{BR} = -15 'C	

SOLUTION:

- From eqn. (1) : δ_{BL} = -0.308 mm
  δ_{BR} = 1.038 mm
- From eqn. (6) : α = 0.0752
  β = 0.0675
- By substituting these values into (5) the final deformations Δ_{BL} and Δ_{BR} can be found:

$$\begin{Bmatrix} \Delta_{BL} \\ \Delta_{BR} \end{Bmatrix} = \begin{bmatrix} 0.934 & 0.066 \\ 0.059 & 0.941 \end{bmatrix} \bullet \begin{Bmatrix} -0.308 \\ 1.038 \end{Bmatrix} \quad \text{or}$$

$\Delta_{BL} = -0.219$ mm

$\Delta_{BR} = 0.958$ mm

- The internal forces and moment which must be resisted by the masonry components are then given by equation (2):

$Q = 19.07$ kN and

$M = 1.907$ kN.m

SOLUTION SUMMARY:

1. Due to this type of load the brick wythe has to resist a compressive stress of :

$$\frac{19.07 \bullet 10^3}{72 \bullet 10^3} = 0.265 \text{ MPa}$$

and the block wythe a tensile stress of:

$$-\frac{19.07 \bullet 10^3}{60.32 \bullet 10^3} - \frac{1.907 \bullet 10^6 \bullet 95}{353.4 \bullet 10^6} = -0.829 \text{ MPa}$$

2. Stresses originating from other load conditions can be superimposed on the previously obtained values since the analysis is linear elastic.

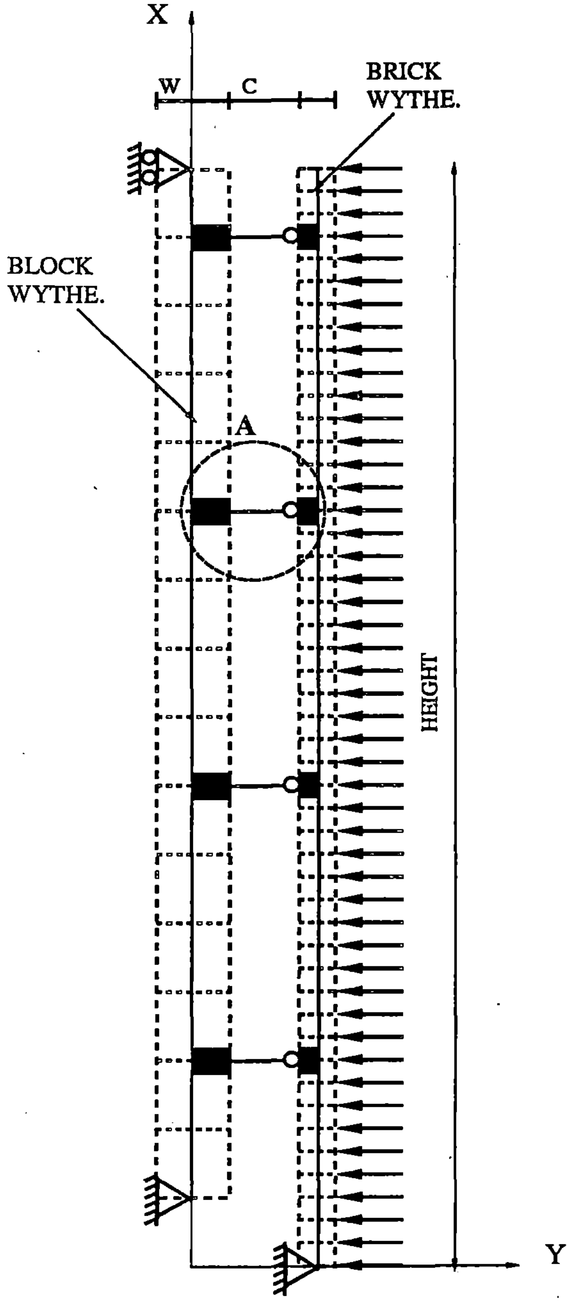
3. A more exact solution of the same example was conducted usmg the S-FRAME computer program and the model described in Section 3.2. The maximum internal forces in the masonry components were found to be at the location of the top support and had the following values : Q = 13.57 kN M = 1.36 kN•M

The deflections at these points were:

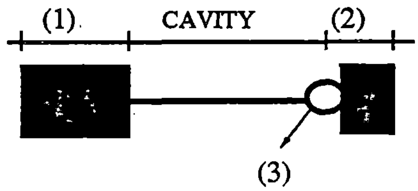
$$\Delta_{BL} = -0.14 \text{ mm}$$

$$\Delta_{BR} = 0.91 \text{ mm}$$

It can therefore be concluded that the approximate method· gave conservative results. In a leter Chapter a wider sample will be used to confirm the above conclusion.



DETAIL A.



(1),(2) For the part of the shear connector embedded in the vertical mortar joint of blocks and for the tie embedded in the mortar bed very large stiffnesses were assumed.

(3) The junction of the tie with the shear connector was modelling by a hinge.

Figure 3.1 Finite Element Model

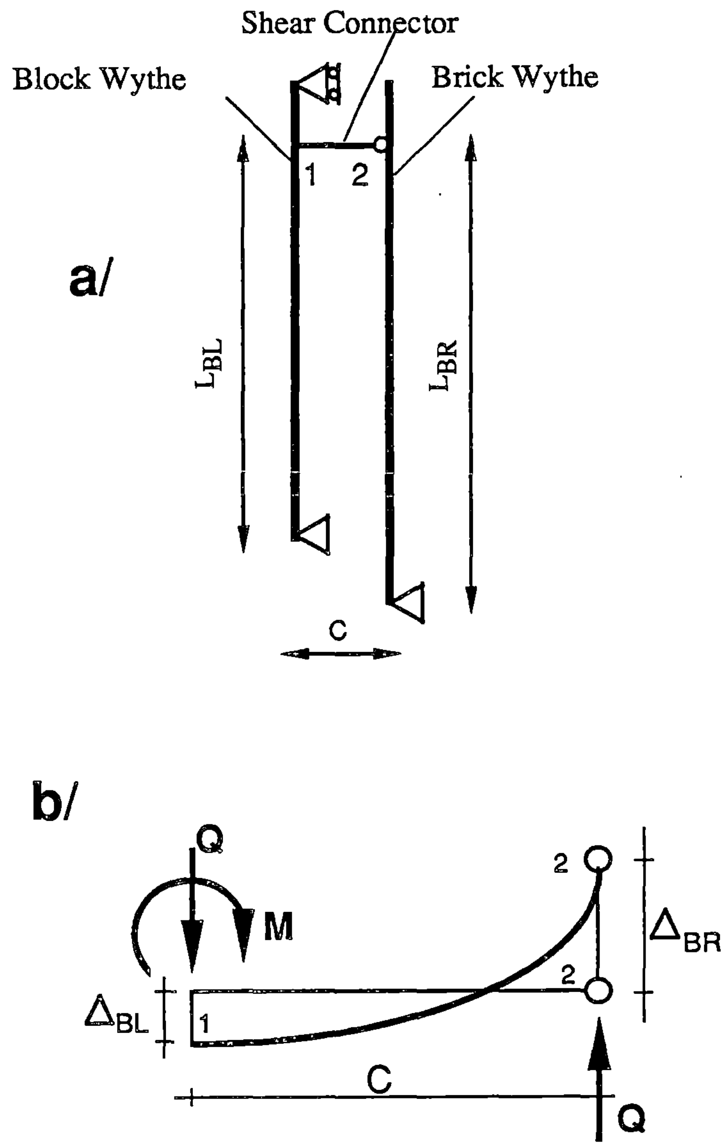


Figure 3.2 ' Approximate Method

CHAPTER 4

EXPERIMENTAL PROGRAM

4.1 Introduction

The experimental program, conducted to investigate the performance of shear connected cavity walls, comprised four different phases:

- The first part consisted of three wall segments (prisms) and was conducted to evaluate the capacity and the mode of failure of shear connectors under horizontal and vertical loads.
- The second part involved the testing of twelve full scale cavity walls subjected to positive lateral pressure.
- The third phase comprised one full scale masonry cavity wall exposed to the climatic conditions.
- The fourth part consisted of twenty wall prisms and was conducted to estimate the pull-out strength of the simplified shear connector.

This chapter describes the materials that were used, the test specimens, the construction sequence, the testing apparatus and the test procedures for the four different phases of the experimental program.

4.2 Materials

The materials that were used in the construction of the various test Specimens are commercially available in the Edmonton, Alberta area, and are typical of those presently used in building construction.

4.2.1 Concrete Block Units

The wall segments and the backup masonry walls were constructed using 150 and 200 standard hollow concrete block units. The 200 mm units have nominal dimensions 200 by 200 by 400 mm. The 150 standard units have nominal dimensions 150 by 200 by 400 mm. Figure 4.1 and Figure 4.2 show schematically the units. The half units- have only one void and 200 mm lerigth instead of 400 mm. The physical properties of the units are listed in Table 4.1.

4.2.2 Clay Brick Units

The facing wythes for all the cavity wall specimens were constructed using burned clay brick units , in accordance with CSA Standard CAN3-A82.l-M1987, "Burned Clay Brick" (Ref. 5). The units had actual dimensions of 90 mm wide by 57 mm high by 190 mm long.

For the unreinforced veneer wall "100 metric standard-wirecut" brick units were used. The reinforced brick veneer walls were constructed by using "100 metric standard-pressed" brick units, which have larger voids and allow placement of wire reinforcement. Figure 4.3 shows schematically the units and their physical properties.

4.2.3 Mortar

The mortar used for all the specimens was premixed mortar with properties according to the specifications given in CSA Standard Al79-M1976, "Mortar and Grout for Unit Masonry" (Ref. 6). Type S mortar, the most common type, was used for all specimens.

Nine 50 mm mortar cubes were cast from each batch, in accordance with ASTM C 109-86 "Test Method for Compressive Strength of Hydraulic Cement Mortars" (Ref, 7). The cubes were subjected to identical curing conditions as their companion prisms and walls. All the cubes were tested at 28 days. The average values are summarized in Appendix A.

4.2.4 Grout

Grout was used to fill the voids of the backup concrete block walls which had rebars in . them. The grout was prepared in the laboratory and consisted of normal (Type 1) portland cement, concrete sand, and 10 mm pea gravel in proportions by weight of 1:2.5:2, It also had a water / cement ratio of 1.

From each batch three test cylinder (150 mm by 300 mm) were cast. They were subjected to identical curing conditions as their companion specimens and tested after 28 days. The average values are given in Appendix A.

4.2.5 Reinforcement

Deformed metric l0M (diameter=l 1.2 mm) bars were used to reinforce some of the backup walls. Their specified tensile yield strength is 300 MPa.

Two of the brick veneer walls were also reinforced. For that" case smooth steel reinforcing wire of gauge 1 (diameter = 7.19 mm) was used with a 230 MPa specified minimum tensile yield strength.

4.2.6 Shear Connectors

The shear connector prototype that was used for these tests 1s shown in Figure 2.9 and was presented in section 2.4.3.

The length, L, of the connector plate varied with the cavity width and block width. The different heights of 60 mm ,70 mm and 75 mm were used. The yield strength of the connector was 230 MPa.

4.2.7 Metal Studs and Metal Tracks

Metal studs and matching metal tracks of 14 gauge metal plate were used for the specimens with the metal stud backup system. Their depth was 150 mm.

4.2.8 Gypsum Board

The metal studs were covered in both sides by a 12 mm thick gypsum board having dimensions of 3000 mm by 1200 mm.

4.3 Test Specimens, Testing Apparatus and Test Procedures.

In the following sections the test specimens, construction sequence, testing apparatus and test procedures used in the different experimental phases are described.

All masonry components were constructed by experienced masons in order to simulate the level of workmanship typical of a well made wall in the field. The wythes were laid in a running bond pattern, having mortar joints with 5 mm raking. All specimens were cured in the laboratory for 28 days prior to testing.

4.3.1 Wall Segments with Shear Connectors

4.3.1.1 Objectives

Three concrete block wall segments with a shear connector embedded within the mortar joint were constructed m order to evaluate the axial and shear strength capacity as well as the mode of failure of the shear connector of Figure 2.9.

In addition, the effects of the type of load applied to the connectors and the holes in which the tie is placed were also investigated. The types of load applied to the connectors were: axial compressive load, vertical upward load and vertical downward load. The location of the tie defines the point at which the load is applied.

4.3.1.2 Specimen Description and Construction

All specimens were identical and consisted of H/15/C/M 200 mm standard concrete blocks. The dimensions of the wall segments were 800 mm by 800 mm. The cross-sectional dimensions of the shear connector were 70 mm high by 1.8 mm thick. The part of the connector outside the mortar joint had a length of 68 mm which corresponds to a cavity of 75 mm. Figure 4.4 shows all the dimensions of the wall segments.

4.3.1.3 Testing Apparatus and Test Procedure

The specimens were rigidly secured within the testing frame. The compressive axial load and shear force (vertical load) were applied at the free end independently. A rod connected to the hole under test and a manually controlled hydraulic jack were used to apply the load. Figure 4.4 and Plates 4.1 and 4.2 show a specimen and the load transfer mechanism. The applied load was measured using a load cell attached to the loading apparatus.

Three strain gauges located at the free part of the connector were used to measure the deformations of the connectors. In order to be able to place the strain gauges, these connectors did not have holes on the main plate. The exact location of the strain gauges is shown in Figure 4.5. The strain gauges were connected to a switch and balance unit and the readings were recorded using a digital strain indicator.

Full Scale Cavity Walls Under Lateral Load

4.3.2.1 Objectives

The objectives of the second phase of the experimental program were

- To evaluate the performance of shear connectors in cavity walls_ subjected to positive lateral pressure.
- To estimate the magnitude of the maximum internal forces carried by the connectors.
- To study the effect of the different backup systems (concrete block wythe or metal studs with gypsum board).

- To investigate the effect of the following geometric parameters:
  - concrete block width (140 mm, 190 mm),
  - cavity width (25 mm9 50 mm9 100 mm),
  - three connector arrangements as shown in Figure 4.6
  - reinforcement of concrete block backup wythe,
  - reinforcement of brick veneer facing wythe.

The most important variables of the specimens are summarized in Table 4.2.

4.3.2.2 Specimen Description and Construction

Twelve specimens forming the second phase of the experimental program were constructed and tested in three series of four walls. Figures 4.7 and Figure 4.8 show the dimensions and the construction details of a typical full size cavity wall specimen.

Two requirements were taken into consideration. The first was to enable the completed wall assemblies to simulate a part of one complete strip of a wall section of an actual building. The second requirement was to allow transportation of the wall assemblies to the testing apparatus without damage.

The 3000 mm high and 1200 mm wide cavity walls were built on a bottom concrete slab having dimensions of 1200 mm by 1000 mm by 200 mm. As in the case of a real structure a 12 mm shelf angle was attached to the simulated floor slab and was used to support the brick veneer.

The first two series had a concrete block backup system and the third a metal stud backup system. The shear · connector used for the full scale specimens consisted of steel plate gauge 15 (thickness = 1.6 mm) with a height of 60 mm for series 1 and 2 and 75 mm for series 3. The length of the connector depended upon the concrete block width and the cavity.

The construction sequence for the masonry cavity specimens (series 1 and 2) was :

1. The first concrete block course was laid in a mortar joint on the concrete slab to ensure a uniform and level surface at the bottom.

2. The remaining units were then laid in a running bond pattern, with the appropriate shear connector arrangement (Figure 4.6 a). For the unreinforced specimens the two top layers of the concrete block wythes were grouted and reinforced horizontally with l0M bars. For the reinforced specimens the 3 m long l0M bars were placed from the top and grout was poured into the cores. The reinforcement was then vibrated to ensure proper grouting.

3. At the specified cavity width away from the block wall, the brick wall was laid, resting on a mortar bed on the shelf angle. The specimens that had the veneer wythe reinforced were constructed using pressed bricks. Steel reinforcing wires 500 mm long were then placed in all the large cores during the construction process. Mortar was used instead of grout to fill the voids.

The construction sequence for the specimens with metal studs backup wythe (series 3) was similar to that previous described except the steps related to the construction of the backup wythe:

1. A metal track with length of 1200 mm was fastened to the bottom concrete slab by means of three expansion anchors per track"

2. All metal stud backup systems were the same and consisted of three 3000 mm long metal studs installed at a spacing of 400 mm on center (Figure 4.8). Finally, the top metal track was positioned and fastened with the metal studs using self -drilling screws. Fourteen gauge metal plate was used for both metal studs and matching metal tracks.

3. The metal studs were braced by one row of 25 mm metal channel running through the service cutouts near the mid-height.

4" The connectors were fastened with four screws to the web of the metal stud using the appropriate shear connector pattern. See Figure 4.6 b

5. Interior and exterior gypsum sheetings were then fastened to the studs by means of 38 mm, self-drilling, "teck" screws spaced at 600 mm. The interior sheet had slots (with dimensions 100 by 10 mm^2) corresponding to the shear connector pattern.

6. At the specified cavity width away from the block wall, the brick wythe was laid, resting on a mortar bed on the shelf angle.

All specimens were air cured in the laboratory at about 30 % relative humidity for at least 28 days before testing. For safety all specimens were tied back from the sides to the bottom slabs by using metal studs, as shown in Plate 4.3. The different materials used in this phase were· described in Section 4.2. The walls are labeled and identified by their corresponding series and wall number (ie. wall specimen N°2 from series 1 will be referred as S1W2).

The first series investigated the effect of concrete block width, reinforcement of the backup wall and cavity width" It consisted of two specimens with hollow backup walls which were identical except that S1Wl had 200 mm standard concrete block units while specimen S1W3 had 150 mm standard blocks. Each used 90 mm wide clay brick units and had 25 mm cavity. Specimens S1W2 and S1W4 were similar to specimens S1W1 and S1 W3 except that they were reinforced and had a cavity of 100 mm.

The second series investigated the effect of shear connector arrangements. In addition, specimens S2W3 and S2W4 had both wythes reinforced in order to force the shear connectors to fail and to find the load at which this will occur. It is important to note that specimen S1W2 was repaired and strengthened by an additional connector at the failed mortar bed and was re-tested as specimen S2Wl.

The third series investigated the performance of a shear connected "brick-metal stud" wall. The different parameters were cavity width and shear connector arrangements. Specimen S3Wl and S3W2 were identical except that S3Wl had shear connector pattern type A and S3W2 type C (see Figure 4.6 a). Specimens S3W3 and S3W4 were identical with S3W2 and S3Wl respectively except that they had a cavity of 50 mm instead of 100 mm. For the masonry cavity walls with large cavitie.s (100 mm) some shear connectors were instrumented by means of strain gauges

In order to obtain an estimation of the deformations and internal forces of the critical connector (endmost shear connectors).

4.3.2.3 Testing Apparatus and Procedure

After a specimen had been cured for 28 days the bottom slab with the test specimen on it was transported by overhead crane and positioned on the testing apparatus. The specimens were lifted by placing a cargo sling under the bottom shelf angle and a chain attached to a hook located on the rear of the bottom concrete slab.

The testing apparatus consisted of four W31 Ox 118 columns fixed rigidly on the floor of the laboratory. The four columns were boxed together by four beams. One side of the test frame was covered by a rigid support wall made of fluted steel decking with 12 mm plywood. That rigid support wall served as a backing for the au bag which was rested on the interior of it. Specimens were positioned with the exterior brick veneer facing the plywood backing, with the air bag between the specimen and backing.. The bottom slab was secured in place by jacking against a beam at the base of the testing frame. The top support consisted of a frame of HSS members bolted to the columns. The top edge of the backup wall was set to an angle (200 mm by 100 mm) which was connected with the top support frame. The testing apparatus is shown in Figure 4.7. Plate 4.4 shows the top support conditions.

Load was applied to the wall system by inflating the au bag. The air bag had two ports near the bottom, one used to inflate the bag and the other was connected to a pressure transducer. The bag was inflated by employing the 690 kPa laboratory air supply m conjunction with a pressure regulator to adjust the air flow.

Instrumentation on the wall systems consisted of 10 linear variable differential transducers (L.V.D.T's). Five LVDT's were connected to each wythe along its height at a distance of 700 mm in order to record the deflected shapes of the wythes at the various load steps. Figure 4.9 shows the location of the LVDT's. All of the LVDT's were fastened within the wooden brace which was clamped to the two rear columns of the testing frame. To provide access to the inner face of the brick veneer, 25 mm diameter holes were drilled through the backup wythe. Steel . rods extended through the pre-drilled holes were attached to the brick wall- by epoxy glue. Thin copper wires were used. to connect the LVDT's to the block wall and veneer extension rods. All LVDT's wires were tensioned by elastic bands. Plate 4.5 shows a full scale cavity wall during test.

The masonry cavity specimens with large cavities (100 mm) had four shear connectors along the height of the wall instrumented with four strain gauges. The strain gauges were located at the middle of the cavity, as shown in Figure 4.10.

The data output from the LVDT's, strain gauges and pressure transducer were simultaneously recorded and stored by a computerized data aquisition system (Data General).

Prior to the actual test, a pressure of 0.3 kPa was applied to the wall and then released. Initial readings of all the measuring devices were taken. The specimens were then loaded to failure. Each specimen, was inspected carefully during and after the test in order to observe the mode of failure.

Attention was also given when disassembling the specimens.

4.3.3 Full Scale Masonry Cavity Wall Exposed to the Climatic Conditions

4.3.3.1 Objectives

As it was stated in section 1.2.3 significant deformations can take place due to environmental factors such as humidity and temperature changes.

Deformations due to environmental humidity include shrinkage of concrete block wall and expansion of brick veneer wall. In addition, as it is common for other materials, masonry will expand when heated and contract when cooled. This will lead to differential deformations between interior walls in a building, which are at -a relative constant temperature, and exterior walls exposed to the weather temperatures.

In the case where the concrete block wythe and the brick veneer are bound together using shear connectors, the opposite vertical movements between the two wythes induce significant forces at the connectors which partially restrain these movements.

The objective of this phase of the experimental program was to investigate the performance of shear connected masonry cavity walls under imposed deformations due to environmental factors such as humidity and temperature changes.

4.3.3.2 Specimen Description and Construction

The full scale cavity wall that was used in this phase of the of the experimental program was similar to the full scale cavity specimens subjected to lateral pressure (Section 3.2). The construction sequence was the same as the described in Section 4.3.2.2.. Only the differences to the previous mentioned specimens will be presented in this section.

The full scale cavity wall (3000 mm high and 1200 mm wide) consisted of a brick veneer and a 200 mm concrete wall backup wythe connected together using shear connectors placed at 800 mm in both directions. The cavity was 75 mm, and 50 mm insulation was attached to the backup wall. The cross-section of the connector was 70 mm by 1.8 mm (gauge 14 plate) and its length 343 mm.

In terms of design of the support, typical support conditions were reproduced by supporting the specimen on a frame consisted of upper and lower concrete slabs separated by four H.S.S. columns.

The three free faces of the frame were closed with plywood walls and 50 mm *SIM* insulation sheets to form a chamber that insulated the backup wall and reproduced the actual conditions of an internal backup wall. Access to the test chamber was possible through a small door.

The specimen was then moved outside of the structural laboratory. Figure 4.11 shows a cross-section of the test chamber as well as the dimensions and support conditions of the specimen. Plate 4.6 shows the specimen standing outside the laboratory.

4.3.3.3 Testing Procedure

A small heater was placed inside the test chamber and because of au leakage and heat loss at the bottom slab the temperature was varied between 15 and 45 'C as it is shown in Figure 4.12 The internal and external temperatures were recorded automatically through an X-Y plotter placed inside the laboratory. The roller paper was unrolled with a speed of 2 cm per hour.

Four shear connectors along the wall were instrumented by means of strain gauges. Each connector had three strain gauges located as shown in Figure 4.13. Another strain gauge ("dummy strain gauge") was installed in a shear connector located next to the specimen and exposed only to the environmental temperatures. The dummy strain gauge was used to compensate. for the thermally induced strains. The thirteen strain gauges were connected to a switch and balance unit and the readings were recorded manually using a digital strain indicator. Measurements were taken once.· or twice per day, depending on the environmental temperature variations, for a period of 6 months (December 1987 to May 1988). The measurements included average daily humidity, internal and external temperatures, and strain gauge readings.

No specific external load was applied to the specimen although from the strain gauge readings it was observed that the wind load sometimes acted as an externally applied lateral load.

4.3.4 Wall Segments with Simplified Shear Connector

4.3.4.1 Objectives

The simplified shear connector was presented in section 2.4.3. The only difference in the performance of the initial connector of Figure 2.8 and the simplified connector of Figure 2.9 would probably be its capacity in pullout force. A total of twenty tests were conducted to evaluate the tension strength and the mode of failure of four different types of simplified shear connectors.

In order to increase the pullout strength of the connector the interlock of the connector with the mortar and the friction of the connector when pulling it out should be improved. Two different ways of achieving the above were considered. The one is by corrugating the horizontal part of the connector which is embedded in the mortar bed of the concrete block wythe, and the other. is by adding holes in the vertical part of the connector plate which is embedded in the vertical face shells. The mortar that will fill this holes will interlock with the connector and improve its pullout strength. Four different type of connectors were tested :

Type A : No holes and no corrugation With
Type B : holes but no corrugation No holes
Type C : but with corrugation With holes
Type D: and corrugation

4.3.4.2 Specimen Description and Construction

All specimens had identical dimensions and consisted of a concrete block wall segment with a simplified shear connector embedded in the middle vertical joint. H/15/C/M 200 mm standard concrete blocks were used for the wall segment which had dimensions 790 mm wide by 390 mm high. The cross-sectional dimensions of the connector were 70 mm high by 1.6 mm wide and its length corresponded to a cavity of 100 mm. Figure 4.14a shows the dimensions of the specimen.

4.3.4.3 Testing Apparatus and Test Procedure

Plate 4.7 shows the testing apparatus and specimen during test. The specimens were tipped over and laid onto a piece of plywood with the connector facing upward. An H.S.S. frame was then positioned at the top of the specimen to provide reaction and to hold the load transfer mechanism. A rod connected to the hole under test and a manually controlled hydraulic jack were used to apply the load. The applied load was measured using a load cell attached to the testing apparatus. See Figure 4.14b.

Two potential modes of failure exist. The first is crushing of the mortar joint or slipping of the connector out of the junction, and the second is yielding of the plate around the hole used by the tie. Since the tests were intended to investigate the junction of the connector with the block wall, the rod was connected to a stronger inner hole (see Figure 4.15) and the specimen was loaded up to failure. Five of the tests were conducted by first applying the load to the hole used by the tie (see Figure 4.15) in order to investigate the critical mode of failure.

Table 4.1 Physical Properties and Dimensions of Concrete Block Units

PROPERTY	200 mm STANDARD BLOCK H/15/C/M	150 mm STANDARD BLOCK H/15/C/M
Width (mm)	190	140
Length (mm)	390	390
Height (mm)	190	190
Min. Face Shell Thickness (mm)	32	26
Moisture Content (%)	10.2	21.9
Absorption (%)	14.3	20.0
Gross Area (mm^2)	74100	54600
Net Area (mm^2)	41500	31700
(%)	56	58
Unit Mass (Kg)	13.4	10.2
Wall Mass (kg/m^2)	192	152
Compressive Strength (MPa)	15	15

Table 4.2 Variables of Full Scale Specimens

WALL No.	BACKUP WIDTH	BRICK VENEER	CAVITY	CONNECTOR ARRANGEMENT
S1W1	190 H(1)	90	25	A
S1W2	190 R	90	100	A
S1W3	140 H	90	25	A
S1W4	140 R	90	100	A
S2W1	190 R	90	100	B
S2W2	140 R	90	50	A
S2W3	190 R	90 R	100	C
S2W4	140 R	90 R	100	C
S3W1	150 M.S	90	100	A
S3W2	150 M.S	90	100	C
S3W3	150 M.S	90	50	C
S3W4	150 M.S	90	50	A

(1) R = Reinforced Concrete Block or Brick Wythe
H = Hollow Concrete Block Wythe
M.S = Metal Stud Backup Wythe

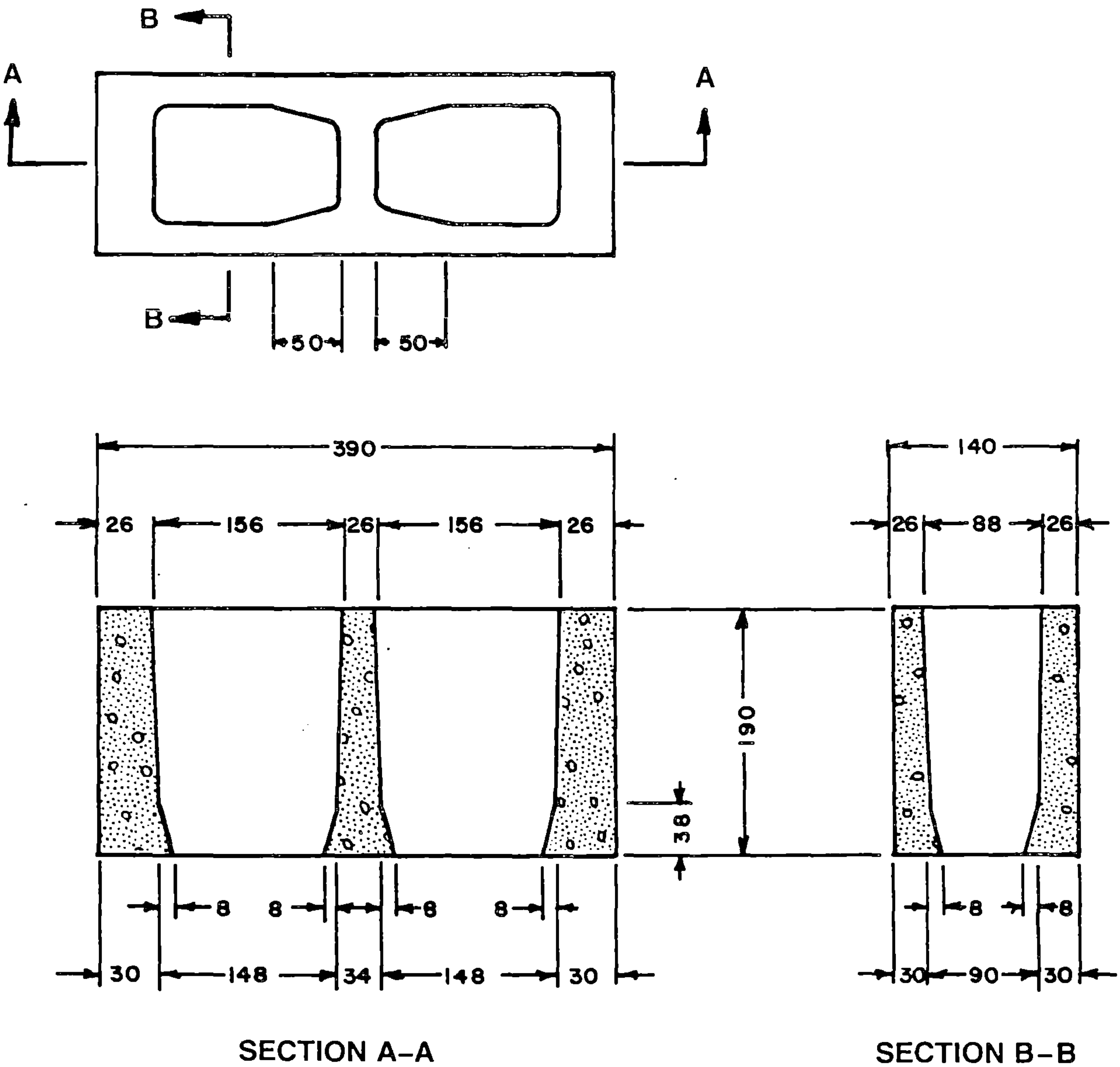


Figure 4.1 Dimensions of 150 Standard Concrete Block Units

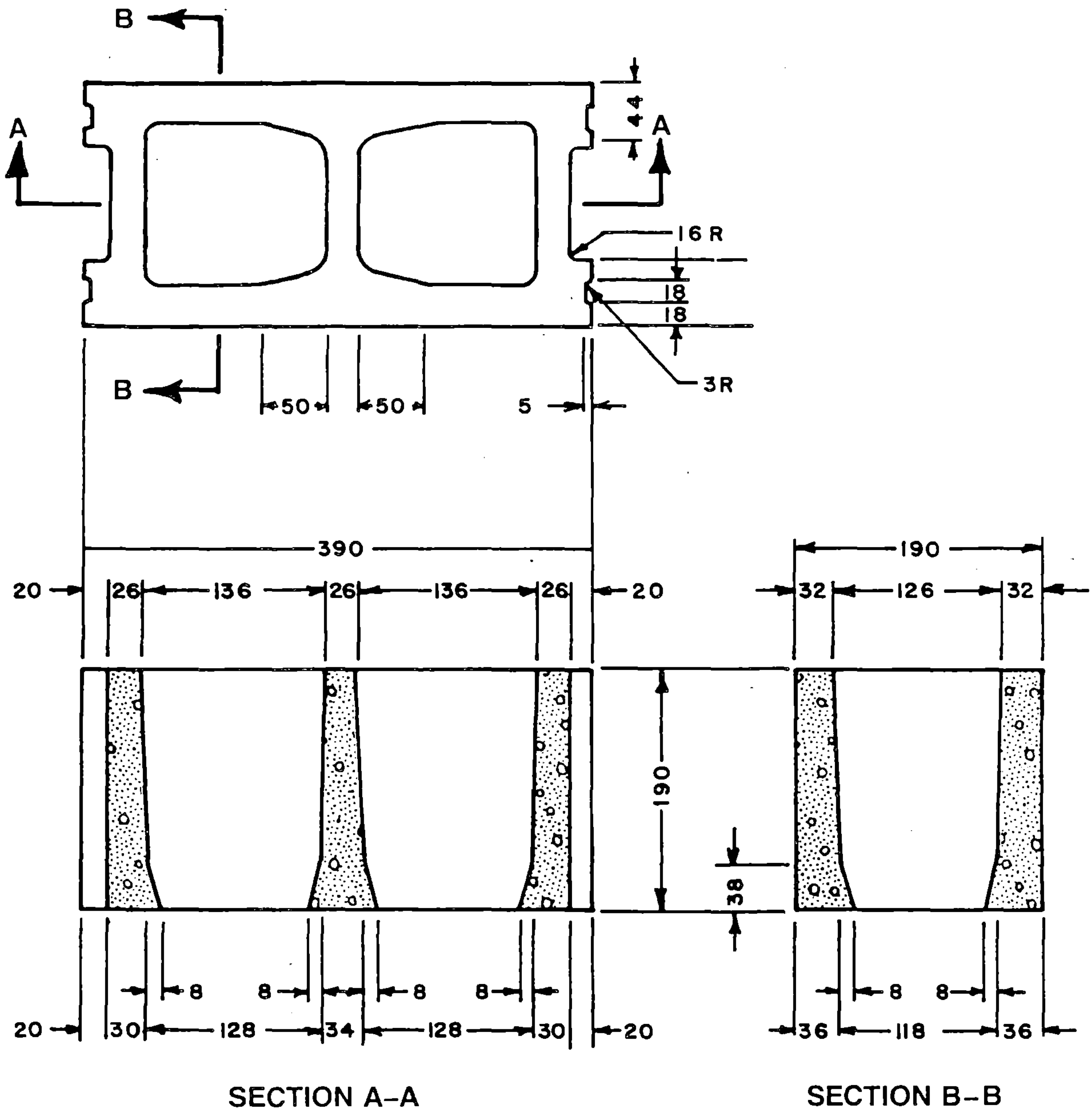
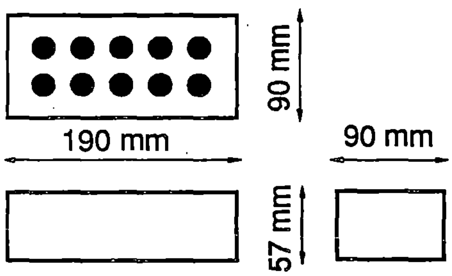


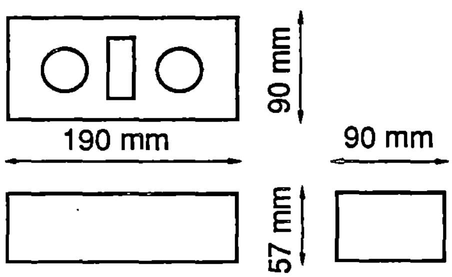
Figure 4.2 Dimensions of 200 Standard Concrete Block Units

100 METRIC STANDARD ◦ WIRECUT



CSA Specification..............................A 82.1M
ASTM Specification................................C 216
Volume of Voids (per cent).......................25
Fire rating...1 hour
(clay brick is non-combustible)
Thermal Resistance.............................R=1.56
Sound Trans. Class.................................46 db
Thermal Expansion Coef. ....3.6 x 10-6
Weight of Wall/Sq. M ....140 kg
Weight per Unit ....1.5 kg
Number of Unit/Sq. M 75

100 METRIC STANDARD • PRESSED



CSA Specification..............................A 82.1M
ASTM Specification................................C 216
Volume of Voids (per cent).......................25
Fire rating...1 hour
(clay brick is non-combustible)
Thermal Resistance.............................R=1.56
Sound Trans. Class.................................46 db
Thermal Expansion Coef. ....3.6 x 10-6
Weight of Wall/Sq. M ....140 kg
Weight per Unit ....1.5 kg
Number of Unit/Sq. M 75

Figure 4.3 Dimensions and Physical Properties of the Clay Brick Units Used in the Tests

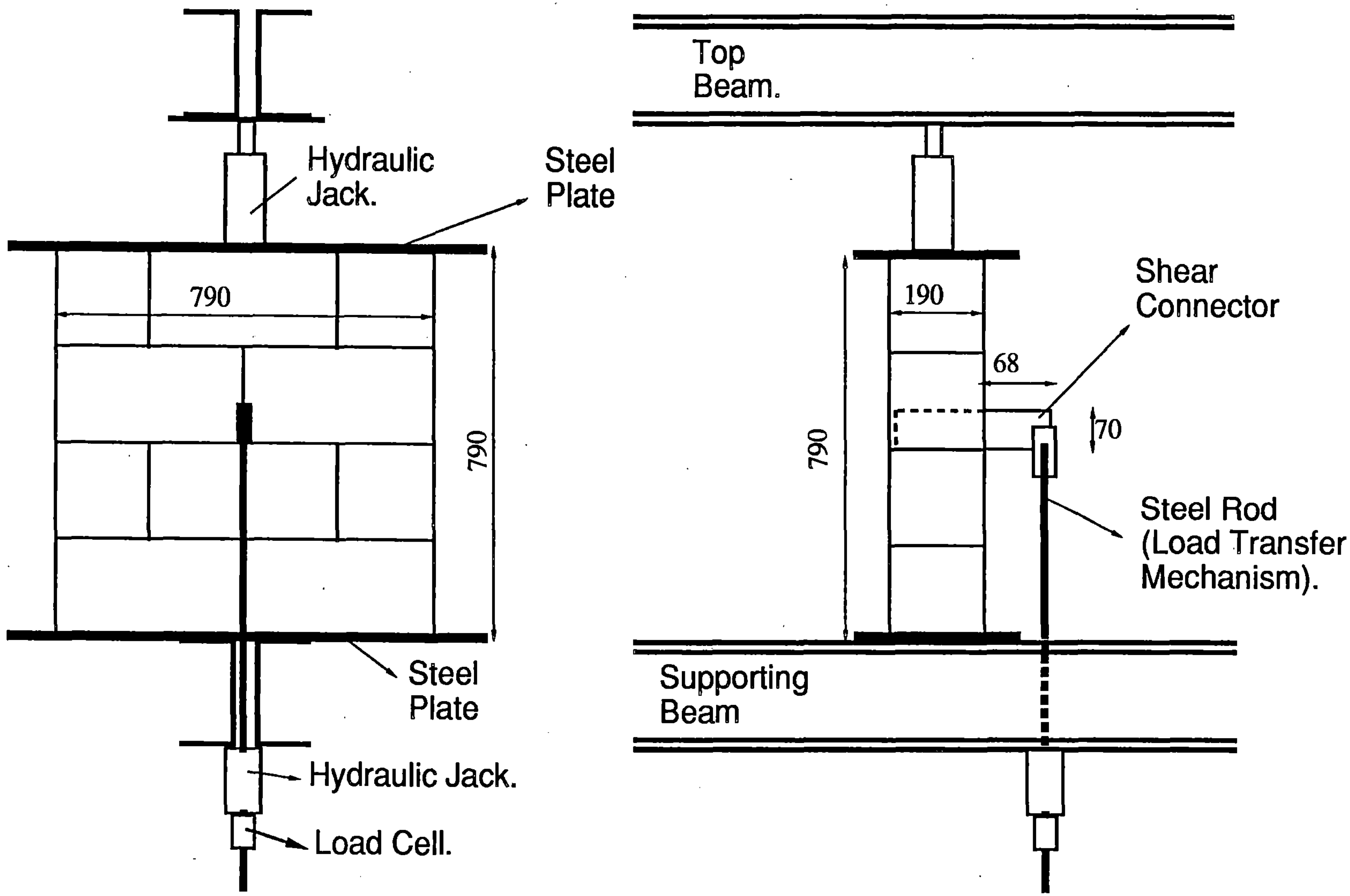


Figure 4.4 Dimensions and Testing Setup for Wall Segments

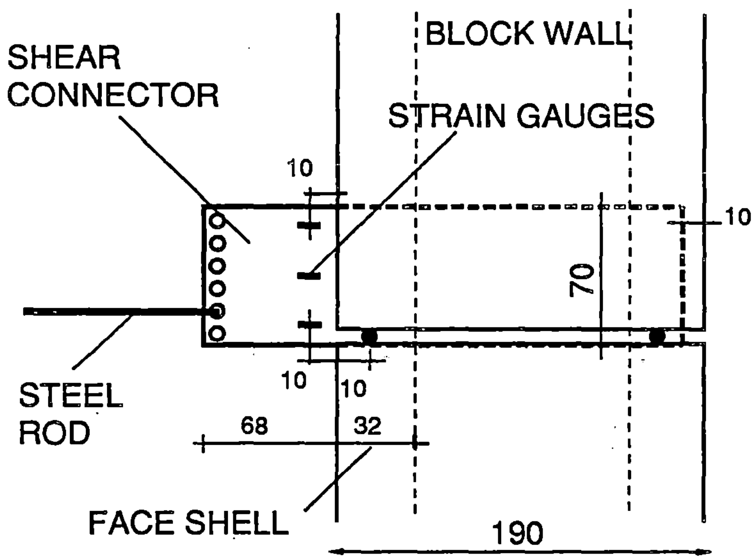
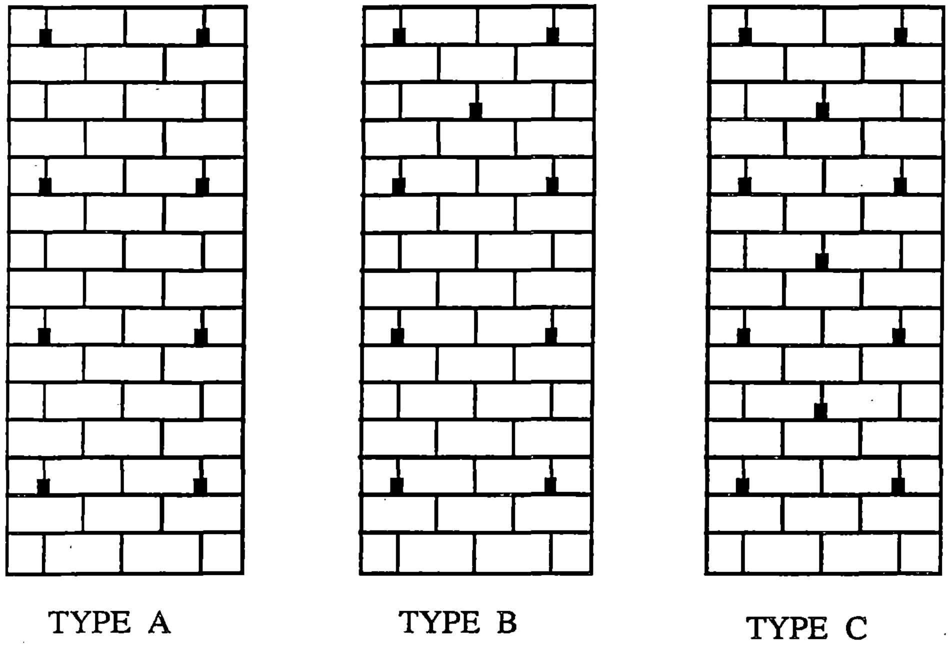
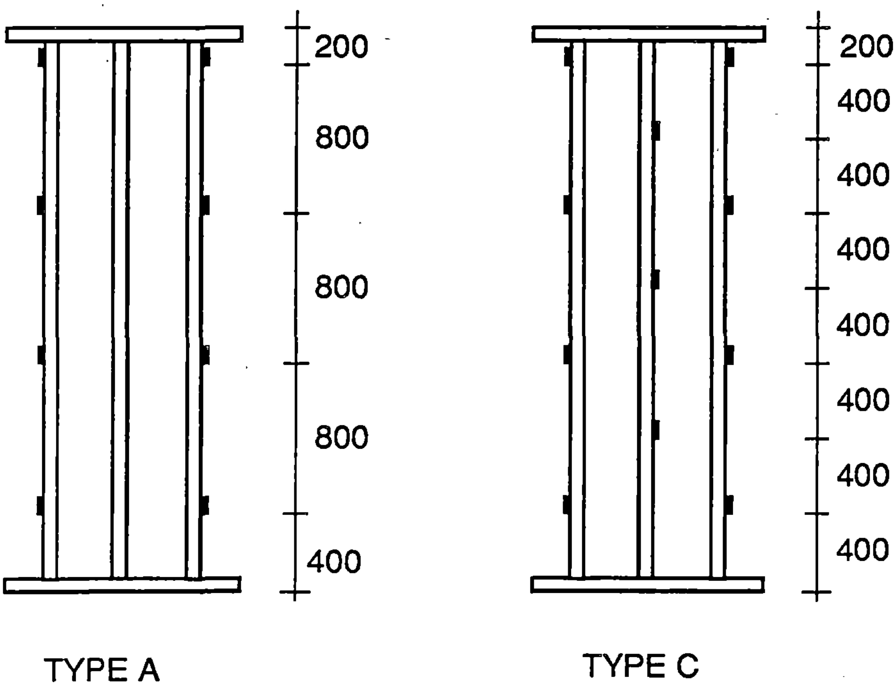


Figure 4.5 Location of Stain Gages in the Shear Connector Plate of the Wall Segments

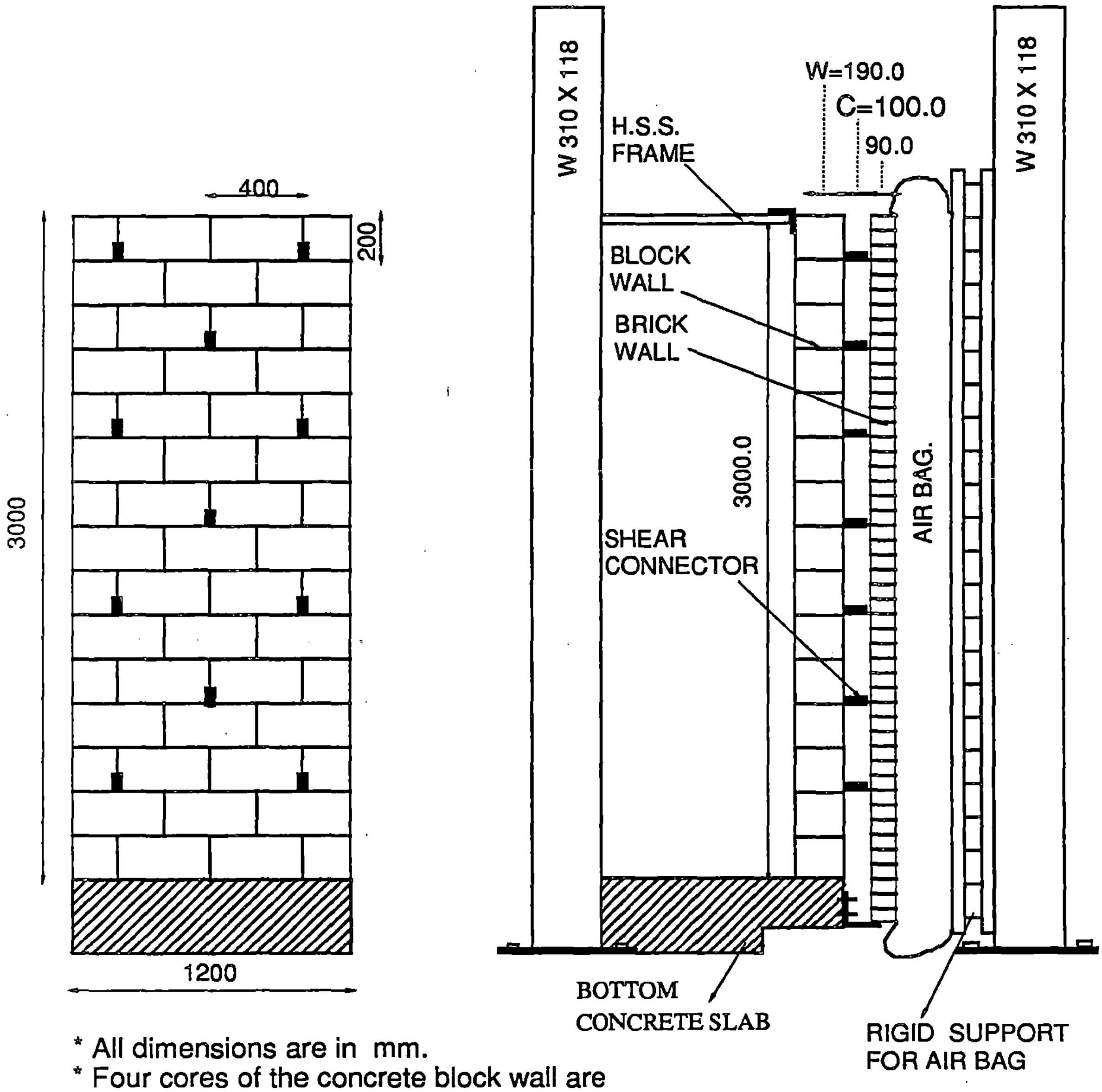


(a) For Masonry Cavity Walls



(b) For "Brick - Metal Studs" Cavity Specimens

Figure 4.6 Shear Connector Arrangements



* All dimensions are in mm.
* Four cores of the concrete block wall are reinforced (#M10 bars) and grouted.
* Brick wall is also reinforced.

Figure 4.7 Specimen Description and Testing Apparatus (S2W3)

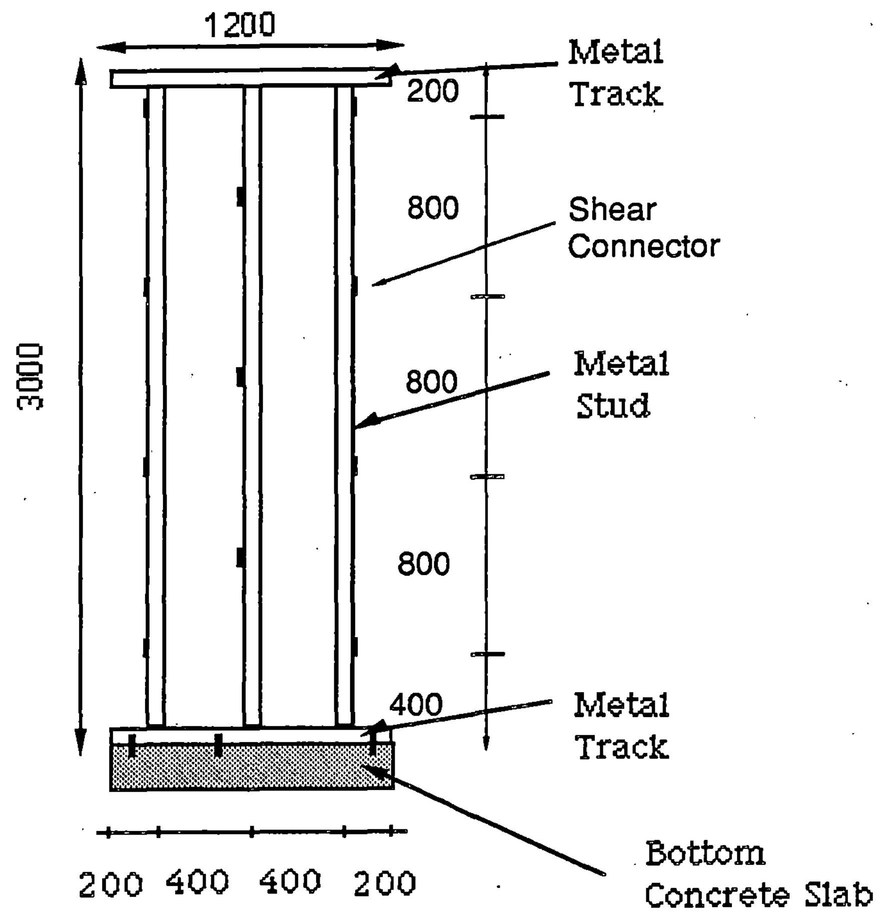


Figure 4.8 Dimensions and Arrangement of Metal Stud Backup Wythe

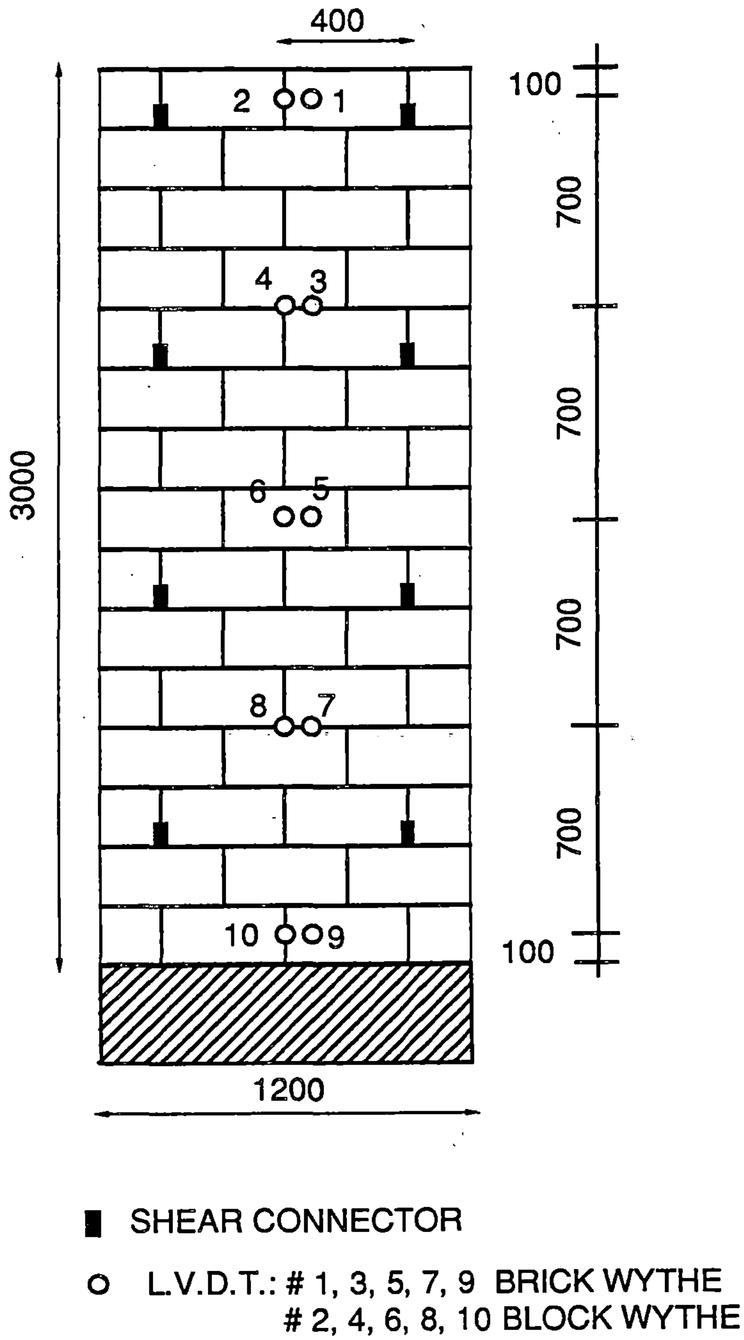


Figure 4.9 Location of L.V.D.T.'s

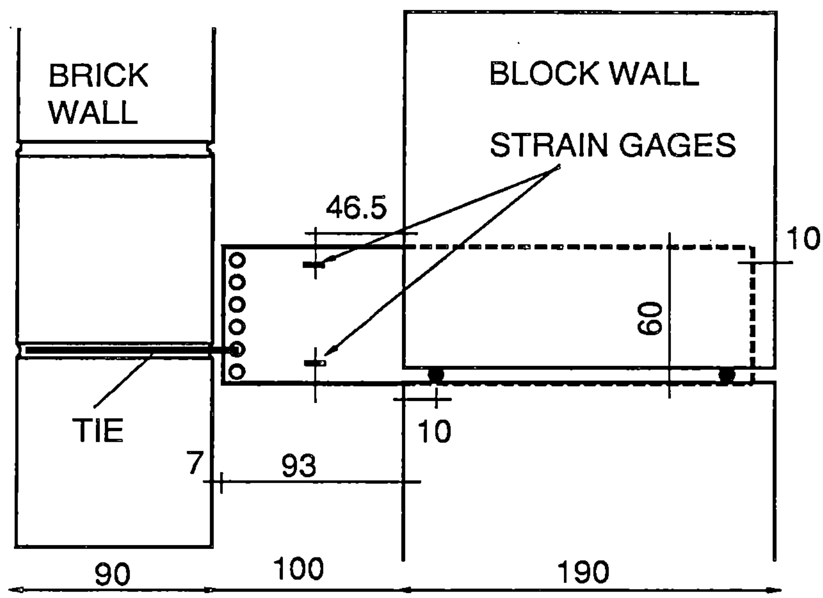


Figure 4.10 Shear Connector and Location of Strain Gages for Full Scale Specimens

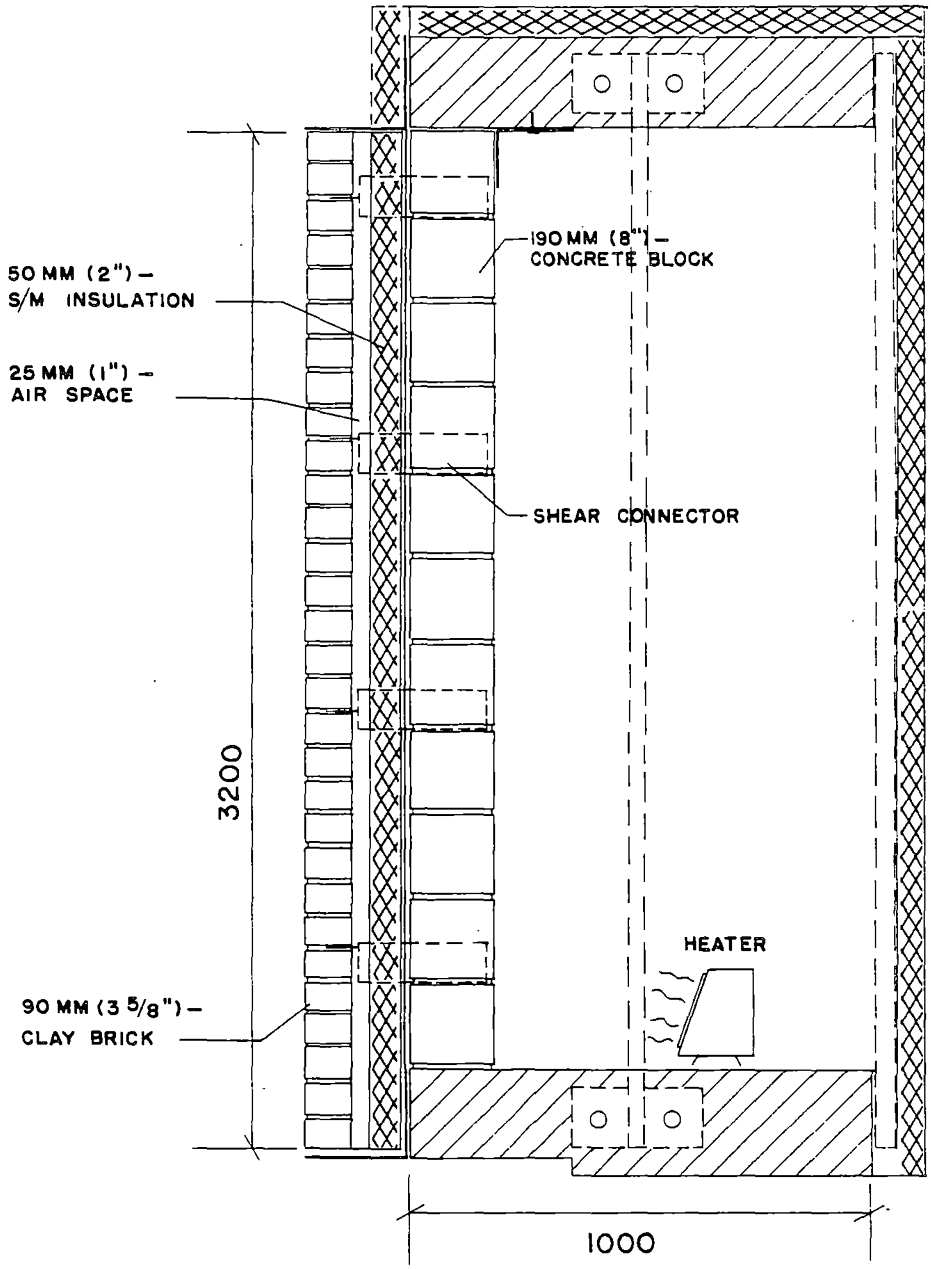


Figure 4.11 Cross Section of Test Chamber and Wall Section

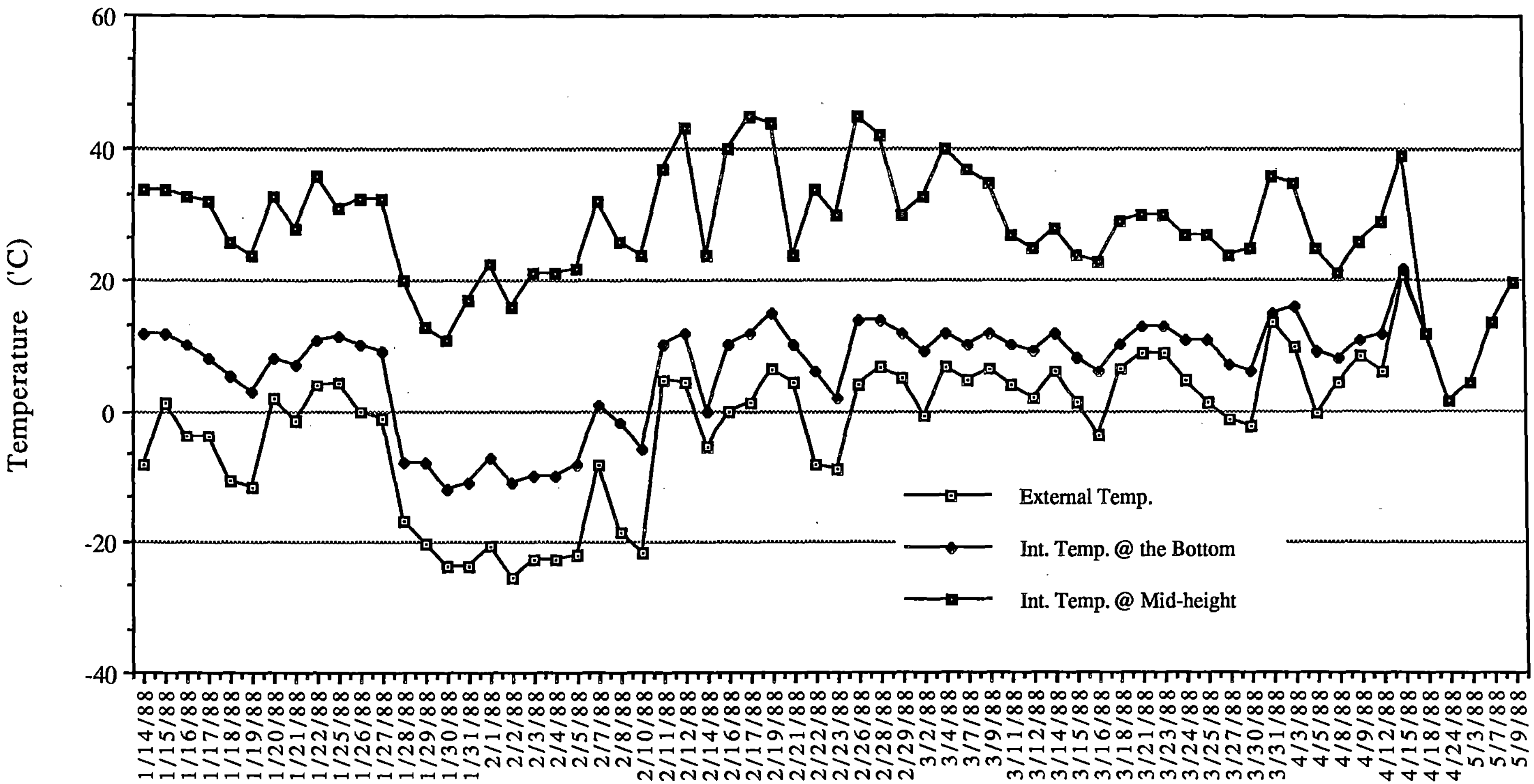


Figure 4.12 Variation of Temperature with Time

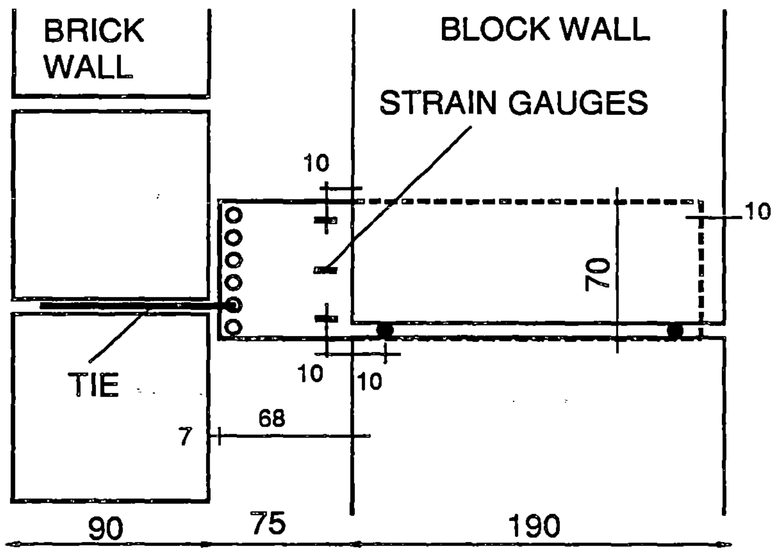


Figure 4.13 Location of Strain Gages in Shear Connectors for the Specimen Exposed to the Climatic Conditions

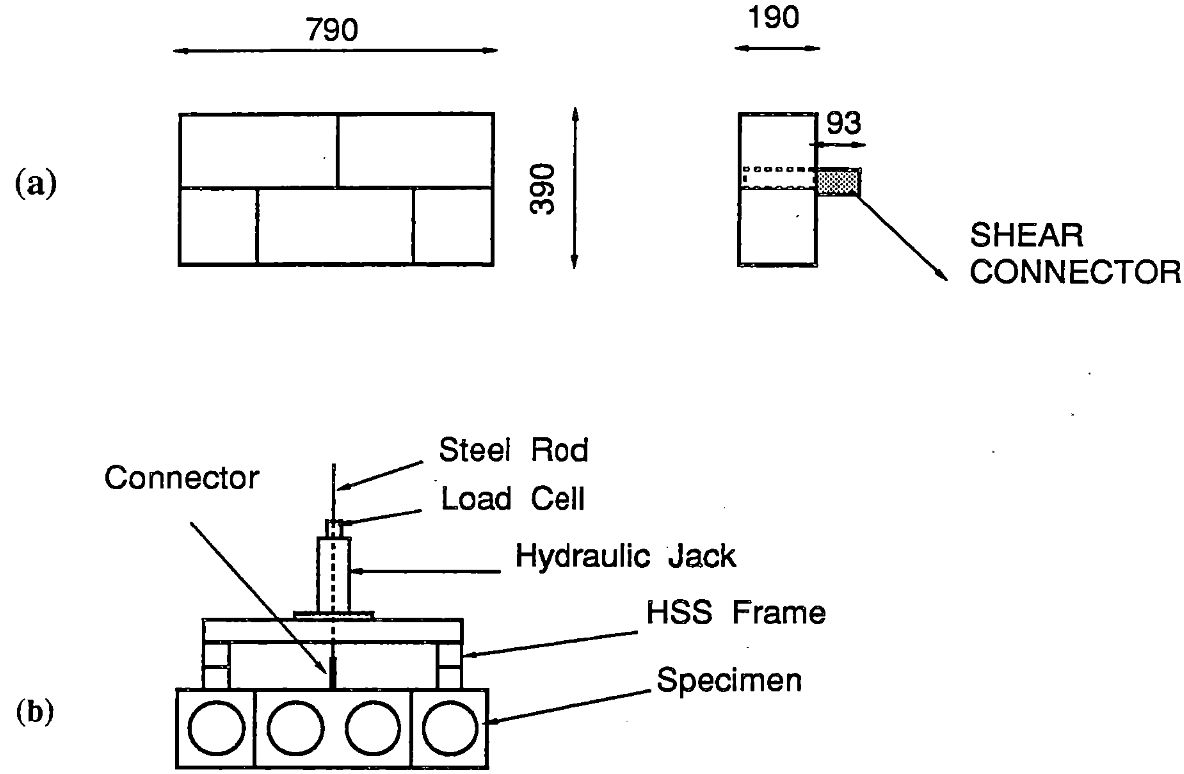


Figure 4.14 (a) Dimensions of Wall Prisms with a Simplified Connector
(b) Testing Apparatus

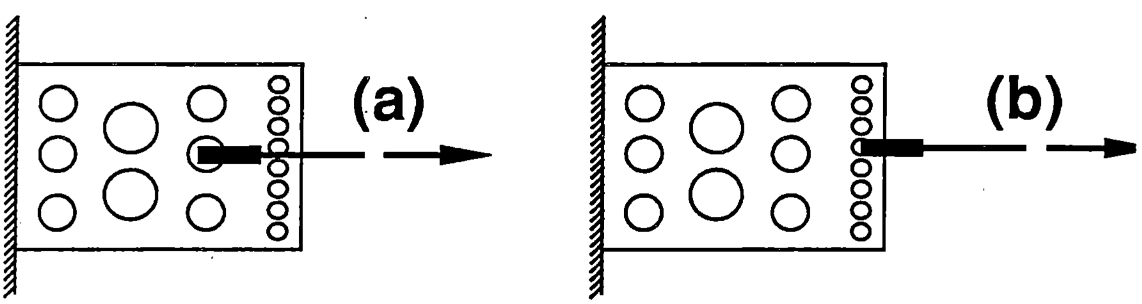


Figure 4.15 Location at which Load was Applied

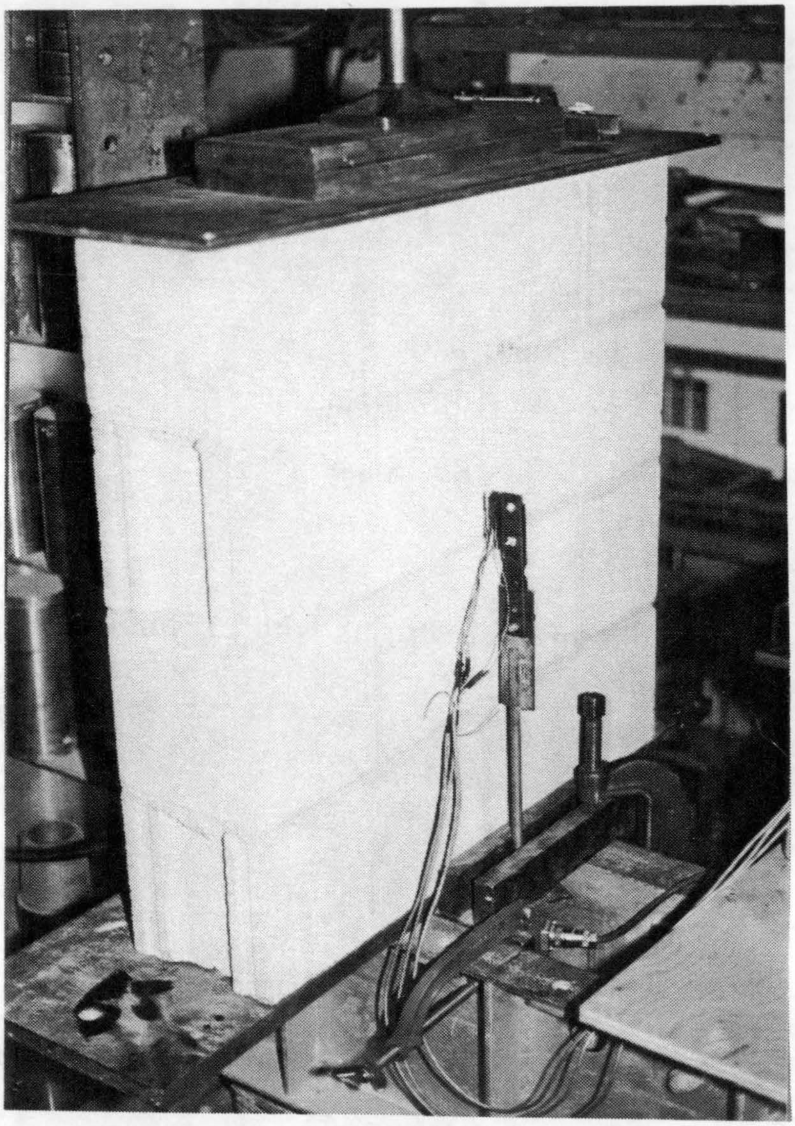


Plate 4.1 Typical Wall Segment Specimen During Test

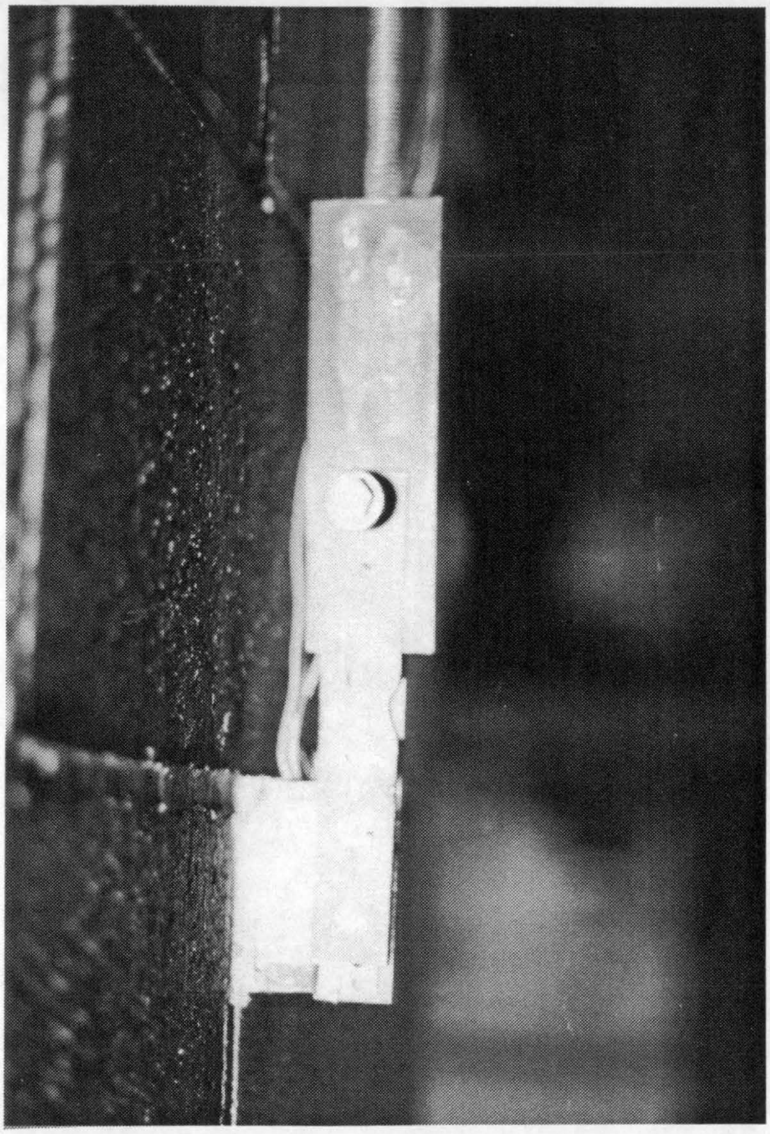


Plate 4.2 Load Transfer Mechanism for Wall Segment Test

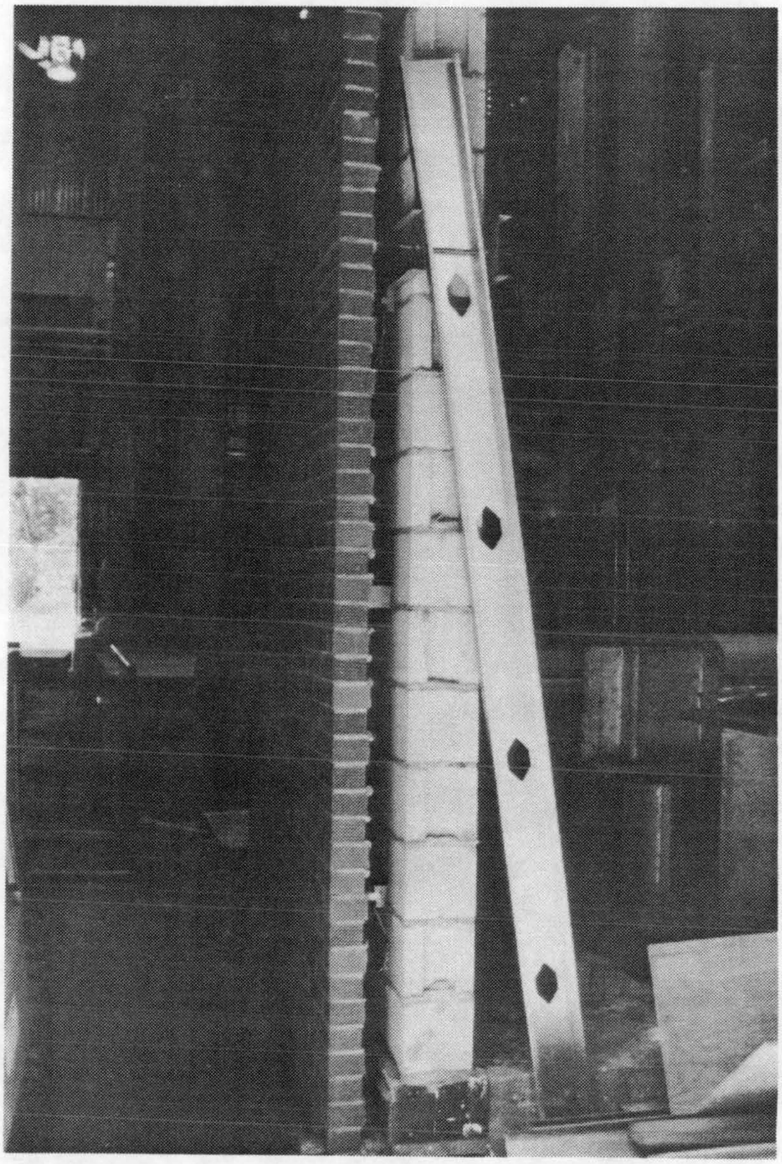


Plate 4.3 Bracing of Specimen During Curing Process

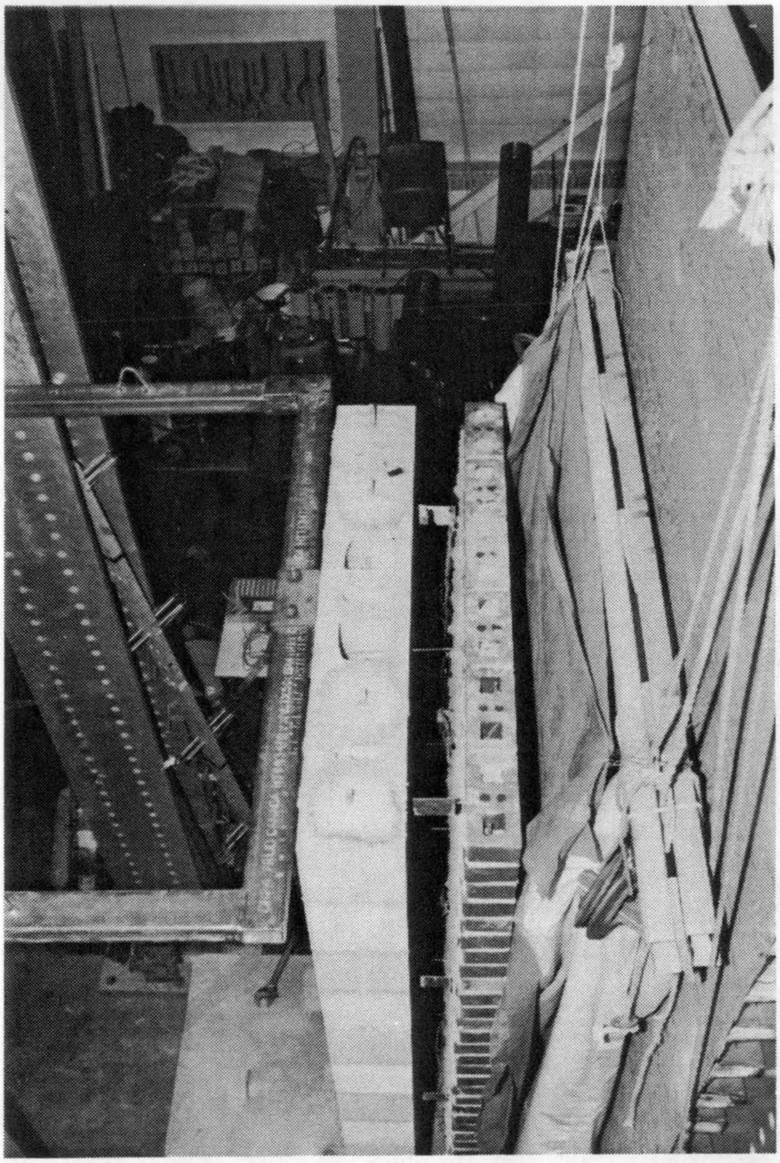


Plate 4.4 Top Support Condition for Full Scale Specimen

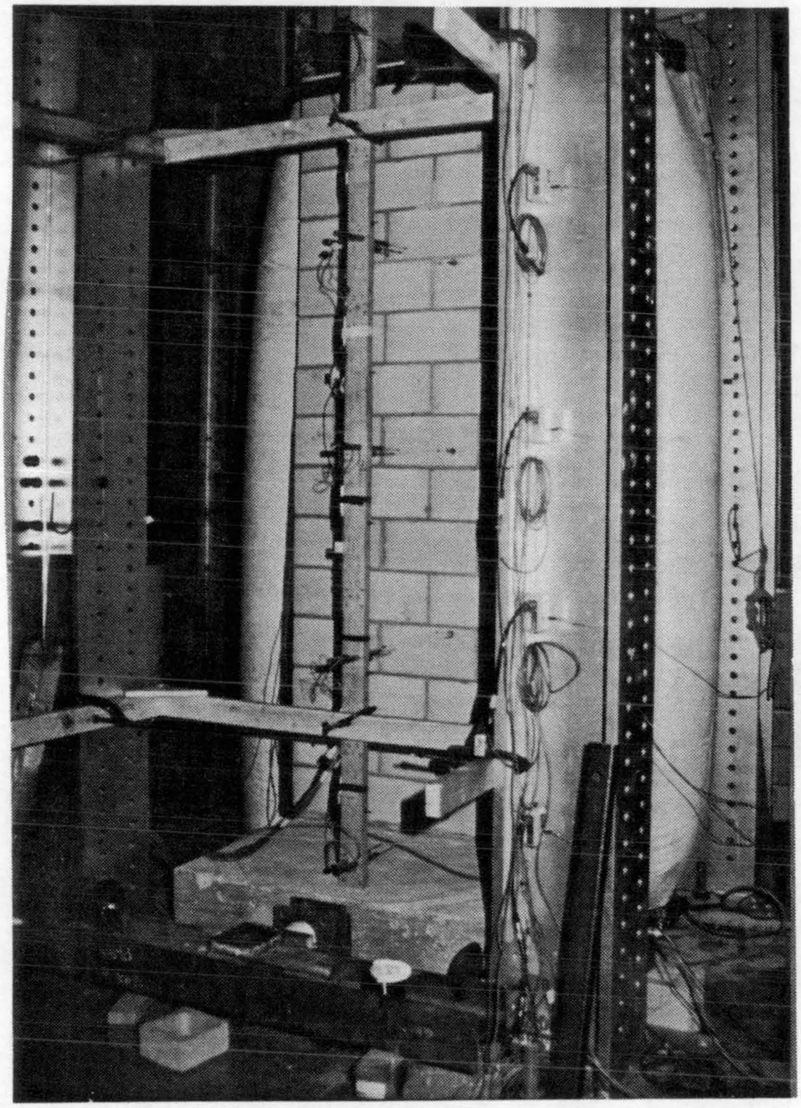


Plate 4.5 Typical Full Size Cavity Wall During Test

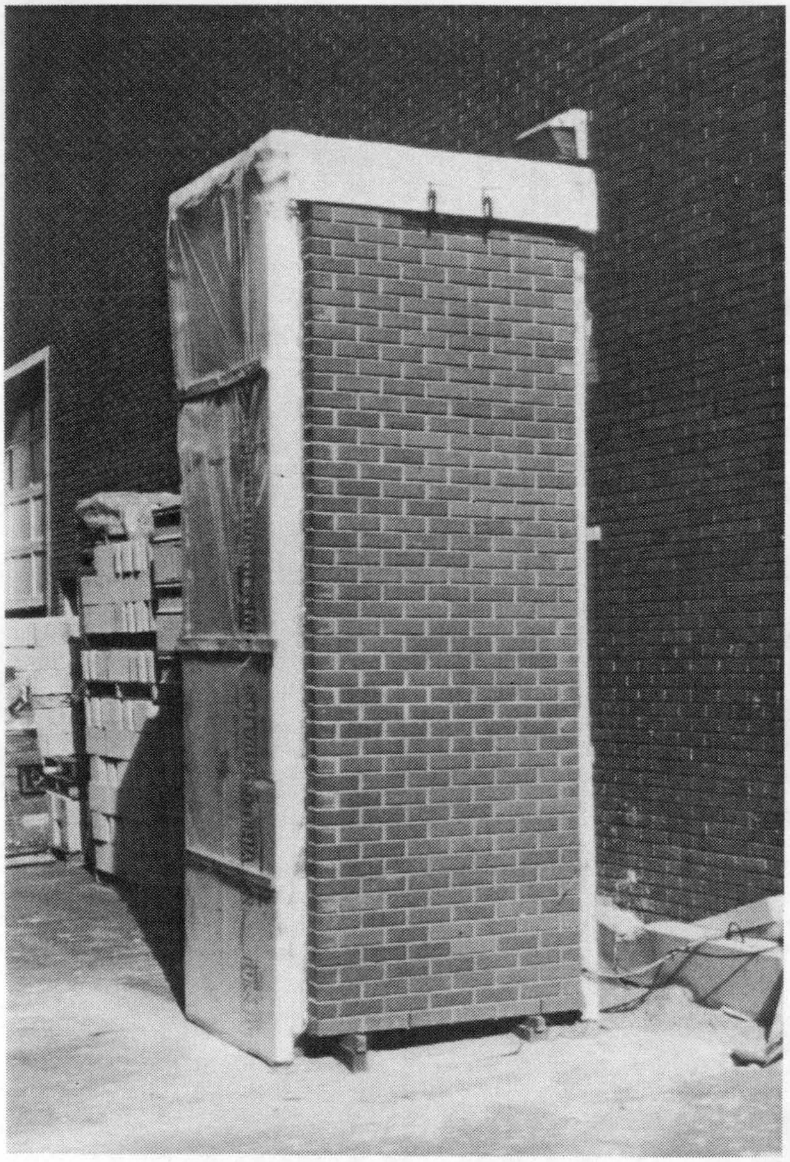


Plate 4.6 Cavity Wall Exposed to Climatic Conditions

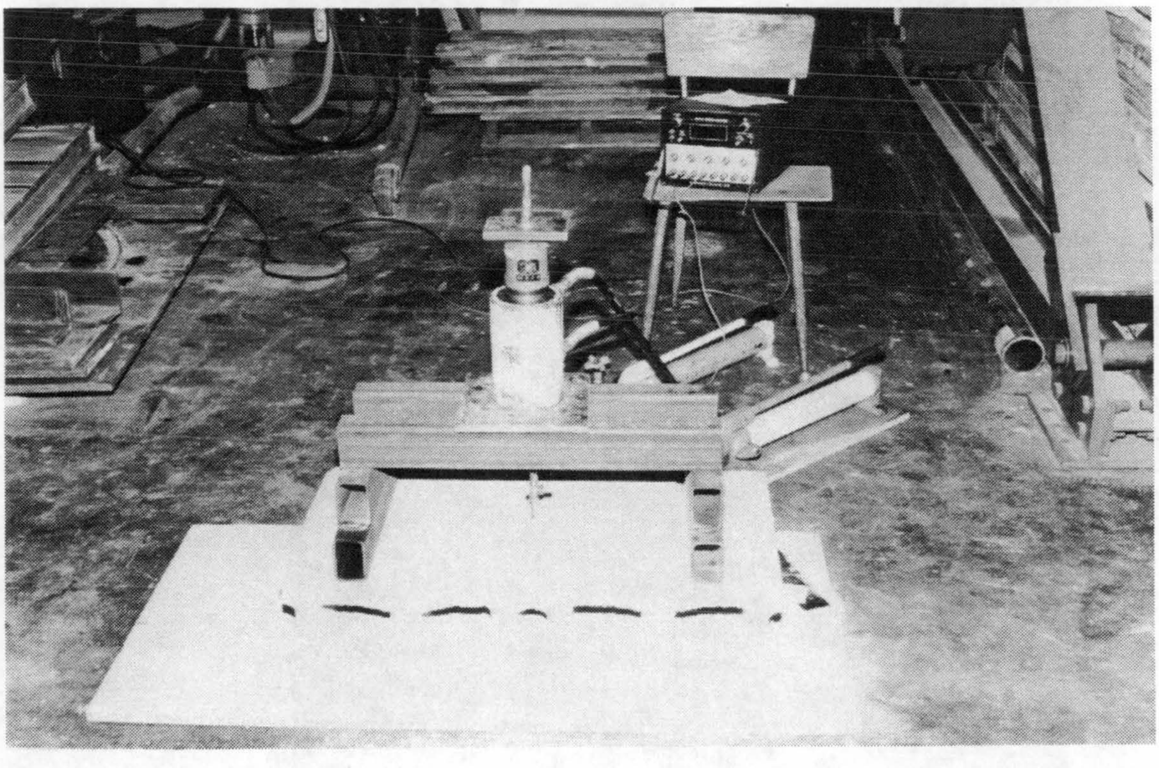


Plate 4.7 Testing Apparatus for Prisms with a Simplified Shear Connector

CHAPTER 5

EXPERIMENTAL RESULTS

5.1 Introduction

In this Chapter, the results from all four phases of the experimental program (Chapter 4) are summarized and presented in tabular, graphic and photographic form.

5.2 Segment Specimens with a Shear Connector

The results of the testing of the three identical wall segments are summarized in Table 5.1. This Table includes information relating to the fastening location (hole), the load type and the maximum (failure) load. Analytical "strain gage vs. load" curves for all the te.sts are given in Appendix B.

The average capacity m upward vertical load was 4.39 kN with a standard deviation of 0.335 kN and a minimum value of 3.78 kN. For the downward shear force the average capacity was 3.26 kN, with a standard deviation of 0.466 kN and a minimum value of 2.45 kN. Finally, the average capacity of the shear connector in axial compression was 5.8 kN.

The failure mode was consistently identified as the yielding of the metal plate around the hole in which the bent rod is attached. Plate 5.1 shows the mode of failure of a connector after several tests. Since for all the tests no cracks appeared at the mortar joint where the connector is embedded it can be concluded that the shear connector is well fixed within the concrete block.

5.3 Full Size Cavity Walls Subjected to Out-of-Plane Pressure

5.3.1 General

The objectives of this phase of the experimental program were presented in Section 4.2.1.

The results of the twelve full scale cavity walls are presented in two types of diagrams. The first type were pressure vs. centerline lateral deflection curves for both the brick and - block wythes · (see Figure 5.1 for example). The second type were deflected shapes along the wall diagrams at different pressure intervals (see Figure 5.2 for example). Unless referred to in this Chapter, all such diagrams may be found for each wall specimen in Appendix C.

Table 5.2 presents the test results for all the full scale cavity walls subjected to positive lateral pressure. For the masonry cavity walls two sets of values for pressure and deflections are presented in that Table. The first value corresponds to the yield strength (which was defined by a significant change in slope) and the second value corresponds to failure (ultimate strength). The point of ultimate strength is defined as the maximum pressure resisted by a wall specimen. For the "brick - metal stud" cavity walls only the ultimate strength is presented since the end of the elastic range is not well defined.

The method used to reduce the data from the strain gages located at the shear connectors of the specimens with large cavities (100 mm) is presented in Section 5.4 and it is shown in Figure 5.12. The results of the strain gage readings will be discussed in this and Chapter 6.

5.3.2 Cavity Wall S1W1

This wall system consisted of a 150 mm wide hollow block backup wythe, a cavity of 25 mm, and a 90 mm wide brick veneer wythe. The shear connector arrangement was type A (see Table 4.2 and Figure 4.6).

The pressure versus centerline deflection and the deflected shapes at different pressures are presented in Figures C-1 and C-2 of Appendix C.

From the pressure - centerline deflection curves it can :be concluded that the wall system behaved elastically up to a pressure of 0.70 kPa. The corresponding maximum deflection at that pressure was recorded at mid-height of the brick wall and was 1.66 mm. At this point, the rate of deflection per load increment increased since cracks started to form at horizontal mortar joints of the block wall. The location of the first cracks were around the concrete block that was set on the top angle. This type of failure can be classified as punching shear failure. That was not the kind of failure that was expected but can be explained by the fact that the backup concrete wythe was ungrouted and unreinforced. The final failure occurred at a pressure of 0.92 kPa, when the previous crack deteriorated. The maximum deflection at failure appeared in the block wall at a height of 2200 mm and was 4.9 mm. The pressure vs. centerline deflection curve remained relatively steep throughout the test and the failure was identified by a sudden drop of the pressure. The sudden failure confirmed the brittle performance of unreinforced hollow masonry wall. Up to failure both wythes deflected the same amount at mid-height. thus composite action was present.

In order to avoid this type of failure specimen S 1 W3 which was also unreinforced had four out of six voids of the top two layers of the block wythe grouted and reinforced.

5.3.3 Cavity Wall S1W2

Specimen S1W2 had a 200 standard reinforced concrete block wall, a cavity of 100 mm, and a shear connector arrangement type A (see Table 4.2 and Figure 4.6). The wall was reinforced by placing two 1OM bars at the second from the edge voids and then grouting these cores.

The pressure vs. centerline deflections and the deflected shapes for both wythes are presented in Figure 5.1 and Figure 5.2 respectively.

From Figure 5.1 it can be concluded that the cavity wall behaved elastically up to a pressure of 1.61 kPa with - corresponding maximum centerline deflection of 0.82 mm. After that pressure cracks started to develop at the horizontal mortar joints of the block wall at mid height. This specimen was able to resist a pressure of 3.6 kPa with a maximum centerline deflection of 10 mm. Failure occurred in the brick wythe at mid height between the first two layers (from top) of connectors when the tensile stresses (due to secondary moments) exceeded the tensile bond strength of the mortar. Plate 5.2 shows specimen S1W2 after failure.

A ductile performance was observed between the yield strength and failure. The explanation for this is the presence of reinforcement in the concrete block wythe.

Referring to Figures 5.1 and 5.2 it can be seen that both wythes acted compositely up to failure since they had the same deflections under a given pressure.

The specimen had four shear connectors instrumented with strain gages. By reducing the strain gage readings, assuming linear strains as shown m Figure 5.12, the axial force transferred through the connectors could be found. From that analysis the endmost shear connectors appeared to carry the largest internal forces. The critical shear connectors, therefore, are the ones located close to the supports. At· the end of the elastic range (pressure = 1.61 kPa) the maximum axial force was recorded at the lowermost shear connector and was found to be 0.93 kN. At the failure load (3.6 kPa) the axial force at the same critical connector was 1.87 kN. This value is approximately one third of the minimum capacity of the two connectors tested in axial compression (5.78 kN) as was described in Section 5.2.

Since only the uppermost part of the brick veneer was damaged that part was removed and repaired by adding a shear connector at the mortar bed where failure occurred. The specimen now had a connector arrangement Type B (see Figure 4.6) and was re-tested as specimen S2Wl.

5.3.4 Cavity Wall S1W3

Wall S1W3 was identical to S1W1 except that 150 standard hollow concrete blocks were used. In order to avoid the undesirable shear failure of S1W1 the top two layers of the block wythe were grouted and reinforced.

The pressure vs. centerline deflection diagrams and the deflected shapes are presented in Figures C-5 and C-6 of Appendix C. The yield strength was well defined to be 0.43 kPa with maximum centerline deflection at the block wall of 1.52 mm. After that point, the deflection increased up to 3.00 mm while the pressure dropped slightly to 0.417 kPa. At that time, tensile cracks in the horizontal mortar joint of the block wall at mid-height opened. After reaching 0.417 kPa and 3.00 mm maximum centerline deflection, the wall system again started to carry increased pressure. The final failure occurred at a pressure of 1.00 kPa when the previous cracks deteriorated. The maximum lateral deflection at failure was recorded at middle - height of the block wythe and was 9.87 mm.

The performance of that specimen can be described as poor and brittle.

5.3.5 Cavity Wall S1W4

Specimen S1W4 was identical to S1W2 except that the backup wythe consisted of 150 standard hollow concrete blocks instead of 200.

The end of the elastic range was not well defined but from the pressure vs. lateral deflection curves it can be interpreted to be around 1.0 kPa with a maximum centerline deflection at the block wythe of 2.49mm.

After this load level the deflections started to increase substantially since tensile cracks began to develop along the block wythe. At a pressure of 2.2 kPa a crack in the horizontal joint of the block wythe at a height of 800 mm started to increase. Shortly after, cracks in the brick veneer mortar. bed at the same height (800 mm) were detected. Failure, finally, occurred at a pressure of 2.39 kPa when, the previous cracks in the lower part of the block and brick wythes deteriorated. The maximum lateral deflection at failure pressure was 49 mm.

Unexpected high deflections at the base of both wythes were observed throughout the test. That occurred as a results of cracks which formed in the mortar joint at the block / concrete interface. In general, the large deflections (in comparison with specimen S1W2) along the wall can be attributed to the following two reasons:· first, the block wall itself was more slender (140 mm wide) than that 'of S1W2 (190 mm wide), · and second, and the .most important reason, · is that the grout that was used to fill the reinforced cores did not fill the cores completely. As a matter of fact, upon disassembly of the failed specimen, the half· lower part of the block wythe was found to be ungrouted. Therefore, half of the block wall acted as if it was unreinforced, resulting in large deflections and premature tensile failure.

From the strain gages located at the shear connectors along the wall it was found that the critical connector was the bottom one. The internal axial force of that connector at failure was 2.67 kN (2.2 times smaller than the capacity of a connector under axial compression).

5.3.6 Cavity Wall S2Wl

As has been mentioned earlier, specimen S1W2 was repaired and strengthened by an additional connector at the failed mortar bed and was re-tested as specimen S2Wl. The two specimens were therefore identical except that S2Wl had a shear connector arrangement type B (see Figure 4.6).

Figure 5.3 and Figure 5A show the pressure vs. centerline deflections and the deflected shapes for both wythes. Figure 5.5 compares the pressure deflection behavior of S1W2 and S2Wl.

Cavity wall S2Wl had a more flexible behavior (larger deflections) in the initial (elastic) range than specimen S1W2. That can be attributed to the fact that specimen S2Wl was re-tested. Since the elastic range was exceeded in the first test (S1W2) it is apparent that minor cracks were formed along the reinforced block wythe. Cracks were, therefore, present from the beginning of test S2Wl, yielding to a reduction in the stiffness. As a result, no elastic range appears in the pressure - deflection curves, as can be seen from Figure 5.3 and Figure 5.5.

The wall assembly failed at a pressure of 4. 73 kPa with a maximum deflection of 16.8 mm at the middle height of the block wythe. The failure was identified as sliding of the concrete block wythe at the block wall / concrete slab interface. Plates 5 .3 and 5 .4 show specimen S2Wl after failure. That kind of failure would not have occurred in an actual cavity wall construction since reinforcement from the slab is used to anchor the block wall and to improve the friction and the integrity of the interface. In order to avoid that undesirable failure for the remaining tests, the bottom of the concrete block wall was secured by an additional HSS beam member connected with adjustable steel rods to the two W shaped columns of the testing frame.

From the strain gage readings it was again found that the critical shear connector was the lowermost and just before failure the axial compression force m that connector was 3.86 kN.

Both wythes acted compositely throughout the test and a ductile behavior of the wall assembly was observed. In addition, upon disassembly of the failed specimen, the shear connectors were found to be undamaged and the backup concrete block well grouted along its height.

5.3.7 Cavity Wall S2W2

Masonry cavity wall S2W2 was identical with specimen S1W4 except that it had a cavity of 50 mm instead of 100 mm.

As expected the performance of the wall assembly and the failure mode were similar. Composite action between the two wythes was achieved throughout the test. From the pressure vs. centerline deflection curves given in Figure C-11 it can be seen that the elastic range is not well defined. At a pressure of 2.75 kPa a ma3or tensile crack in the horizontal mortar bed was formed. The corresponding maximum deflection was 26 mm. That crack caused the wall assembly to fail at a pressure of 3.09 kPa with a centerline deflection of 43.7 mm. Plate 5.5 shows specimen S2W2 at the end of the test.

Large deflections at the base of both wythes and along the height of the cavity wall were recorded during the test. These deflections were similar to those observed in specimen S1W4. Explanation for the above were given in Section 5.3.5

Although special care was taken in reinforcing and grouting the specimen after disassembling the wall system it was found that the five last courses of the concrete block were ungrouted. No damage was detected to the shear connectors.

5.3.8 Cavity Wall S2W3

The special features of specimens S2W3 and S2W4 is that they had both wythes reinforced and a shear connector arrangement type C (see Figure 4.6). Cavity wall S2W3 had a 200 standard reinforced concrete block wythe and a cavity of 100 mm (see Table 4.2). Four voids out of the six in the concrete block wythe were reinforced using l0M bars and were then grouted. The sixteen cores of the brick wythe were reinforced using steel reinforcing wire 7.19 mm diameter. Plate 5.6 shows the reinforcement of both wythes.

The pressure vs. centerline deflections and the deflected shapes for both wythes are given in Figure 5.6 and Figure 5.7. From Figure 5.6 it can be seen that the elastic range is not well defined. The first major crack appeared at a pressure of 3.6 kPa at the tensile face of the middle height mortar bed of the block wythe. The corresponding centerline deflection was 4.8 mm. The slope of the pressure - deflection curve was then decreased and remained constant up to failure which occurred at a pressure of 6.2 kPa with a corresponding centerline deflection at the brick wall of 16.3 mm. At that pressure the two uppermost shear connectors buckled and the brick veneer rotated about the mortar joint corresponding to the shear connectors located at 1600 mm from the bottom. This rotation resulted in a sequential buckling of the connectors located at the half upper part of the wall. It was noted that all the failed connectors were buckled in the same side. Plates 5.7 and 5.8 show specimen S2W3 after failure. Composite action between the two wythes was observed throughout the test as we can see from Figure 5.7.

By reducing the strain gage readings it was found that the buckling compressive force of the top connectors was 5.2 kN. The theoretical buckling load for that connector was calculated using the interaction equation (Section 13.8.3) given by the CAN 3-Sl6.1-M84 "Steel Structures for Buildings" (Ref.8) and was found to be 2.67 kN. It is known that the above theoretical approach is conservative and can therefore be used safely for design.

5.3.9 Cavity Wall S2W4

Specimen S2W4 was identical to S2W2 except that the backup wythe consisted of 150 standard hollow concrete blocks instead of 200.

The pressure vs. centerline deflection and the deflected shape curves are given in Figures C-15 and Figures C-16 of Appendix C. Since both wythes were reinforced the wall assembly had a ductile performance · and therefore the end of the elastic range was not well defined. At a pressure of approximately 1.5 kPa a crack appeared in the horizontal joint of the block wythe at a height of 800 mm. The corresponding. centerline deflection was 3.81 mm. After this pressure the deflections started to increase substantially since tensile cracks began to develop along the block wythe. Failure, finally occurred at a pressure of 3.16 kPa when the previous cracks deteriorated. The maximum lateral deflection at failure pressure was 18.6 mm. After that peak point the pressure dropped at 2.65 kPa and the wall assembly experienced a ductile behavior with constant pressure up to a deflection of 58 mm (recorded at the block wythe at 800 mm height). At that point the air bag pressure started to drop and the specimen was unable to carry more load.

Since both wythes were reinforced it was expected that the connectors would buckle before the masonry components failed. The premature cracking of the concrete block wythe can be explained by the fact - that upon disassembly of the specimen, the half lower part of ·the block wythe was found to be ungrouted (see Plate 5.9). Therefore, half of the block wall acted as if it was unreinforced, resulting in large deflections and premature failure.

From Figures C-15 and C-16 it can be seen that the two wythes acted compositely throughout the test.

5.3.10 Cavity Wall S3Wl

All specimens of the third series consisted of a metal stud backup system with a brick veneer facing wythe. Specimen S3Wl had 100 mm cavity and shear connector type A (see Figure 4.6 b). The pressure vs. centerline deflections and the deflected shapes of the two wythes are shown in Figures 5.8 and 5.9 respectively.

As can be seen from these Figures, the two wythes did not act compositely since the brick veneer experienced larger deflections. At the location of the connectors the wythes underwent the same deflections, but between the connectors they had different deflections. An explanation for this is that the stiffnesses of the two wythes are different and under the same pressure they undergo different deformations. By placing connectors in closer proximity the specimens will be forced to undergo the same deflections.

At a pressure of 4.2 kPa the two uppermost shear connectors buckled and the brick veneer rotated about the mortar joint located at 600 mm from the bottom. This rotation resulted in a sequential buckling of the rest of the connectors. The maximum deflections recorded just before failure at tlie center of the backup and brick wythe were 9.87 and 3.96 mm respectively. Plate 5.10 shows specimen S3Wl after failure.

Upon unloading the specimen it was found that the metal stud backup system, returned to its undeformed configuration proving an elastic behavior.

5.3.11 Cavity Wall S3W2

This specimen was identical with specimen S3Wl except that S3W2 had a shear connector arrangement type C instead of type A. The pressure vs. centerline deflections and the deflected shapes are given in Figures C-19 and C-20 (Appendix C) respectively.

From these Figures it can be seen that both wythes acted compositely throughout the test. The specimen seemed to have behaved elastically up to failure, which occurred suddenly and m a similar manner to that of specimen S3Wl. At a pressure of 6.48 kPa the top shear connector buckled and the brick veneer rotated about the bottom mortar joint towards the backup, forcing all the other connectors to buckle. The c orresponding maximum mid-height deflection at failure was 14.34 mm.

Large deflections were recorded at the base of both wythes at the end of the test. That probably occurred when the three expansion anchors, used to connect the backup system with the bottom concrete slab, exceeded their slip resistance capacity. At the same time cracks were formed in the mortar joint of the brick / shelf angle interface.

5.3.12 Cavity Wall S3W3

This specimen was identical with S3W2 except that the cavity was 50 mm instead of 100 mm. The pressure versus centerline deflections and the deflected shapes, of both wythes are shown in Figures 5. 0 and 5.11 respectively.

From the two previous test it was found that the critical limit state for this senes was the buckling of the connectors and not the failure of the wythes. Therefore as the cavity decreases the effective length of the connector decreases and its buckling load increases. A higher ultimate capacity was therefore expected for the connector components of the assembly.

The mode of failure was similar with that of the previously described test of this series. The only . difference is that the failure of S3W3 occurred at a pressure of 8.34 kPa. The corresponding maximum deflection was recorded by the centerline LVDT and was 20.51 mm. From Figures 5.10 and 5.11 it can be seen that both wythes acted compositely throughout the test.

Upon disassembly of the failed specimen, as was the case for all specimens of series 3, the metal stud backup wythe was found to be undamaged. Plate 5.11 shows specimen S3W3 after failure.

5.3.13 Cavity Wall S3W4

Specimen S3W4 was identical with S3Wl except that the cavity was 50 mm instead of 10 0 mm. Figures C-23 and C-24 (Appendix C) show the load vs centerline deflection and deflected shapes of both wythes.

As was the case for S3Wl, these Figures show that the two wythes did not act compositely. The deflections of the brick wythe were at least twice those of the backup system. Failure was similar to the one described in section 5.3.10, and occurred at a pressure of 3.9 kPa with maximum centerline deflections of 13.47 mm and 5.25 mm at the brick wythe and backup respectively. After failure the backup system was found to be undamaged. The specimen after failure' is shown in Plate 5.12.

5.4 Masonry Cavity Wall Exposed to Climatic Conditions

The deformations induced to the masonry cavity wall exposed to the climatic conditions were due to the

- Shrinkage of concrete block wythe.
- Expansion of brick veneer.
- Thermal deformations.

Therefore relative deformations between the two wythes were present.

From the strain gage readings which were recorded during the testing period of time (10 months), it can be concluded that the shear connectors partially restrained these opposite vertical movements and induced forces in the masonry components.

The environmental humidity, external temperature, internal temperature, and material properties of the masonry components and shear connector plates are some of the parameters that affect the amount of the internal forces induced in the masonry components. It was therefore difficult to isolate one parameter and investigate its effect on the internal forces of the shear connected cavity wall. After the data were reduced to allow for the thermally induced strains on the connectors, a linear distribution was assumed for the deformations of the connectors. By using linear regression a linear strain distribution that best fits the three strain gage readings was found. Based on this strain distribution and assuming an elastic modulus of 200 GPa for the connector material (mill galvanized thick plate). a linear stress distribution was found. Finally, by imposing equilibrium relations at the cross-section with the strain gages the shear and axial forces at the connection of the tie with the connector can be calculated. Figure 5.12 shows the method used to reduce the· data. Although the calculated internal forces were not very accurate, since the real strain distribution is not exactly linear, an estimation of their magnitude was obtained.

The maximum internal forces were recorded at the top connector. Figure 5 .13 shows a typical diagram of the difference of temperature (internal minus external) versus the shear forces at the critical top connector. The maximum calculated shear force was 3.50 kN and corresponded to a temperature difference of 40°C. Although, other parameters (such as humidity and material properties) affected these values Figure 5.13 shows that as the temperature difference increases the shear force also increases. This shear force induced on the connector relates to the axial force transferred to the masonry components. It was also found from the results that the brick wall was subjected to compressive forces, while tensile forces were generated in the backup concrete block wall. Therefore, such a system has the advantage of generating tensile forces in the block wall, which can be taken care of by suitable reinforcement.

In addition to that, no damage was observed to the shear connectors or wall components of the tested specimen.

5.5 Wall Segments with a Simplified Shear Connector

These test were conducted to estimate the tensile strength of the simplified shear connector. The test results for the four different types of shear connectors and the two different types of loads described m section 4.3.4 are summarized in Table 5.3.

As it can be seen from the average values and the standard deviations the best and most consistent performance was observed for type D. In addition to that, by comparing shear connectors type B with type C it can be concluded that the corrugation seems to be more effective than the holes in improving the tension capacity of the connection.

The failure mode for specimens with a shear connector type B, C or D and the load applied at location indicated by (a) (Table 5.3) was identified as cracking of the mortar joint at the junction of the connector with the block wythe. For specimens with connector type A and the same load condition, the failure occurred when the connector pulls out of the joint without any damage to the masonry wall segments. When the load was applied at location (b) indicated on Table 5.3 and for all types of connectors the steel plate around the hole, used to connect the tie to the shear plate, failed by yielding. The connection of the plate with the block wall did not fail. Therefore the capacity of the system can be increased by increasing the thickness of the plate or the metal between holes.

Table 5.1 Shear Connector Wall Segment Test Results

Specimen	Hole of Load Application [1]	Load Type	Maximum Load (kN)
1	2	Upward Vertical	4.45
	3	Horizontal Comp.	5.78
	4	Downward Vert.	3.50
	4	Upward Vertical	4.70
	6	Horizontal Comp.	5.83
	7	Upward Vertical	4.20
2	1	Downward Vert.	2.45
	2	Downward Vert.	2.89
	3	Upward Vertical	4.23
	5	Downward Vert.	3.56
	6	Upward Vertical	4.45
3	1	Upward Vertical	4.89
	1	Downward Vert.	3.11
	2	Upward Vertical	3.78
	6	Downward Vert.	3.34
	7	Downward Vert.	4.00

[1] Hole numbering is from bottom of the connector (hole 1) to the top (hole 7)

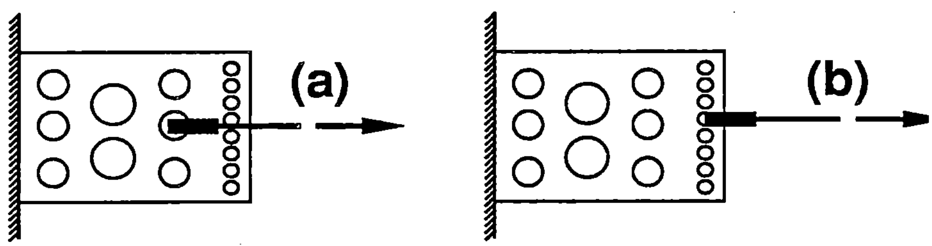
Table 5.2 Summary of Full Sized Specimen Test Results

Specimen	Yield Strength [1]		Ultimate Strength [2]	
	Lateral Pressure (kPa)	Maximum Deflection (mm)	Lateral Pressure (kPa)	Maximum Deflection (mm)
S1W1	0.70	1.66	0.92	4.9
S1W2	1.61	0.82	3.6	10.0
S1W3	0.43	1.52	1.0	9.9
S1W4	1.0	2.49	2.4	49.0
S2W1	1.5	2.8	4.7	16.8
S2W2	1.5	8.0	3.09	43.7
S2W3	3.6	4.8	6.2	16.3
S2W4	1.32	3.02	3.16	18.6
S3W1			4.2	9.87
S3W2			6.48	14.34
S3W3			8.34	20.14
S3W4			3.9	13.47

1 Pressure and deflection at the end of the elastic range.
2 Maximum pressure and corresponding deflection resisted by the wall assembly.

Table 5.3 Simplified Shear Connector Test Results

	Type A		Type B		Type C		Type D	
	(a) (kN)	(b)(1) (kN)	(a) (kN)	(b) (kN)	(a) (kN)	(b) (kN)	(a) (kN)	(b) (kN)
1.	4.78		5.81		6.86		6.95	
2.	3.56		5.87		6.62		7.25	
3.	3.14		5.84	3.55	5.15		7.02	
4.	3.52		5.80	4.38	5.46	3.45	7.95	
5.	5.41	3.92	4.31	4.30	5.82		6.45	
AVG	4.082		5.526		5.982		7.124	
σ_D	0.863		0.608		0.659		0.489	



(1) The average tensile strength for load applied at (b) was 3.92 kN with a standard deviation of 0.378 kN.

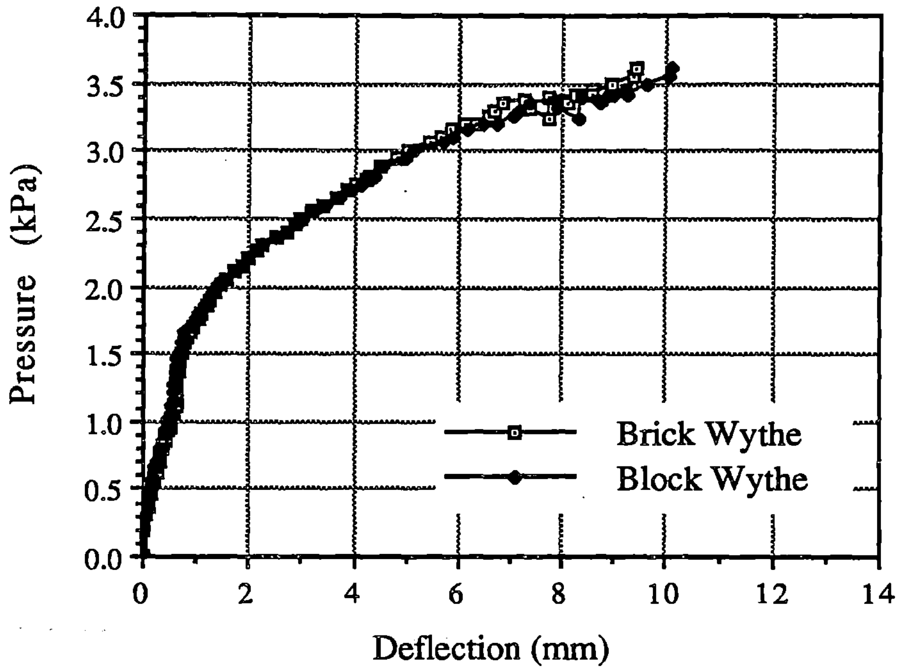


Figure 5.1 Pressure - Centerline Deflection of S1W2

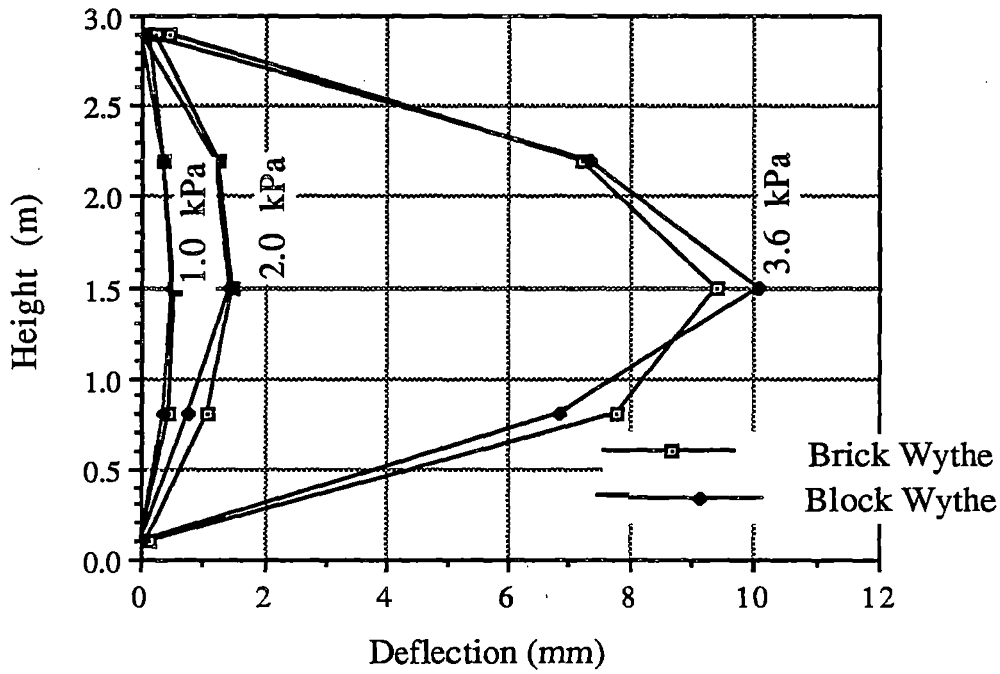


Figure 5.2 Deflected Shapes for S1W2

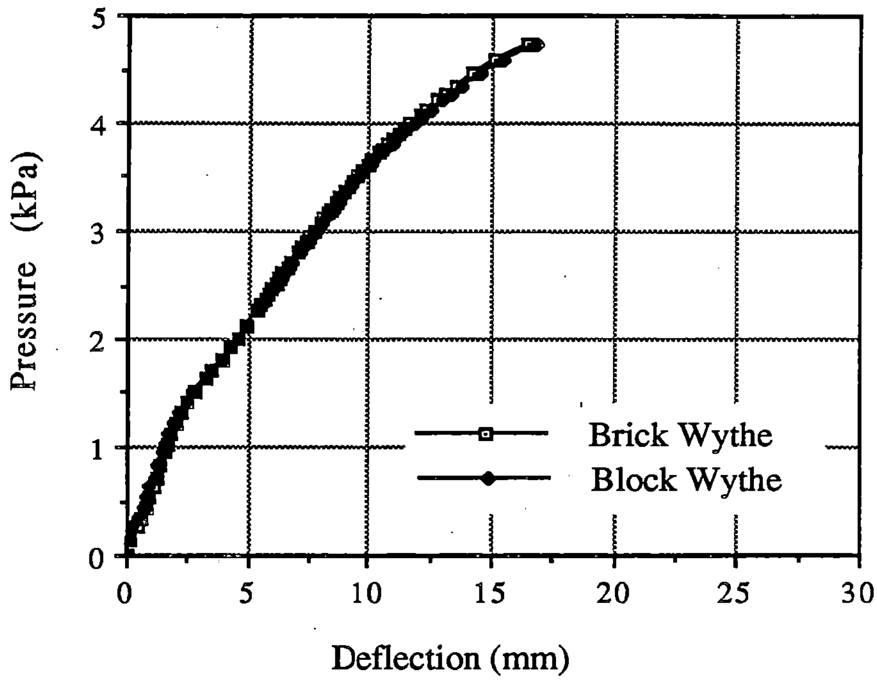


Figure 5.3 Pressure - Centerline Deflections for S2W1

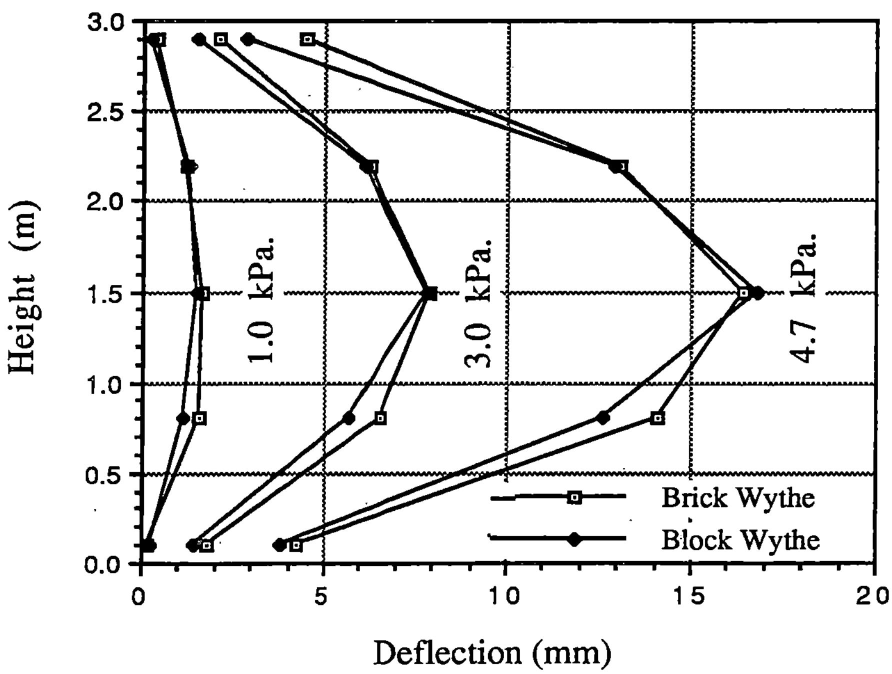


Figure 5.4 Deflected Shapes for S2W1

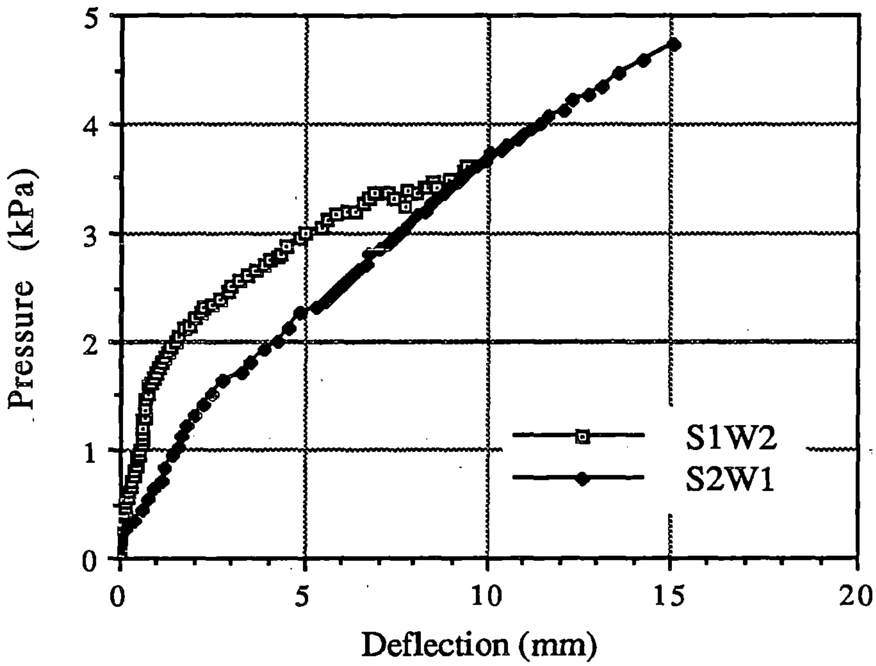


Figure 5.5 Comparison of Centerline Deflection between Wall S1W2 and S2W1

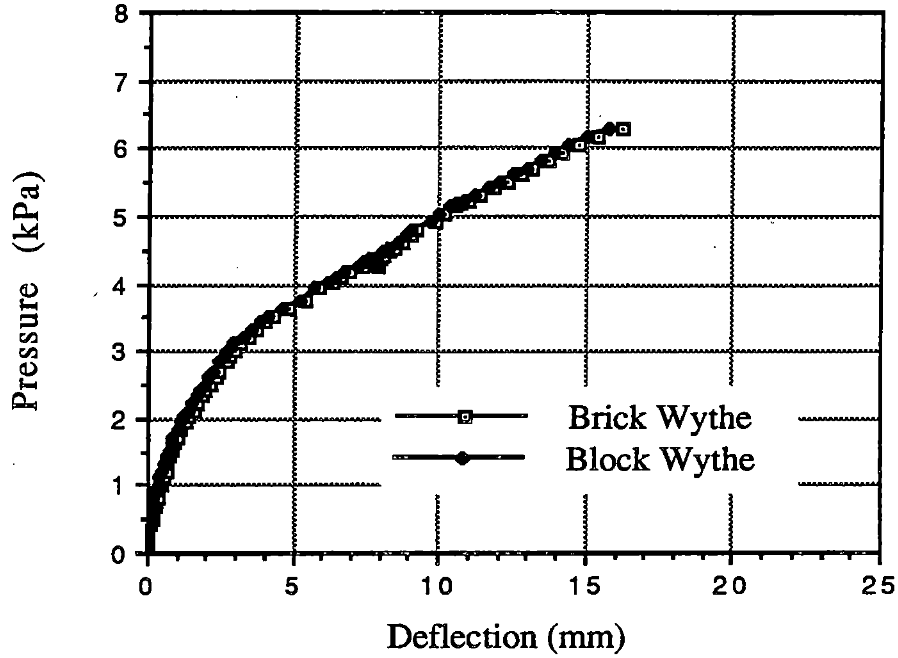


Figure 5.6 Pressure - Centerline Deflections of S2W3

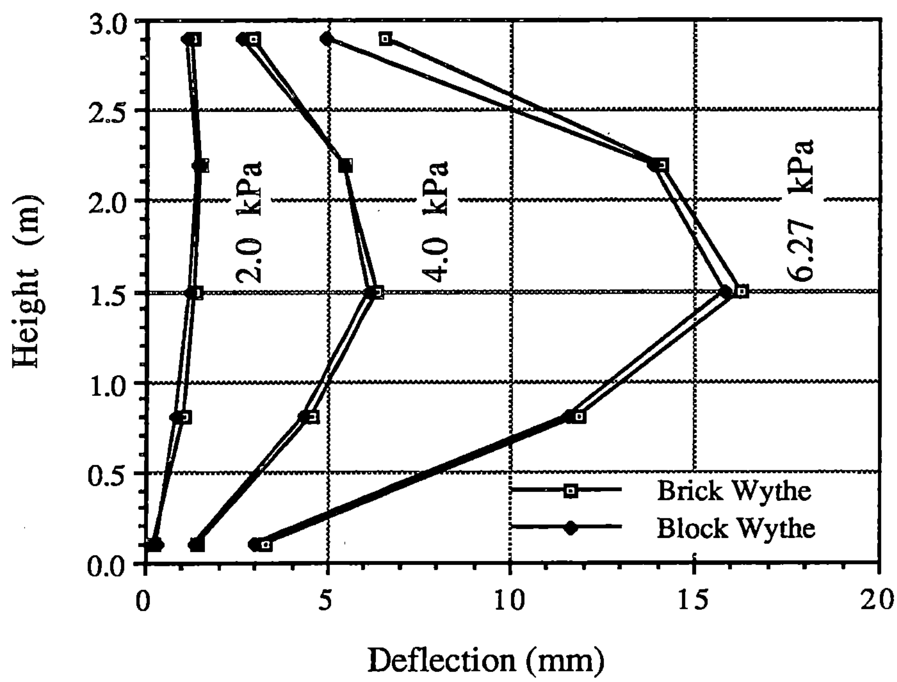


Figure 5.7 Deflected Shapes for S2W3

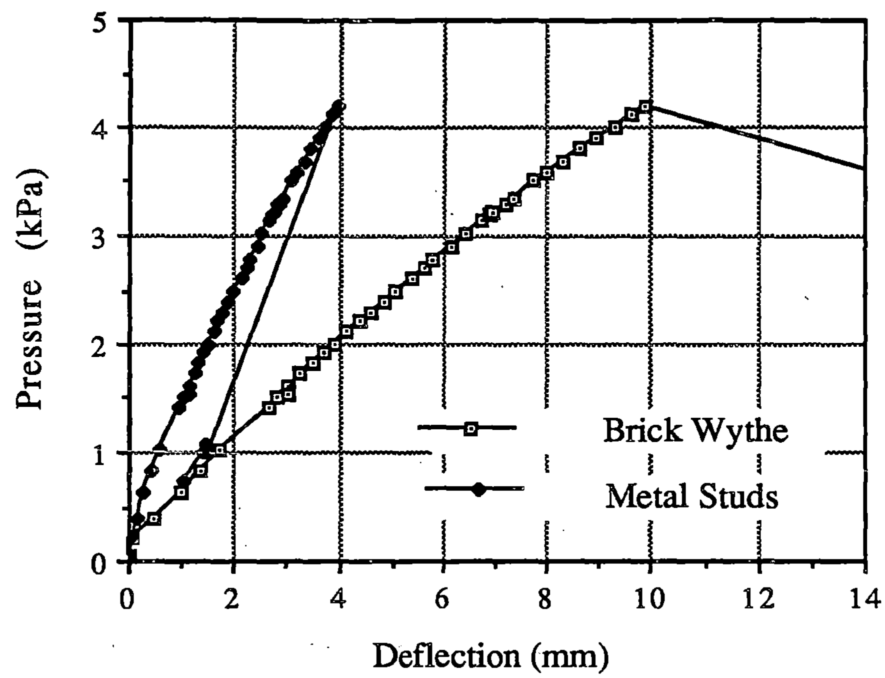


Figure 5.8 Pressure - Centerline Deflections of S3W1

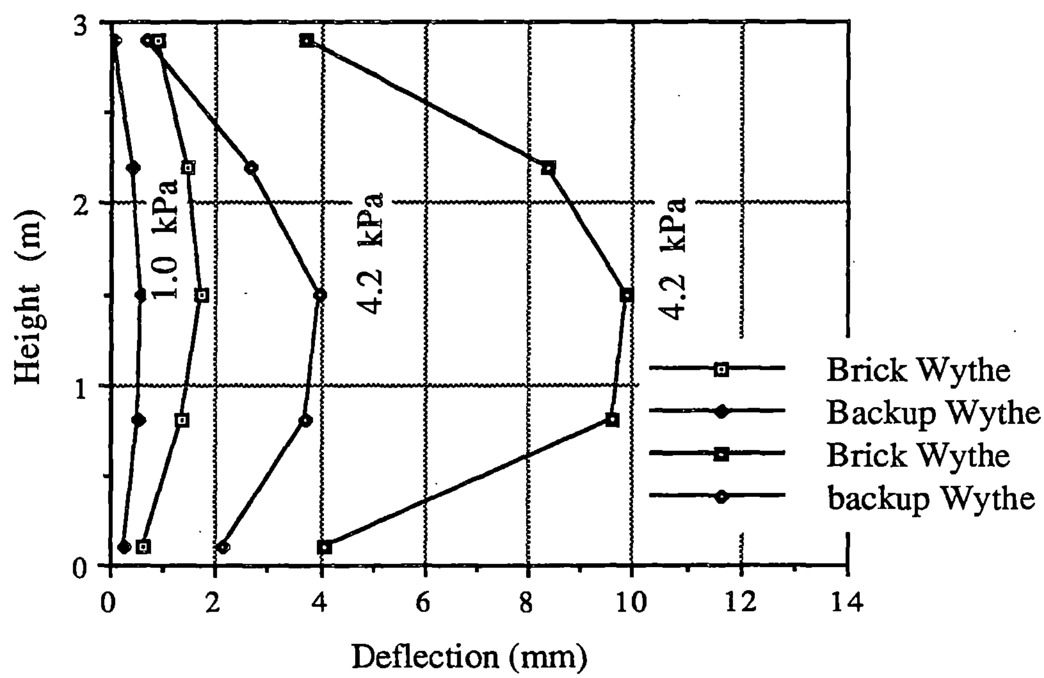


Figure 5.9 Deflected Shapes of S3W1

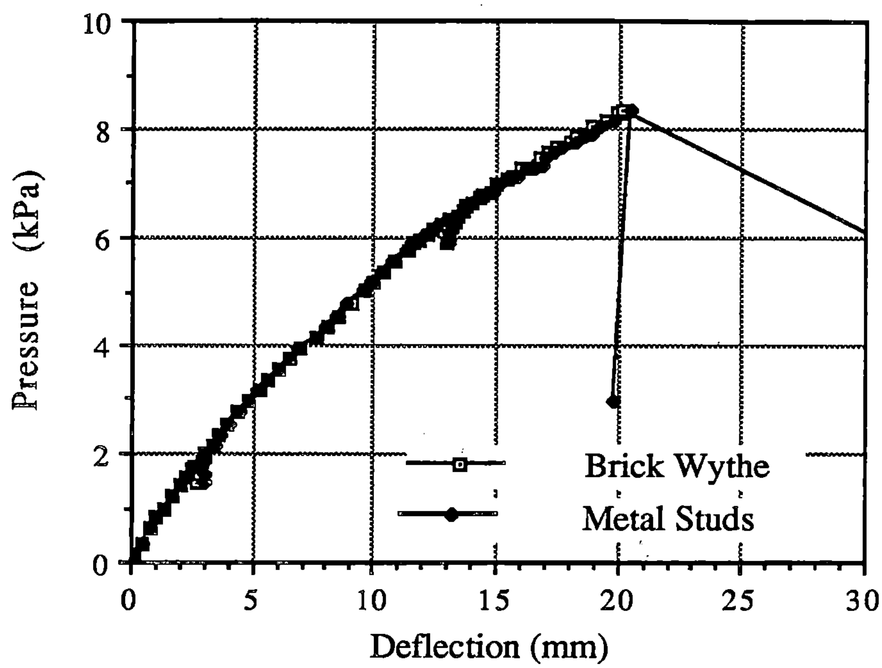


Figure 5.10 Pressure - Centerline Deflections of S3W3

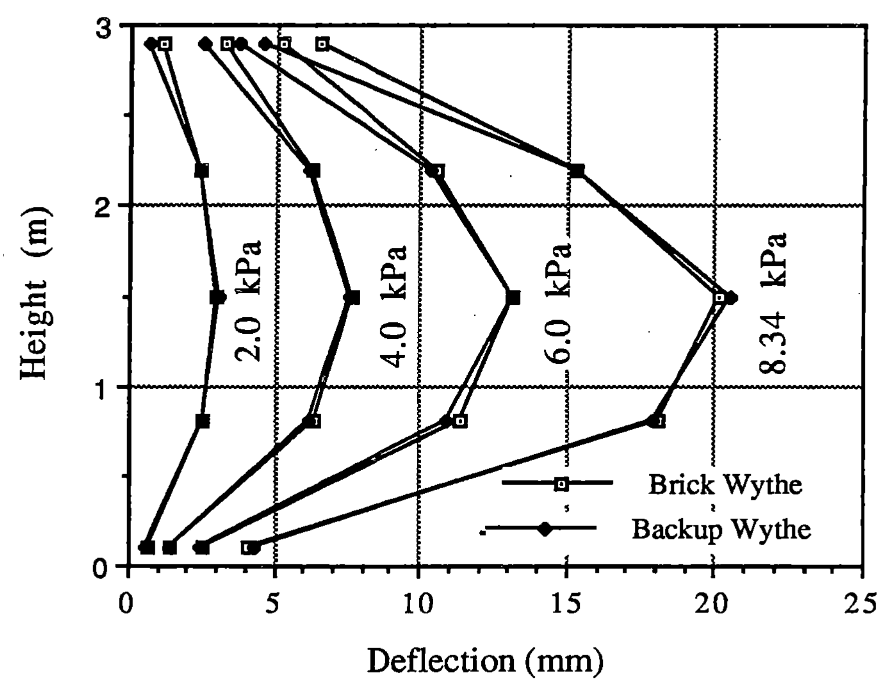


Figure 5.11 Deflected Shapes for S3W3

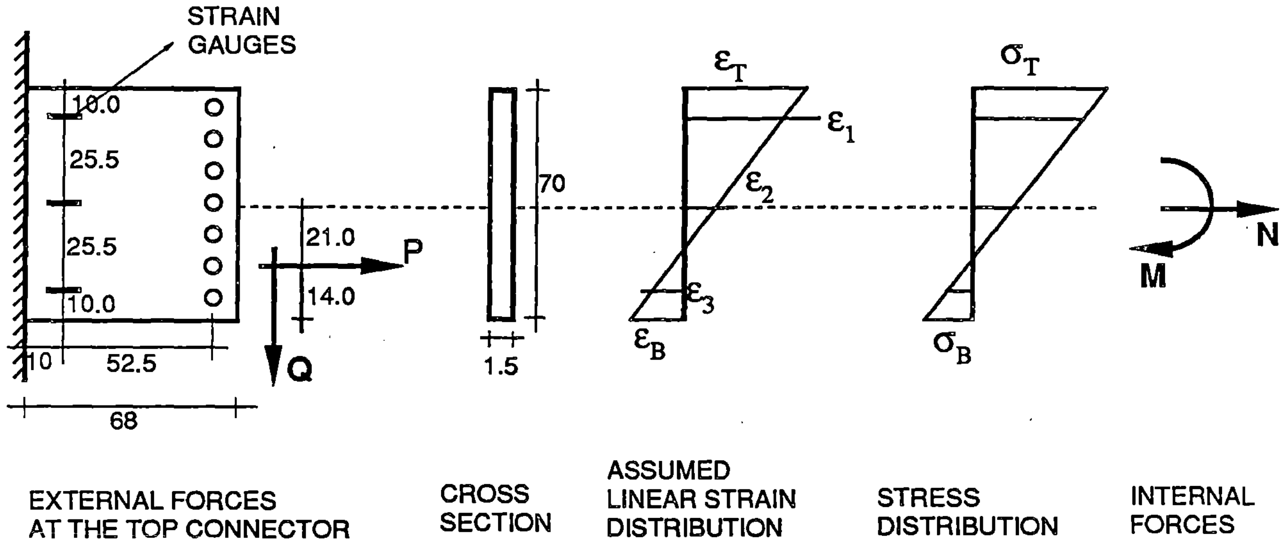


Figure 5.12 Method Used to Reduce the Strain Gages Readings

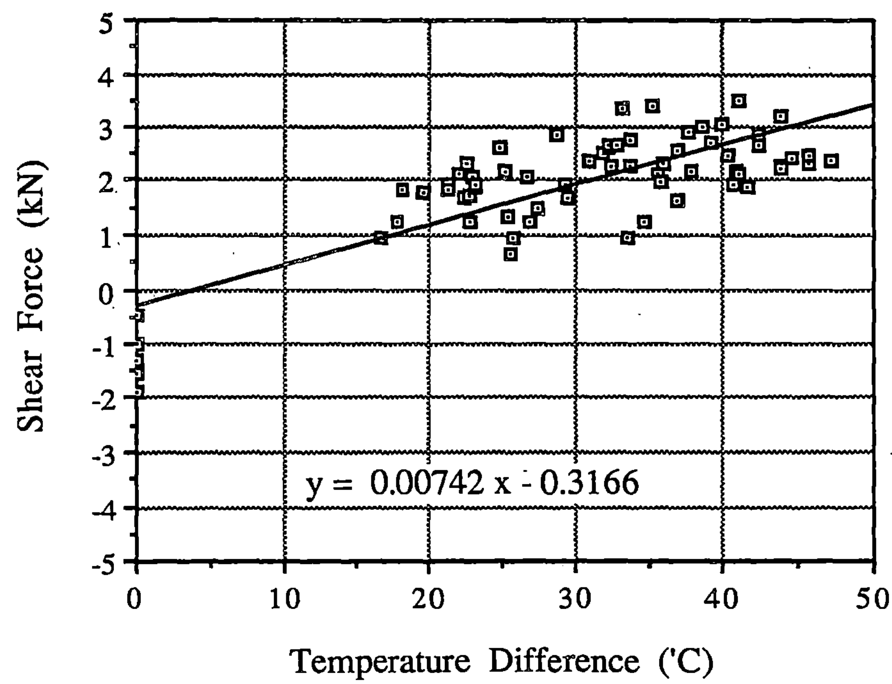


Figure 5.13 Test Results for Masonry Cavity Wall Exposed to Climatic Conditions and Material Properties

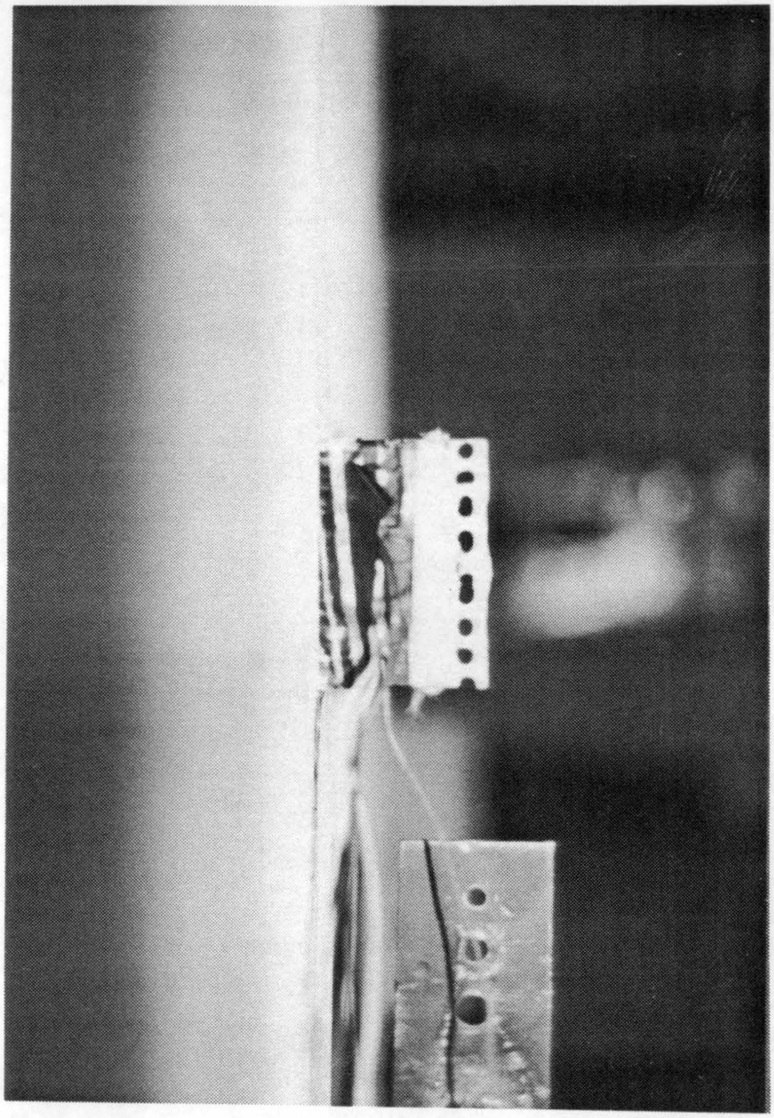


Plate 5.1 Mode of Failure of a Connector Embedded in Concrete Blocks

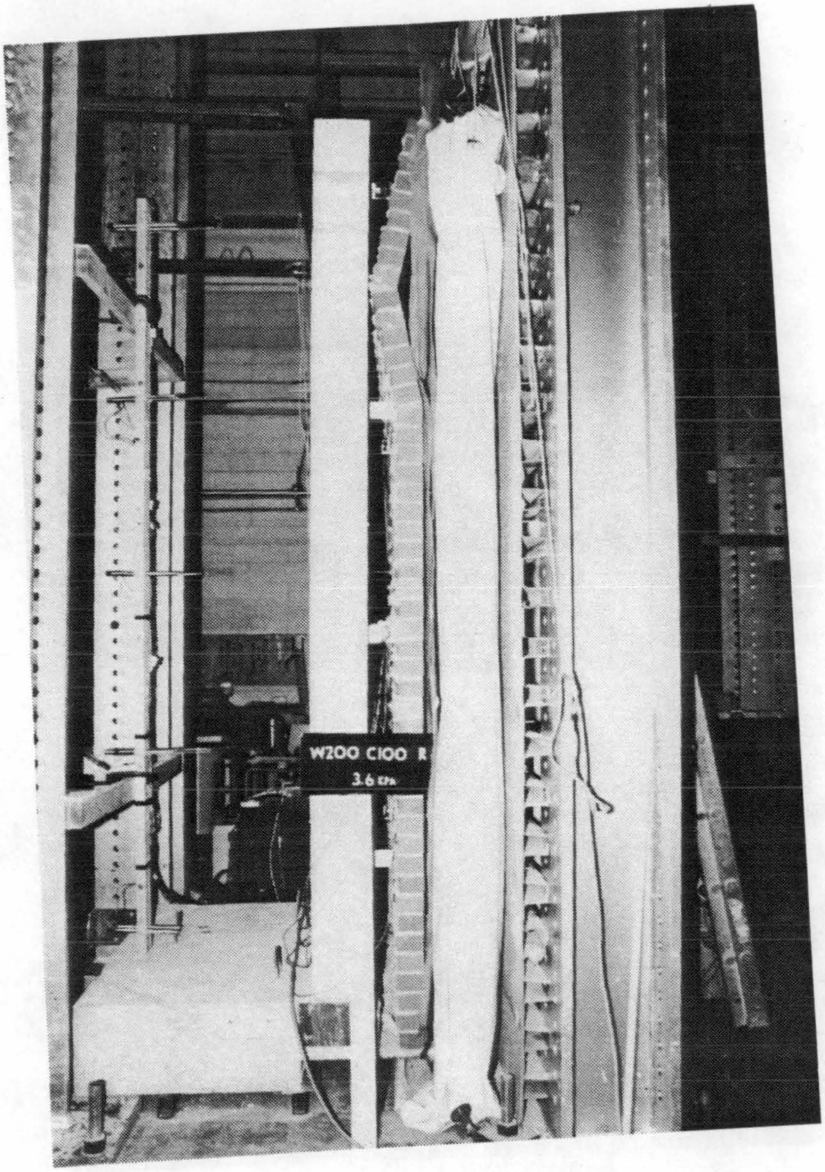


Plate 5.2 Specimen S1W2 after Failure

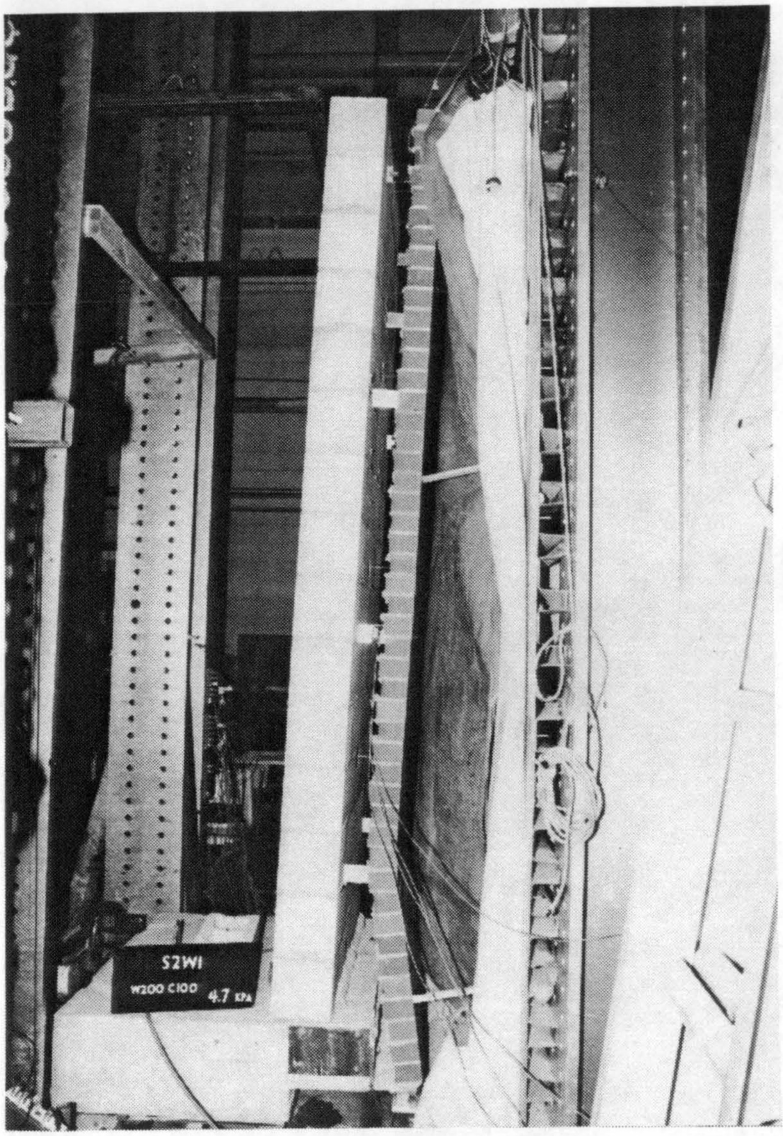


Plate 5.3 Specimen S2W1 after Failure

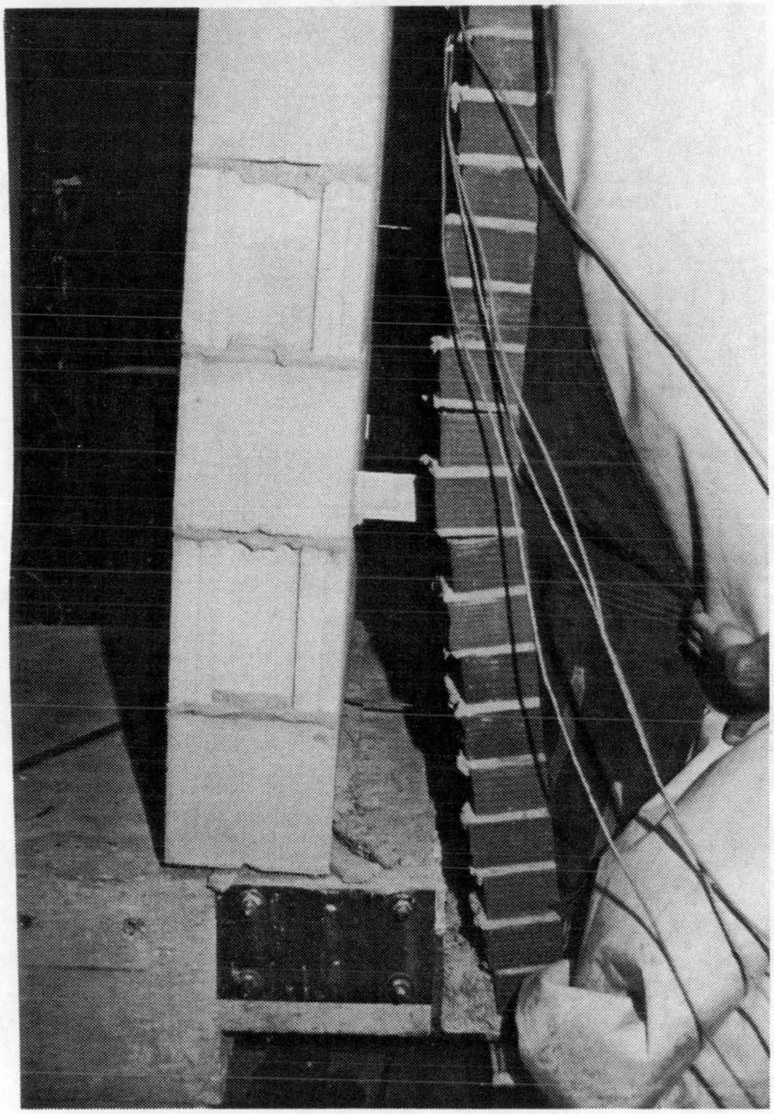


Plate 5.4 Close View of Failure for Specimen S2W1

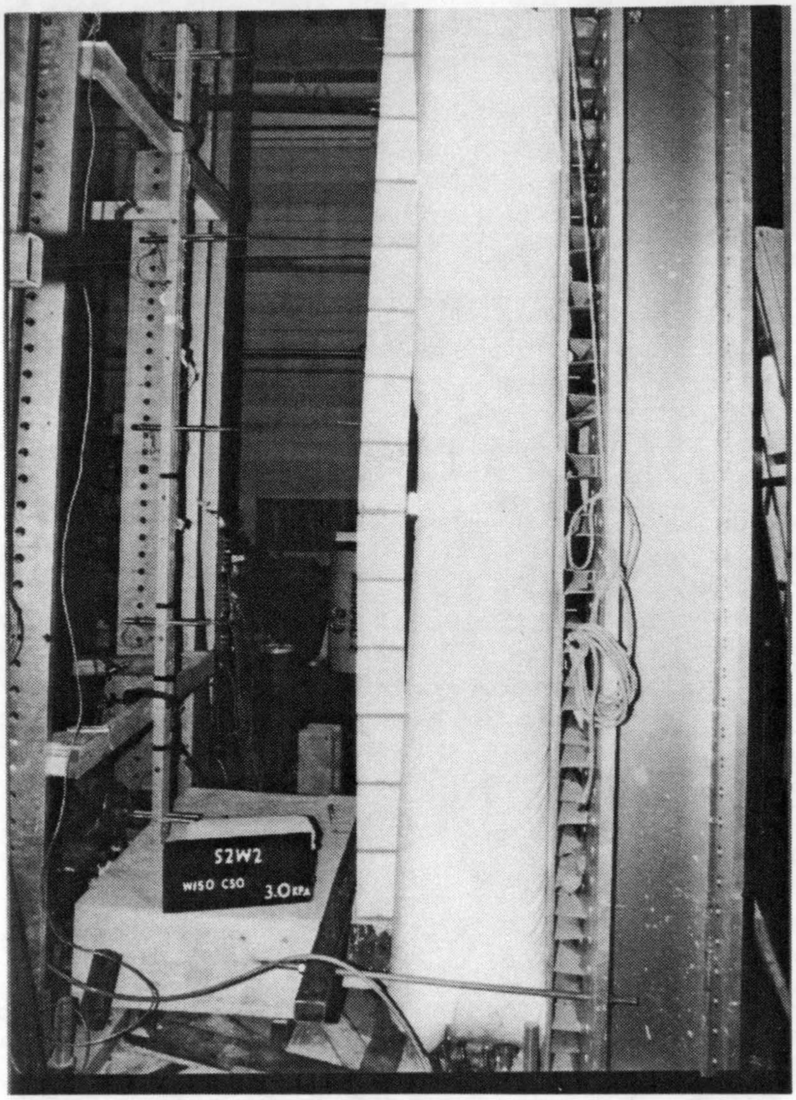


Plate 5.5 Specimen S2W2 after Failure

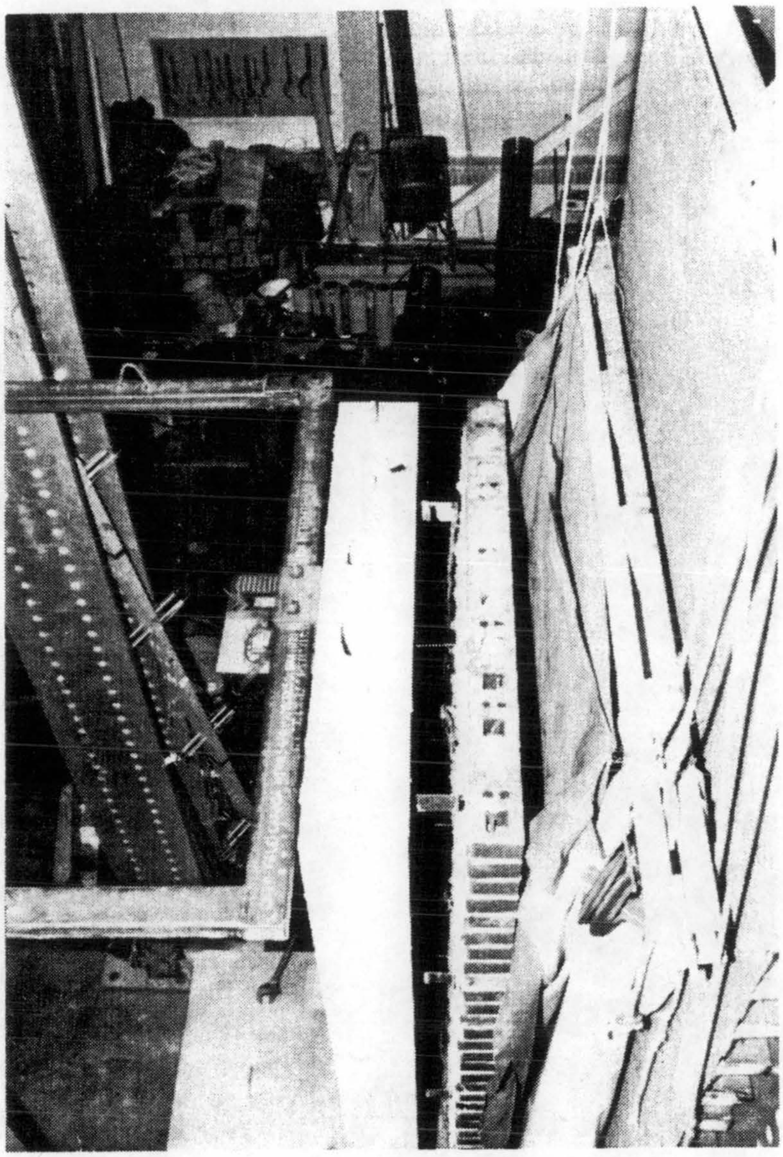


Plate 5.6 Reinforcement and Grouting of Specimen S2W3

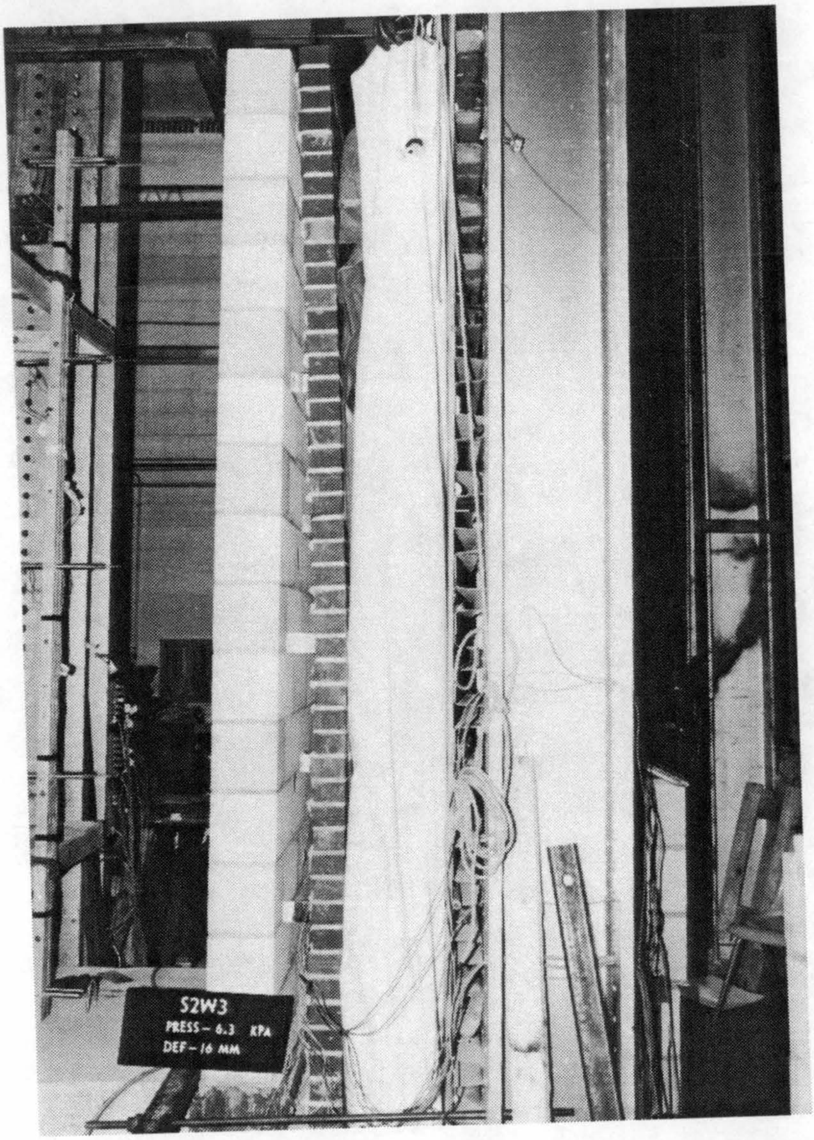


Plate 5.7 Specimen S2W3 after Failure

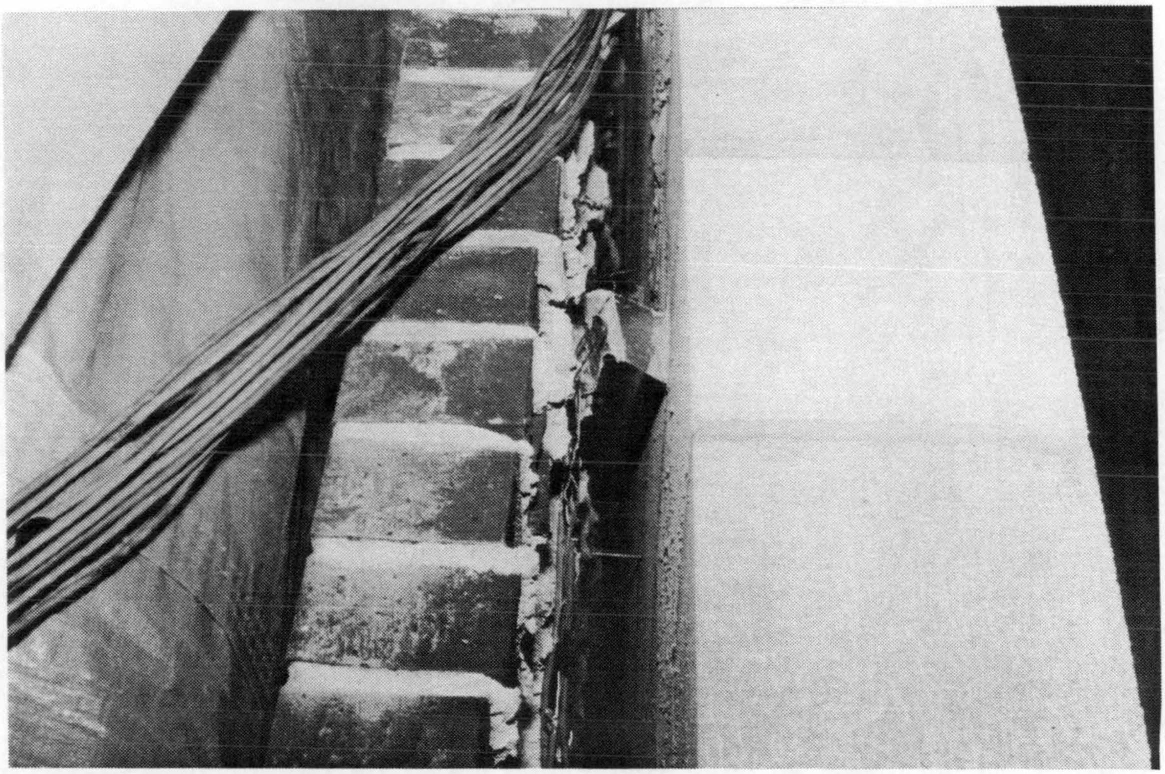


Plate 5.8 Close View of the Buckled Top Connector of Specimen S2W3

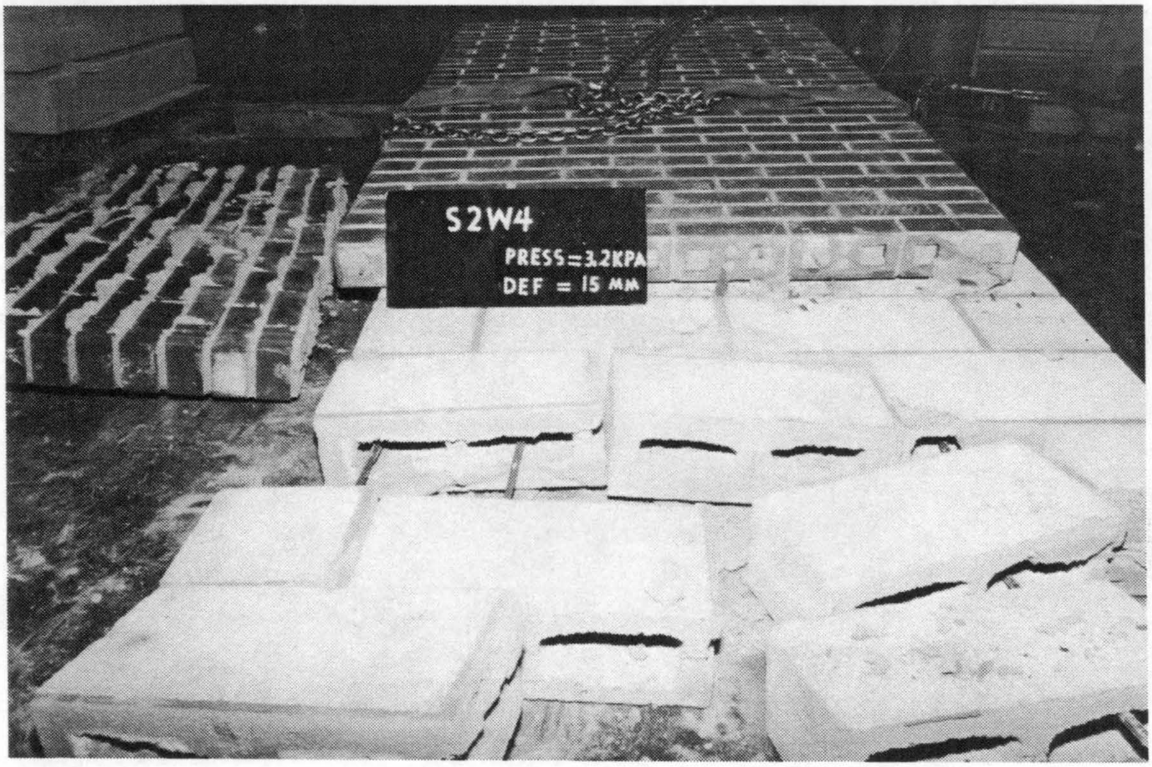


Plate 5.9 Disassembling Specimen S2W4

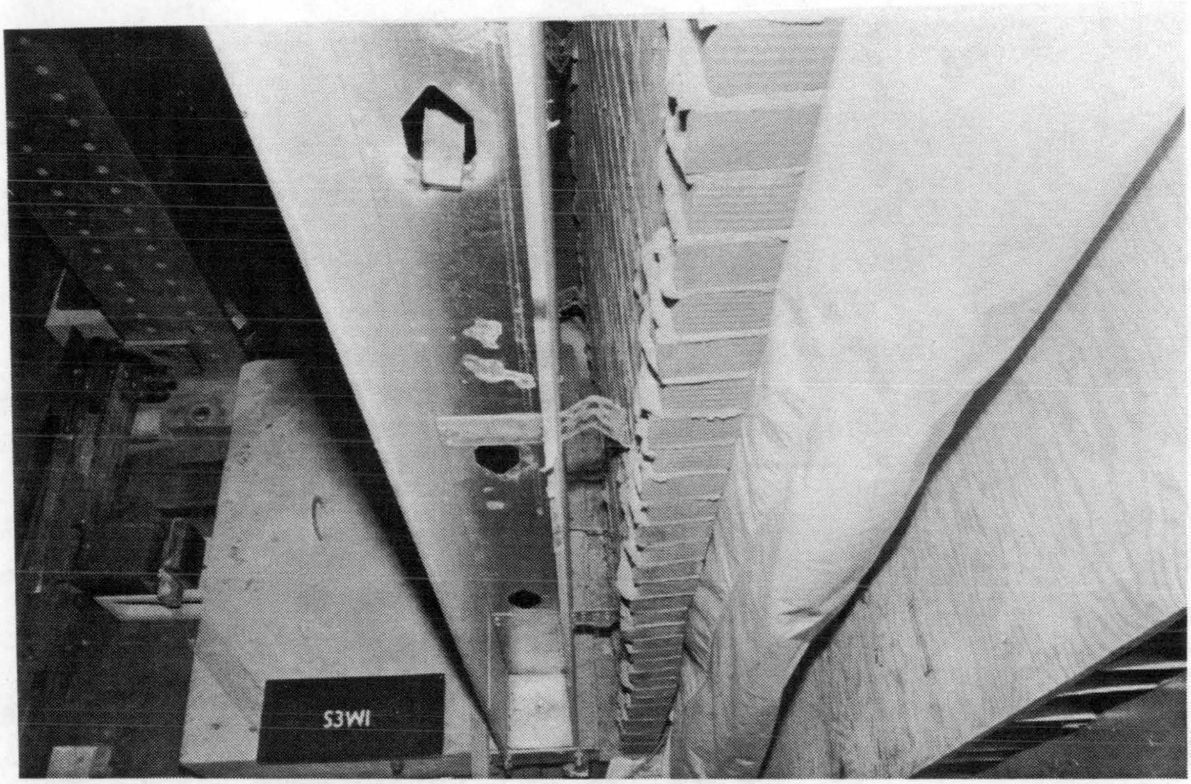


Plate 5.10 Specimen S3W1 after Failure

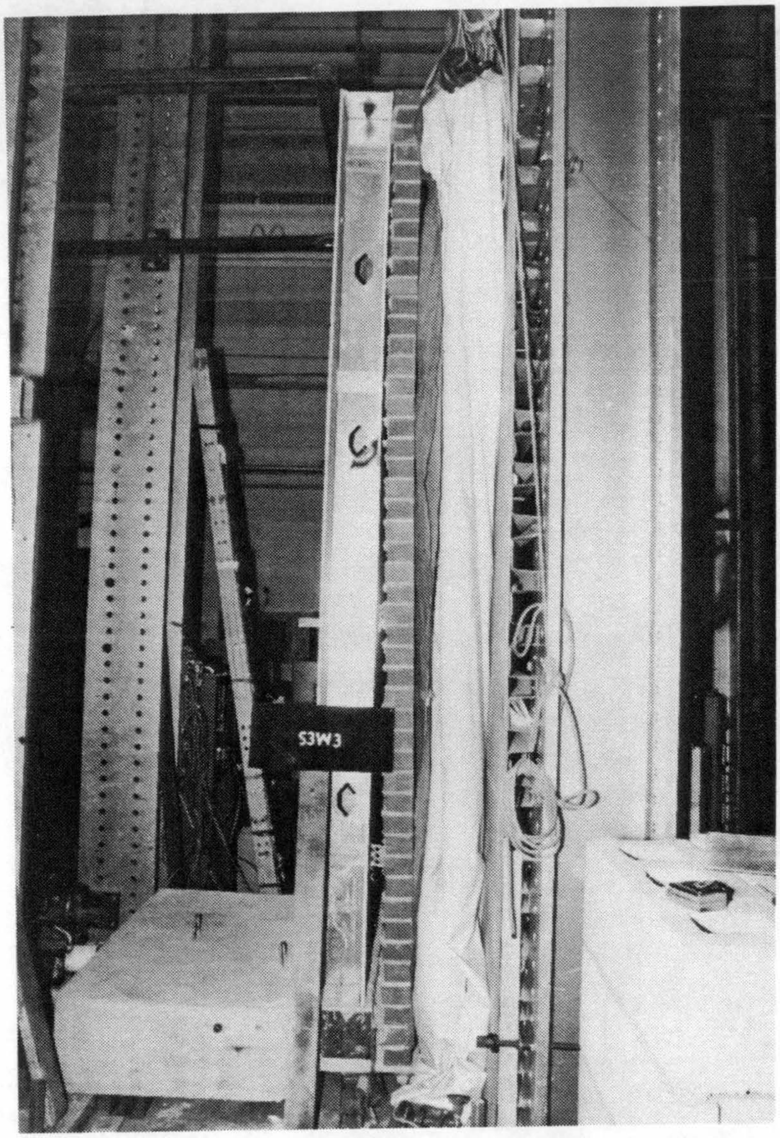


Plate 5.11 Specimen S3W3 after Failure

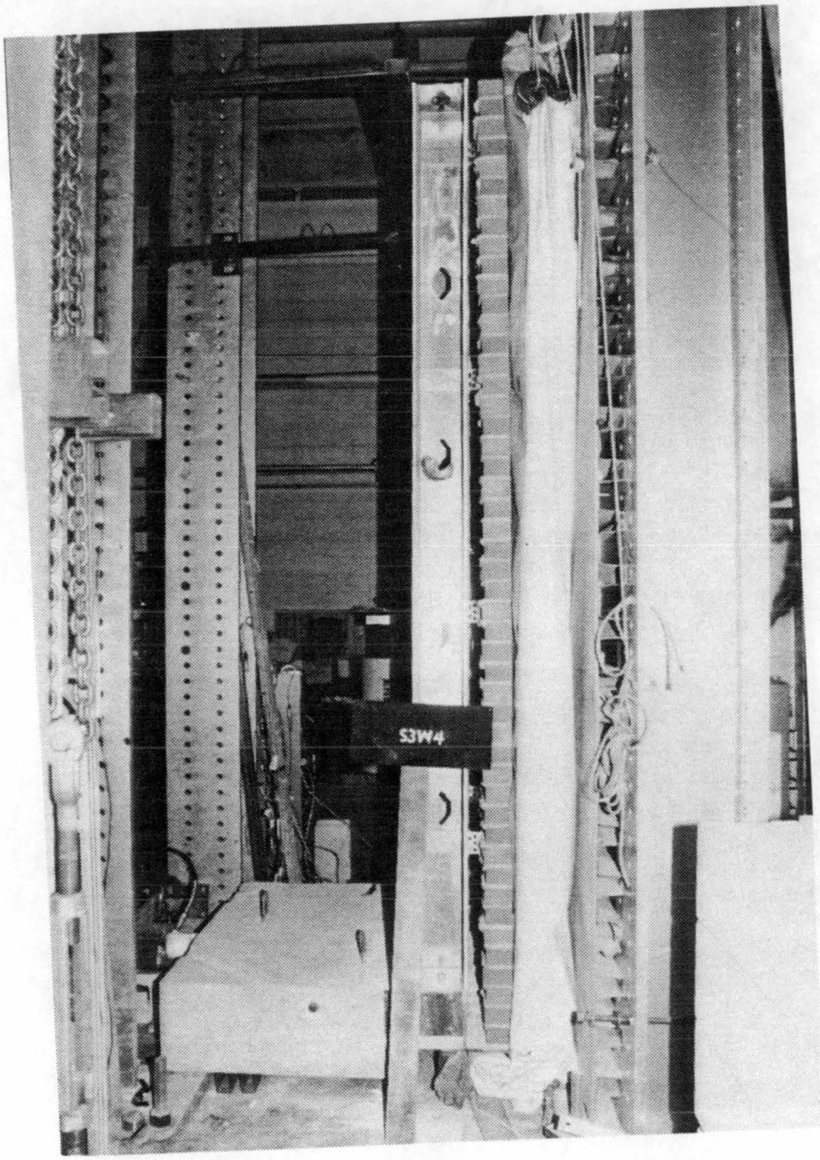


Plate 5.12 Specimen S3W4 after failure

CHAPTER 6

DISCUSSION AND ANALYSIS OF TEST RESULTS

6.1 Introduction

This Chapter consists of two parts. The first part examines and discusses the test results and especially the effects of the parameters under investigation (Sections 4.3.2.1 and 4.3.3.1) on the performance of a cavity wall.

In the second part a companson between the finite element analysis (described in Section 3.2) and the test results (Chapter 5) is presented for the case of masonry cavity walls. The objective was to confirm the validity of the finite element model (F.E.M.) in computing the internal forces transferred by the shear connectors. Since an elastic finite element analysis was conducted the comparison will be made only in the elastic range.

6.2 Discussion of Test Results

6.2.1 Shear Connector Plate

From the average values obtained by the wall segment tests (Section 5.2) it can be concluded that the capacity of the shear connector depends upon the type of load. The obtained average values were:

- Axial Compression: 5.8 kN

- Upward Vertical Load: 4.39 kN
- Downward Vertical Load: 3.26 kN.

From the small values of the standard deviations (Section 5.2)

It can be seen that the hole at which the tie is placed does not significantly affect the capacity of a connector under a given load. The mode of failure, which was identified as yielding of the metal plate around the hole at which the connector is placed, confirmed that the connector is fixed within the concrete block.

From the pull-out tests conducted on the simplified shear connector (see Sections 4.3.4 and 5.5) it was found that this connector is well confined within the mortar joints of the concrete block wall. The· steel plate around the· hole used by the connector, for all connector types, will yield before the connection of the connector with the concrete block fails. The simplified shear connector type D, which consists of corrugation and holes, had the best behavior and was able to resist an average of 7.12 kN before the connection failed (note that the steel plate will yield at an average tensile .force of 3.92 kN).

6.2.2 Effect of Concrete Block Width

By comparing specimen S1W2 with S1W3 (Figure C-3 and C-5) it can be concluded that for the case of hollow concrete block backup wythe the width of the concrete block does not significantly affect the overall behavior of the cavity wall, since the moment of inertia of the backup wythe does not increase much when the width of the block wall is increased. The poor performance observed for both specimens is attributed to the low tensile strength of ungrouted and unreinforced concrete block walls.

That is not the case for the specimens with reinforced concrete block wythe. Comparing _ S1W2 with S1W4 (Figure 6.1) and S2W3 with S2W4 (Figures C-13 and C-15), it can be seen that increasing the width of the concrete block results in an increase of the load carrying capacity of the wall system in the order of 50 % to 90 %. In addition, the maximum deflection at failure decreased by an average of two times due to changing from a 150 to 200 mm nominal block size.

For the case of 150 mm standard concrete block wythe (as presented 1n Sections 5,3.5 and 5,3.9) the grouting was poor, resulting in large deflections and premature failure, It is therefore important for the case of reinforced concrete block walls to ensure proper grouting.

6.2.3 Use of Vertical Reinforcement

A comparison of S1W1 and S1W3 with all the other specimens demonstrates that vertical reinforcement in the block backup wall results in a composite wall system with a load carrying capacity at least twice that of a hollow block backup wythe. That can be attributed to the following reason. For the type of load under consideration, the concrete block wythe is subjected to tensile stresses and therefore reinforcement is essential in carrying these tensile forces. The effect of grouting and reinforcement is shown in Figure 6.2 which compares specimen S1W1 with S1W2.

6.2.4 Effect of Shear Connector Arrangement

The variation of the connector pattern was investigated throughout this study and it was found that the spacing of the connectors and their location within the assembly must be considered as a relevant factor in the performance of a shear connected cavity wall system.

High axial and shear forces are attracted by the connectors near the supports. That can be confirmed by comparing specimen S 1 W2 with S2Wl. The load carrying capacity of S2Wl was improved by 30% just by adding one shear connector at the top middle course of the failed mortar joint of wall S 1W2.

As expected, by decreasing the spacing between the connectors the performance of the wall assembly is improved. A comparison of S 1 W2 and S 1 W 4 with S2W3 · and S2W 4 showed that by reducing the distance between connectors in half .the ultimate load capacity is doubled. ·The lateral deflections at comparable pressures were also decreased dramatically as a result of reducing the distance between connectors.

For the case of cavity walls with metal stud backup system it was found that the distance between the connectors affects significantly the composite performance of the wall assembly. For the case of connectors placed every 800 mm in both directions no composite action between the two wythes was observed. For a given pressure the deflections of the brick veneer were at least twice those of the backup system. By placing the connectors according to configuration C (see Figure 4.6 (b)) composite action between the two wythes is achieved. In addition to that the ultimate capacity increases significantly for the second case. Figure 6.3 shows the effect of the shear connector arrangements by comparing specimens S3W1 with S3W2.

6.2.5 Deflections of Masonry Cavity Walls

The allowable deflection for a masonry cavity wall is related to the crack width. In order to limit the maximum crack width to 0.5 mm it was found (Ref. 8) that the allowable deflection should be of the order of L/720. For the tested specimens this corresponds to an allowable deflection of 4.2 mm.

The deflections of the reinforced full scale specimens, corresponding to a pressure of 0.8 kPa (conservative value for wind load) were compared with the allowable deflection (4.2 mm). An average safety factor of 3 was found. That reduction in deflectfon will minimize crack width and moisture penetration, aspects that are very important for the design of masonry structures.

6.2.6 Effect of Backup System

Two different types of backup systems were used throughout this study: concrete block wall and metal studs - gypsum board assembly.

Figures 6.4 and 6.5 show the effect of the backup system for different shear connector patterns. As it can be seen from Figure 6.4, which compares specimen S1W2 with S3W1, for connector arrangement type A better performance is obtained when concrete blocks are used for the backup system although the ultimate capacity of the metal stud cavity wall was higher. As was stated in Section 6.2.5, for this type of connector pattern composite action is not present when metal studs are used for the backup system.

When connector arrangement type C is used very good performance for both concrete blocks and metal studs backup systems were observed. Comparison of S2W3 and S3W2 showed that both specimens acted compositely up to failure. At the serviceability limit state better performance was observed for the masonry cavity wall (smaller deflections). From· the point of view of ultimate capacity the metal stud cavity wall resisted higher pressure than its masonry counterpart.

6.2.7 Effect of Temperature Difference Between the Two Wythes

From the cavity wall that was exposed to the climatic conditions it was found that internal stresses are generated due to material properties and environmental factors. More specifically, Figure 5.8 showed that there is a trend between the temperature difference and the shear forces at the connectors. Increasing the temperature difference increases the shear force is also increase.

6.3 Comparison of Test Results with Analytical Study

Two different approaches exist in comparing the test results with the analysis. The one is by comparing the deflections for a given pressure and the other by comparing the internal forces in certain elements for a given pressure.

It is believed that for this study the most appropriate way of companng the results was the second approach for the following reasons. The deflections in the elastic range (for both wythes) that were obtained by the analytical and experimental studies were very small (less than a 1.0 mm) and therefore cannot be used for a comparison (since a difference of deflections on the order of 0.5 mm will yield a 50 % error). Secondly, since a cavity wall system is. very stiff as can be concluded from the small deflections, the internal forces must be large and therefore that is the . parameter that first concerns an engmeer.

Since this research 1s focusing on the study of the load transferred by the connectors to the backup system, the comparison was carried out for the axial force at the bottom shear connector . which is the critical one. The comparison was made only for the specimens that had their shear connectors instrumented with strain gages. These are the specimens with 100 mm cavity (see Table 4.2).

The strain gages readings were reduced by assuming a linear strain distribution. Then using a linear constitutive law with E = 200 GPa a linear stress distribution was found. Finally, using equilibrium equations the corresponding internal forces were calculated.

The comparison is presented in Table 6.1. The maximum .axial force transferred by the connectors is reasonably well predicted by the analytical model. The average test - to - predicted ratio was 1.069 with a standard deviation of 18 %.

For the completeness of the study Figure 6.6 shows a companson of deflected shapes at different pressures for specimen S 1 W2. The deflections obtained by the F.E.M. were found to be half the corresponding experimental values.

Table 6.1 Axial Compressive Force on Bottom Shear Connector (in kN)

Pressure	S1W2		S1W4		S2W3	
(kPa)	F.E.A.	Test	F.E.A.	Test	F.E.A.	Test
0.5	0.28	0.29	0.32	0.27	0.20	0.28
1.0	0.58	0.51	0.63	0.64	0.41	0.52
1.5	0.86	0.82			0.62	0.72

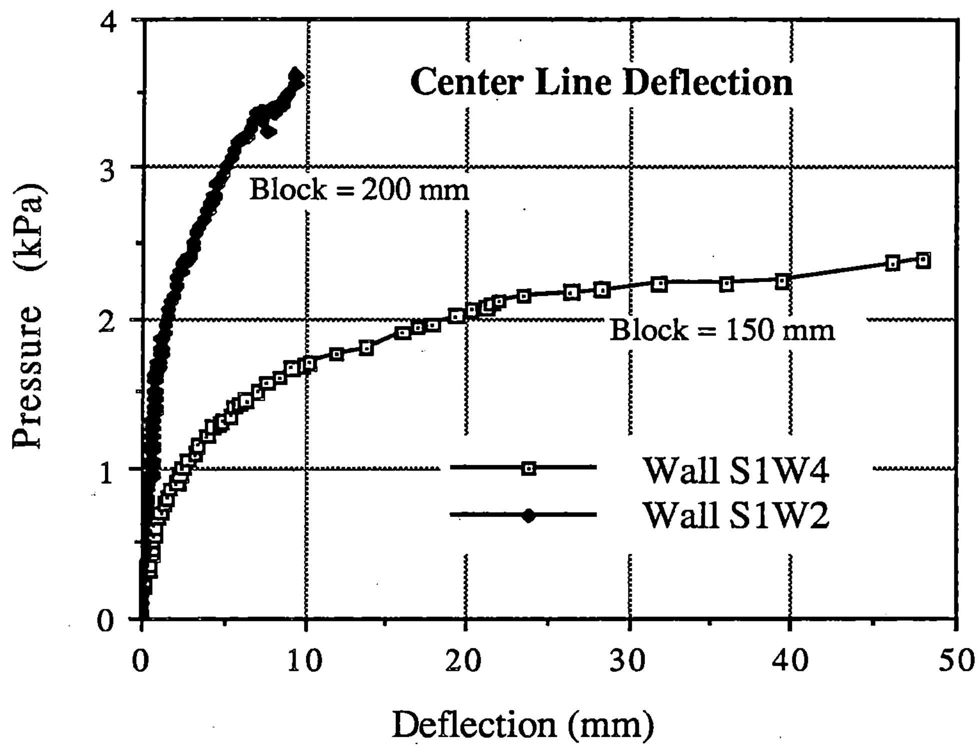


Figure 6.1 Effect of Concrete Block Width

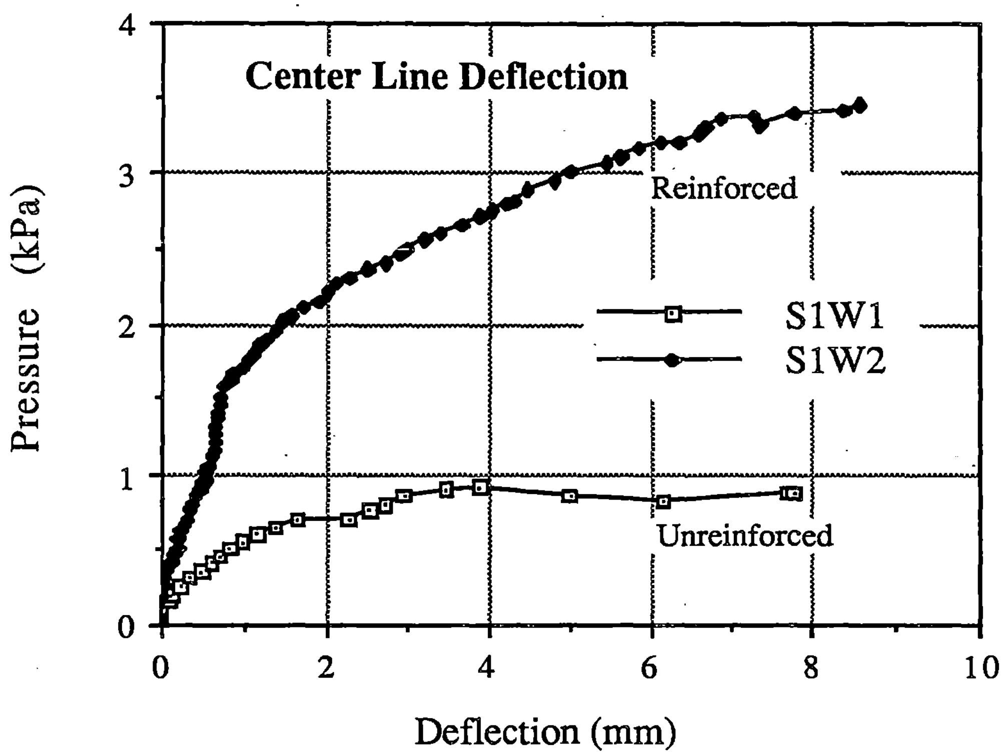


Figure 6.2 Effect of Grout and Reinforcement

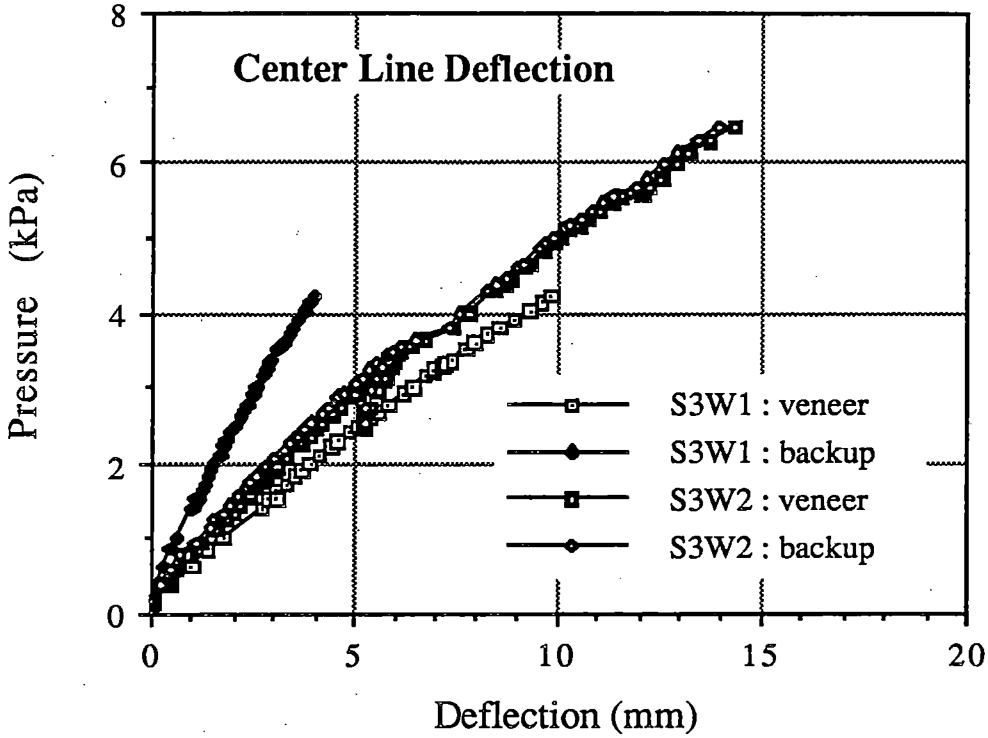


Figure 6.3 Effect of Shear Connector Arrangement

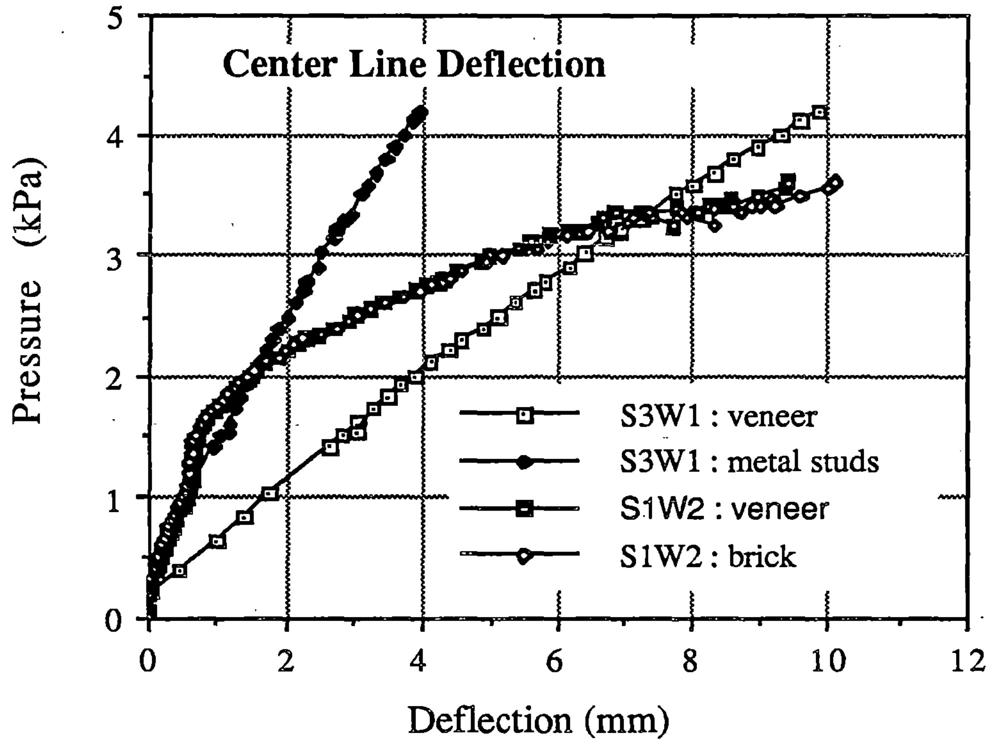


Figure 6.4 Effect of Backup System for Connector Arrangement Type A

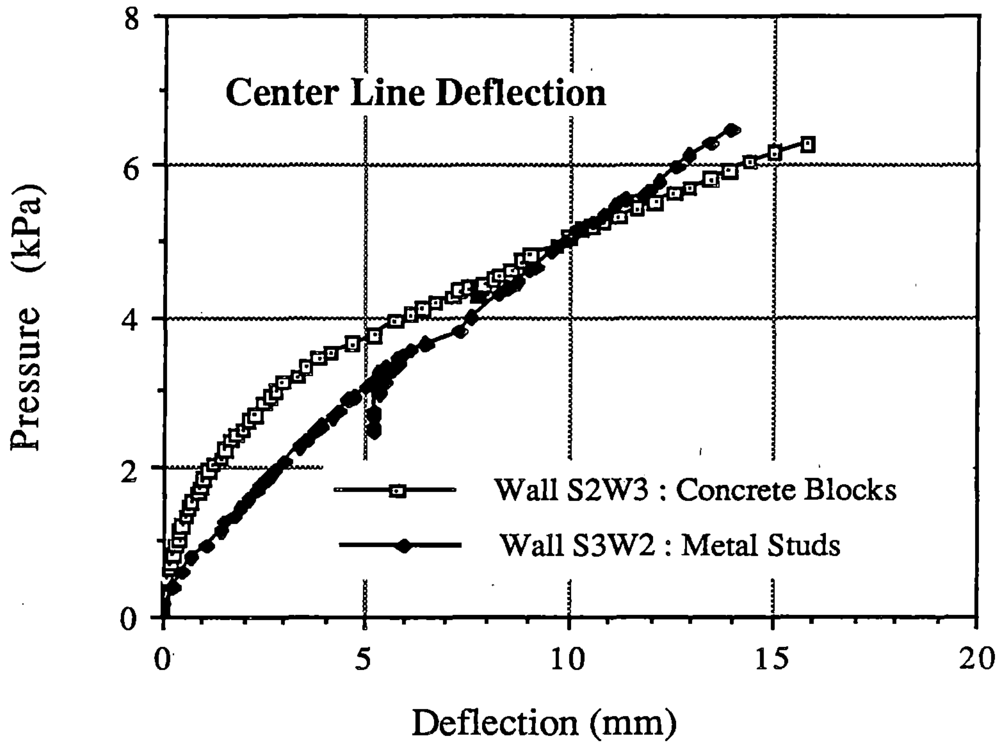


Figure 6.5 Effect of Backup System for Connector Arrangement Type C

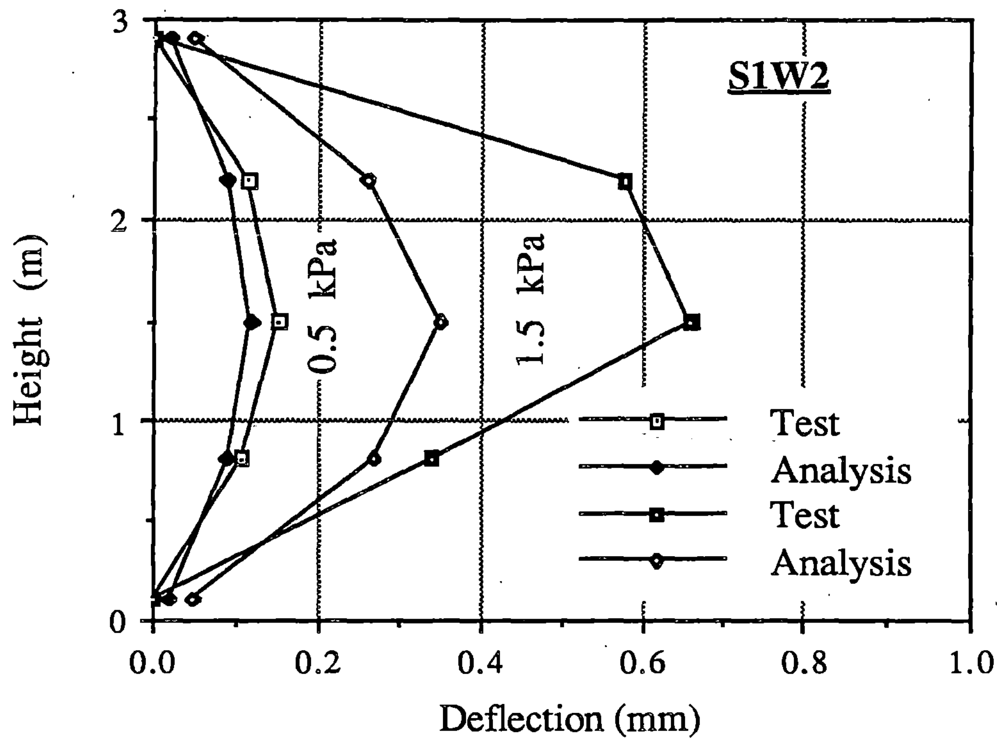


Figure 6.6 Comparison of Deflections Between Experimental and Analytical Results for S1W2

CHAPTER 7

PARAMETRIC ANALYSIS

7.1 Introduction

The comparison of the finite element results with the experimental results presented in section 6.3 indicates that the analytical model was valid in the elastic range and could be used for parametric studies.

This chapter presents the results of a parametric study that was carried out to investigate the effects that certain geometric parameters, material properties and climatic conditions have on the internal forees developed at the connectors of a masonry cavity wall.

Through this parametric analysis the approximate method (presented in section 3.3) used to calculate the internal forces due to opposite movements between the two wythes was compared with the more exact finite element analysis.

The results of this parametric study are summarized m graphical form. Similar graphs can be developed for a large variety of geometric parameters and material properties and can therefore be used as design charts for shear connected masonry cavity walls.

7.2 Effect of Geometric Parameters on the Performance of a Cavity Wall Subjected to Lateral Pressure

From the experimental results of masonry cavity walls subjected to out-of-plane pressure and with due consideration given to ease of construction and placement of insulation and accessories, it was found that the connector arrangement type A is the most appropriate. The connector pattern was therefore kept constant in this study and connectors were assumed to be placed every 800 mm m both directions.

The geometric parameters that were investigated consisted of:

- height of wall
- cavity width
- concrete block thickness.

The values of the different parameters are presented in Table 7.1. The modulus of elasticity of the masonry components was assumed to be 7.5 GPa. The cross-section of the shear connector was 1.5 x 60 mm^2 with modulus of elasticity 200 GPa. Finally, a nominal uniform pressure of 1.0 kPa was used for all the cases analyzed. The results are summarized in the graphs of Figure 7.1 to Figure 7.3.

7.2.1 Axial Force in the Connectors

As will be discussed in Section 7.2.3, the maximum axial force occurs at the endmost shear connectors. From Figures 7.1a, 7.2a, 7.3a it can be concluded that the larger the cavity and the taller the wall the higher the axial compression transferred by the connectors. These graphs indicate also that as the concrete block width increases the axial force on the connector decreases since more load can be attracted by the backup wythe.

7.2.2 Shear Force in the Connectors

As was expected, the maximum shear forces occur again at the endmost connectors. Figures 7.1 b, 7.2 b and 7.3 b indicate that the smaller the cavity and the taller the wall the higher the shear force transferred by the connectors. The concrete block width has again a beneficial effect as can be seen from these graphs.

7.2.3 Variation of Internal Forces in the Connectors Along the Wall

Figures 7.4 a and 7.4 b show the variation of connector forces (axial and shear) along the wall height. These graphs indicate that the connector forces are not uniformly distributed along the wall, particularly for tall walls (more flexible backup system). The maximum values for both axial and shear forces appeared at the endmost shear connectors. Therefore, the critical location for the connectors is the region adjacent to the supports. The behavior of a cavity wall subjected to uniform lateral load is similar to that of beam under uniform load where it is known that the maximum shear forces occur close to the supports.

7.3 Parametric Analysis for Masonry Cavity Walls Subjected to Opposite Movements

Table 7.2 summarized the values of two important parameters that affect the internal forces developed in the critical top connector due to the opposite movements of the two wythes. These are:

- cavity width
- differential strain due to material properties

A differential strain due to temperature change will have a similar effect to the maximum internal forces, except that the movement will be opposite.

The three first cases of Table 7.2 investigated the effect of the differential strains which are expressed with the linear shrinkage strain of block wythe and the linear expansion strain of the veneer. The temperature change and the cavity were kept constant. Case 4 to case 7 investigated the · effect of the cavity for a given differential movements of the two wythes.

A 3 m tall masonry cav.ity wall having 200 mm nominal standard concrete block backup system and a 90 mm wide brick veneer facing wythe was used for this analysis. The shear connectors were placed every 800 . mm in both directions. The geometric and material properties for a strip of a wall section 800 mm wide are given in Figure 7.5.

The seven cases presented in Table 7.2 were analyzed using both the finite element model and the simplified method (section 3.3). This comparison was used to check the validity and the limitations of the approximate method.

The results of this parametric study are summarized in Figures 7.6 and 7.7 which give the shear force at the critical top connector with respect to the variable under investigation.

7.3.1 Effect of Differential Strain

Figure 7.6 shows a linear relation between the differential strain of the two wythes due to material properties and the shear force at the top connector. More specifically, as the differential strain between the two wythes increases the shear force increases linearly.

Thermal deformations are most of the times opposite to the movements from material properties. For the case when the movements due to material properties are large the thermal deformations can have a beneficial effect.

7.3.2 Effect of Cavity

The internal forces that are generated m the connectors and masonry wythes due to the opposite vertical movements depend on the restraint provide by the connectors. The stiffer the connector the larger are the restraint and the internal forces. As the cavity decreases the bending stiffness of the connector, which can be expressed as 3(EI)sc/C3, increases substantially resulting in large internal forces (Figure 7.7). This graph shows that the internally generated forces on walls with cavity less than 50 mm are very large. It is therefore recommended that shear connected cavity walls incorporate cavities larger than 50 mm.

7.3.3 Comparison of the Finite Element Analysis with the Simplified Method

The approximate method models only the top connector and as such .it is expected to give , larger internal forces than the actual values, since the restraint provided by all the other connectors along the wall reduce the internal forces. The parametric analysis confirmed the above and showed that the simplified method gives always conservative results for both final deformations and internal forces. For large differential deformations (Figure 7.6) and small cavities (Figure 7.8) the approximate method deviates from the more exact finite element analysis. Therefore this simplified method is applicable only for differential strain smaller than 0.06 % and cavity larger than 75 mm.

Table 7.1 Parameters Studied for Cavity Walls Subjected to Lateral Pressure

Wall Height (mm)	Cavity (mm)	Block Width (mm)
3000	25	200
3800	50	250
4600	75	300
5400	100	
6200		
7000		

Table 7.2 Parameters Studied for Cavity Walls Exposed to the Climatic Conditions

	$\varepsilon_{SH,BL}$ (%)	$\varepsilon_{EXP,BR}$ (%)	ΔT_{BL} ('C)	ΔT_{BR} ('C)	Cavity (mm)
1	-0.015	+0.020	0	0	75
2	-0.025	+0.045	0	0	75
3	-0.040	+0.060	0	0	75
4	-0.025	+0.045	0	0	25
5	-0.025	+0.045	0	0	50
6	-0.025	+0.045	0	0	75
7	-0.025	+0.045	0	0	100

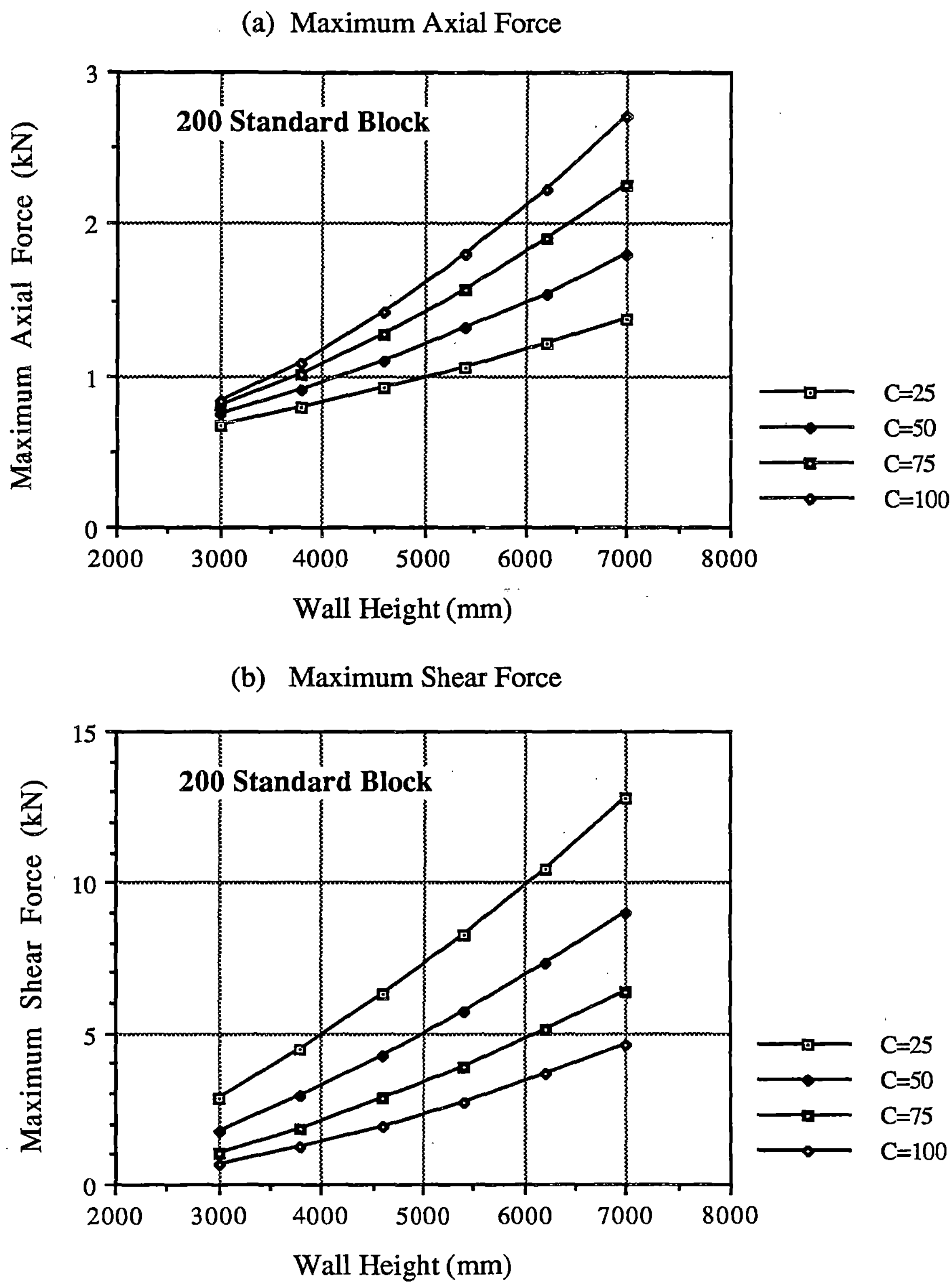


Figure 7.1 Internal Forces at the Bottom Connector for Cavity Wall with 200 Standard Blocks Subjected to Lateral Pressure

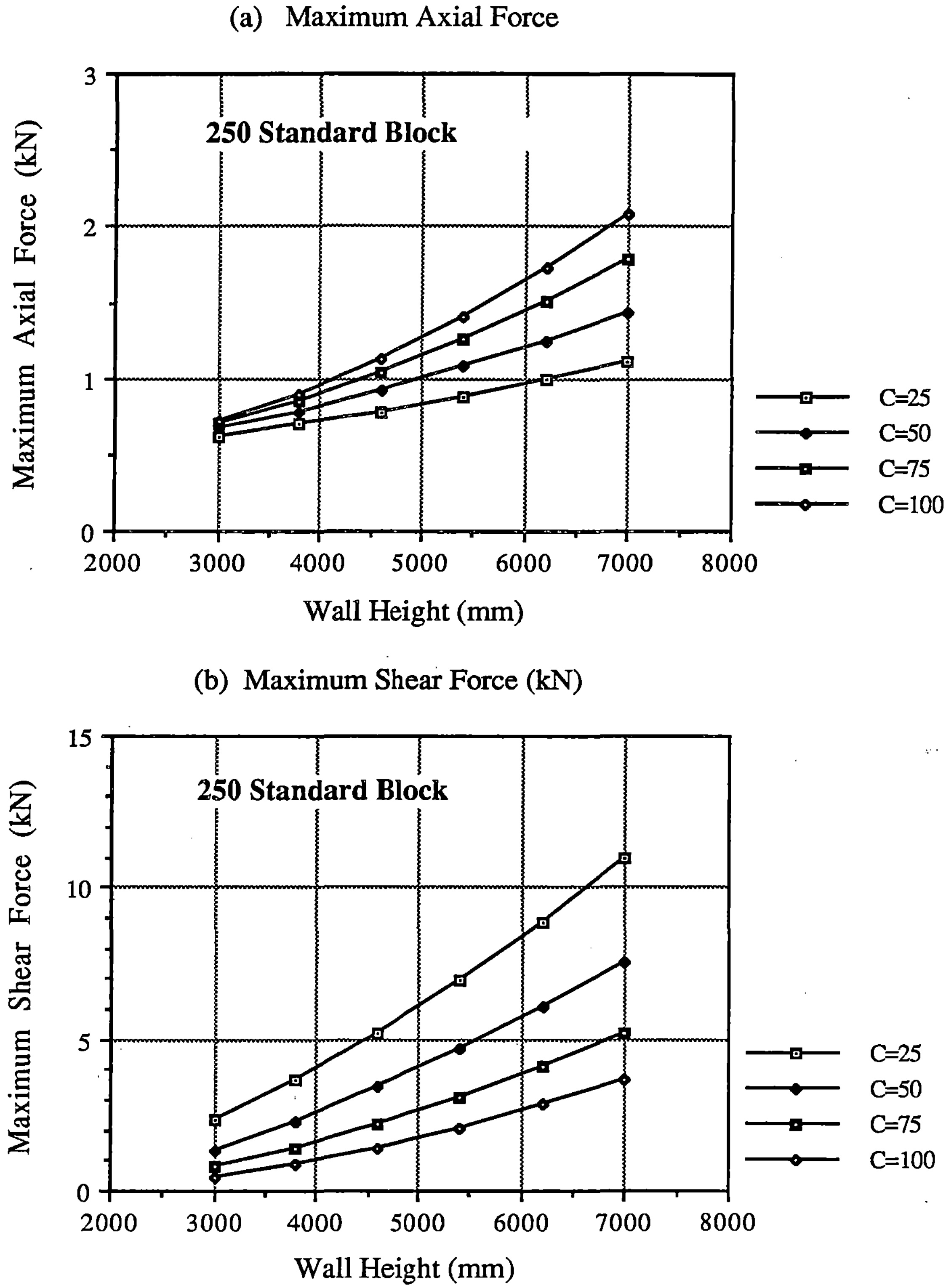
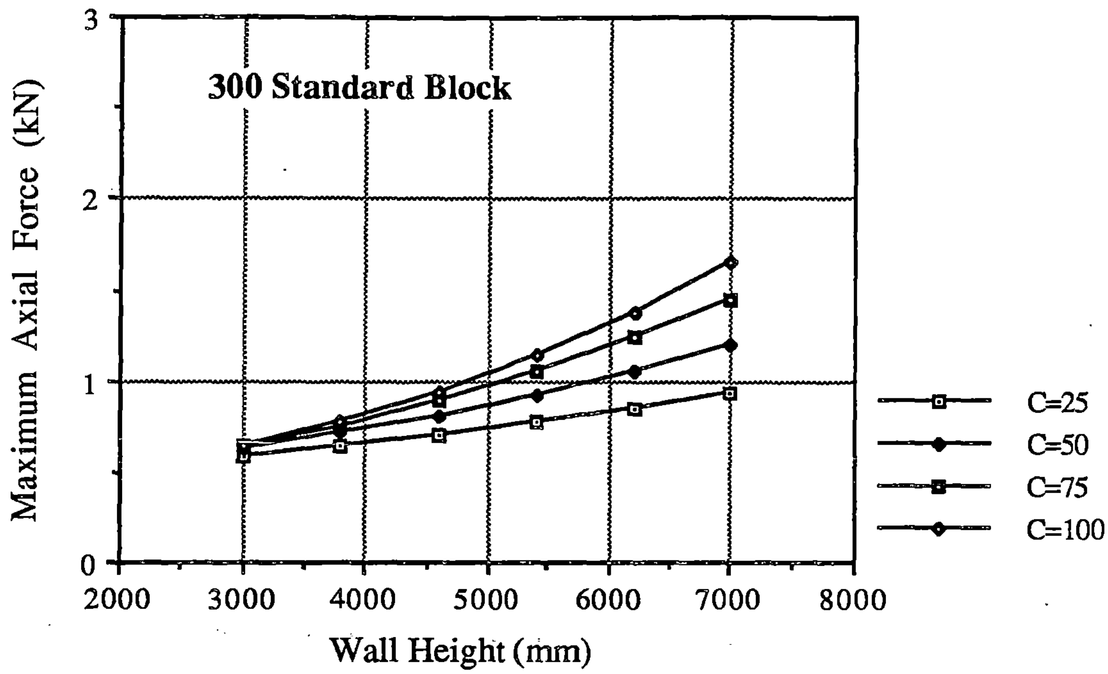


Figure 7.2 Internal Forces at the Bottom Connector for Cavity Wall with 250 mm Standard Block Subjected to Lateral Pressure

(a) Maximum Axial Force



(b) Maximum Shear Force

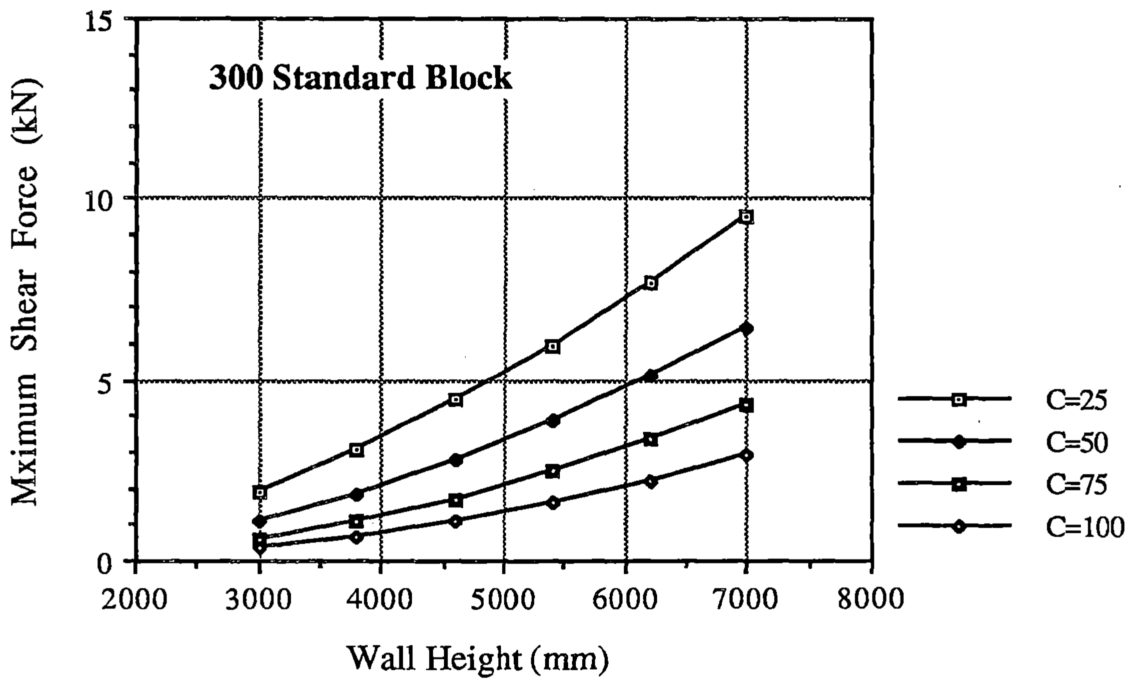


Figure 7.3 Internal Forces at the Bottom Connector for Cavity Wall with 300 Standard Blocks Subjected to Lateral Pressure

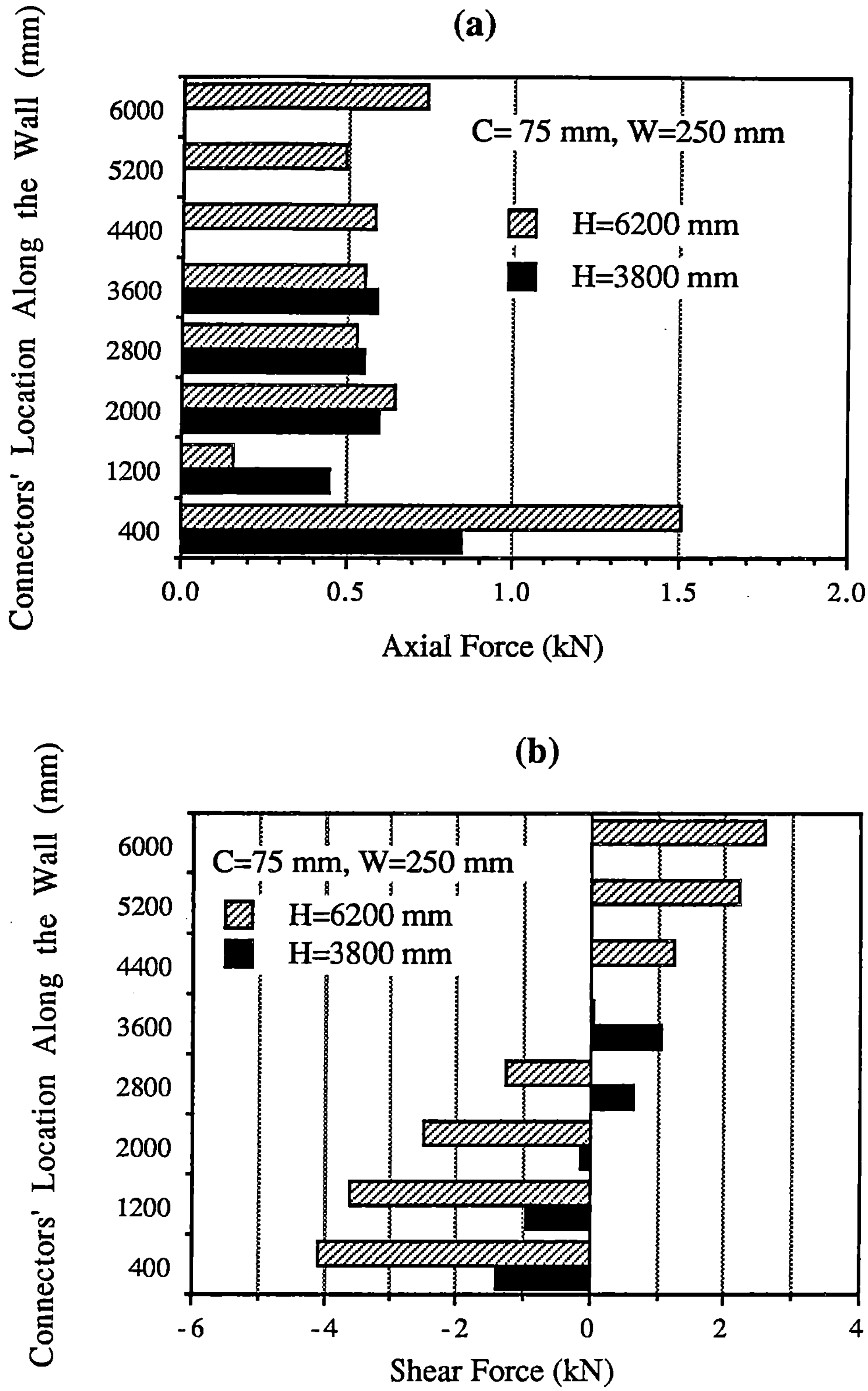
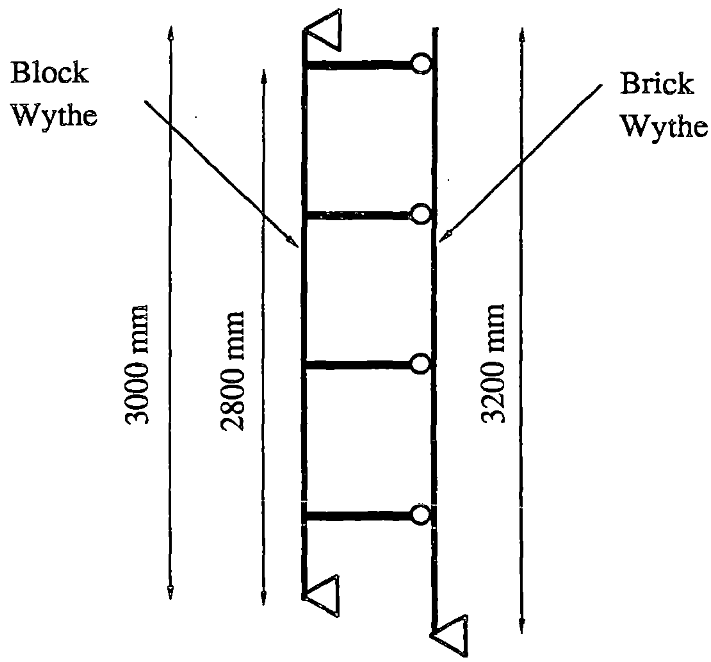


Figure 7.4 Variation of Forces on Connectors Along the Wall



Brick Wythe	E_m = 10x10^3 MPa
	A_{BR} = 72x10^3 mm^2
	I_{BR} = 48.6x10^6 mm^4
Block Wythe	E_m = 10x10^3 MPa
	A_{BL} = 60.32x10^3 mm^2
	I_{BR} = 353.4x10^6 mm^4
Connector	E_{SC} = 200x10^3 MPa
	A_{SC} = 90 mm^2
	I_{SC} = 27000 mm^4

Figure 7.5 Geometric and Material Properties for Cavity Wall Subjected to Opposite Vertical Movements

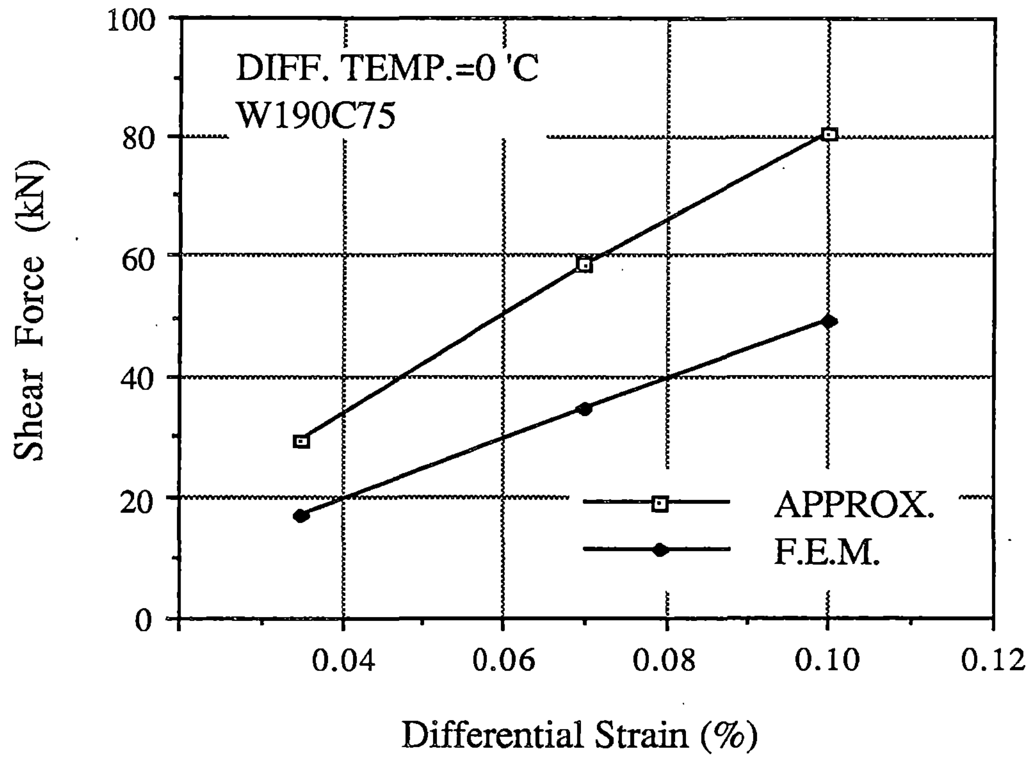


Figure 7.6 Effect of Material Properties

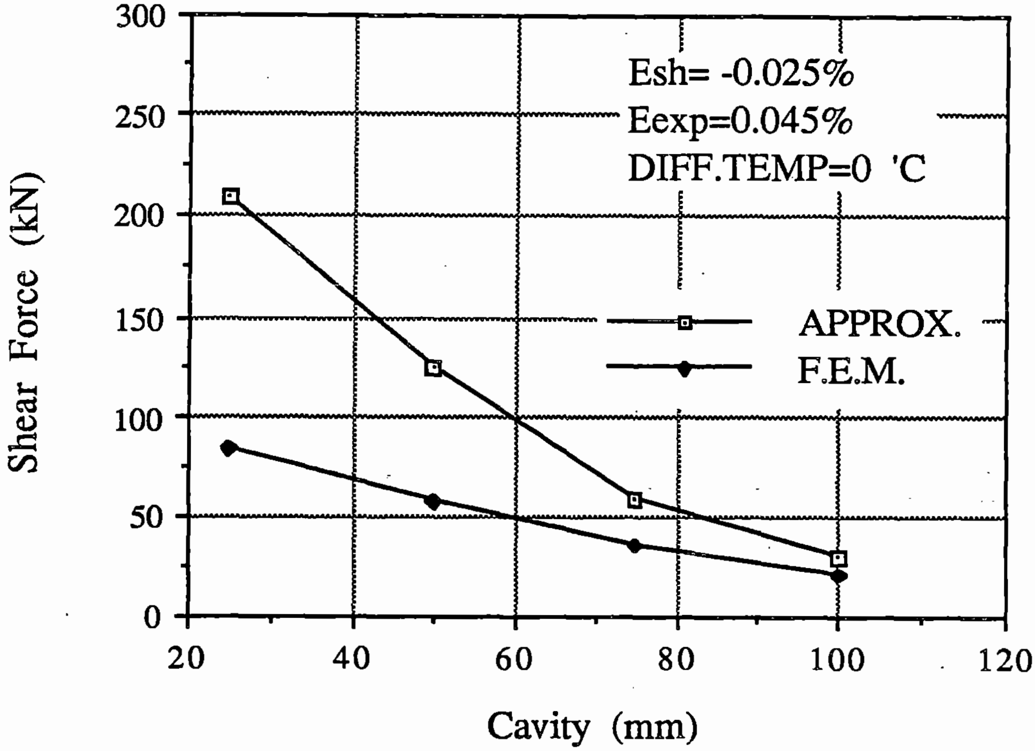


Figure 7.7 Effect of the Cavity

CHAPTER 8
SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

8.1 Summary

The object of the study reported herein was to investigate the performance of cavity walls incorporating individual adjustable shear-resisting connectors.

The experimental program consisted of :

- Twenty three concrete block wall segments with a shear connector 'embedded' in the vertical mortar joint. The scope of this series of tests was to estimate the capacity and investigate the mode of failure of shear connectors under different types of loads.
- Twelve full scale cavity walls subjected to positive lateral pressure. Factors explored in this phase of the study included backup system (concrete block wythe or metal studs with gypsum boards), cavity width, shear connector arrangement and vertical reinforcement. Furthermore, the internal force distribution amongst the connector was also investigated.
- A full size masonry cavity wall exposed to climatic conditions. The objective was to evaluate the internal forces induced in the masonry components, due to the restraint provided by the connectors to the opposite movements between the two wythes.

The analytical study included a finite element model that was developed to calculate the internal forces and deformations of masonry cavity walls subjected to in-plane loads, out-of-plane loads and

and vertical deformations. Comparison of the analytical model with experimental data indicated that the former was valid and could be used for parametric studies. The analytical model was therefore used to examine the effect that certain geometric parameters. material properties and climatic conditions have on the internal forces of masonry cavity walls

Finally, a simplified method was proposed to calculate the maximum internal forces and deformations of a masonry cavity wall subjected to differential vertical movements. The validity of the simplified method was evaluated by comparing the method with the more exact finite element analysis.

8.2 Conclusions

Based on both analytical and experimental studies presented herein, the following conclusions are revealed :

(a) For cavity walls subjected to out-of-plane pressure.

1. The shear connectors are capable of transferring shear and axial load from the brick veneer to the backup system.

2 From the deflected shapes of the full scale specimens under lateral pressure is concluded that composite action between the wythes is achieved when shear connectors are used. Under nominal wind pressure the deflections are consistently less than the allowable L/720 ; such reduction in deflection will minimize crack width and moisture penetration in actual structures.

3. The failure for masonry cavity walls without reinforcement occurs when the tensile stresses exceed th.e tensile bond strength of the mortar. From _the tests, no damage was observed to the shear connectors. The critical limit state for shear connected masonry cavity walls is therefore the tensile failure of the mortar bed of the backup system.

4. For cavity walls having metal studs as a backup system the failure was identified as buckling of the connectors.

5. The connector is well embedded within the concrete block wall. Thus, the steel plate around the hole used by the tie will yield before the junction of the connector with the concrete block wythe fails.

6. The elastic computer model that was suggested gave good predictions for the internal forces at low loads. This 2-D program can accommodate in-plane eccentric loads, out-of-plane pressure and opposite movements of the two wythes.

7. The load transferred between the two wythes through the connectors is not uniformly distributed along the height of the wall as commonly assumed in simplified

design approaches. The more flexible the backup wythe (smaller concrete block width, taller wall), the higher the non-uniformity of the forces in the connectors. Highest forces are attracted by the connectors near the supports.

8. Although the ultimate capacity of a cavity wall with metal studs is higher than that of a masonry cavity wall, better performance was achieved at service pressures when concrete blocks are used instead of metal studs. Especially for the recommended spacing of connectors, 800 mm in both directions, the two wythes did not act compositely' for the case- of a metal -stud backup._ system.

9. Load carrying capacity increases with concrete block width. vertical reinforcement and cavity size.

10. The simplified shear connector has the same performance as that of the initially proposed shear connector, but it is easier to place since it consists of two parts instead of four.

<u>(b) For masonry cavity walls exposed to climatic conditions:</u>

11. Both the experiment and analytical studies showed that the effects of thermal and moisture deformations on shear connected cavity walls cannot be neglected. The forces induced in the masonry components due to the restraints provided by the connectors should be calculated using a rational approach and the adequacy of the materials in resisting these forces must be checked.

12. For all cases investigated and for positive uniform lateral pressure it was found that the critical shear connectors are those located at the top.

13. The analysis confirmed that the brick veneer is always subjected to compressive force while tensile forces are generated in the concrete block wythe. Minimum reinforcement is, therefore, required for the backup wythe, to accommodate tension.

14. The approximate method proposed for calculating the internal forces and deformations on the masonry components due to environmental conditions yields adequate results. This method was compared with the more exact finite element analysis and was found to give conservative results for both the final deformations and the internal forces. For cavities smaller than 50 mm the internal forces calculated using the simplified method were approximately twice the values obtained from the finite element analysis.

15. From the parametric analysis it was found that the internally induced forces on walls with cavities less than 50 mm are very large. It is therefore recommended that shear connected cavity walls incorporate cavities no less than 50 mm.

16. Finally, the placing of connectors at 800 mm in both directions will facilitate construction and placement of accessories such as insulation, air-vapor seals etc.. No adverse effects were found in this study· for such a connector arrangement.

8.3 Recommendations for Further Studies

1. Using the computer model proposed in this study, design curves can be obtained by varying the different parameters involved. These charts will also help to establish suitable guidelines for the design of cavity walls.

2. The performance of a cavity wall under in-plane load applied to the backup wythe must be investigated. Especially, the contribution of the veneer to the stability of the wall assembly should be studied and the moment magnifier method should be revised to account for the contribution of the veneer.

3. Further tests need to be conducted to examine the stresses induced in masonry cavity walls due to climatic conditions and material properties.

4. Tests should be carried out to evaluate the capacity of shear connectors subjected to biaxial forces and interaction diagrams should be developed for the capacity of the connectors depending on factors such as cavity width gage of the steel plate, hole used by the tie and ratio of axial to shear force.

5. Further studies are needed for shear connected cavity walls with metal stud backup system. Factors such as stud thickness, stud width and stud spacing should be investigated.

6. The performance of cavity walls subjected to suction pressure (negative lateral pressure) should also be investigated.

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8. Hatzinikolas, M. A., R. Lee, J. Warwaruk, "Factors Affecting the Performance of Metal Stud Walls Used as a Backup System to Masonry Veneer", Prairie Masonry Research Institute, Edmonton, Alberta, December, 1987.
9. Hatzinikolas, M. A., Longworth, J., Warwaruk, J., "Strength and Behavior of Metal Ties in 2-Wythe Masonry Walls", Alberta Masonry Institute, Edmonton, Alberta, 1981.

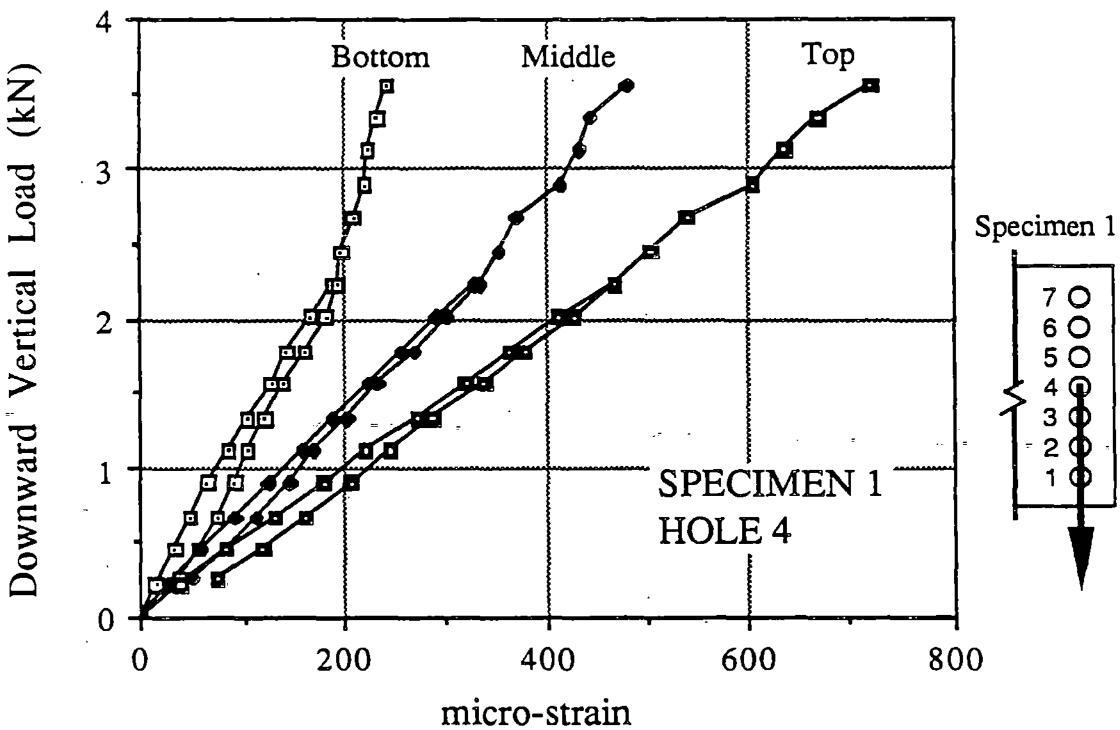
APPENDICES

APPENDIX A - Material Test Results

Table A-1 Summary of Masonry Material Tests

Series	Mortar Cube Compressive Strength (MPa)	Grout Cylinder Compressive Strength (MPa)
1	10.93	22.3
2	12.37	21.7
3	6.43	–

APPENDIX B - Shear Connector Wall Segment Test Results



* Numbering of hole is from bottom to top.

Figure B-1 Load - Strain Gage Readings for Specimen 1 with Downward Load Applied at Hole No 4

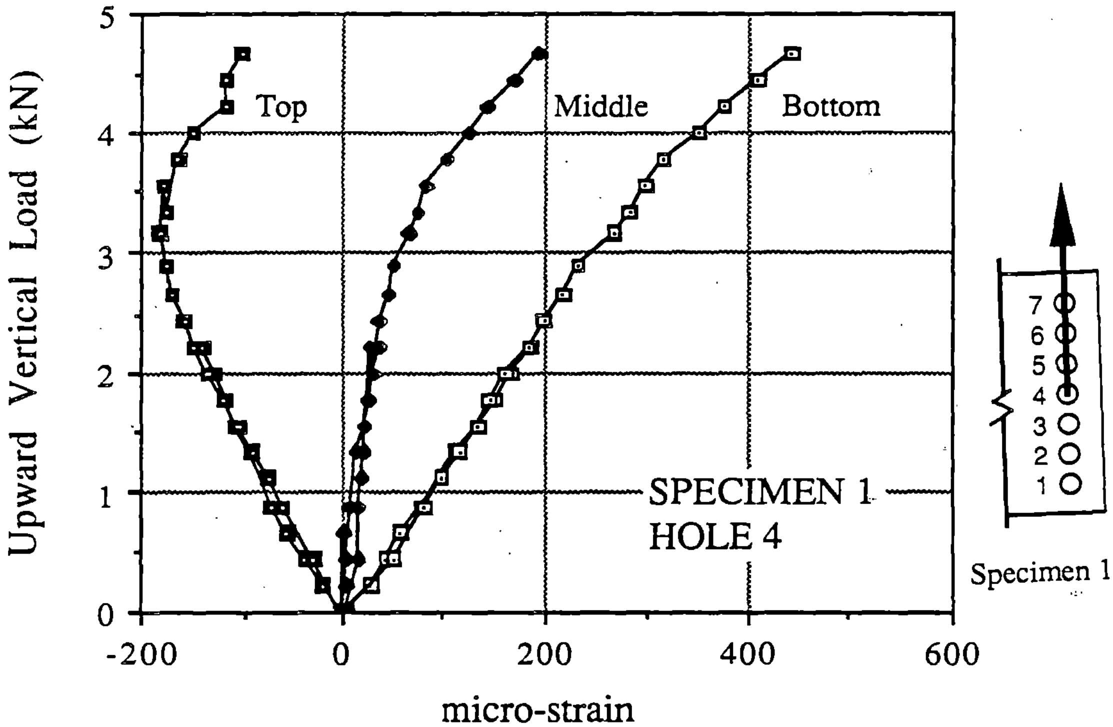


Figure B-2 Load - Strain Gage readings for Specimen 1 with Upward Load Applied at Hole No 4

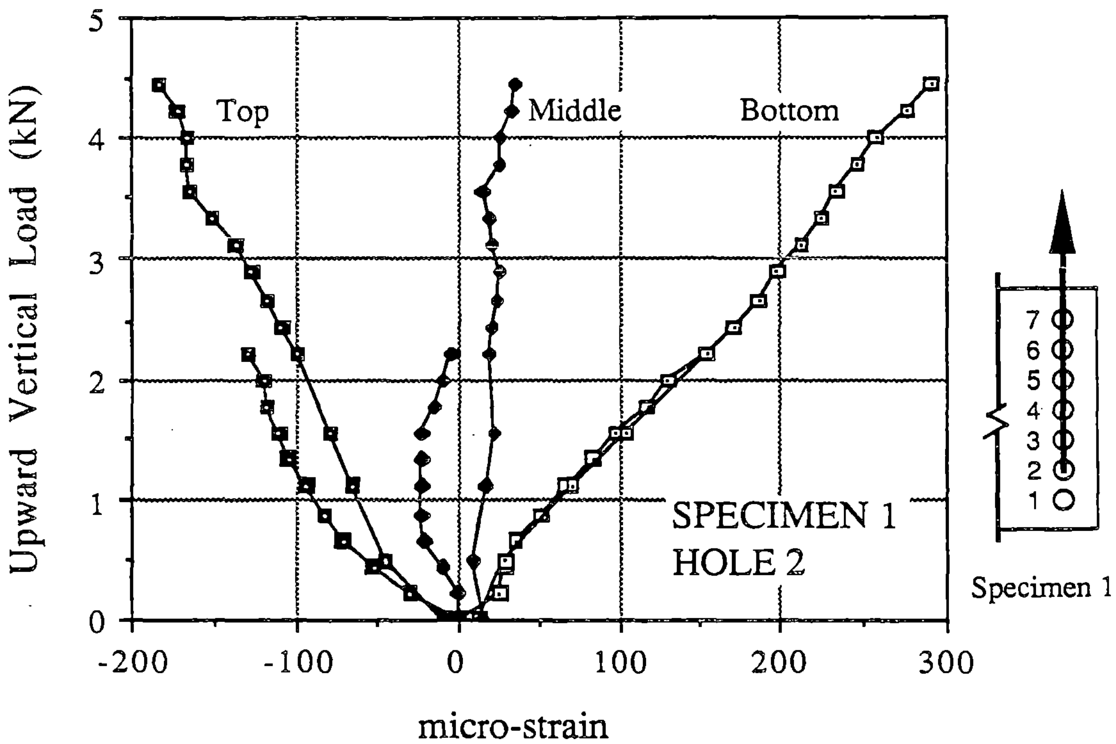


Figure B-3 Load - Strain Gages Readings for Specimen 1 with Upward Load Applied at Hole No 2

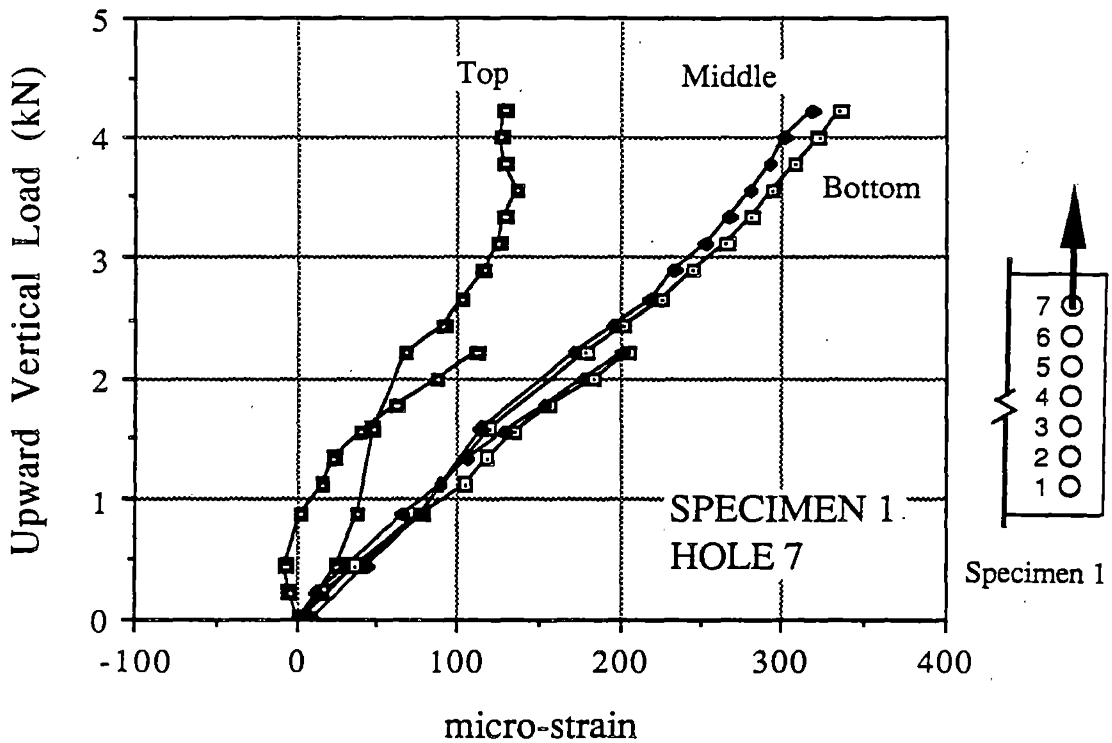


Figure B-4 Load - Strain Gages Readings for Specimen 1 with Upward Load Applied at Hole No 7

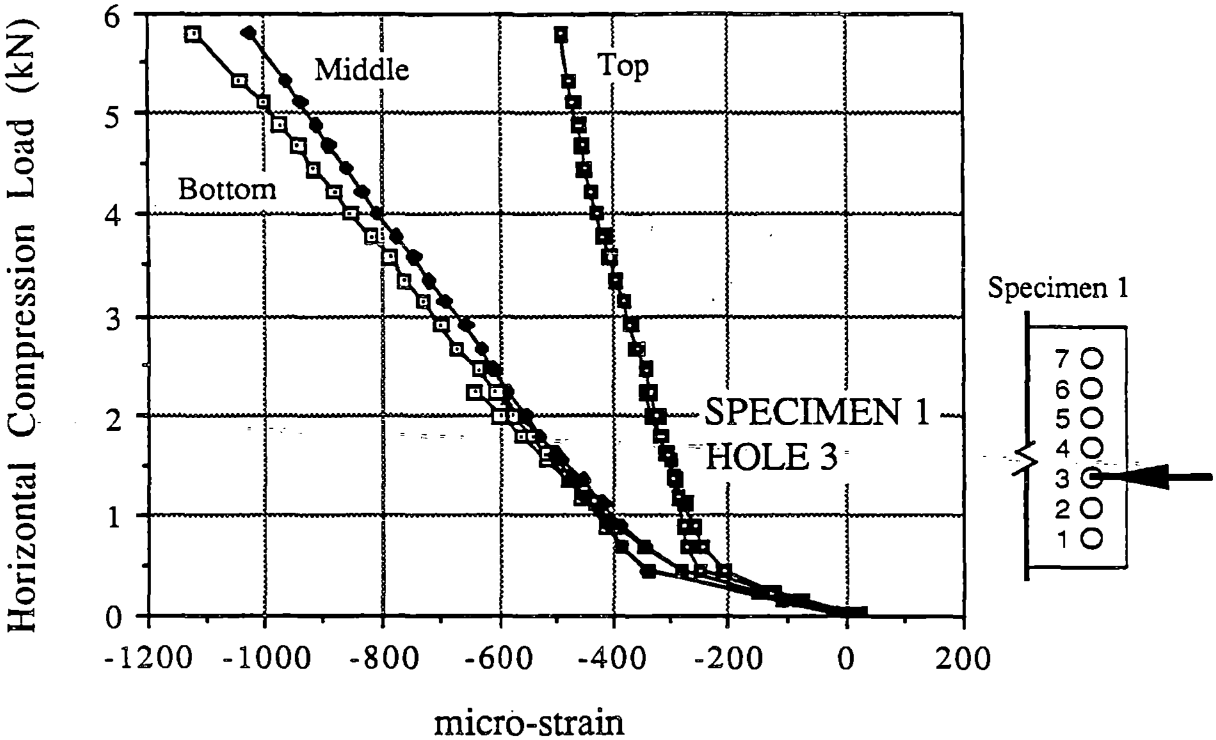


Figure B-5 Load - Strain Gages Readings for Specimen 1 with Compressive Load Applied at Hole 3

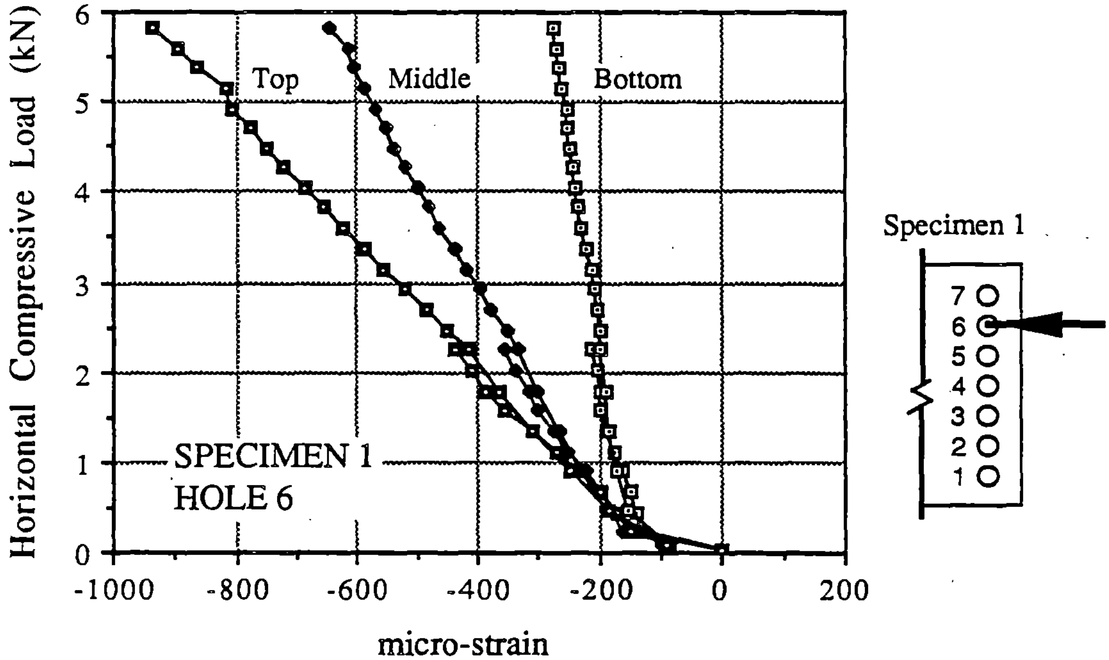


Figure B-6 Load - Strain Gage Readings for Speciemen 1 with Compressive Load Applied at Hole No 6

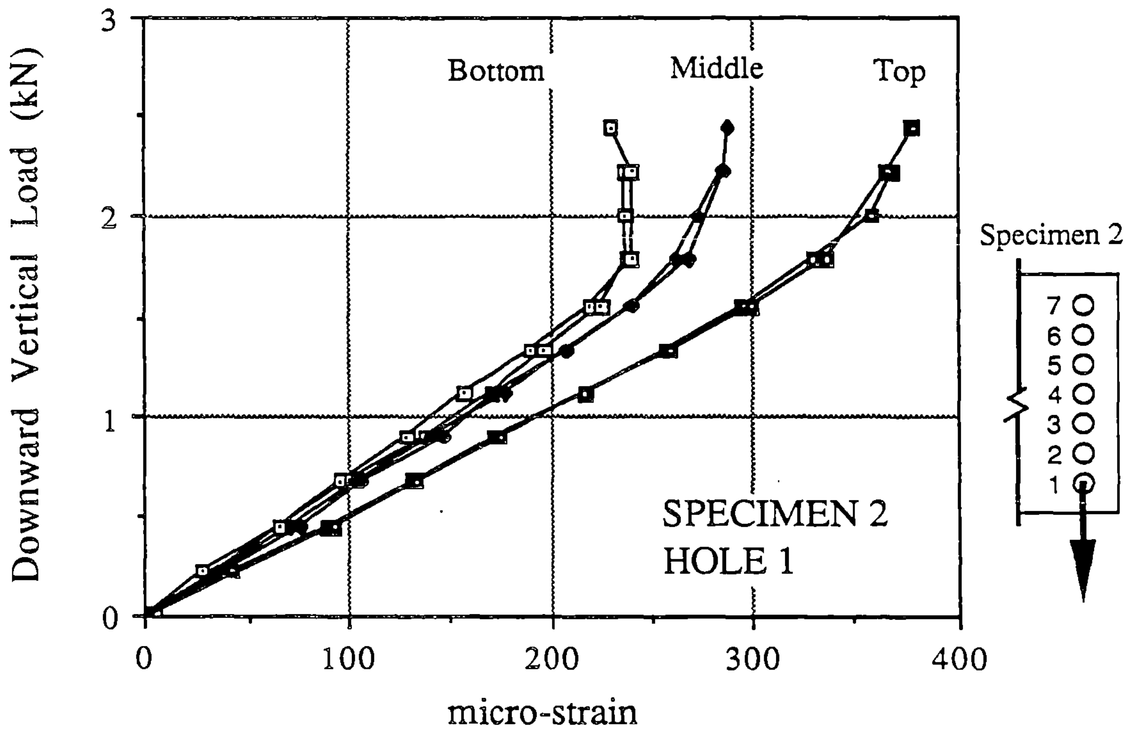


Figure B-7 Load - Strain Gage Readings for Specimen 2 with Downward Load Applied at Hole No 1

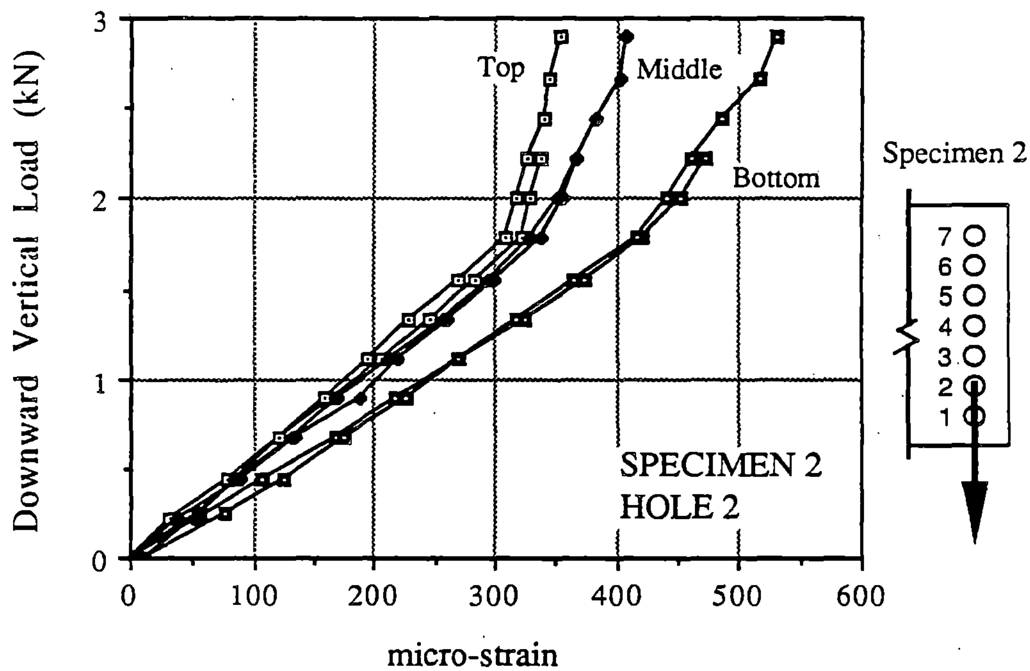


Figure B-8 Load - Strain Gage Readings for Specimen 2 with Downward Load Applied at Hole No 2

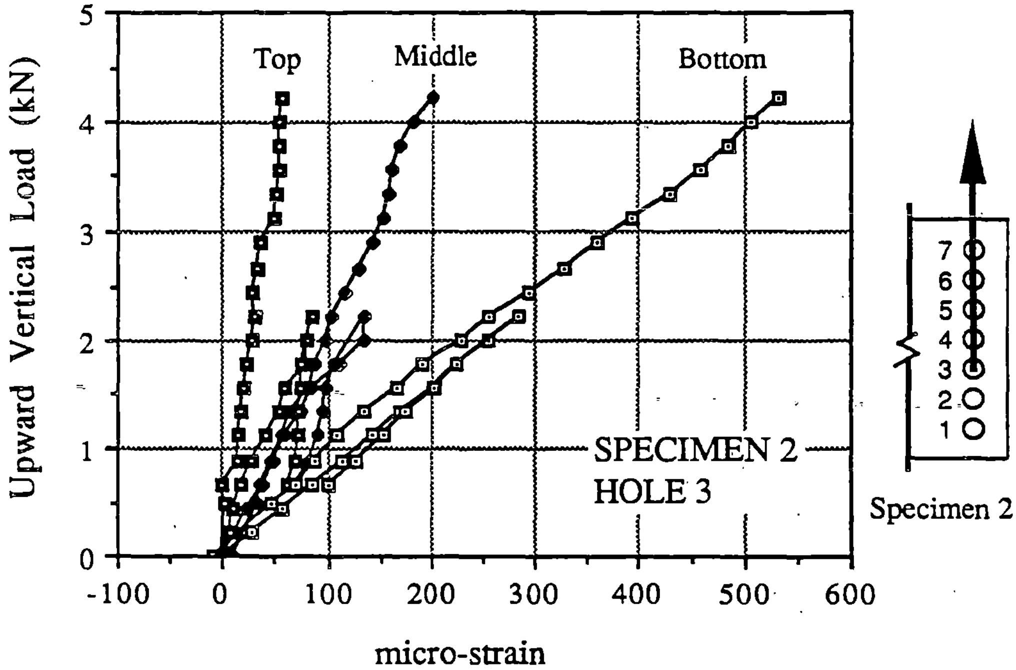


Figure B-9 Load - Strain Gage Readings for Specimen 2 with Upward Load Applied at Hole No 3

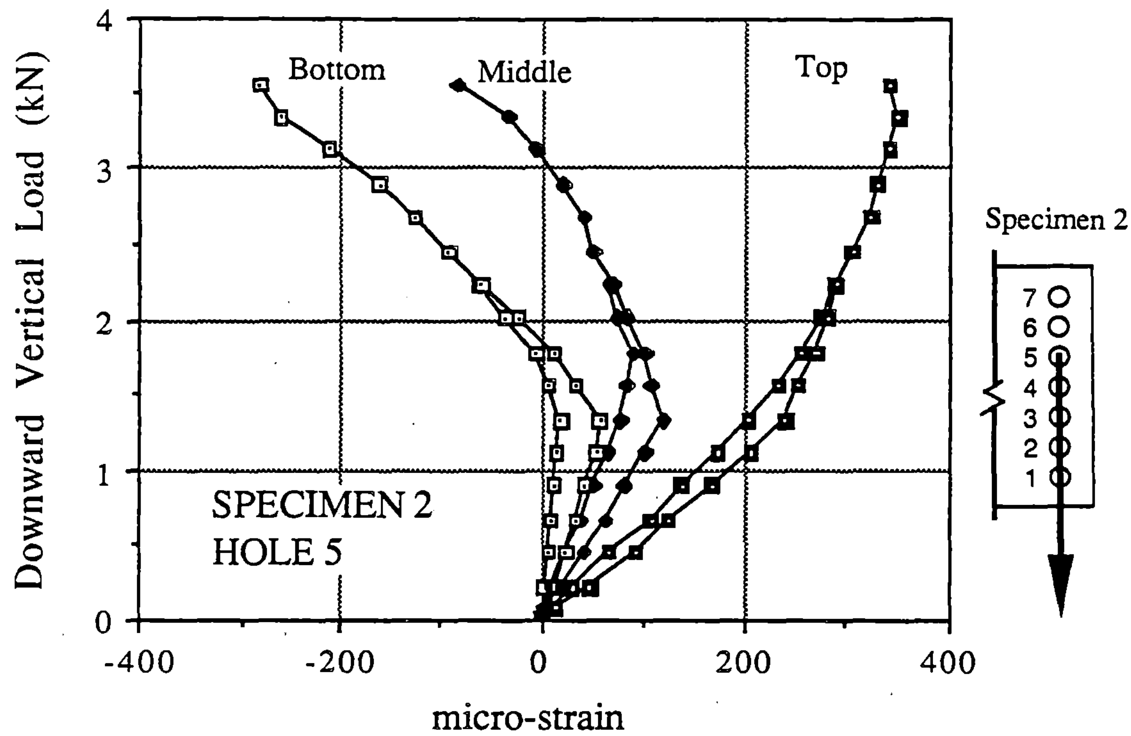


Figure B-10 Load - Strain Gage Readings for Specimen 2 with Downward Load Applied at Hole No 5

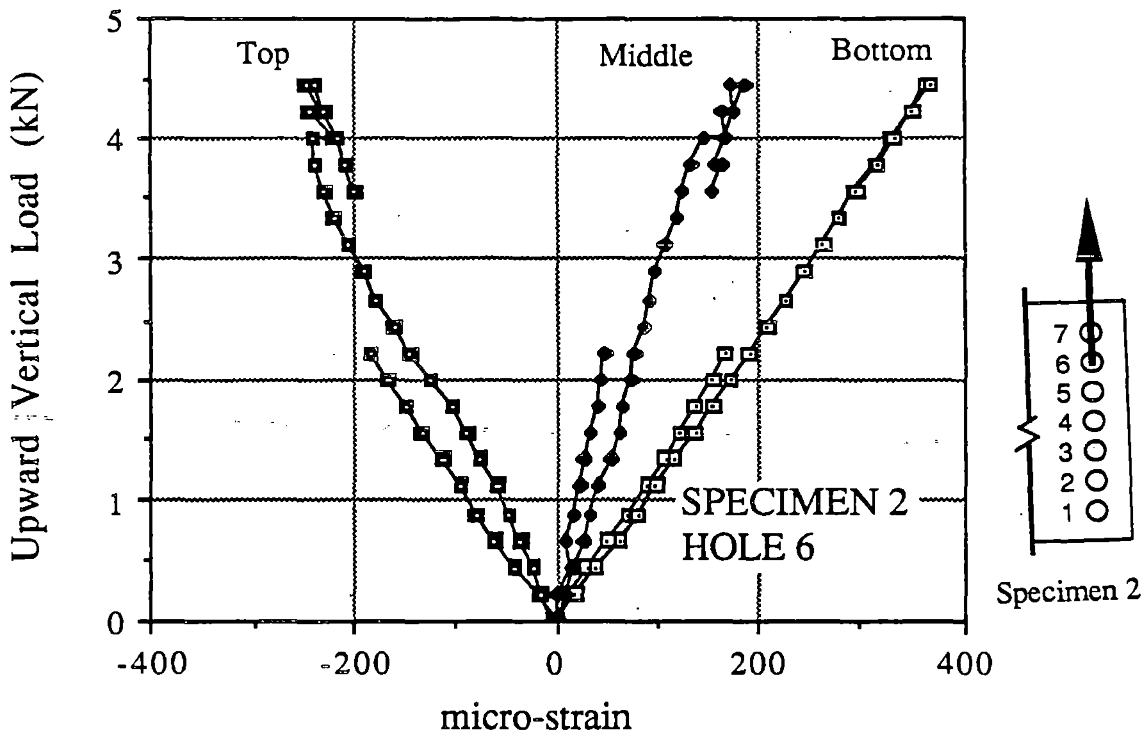


Figure B-11 Load - Strain Gage Readings for Specimen 2 with Upward Load Applied at Hole No 6

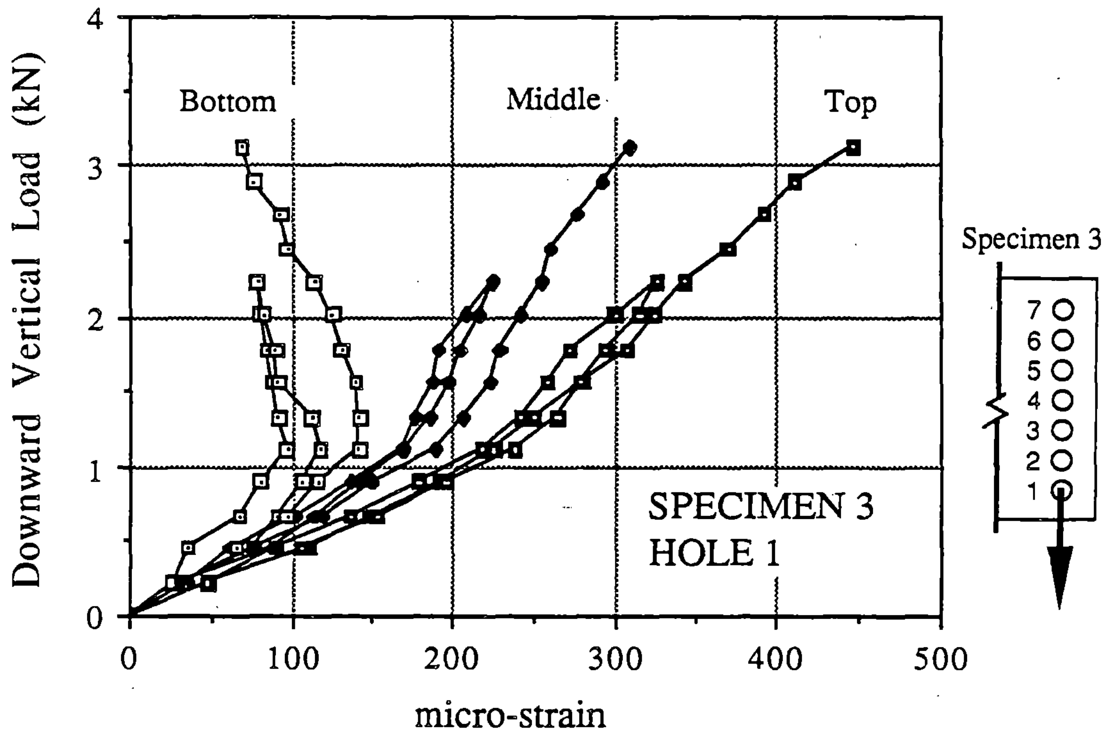


Figure B-12 Load - Strain Gage Readings for Specimen 3 with Downward Load Applied at Hole No 1

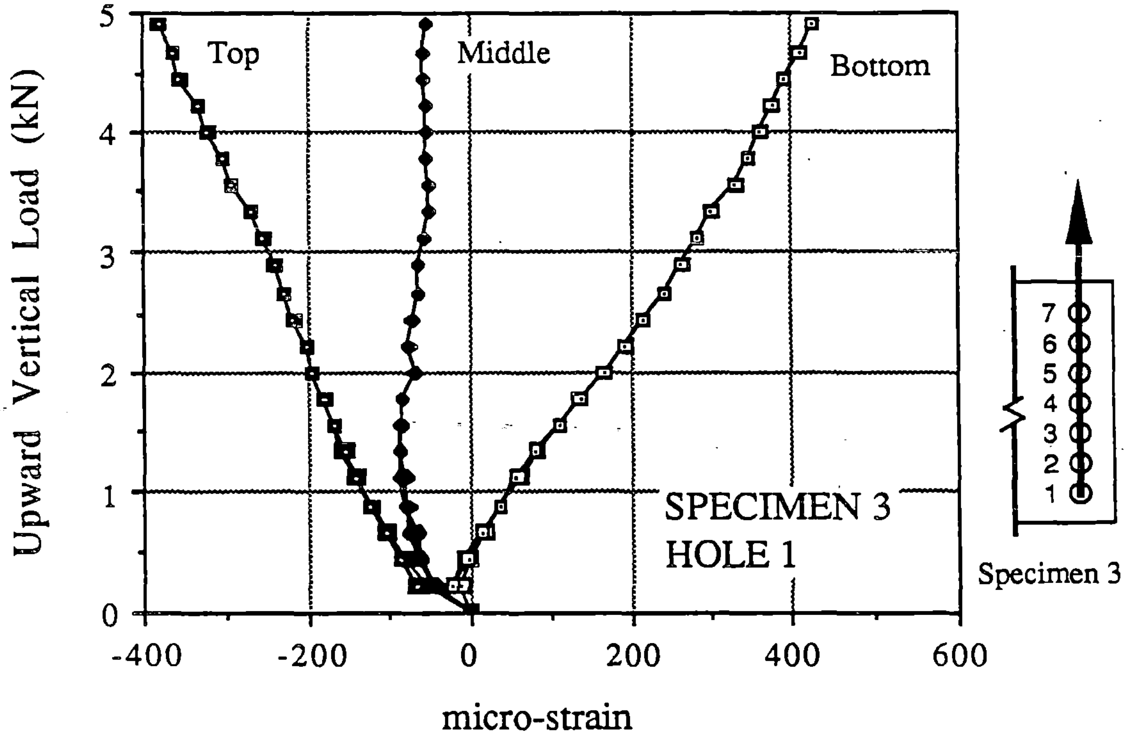


Figure B-13 Load - Strain Gage Readings for Specimen 3 with Upward Load Applied at Hole No 1

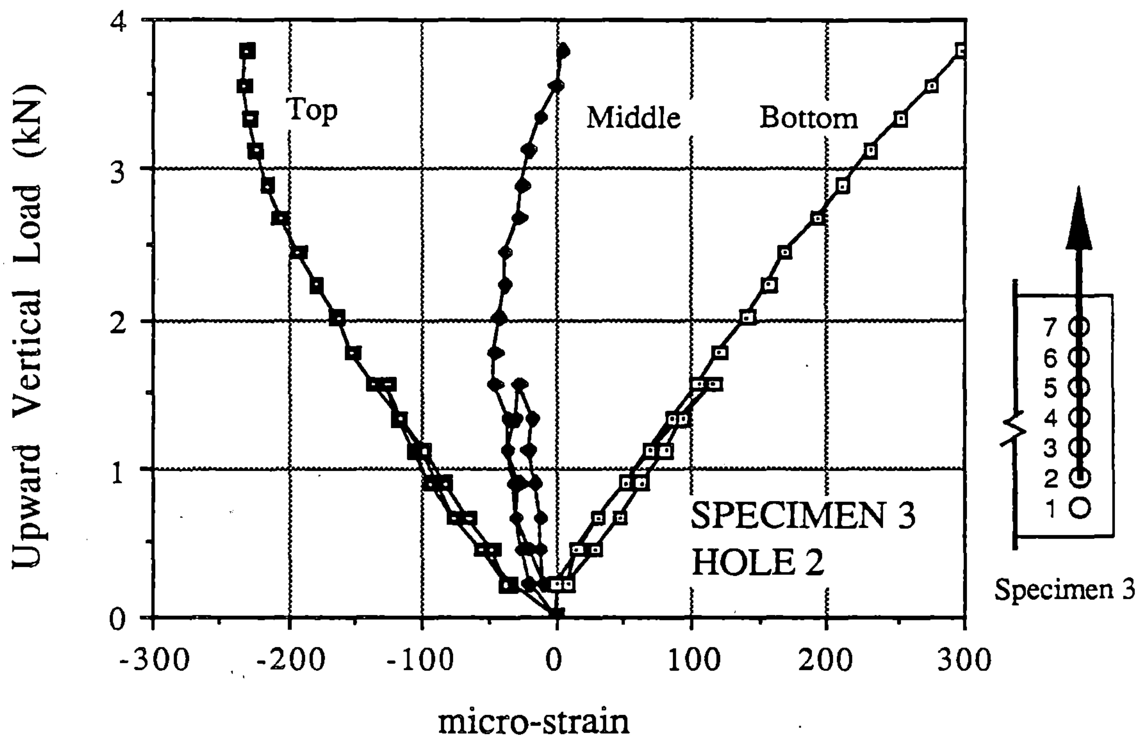


Figure B-14 Load - Strain Gage Readings for Specimen 3 with Upward Load Applied at Hole No 2

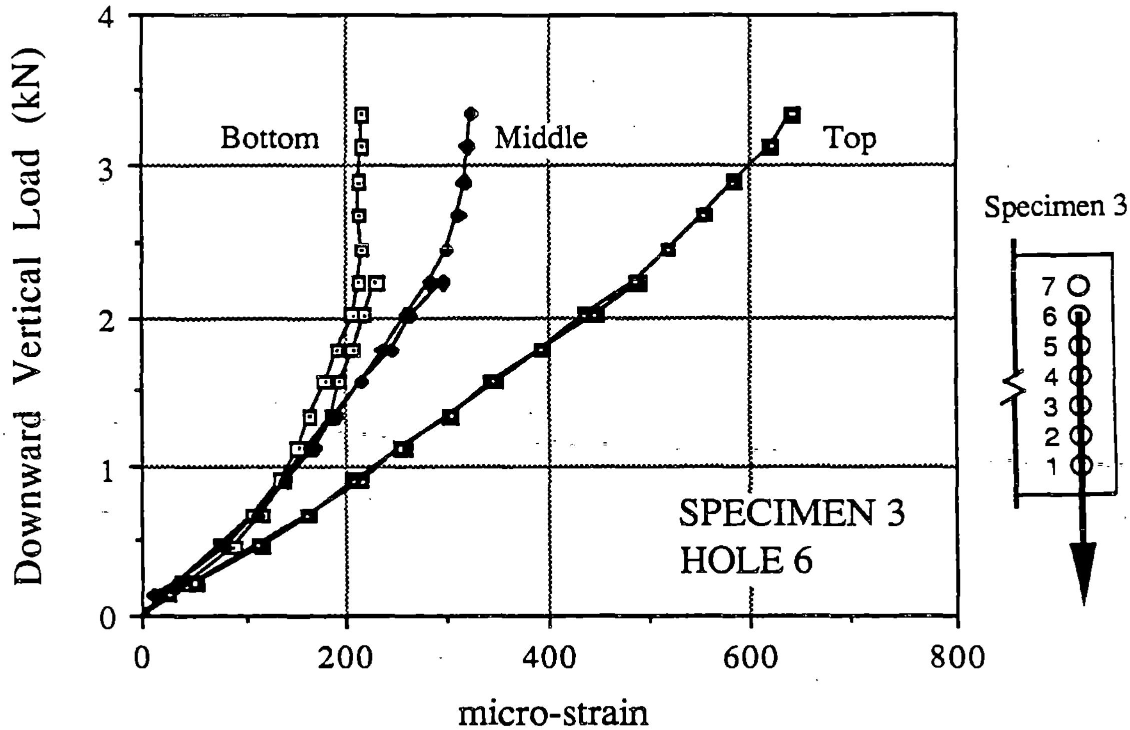


Figure B-15 Load - Strain Gage Readings for Specimen 3
with Downward Load Applied at Hole No 6

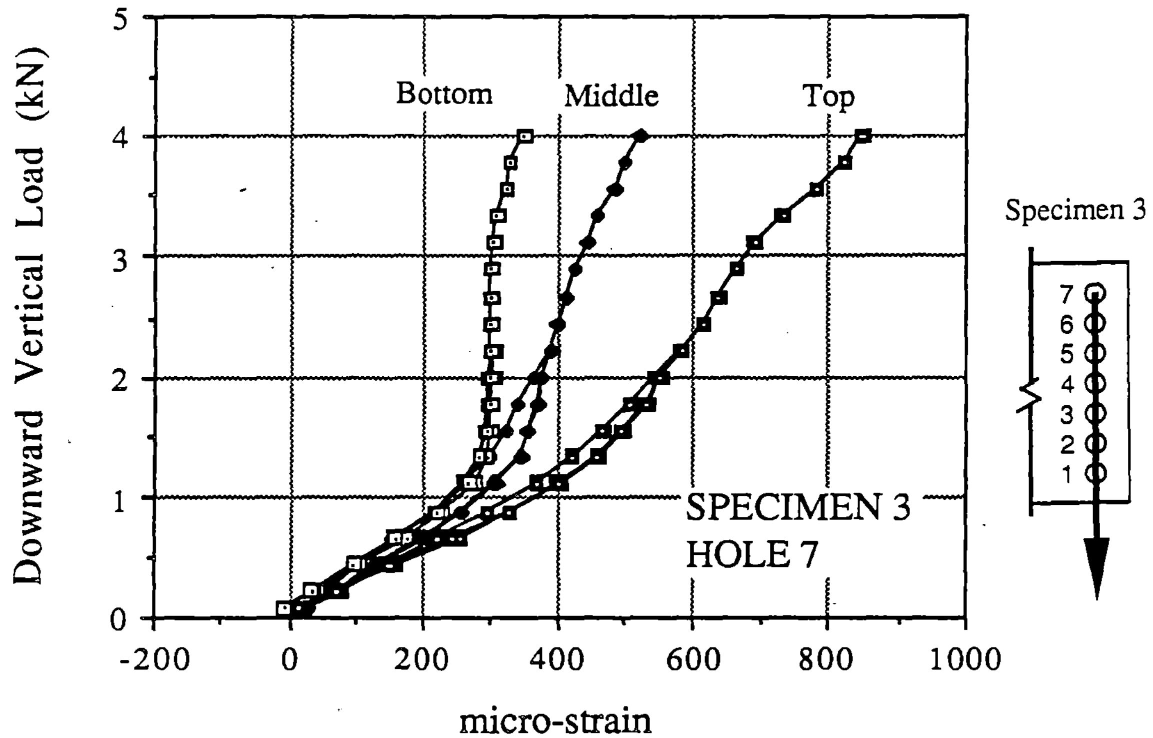


Figure B-16 Load - Strain Gage Readings for Specimen 3 with Downward Load Applied at Hole No 7

APPENTIX C - Full Scale Cavity Wall Test Results

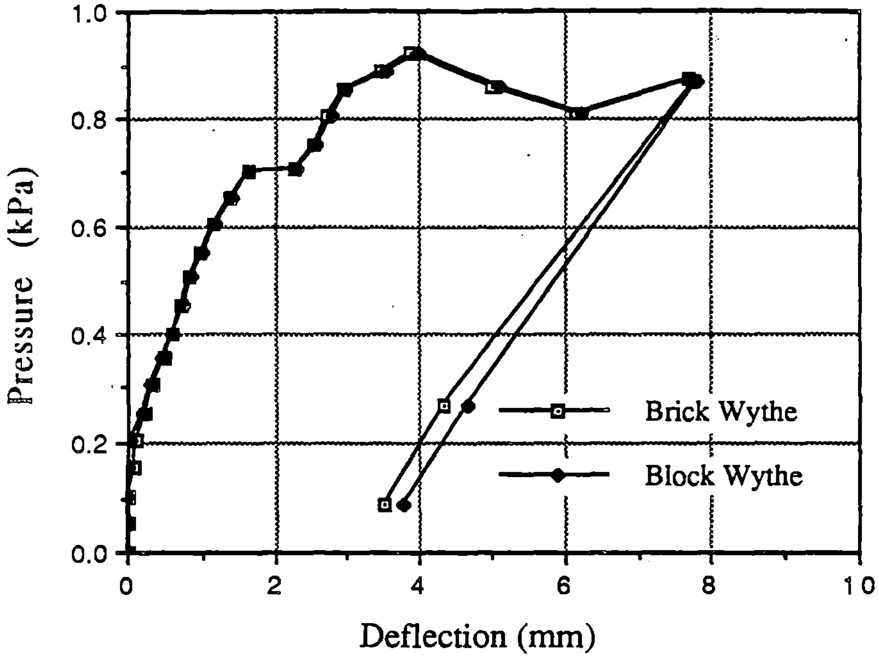


Figure C-1 Pressure - Centerline Deflection for S1W1

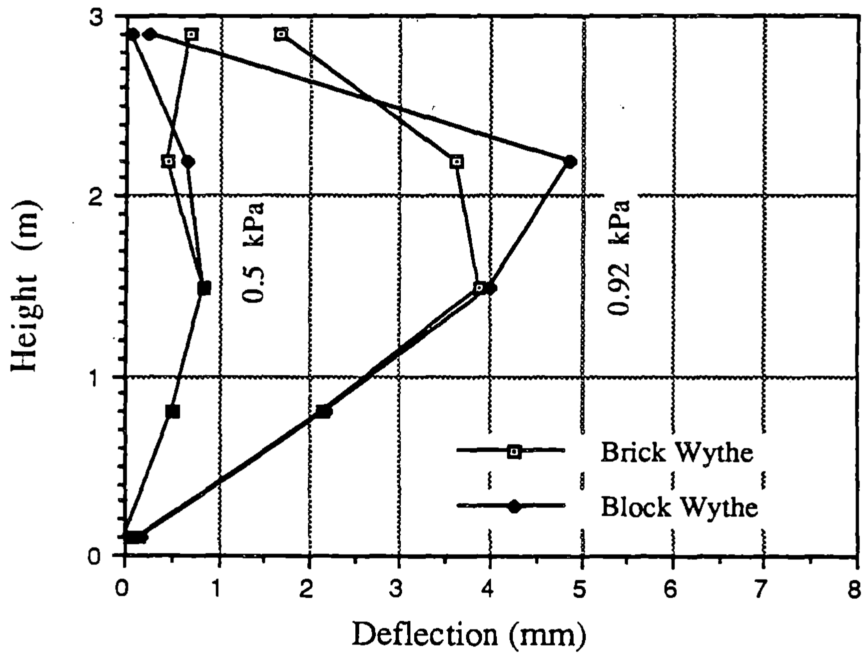


Figure C-2 Deflected Shapes for S1W1

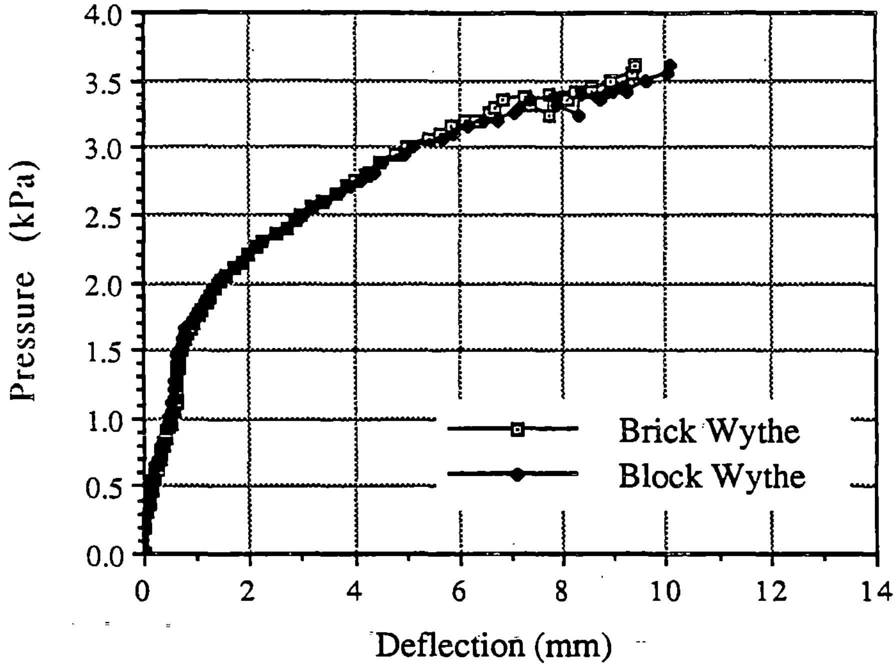


Figure C-3 Pressure - Centerline Deflections for S1W2

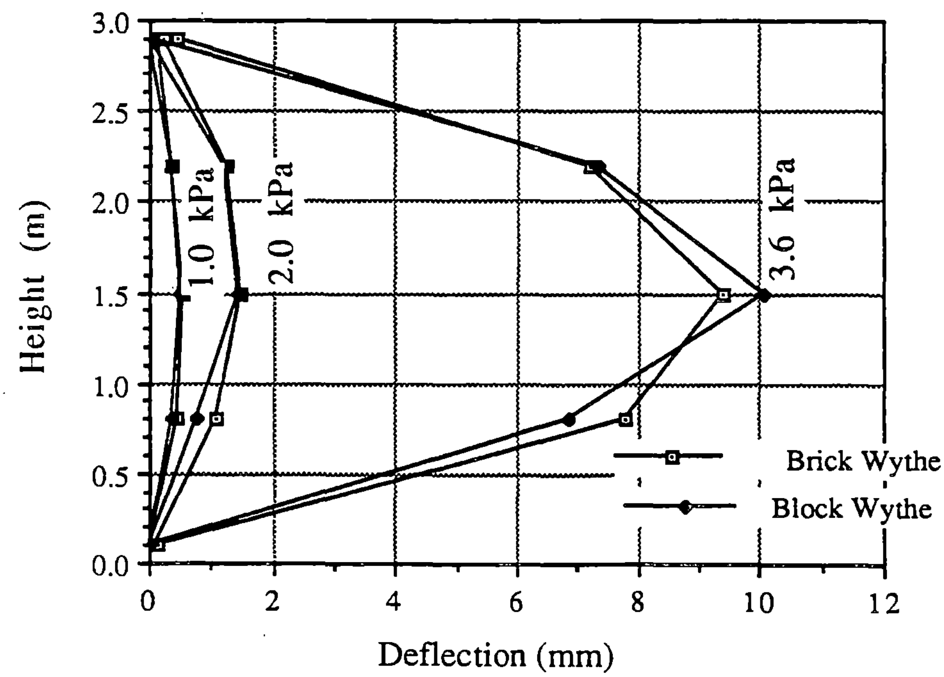


Figure C-4 Deflected Shapes for S1W2

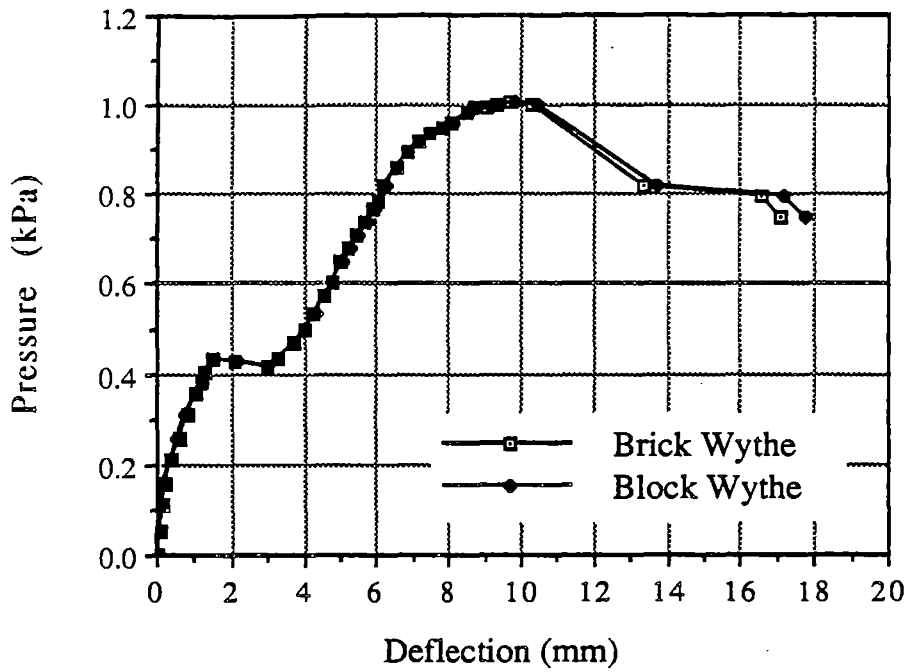


Figure C-5 Pressure - Centerline Deflection for S1W3

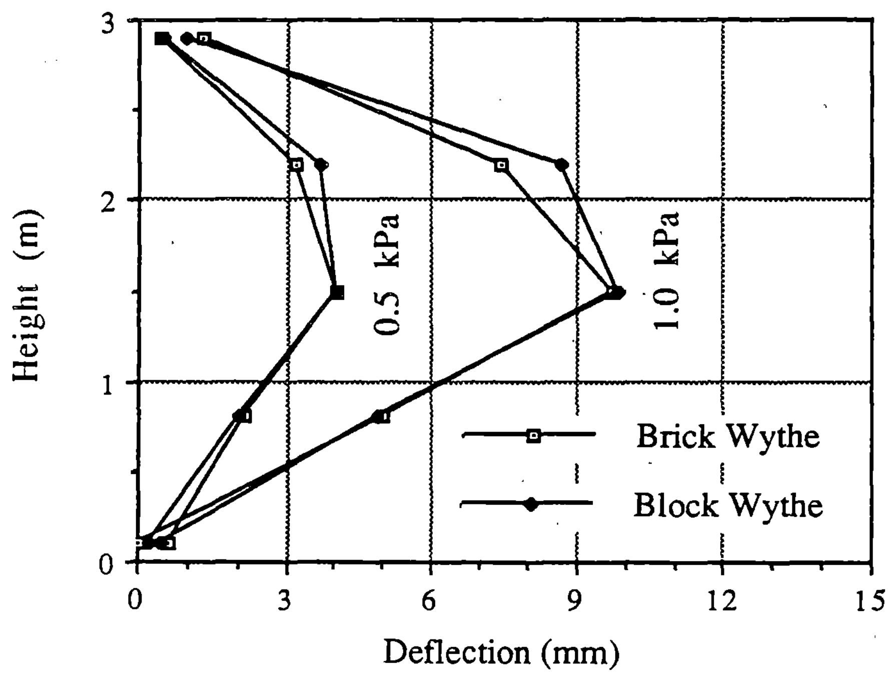


Figure C-6 Deflected Shapes for S1W3

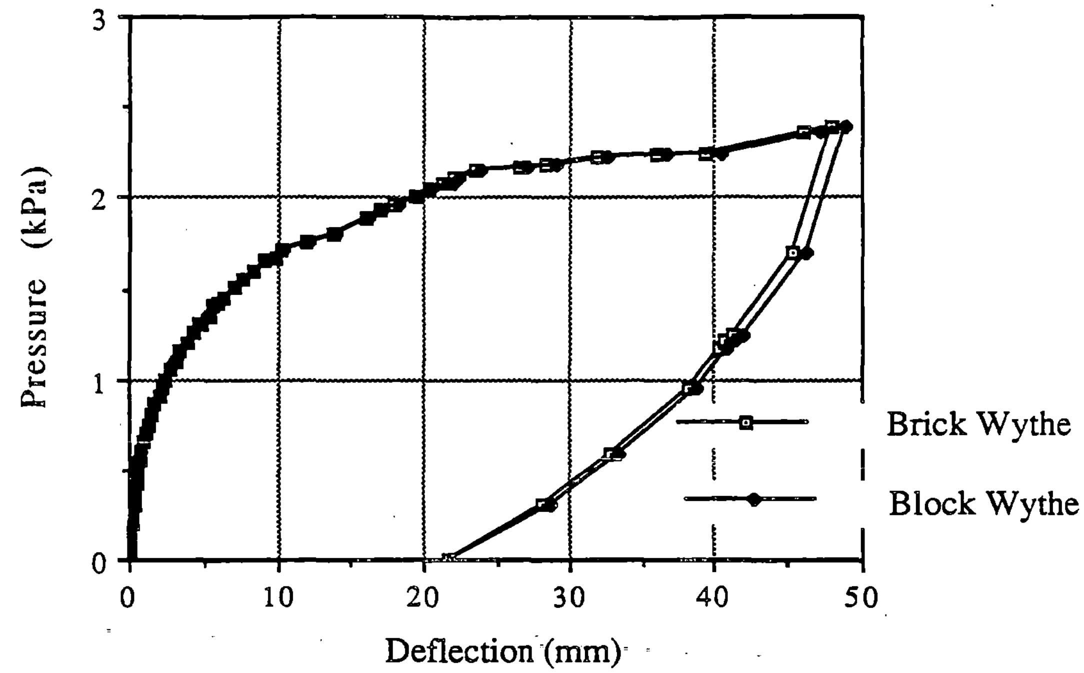


Figure C-7 Pressure - Centerline Deflection for S1W4

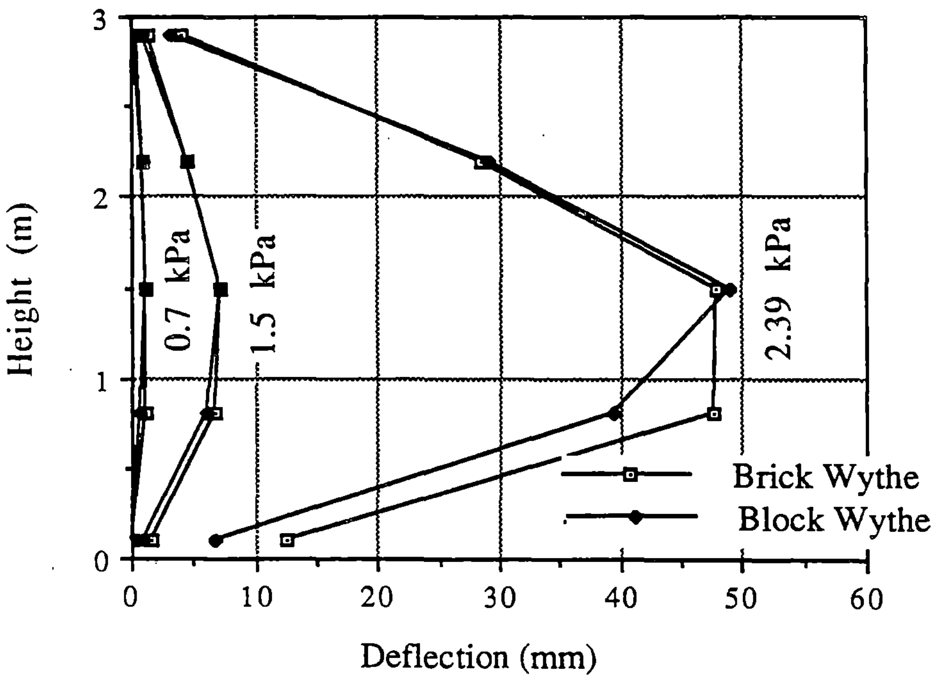


Figure C-8 Deflected Shapes for S1W4

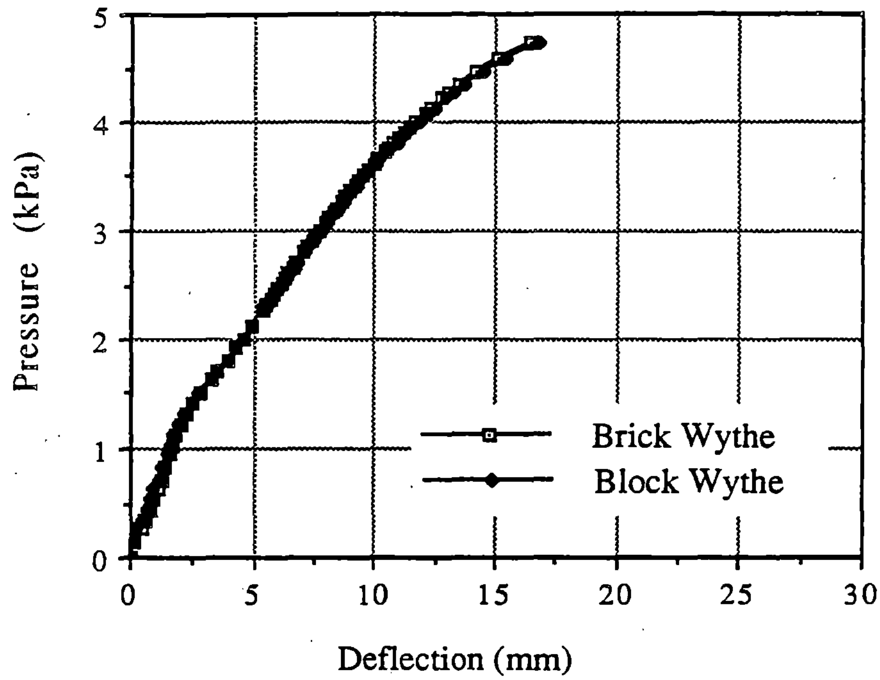


Figure C-9 Pressure - Centerline Deflection for S2W1

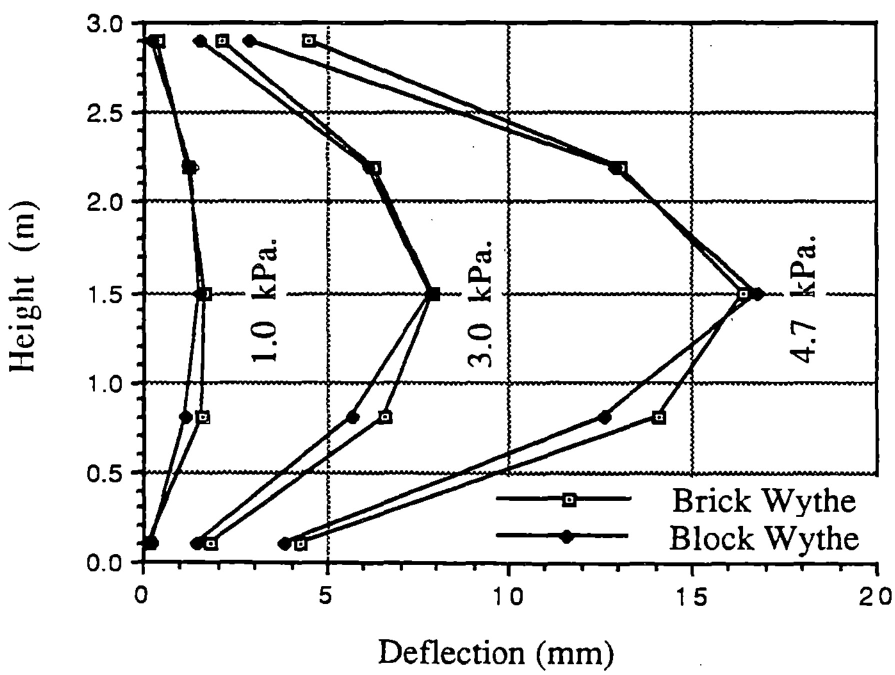


Figure C-10 Deflected Shapes for S2W1

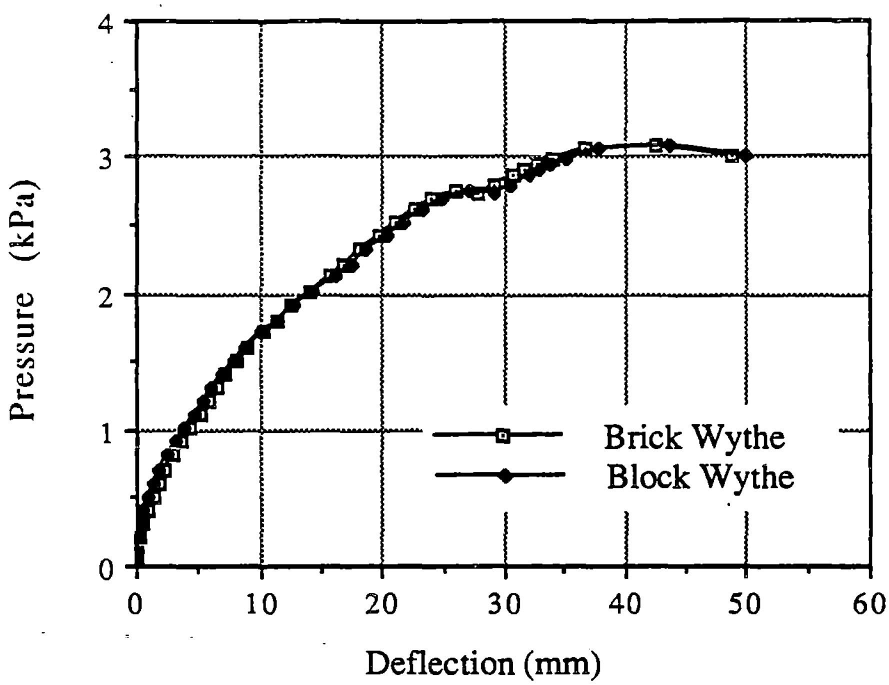


Figure C-11 Pressure - Deflection of S2W2

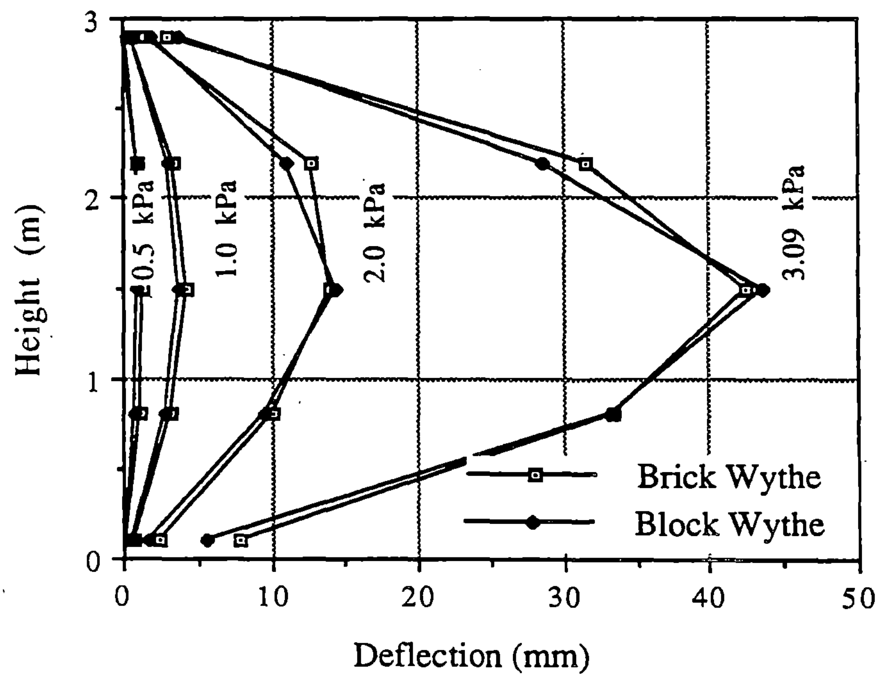


Figure C-12 Deflected Shapes for S2W2

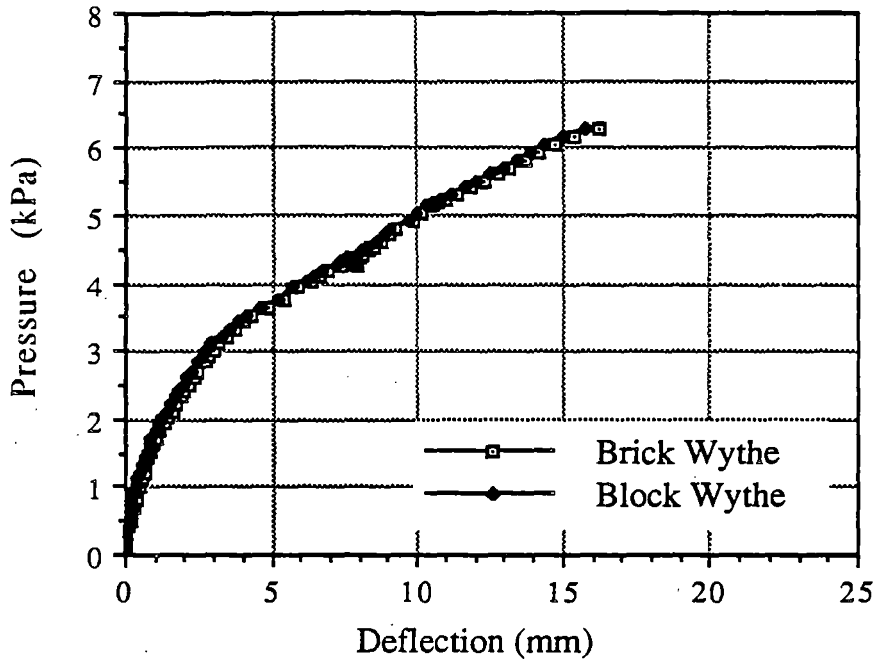


Figure C-13 Pressure - Centerline Deflection for S2W3

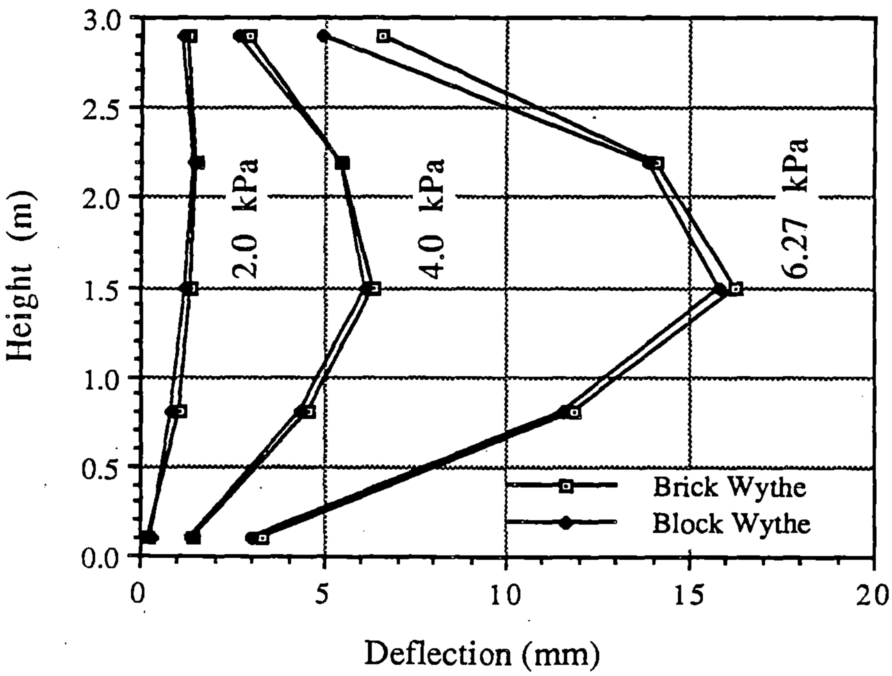


Figure C-14 Deflected Shapes for S2W3

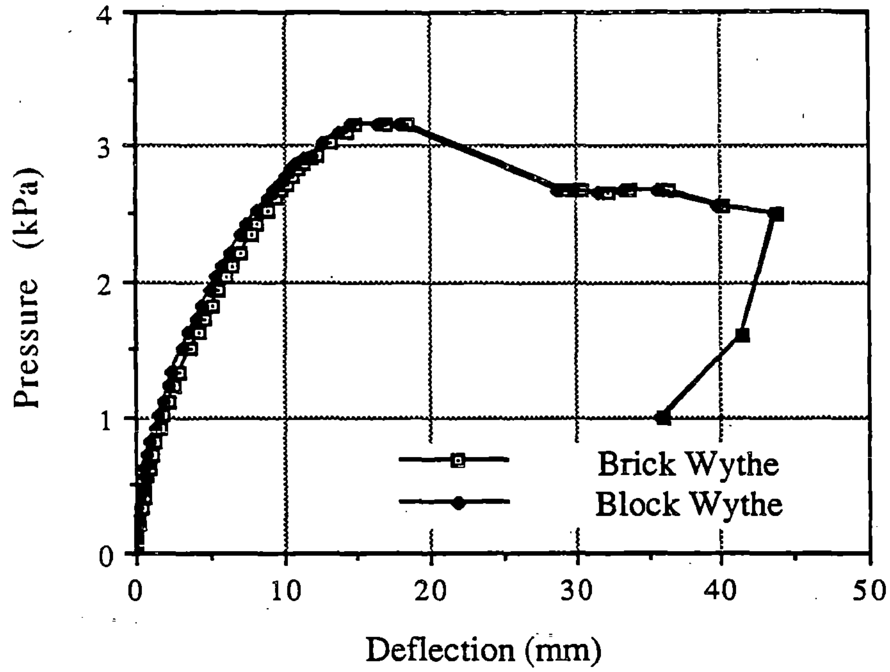


Figure C-15 Pressure - Centerline Deflection for S2W4

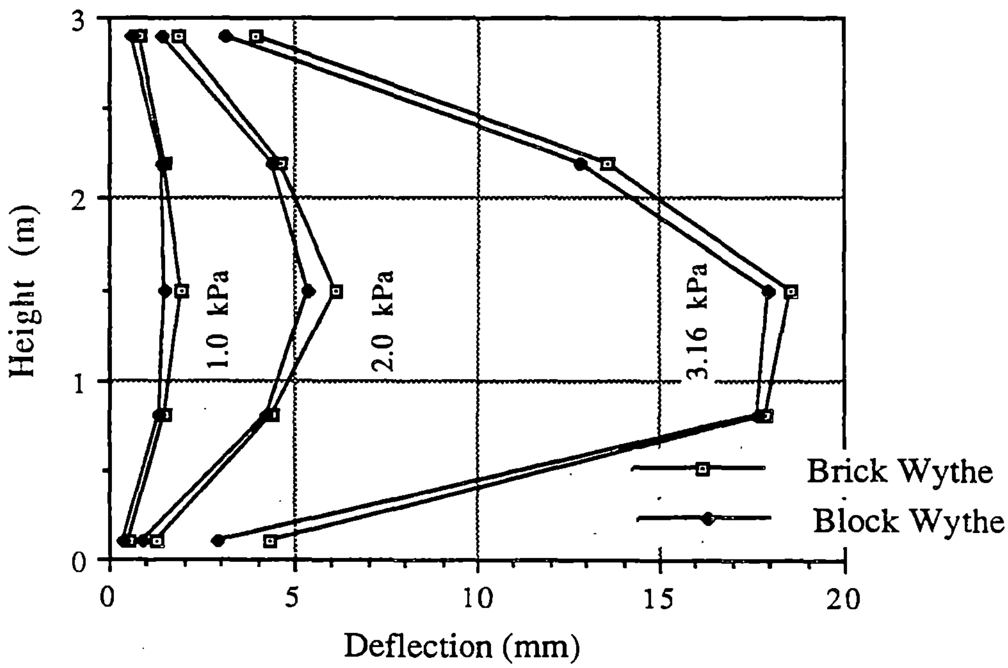


Figure C-16 Deflected Shapes for S2W4

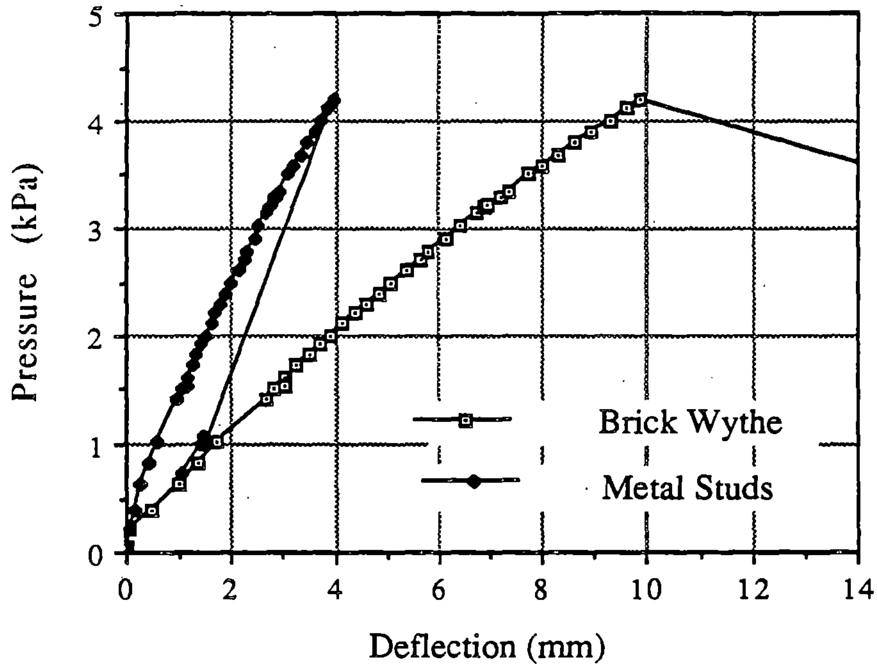


Figure C-17 Pressure - Centerline Deflection for S3W1

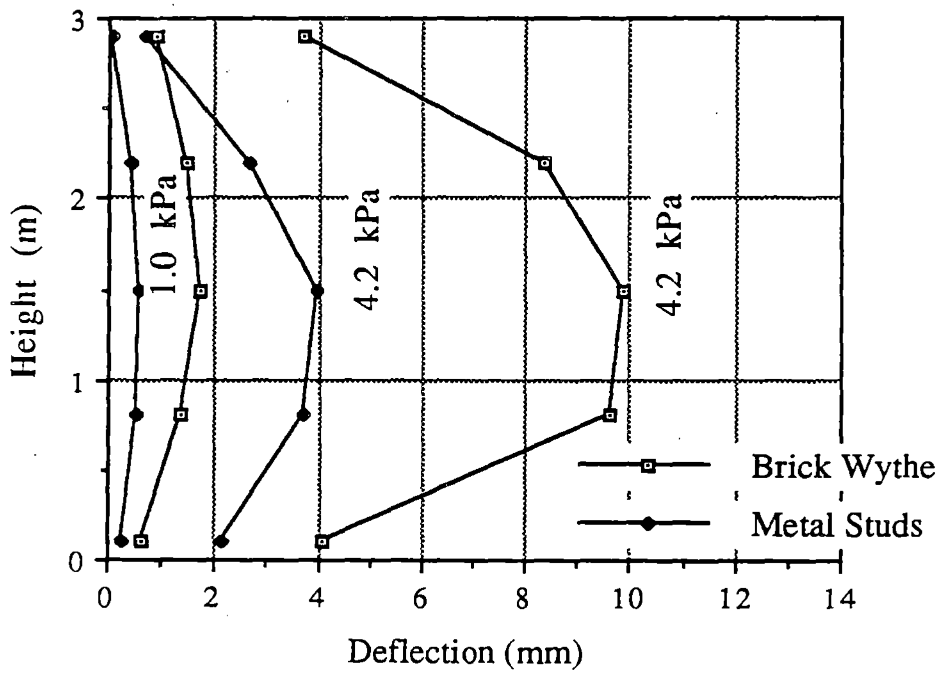


Figure C-18 Deflected Shapes for S3W1

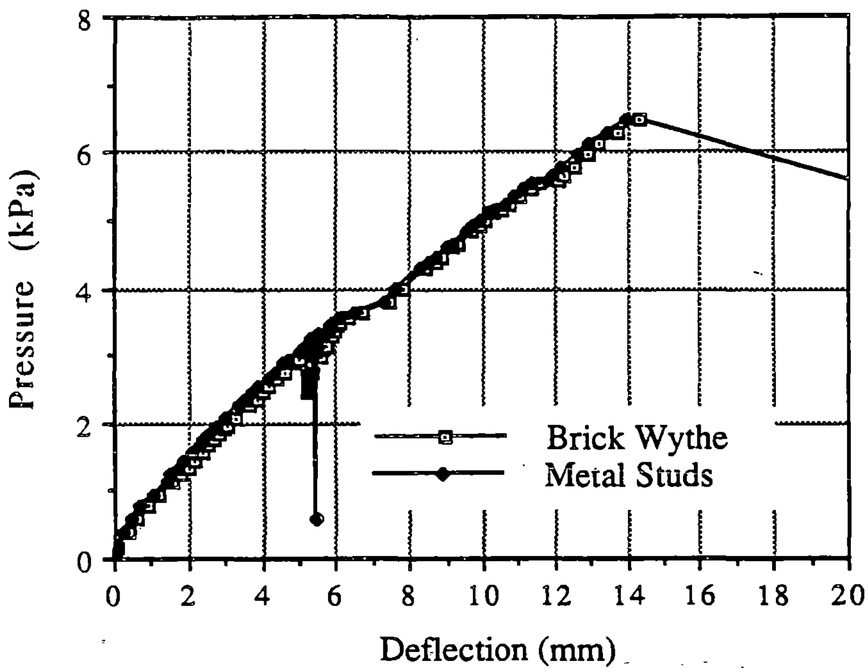


Figure C-19 Pressure - Centerline Deflection for S3W2

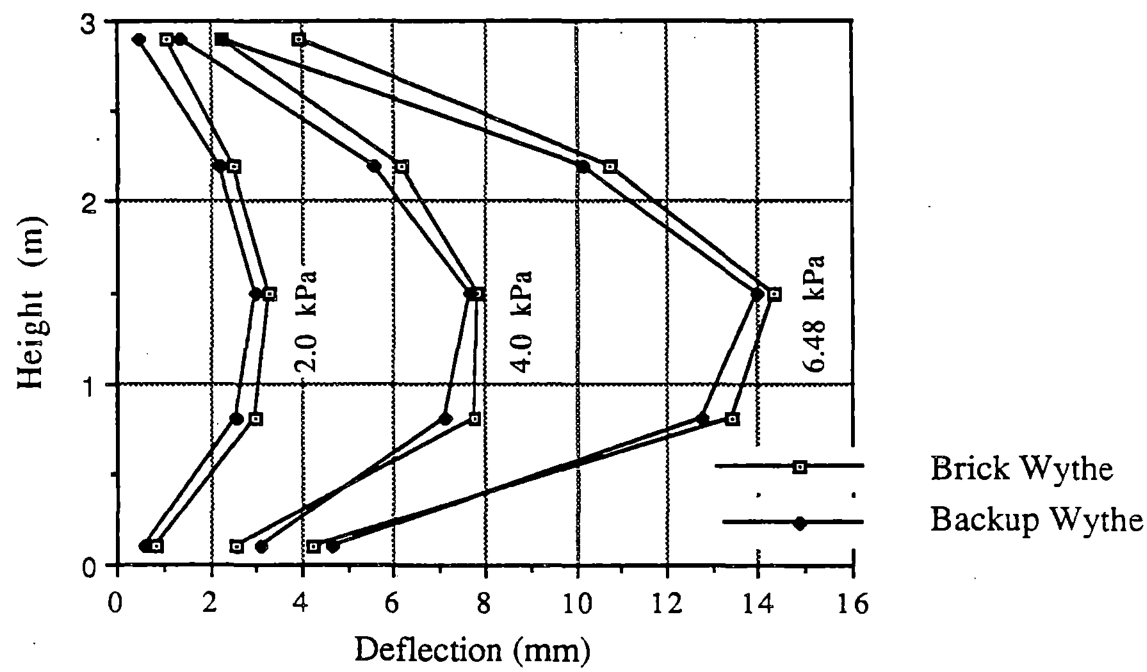


Figure C-20 Deflected Shapes for S3W2

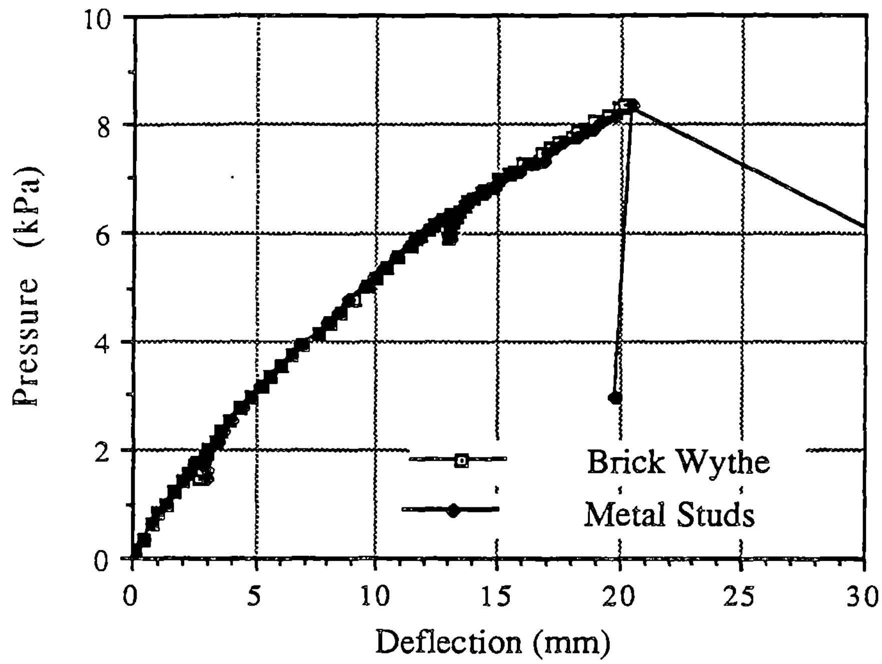


Figure C-21 Pressure - Centerline Deflection for S3W3

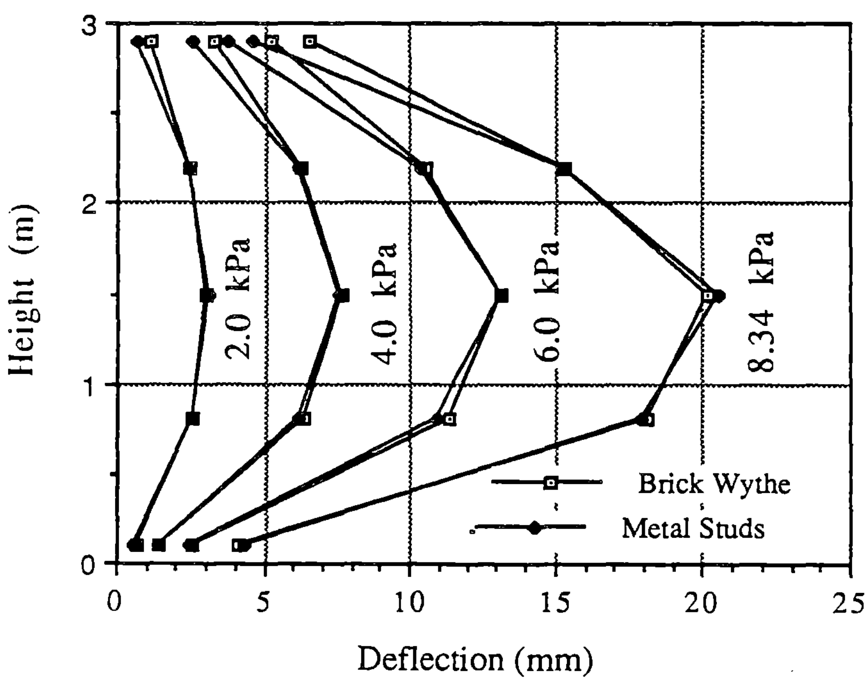


Figure C-22 Deflected Shapes for S3W3

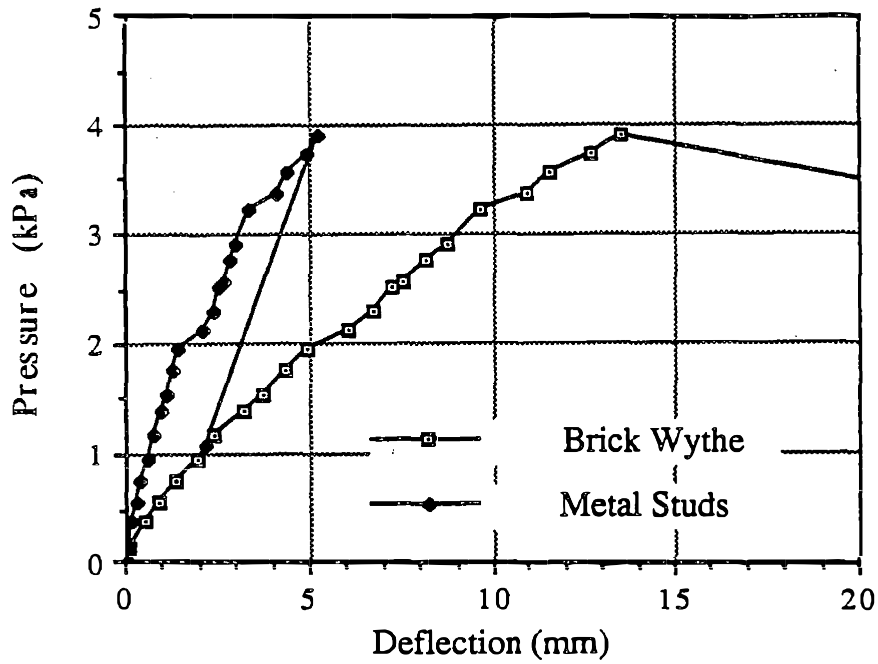


Figure C-23 Pressure - Deflection Behavior for S3W4

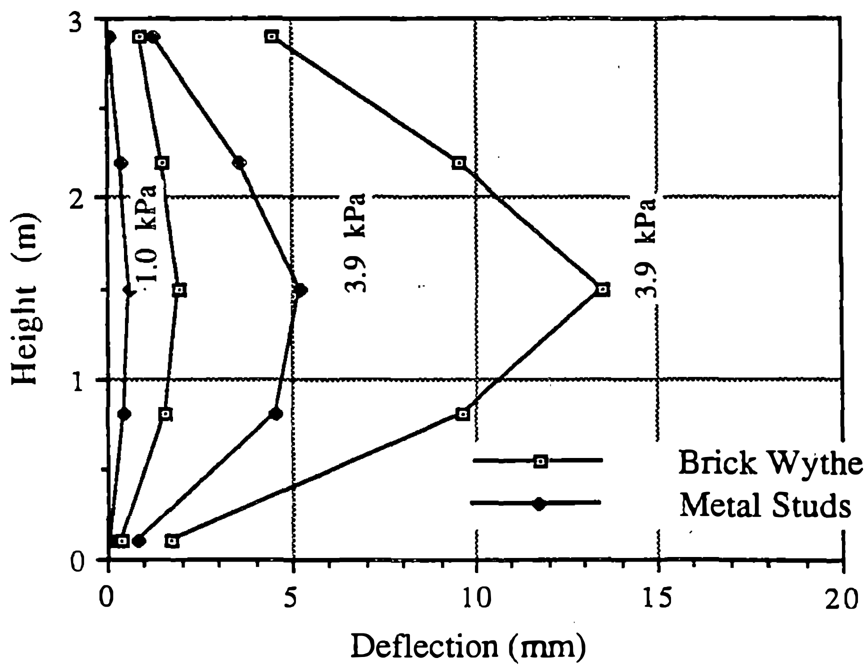


Figure C-24 Deflected Shapes for S3W4