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THE UNIVERSITY OF ALBERTA

SHEAR CONNECTORS IN MASONRY CAVITY WALLS

by

KENNETH WILLIAM PACHOLOK

A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES AND RESEARCH

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DEPARTMENT OF CIVIL ENGINEERING

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DEDICATION

For Mom and Dad.

ABSTRACT

Current design procedures for a masonry cavity wall system do not consider the exterior brick veneer to act as a structural component. A connector was developed to transfer shear between the brick veneer and the back-up wall, thus creating a composite wall system.

A theoretical analysis was undertaken and a model developed to predict the elastic range behaviour of cavity wall systems which use the shear connector.

Testing was conducted on full scale masonry cavity walls subjected to positive lateral loads. A total of eight walls were tested. Three of the walls used conventional wire truss joint reinforcement. The five remaining walls used the proposed shear connector with varied cavity thickness and concrete block back up wall thickness.

The test results demonstrated the superior performance of the walls utilizing the shear connector prototype. The use of the shear connector, as opposed to a conventional type of connector, in the wall systems resulted in increased strengths, and decreased lateral deflections at comparable loads. At the same time, significant increases in the ductility of the wall systems with 190 mm thick block back-up walls were attributed to the presence of the shear connectors.

By utilizing the shear-resisting connector, composite load carrying action is achieved between the brick veneer and the back-up wall. This action results in a superior wall system with an improved load-resisting capacity.

ACKNOWLEDGEMENTS

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TABLE OF CONTENTS

Page

Page

Page

Page

Page

LIST OF TABLES

LIST OF FIGURES

Page

Page

Page

LIST OF PLATES

Page

1. INTRODUCTION

1.1 General

A masonry cavity wall consists of an exterior brick veneer, a cavity composed of an air space and insulation, and an interior back-up wall. In most cases, the back-up wall is made of concrete masonry units, which we shall consider henceforth. The two masonry wythes are tied together by one of many different types of connectors available today. This type of wall system has become very popular due to its superior resistance to moisture penetration, fire, and sound transmission. Excellent thermal resistance is achieved when this type of wall system is insulated.

To maximize the benefits of a cavity wall system is to take full advantage of its structural, weather resistant, and aesthetic properties. However, current design codes such as CAN3-S304-M84, "Masonry Design for Buildings"[1], regard the exterior brick veneer as a weathering surface and an aesthetic skin for the building. No consideration is given to utilizing the veneer as a structural component. Lateral loads acting on the brick veneer, caused by wind or earthquake, must be transferred entirely to a back-up wall system by means of appropriately designed ties. Considering these loads, the design codes incorporate time-tested, empirically-based methods to determine the strength and size requirements of the wall system components. The concrete block back-up wall is at present designed to resist all of the applied loads. Therefore, by relying solely on the back-up wall for load resistance, the veneer is not structurally utilized.

The performance of such a system can be ch and improved greatly by forcing the walls into composite action. By providing a connector capable of resisting shear between the veneer and the concrete back-up wall, this composite action could be achieved.

This study, conducted at the University of Alberta, describes the development of such a shear resisting connector and reports the findings of a preliminary test program carried out to evaluate this system.

1.2 Objectives

The main objectives of this research are:

1. To develop a shear-resisting connector which would force composite action between the brick veneer and the block back-up wall. This would result in a wall system with an increased effective width, and therefore, an improved load resisting capacity.
2. To establish an analytical model to help predict the behaviour of a cavity wall system utilizing shear connectors.
3. To experimentally determine the effects on a cavity wall system by the use of shear connectors as compared to those of conventional-type reinforcement. These experimental results would then be compared to the predictions made by the analytical model.
4. To develop a foundation from which future shear connector research may be expanded.

1.3 SCOPE

A total of eight full size cavity wall specimens were tested at the I.F. Morrison Structures Laboratory of the University of Alberta. The walls were loaded with positive lateral pressure with no axial forces being applied to the block back-up walls. Parameters investigated included back-up wall block size, cavity width, connector type, and vertical reinforcement. Ancillary tests consisted of various prism tests to determine material properties. The experimental data includes applied loads, measured deflections, and observations of the cracking patterns of the walls. From this data, specimen behaviour is analyzed and conclusions are drawn.

1.4 PERSONNEL

The testing was carried out by graduate student Kenneth Pacholok and Chief Laboratory Technician Larry Burden, working under the direction of Dr. M. Hatzinikolas and Dr. J. Warwaruk. Test results were analyzed and the report written by Pacholok under the supervision of Hatzinikolas and Warwaruk.

2. REVIEW OF PREVIOUS INVESTIGATIONS, CONNECTOR TYPES, AND DESIGN PROCEDURE

2.1 Introduction

The following chapter provides a review of relevant previous investigations on connectors used in masonry cavity walls and compares various types of commercially available connectors. In addition, current design procedures specific to cavity wall connectors are presented and discussed.

2.2 Review of Previous Investigations

2.2.1 Conventional-Type Connectors

There has been a considerable amount of research conducted studying conventional-type connectors used in cavity wall systems. Results from these studies have served two purposes. Firstly, these tests have indirectly helped to maintain some form of regulation by recognizing connector types which may prove hazardous if used in situations not specified by the manufacturer. Secondly, the test data from such studies has resulted in recommendations for standardized design code procedures.

Increasingly intricate masonry design in recent years led to an increase in the number of cladding failures. This led the Canadian Standards Association to develop a code exclusive to the design and specification of masonry connectors. The first draft has been completed and is entitled CAN3-A370-M84, "Connectors for Masonry"[2].

Hastings(1980)[3] stated five new developments which emphasized the need for this specific connector design code:

1. The advanced level of design of masonry structures based on engineering analysis had surpassed the level of connector design.
2. Stricter requirements for wind and seismic design required new designs of connectors which could withstand increased lateral and uplift forces.
3. Novel applications of masonry were becoming commonplace. An example of this was the use of masonry veneer attatched to steel wall studs in high-rise buildings.
4. The current trend to provide greater insulation widths to reduce heat loss was resulting in increased cavity widths. This required specialized connectors which could successfully bridge such large cavities.
5. The detailing of some buildings combined with higher internal humidities resulted in increased moisture trapped within the cavity. This underlined the need for greater corrosion resistance in some wall systems.

The masonry connector code considers the following aspects of connector design:

1. Materials
2. Corrosion resistance
3. Fabrication
4. Resistance to water penetration at connectors
5. Configuration (shape and style of connectors)

6. Spacing of connectors
7. Strength requirements
8. Methods of testing

This code classifies connectors into one of two categories: standard and non-standard connectors. Standard connectors are those meeting the requirements set out in the code. Non-standard connectors, or those specifically designed for a particular project, must conform to the criteria as specified in the code, including testing under simulated loading conditions.

Other studies were conducted to complement the existing information on masonry connectors. One such investigation pertinent to this study was "Strength and Behaviour of Metal Ties in 2-Wythe Masonry Walls", by Hatzinikolas, Longworth, and Warwaruk(1980)[4]. Tests were performed on cavity walls using eight different commercially available connectors, such as the ones described in section 2.3. As well as developing an analytical approach to predicting the failure load of such connectors, this paper recommended correct connector placement and indicated poor connector designs.

Another study on connector design was conducted by Warren, Ameny, and Jessop(1983)[5]. This paper was a comprehensive collection of previous research. It also addressed many of the common problems encountered during design.

2.2.2 Shear-Resisting Connectors

The concept of utilizing a shear-resisting connector to improve the quality of masonry cavity walls is not new. However, there has been little success to date as a result of complications pertaining to moisture resistance and differential wythe movement.

A study has been conducted by Mullins and O'Connor of the University of Queensland, Brisbane, Australia. A research paper entitled "The Use of Steel Reinforcement Systems To Improve the Strength and Stiffness of Laterally Loaded Cavity Brick Walls(1987)"[6], was prepared by the above noted group to report preliminary results of their test program. introduced a connector prototype which they called a shear con . As seen in Figure 2.1, it consists of a length of sheet metal which is placed continuously within the height of the cavity, perpendicular to the two wythes. The wythes are connected to this cavity portion by tabs extending into the head joints at every other course. Standard wire ties were used in conjunction with this shear connector. The cavity wall system they tested differs from the system used in this program. Theirs consisted of two identical brick wythes separated by a cavity. The experimental program included numerous prism tests to determine material properties, as well as three full scale wall panel tests. Each of the full scale wall tests used a different type of connector. The panel tests compared wall systems with one of the following connector types: (1) standard wire ties, (2) an unbonded intermediate steel mullion, and (3) the vertical shear connector prototype. The testing showed that the use of the shear-resisting connector resulted in a significant increase in both the pre- and post-cracking strength and the out-of-plane stiffness.

These results were encouraging for the Australians. However, this design is not practical from a construction standpoint. In order for this design to be effective, the tabs must be secured within the head joint of the two wythes. Therefore, both wythes must be identical in both unit size and course height. However, construction procedures render this very impractical. In Canada, often the veneer consists of a different size of unit with respect to the back-up system. Even if both wythes are made of the same unit size, it is very difficult for the mason to construct the bed joints at identical heights. As a result, a new connector design must be developed to overcome both the dimensional and thermal-related problems associated with cavity wall systems.

2.3 Commercially Available Connector Types

Many different types of connectors and variations thereof, are commercially available. Several manufacturers produce their own line of masonry accessories. As a result, there exists numerous varieties of each particular connector type. The majority of these varieties differ only by small details. These details are incorporated to improve over similar designs and/or claim exclusivity over a particular connector patent. However, with these differences aside, they all perform the same basic function. That is, they transfer lateral load acting on the exterior veneer to the back-up wall. For the sake of simplicity and brevity, only commonly used general designs will be presented.

Typical connector types can be classified into one of the following categories:

(i) continuous welded connectors
- non-adjustable
- adjustable

(ii) individually placed connectors
- non-adjustable
- adjustable

(iii) specially designed connectors

Specially designed connectors are those custom manufactured for a specific job. These must be designed utilizing engineering principles. Due to the relatively rare use of such connectors, they shall not be considered in this report.

Commercially available connectors come in a variety of protective finishes, as well as sizes to accommodate the design specifications outlined in reference 2.

2.3.1 Continuous Welded Connectors

Continuous welded connectors consist of two or more parallel longitudinal steel rods that are embedded in the mortar bed joints between courses. Cross rods are weld-connected to the longitudinal rods resulting in a particular pattern; thereby connecting the two wythes in the horizontal plane. It is recommended by reference 4 that the three rod tie of Figure 2.2(i) be positioned such that the longitudinal rods be placed at centreline in the face shells of the block walls and at centreline of the brick bed joint for maximum wall system strength. This type of connector is also commonly referred to as joint reinforcement.

This connector type provides one or more of the following functions. It transfers the load from the veneer to the back-up wall. It helps control cracking due to shrinkage and differential movement. It also provides some reinforcement for structural beam action. The latter two roles may work in conjunction or separately with the tying action. We shall isolate and ignore the latter two reasons for the purpose of this study.

2.3.1.1 Non-Adjustable Continuous Welded Connectors

These connectors generally exist in one of three patterns:

(i) ladder

(ii) truss

(iii) ladder/truss with rectangular ties

2.3.1.1.1 Non-Adjustable Ladder Pattern

With the ladder pattern, the cross rods are welded perpendicular to the longitudinal rods as in Figure 2.2. Depending on the design, there may be a longitudinal rod sitting on one or both of the face shells of the block wall; and one or more longitudinal rods placed in the bed joint of the brick veneer. The information presented by reference 4 suggested that type 2.2(i) be chosen over types 2.2(ii) or 2.2(iii). This recommendation is based on positive and negative lateral load tests conducted on masonry prisms. Connector 2.2(ii) does not benefit from the strength gain that would be supplied by a longitudinal rod on the outer face shell of the block wall. Connector 2.2(iii) has its longitudinal rods placed on the brick such that any degree of lateral tension or compression supplied to the wall would eventually result in the mortar spalling off in the bed joint of the brick wall. This spalling would render the connector useless and expose the brick wall to the elements.

2.3.1.1.2 Non-Adjustable Truss Pattern

The truss pattern is similar to the ladder pattern with the exception that the cross rods are welded on angles to the longitudinal rods, thereby forming a truss shape. A truss pattern connector is shown in Figure 2.3. As with the ladder pattern, truss pattern 2.3(i) is recommended over 2.3(ii) or 2.3(iii).

2.3.1.1.3 Non-Adjustable Ladder/Truss Pattern With Rectangular Ties

As the name suggests, the pattern is similar to either the ladder-type(see Figure 2.4(i)) or the truss-type(see Figure 2.4(ii)) connectors previously described. The difference is that the patterns as described in 2.3.1.1.1 and 2.3.1.1.2 exist only within the concrete block, with rectangular box-shape ties welded to the block longitudinal rods. These box-shape ties extend across the cavity and sit in the bed joint of the brick veneer, thereby connecting the two wythes.

2.3.1.2 Adjustable Continuous Welded Connectors With Rectangular Ties

These connectors are identical to their non-adjustable counterparts with the exception of the allowable movement of the rectangular ties with respect to the rest of the connector. The manufacturers recommend these connectors to be used where there may be misalignment of mortar joints in the outer and inner wythes. This misalignment may be due to factors such as construction procedures and sub-trade responsibilities. Another benefit of some adjustable ties is that they help hold rigid insulation in place in a cavity.

Examples of this type of connector can be seen in Figures 2.5 and 2.6. These connectors do not provide any resistance to movement in the vertical direction, thereby accommodating differential movements of the two wythes On the other hand, they do not resist shear and may tend to fail if excessively large vertical movements are allowed to take place. The connector in Figure 2.5(i) should not be used in cavity wall construction because it does not resist lateral movement of the outer wythe with respect to the inner wythe.

2.3.2 Individually Placed Connectors

As the name implies, these connectors are placed individually at spacings specified in reference 1.

2.3.2.1 Non-Adjustable Individually Placed Connectors

As with the non-adjustable continuous connectors of 2.3.1.1, these are used where the courses of the two wythes are expected to line up. There are three basic shapes of connector:

(i) corrugated strip

(ii) z-shape rod

(iii) rectangular(box)

As seen in Figures 2.7(i) to 2.7(iii), these connectors are very simplistic in design. They sit horizontally within the bed joint of the two wythes. The connector extends from the midpoint of the brick veneer, across the cavity, to the midpoint of the furthest face shell of the back-up wall.

2.3.2.1.1 Corrugated Strip Ties

The corrugated strips, due to their weak structural strength, are not recommended for use in walls exceeding one storey in height, walls comprised of hollow masonry units, or wall systems with cavity widths exceeding 25 mm.

2.3.2.1.2 Z-Shape and Rectangular Connectors

Although both the z-shape and the rectangular conn
quite similar, the latter is recommended by reference 2 for
hollow masonry units.

2.3.2.2 Adjustable Individually Placed Connectors

These connectors are incorporated in cases where the two wythes are to be tied together, but the coursing does not line up. There are basically four types:

(i) corrugated strip with twisted z-type

(ii) adjustable z-type

(iii) rectangular with rectangular rod

(iv) plate type with rod

2.3.2.2.1 Corrugated Strip With Twisted Z-Type

This type of connector as shown in Figure 2.8(i) is commonly used in small residential and commercial buildings. These also are not recommended due to the weak structural strength of the corrugated portion when subjected to positive lateral loading.

2.3.2.2.2 Adjustable Z-Type

Similar to the unadjustable z-type, this connector can allow for vertical movement and discrepancies in the coursing of the two wythes. These are quite unstable and should only be used with solid masonry units. Refer to Figure 2.8(ii).

2.3.2.2.3 Rectangular With Rectangular Rod

As it can be seen from Figure 2.8(iii), this connector differs from the tie shown in Figure 2.7(iii) by the incorporation of an adjustable joint at midpoint in the connector. It can be used with hollow masonry units to transfer loads to the back-up wall.

2.3.2.2.4 Plate Type With Triangular-Shaped Rod

This type of connector is shown in Figure 2.9. It is designed to be used with a steel stud back-up system. The deformed plate is connected to the back-up studs, extends across the cavity, where a triangular-shaped rod is inserted in the slot of the plate and embedded within the bed joint of the brick veneer. As a result of the long vertical slot in the deformed plate, no resistance to vertical differential movement of the two wythes is provided.

2.4 Current Design Procedure

2.4.1 Standard Connectors

Standard connectors, such as the ones shown in Figures 2.2 through 2.8, are recommended to meet the criteria listed in reference 2. For each type of connector, a set of guidelines are prescribed. For example, for the standard connector used in this study as shown in Figure 2.3(i), the following recommendations are stated in Clause 9.2.5. The code sets limitations on configuration, fabrication, strength requirements, corrosion resistance, construction procedure, and connector spacing.

2.4.2 Non-Standard Connectors

Non-standard connectors are considered to be those not conforming to the requirements of Clause 9 of reference 2. They can be either completely different types of connectors such as the proposed shear connector of this study, or modified versions of standard connectors. Testing of all non-standard connectors is recommended by reference 2, with test methods described in Clause 11. These connectors may be used providing they meet the standard connector requirements for strength or corrosion recommended by both references 1 and 2. However, if strength resistance is found to be stronger or weaker with respect to their standard counterparts, the connector spacing may be decreased or increased, respectively.

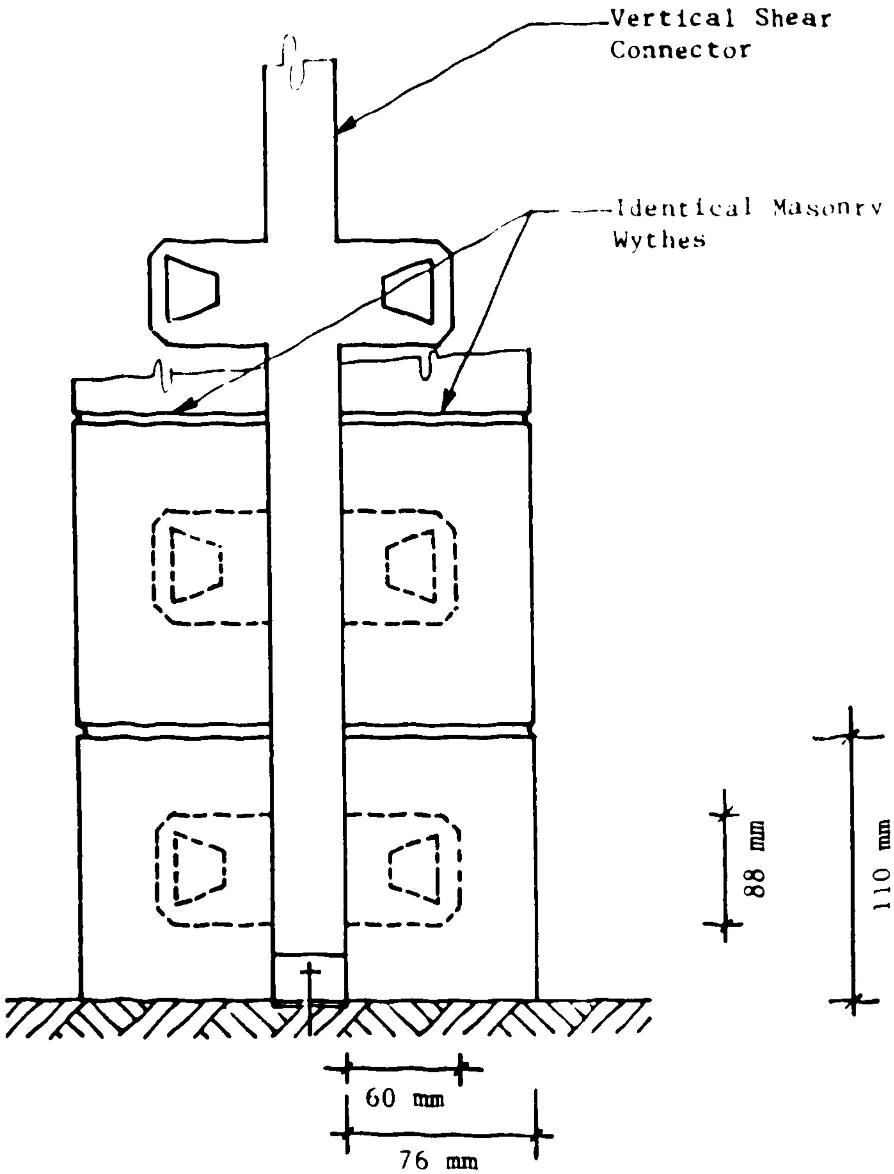
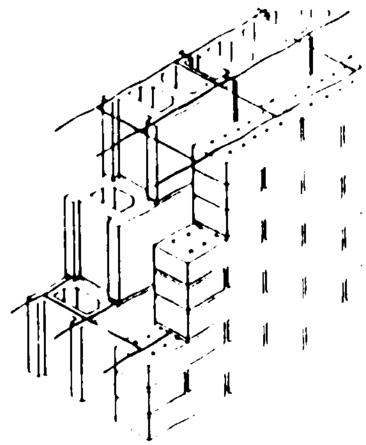
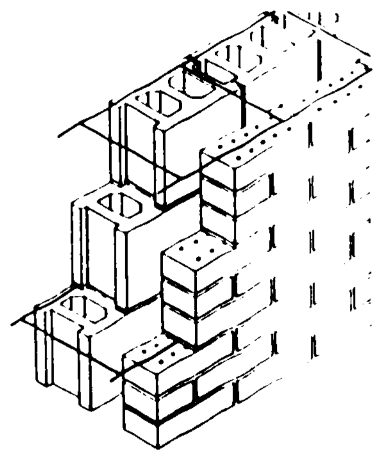


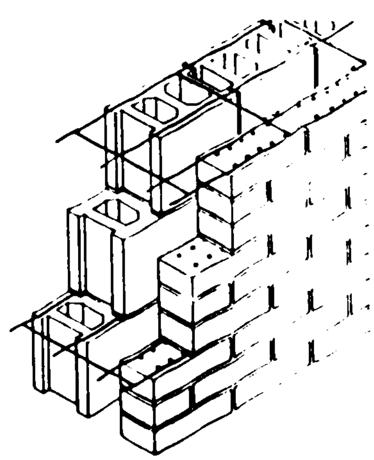
Figure 2.1 Vertical Shear Connector



(i) 3-rod variation

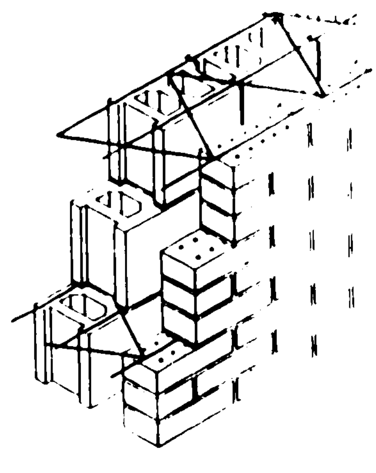


(ii) 2-rod variation

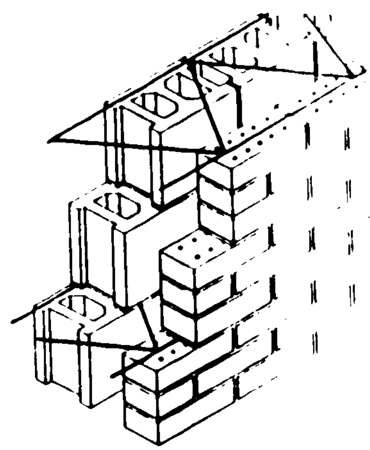


(iii) 4-rod variation

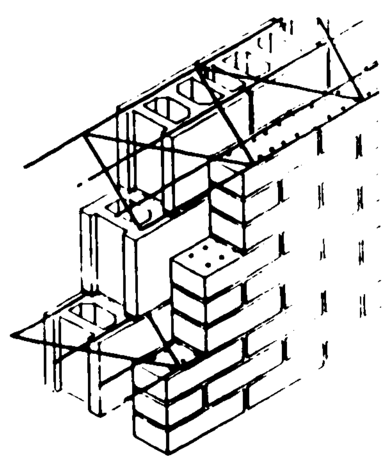
Figure 2.2 Non-Adjustable Ladder Pattern Connectors



(i) 3-rod variation

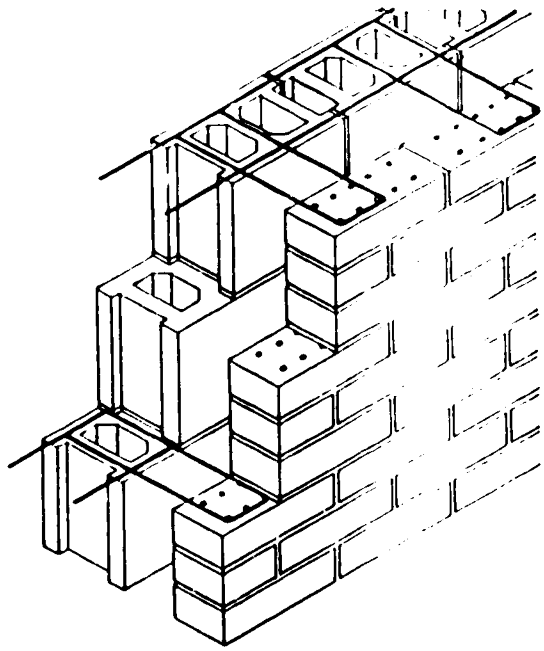


(ii) 2-rod variation

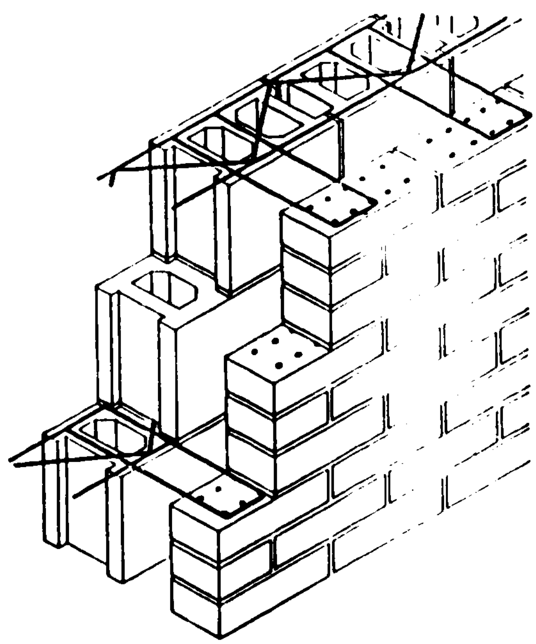


(iii) 4-rod variation

Figure 2.3 Non-Adjustable Truss Pattern Connectors

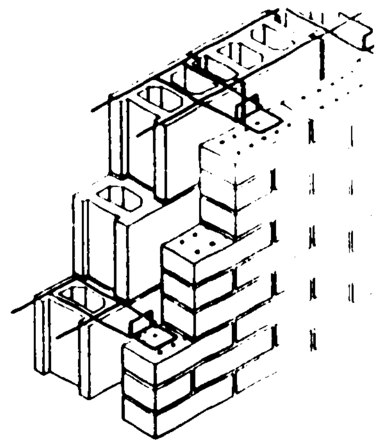


(i) ladder-type variation

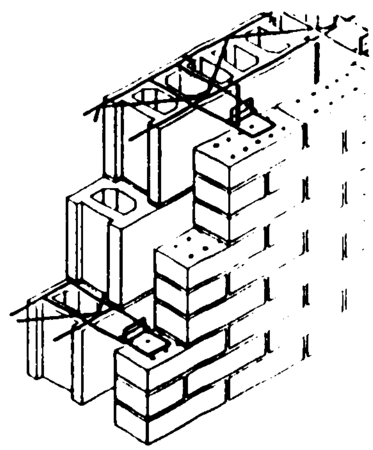


(ii) truss-type variation

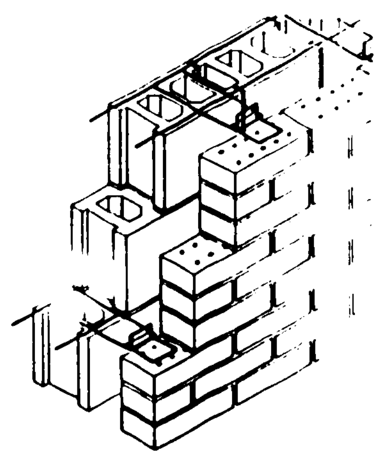
Figure 2.4 Non-Adjustable Ladder/Truss Pattern With Rectangular Ties



(i) ladder-type variation A

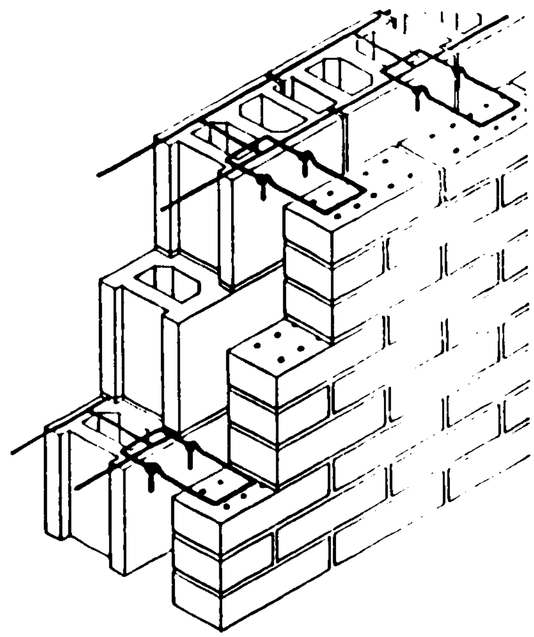


(ii) ladder-type variation B

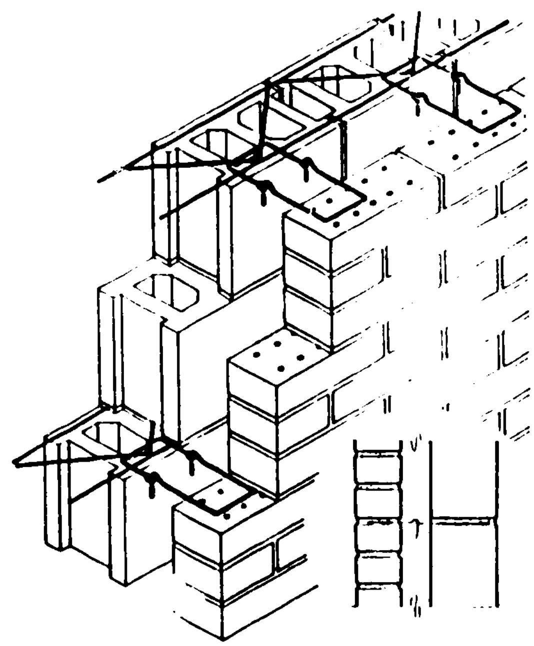


(iii) truss-type variation

Figure 2.5 Adjustable Ladder/Truss Pattern With Rectangular Ties

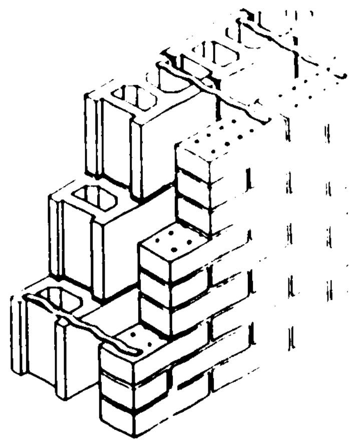


(i) ladder-type variation

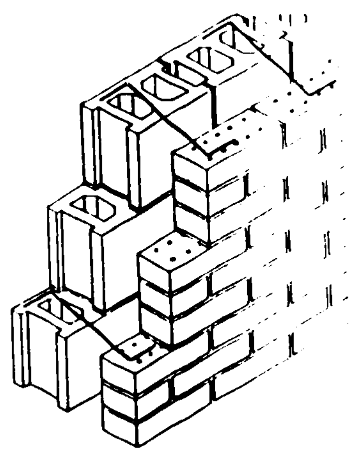


(ii) truss-type variation with superimposed cross-sectional view

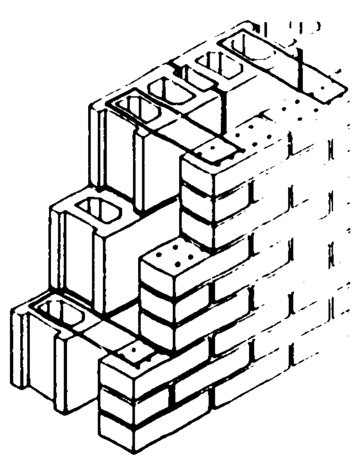
Figure 2.6 Adjustable Ladder/Truss Pattern With Rectangular Ties



(i) corrugated strip

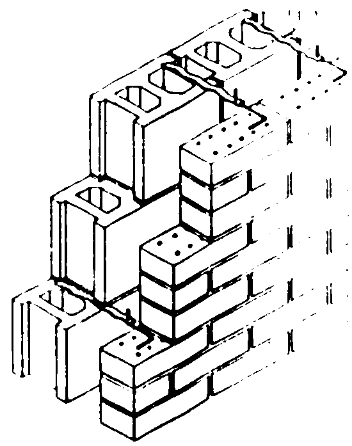


(ii) z-shaped rod

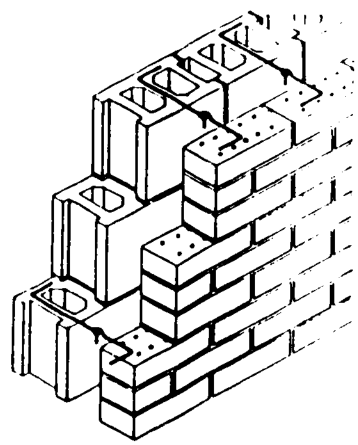


(iii) rectangular(box)

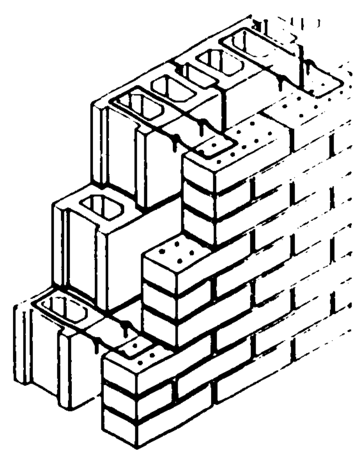
Figure 2.7 Non-Adjustable Individually Placed Connectors



(i) corrugated strip with twisted z-type



(ii) adjustable z-type



(iii) rectangular with rectangular rod

Figure 2.8 Adjustable Individually Placed Connectors

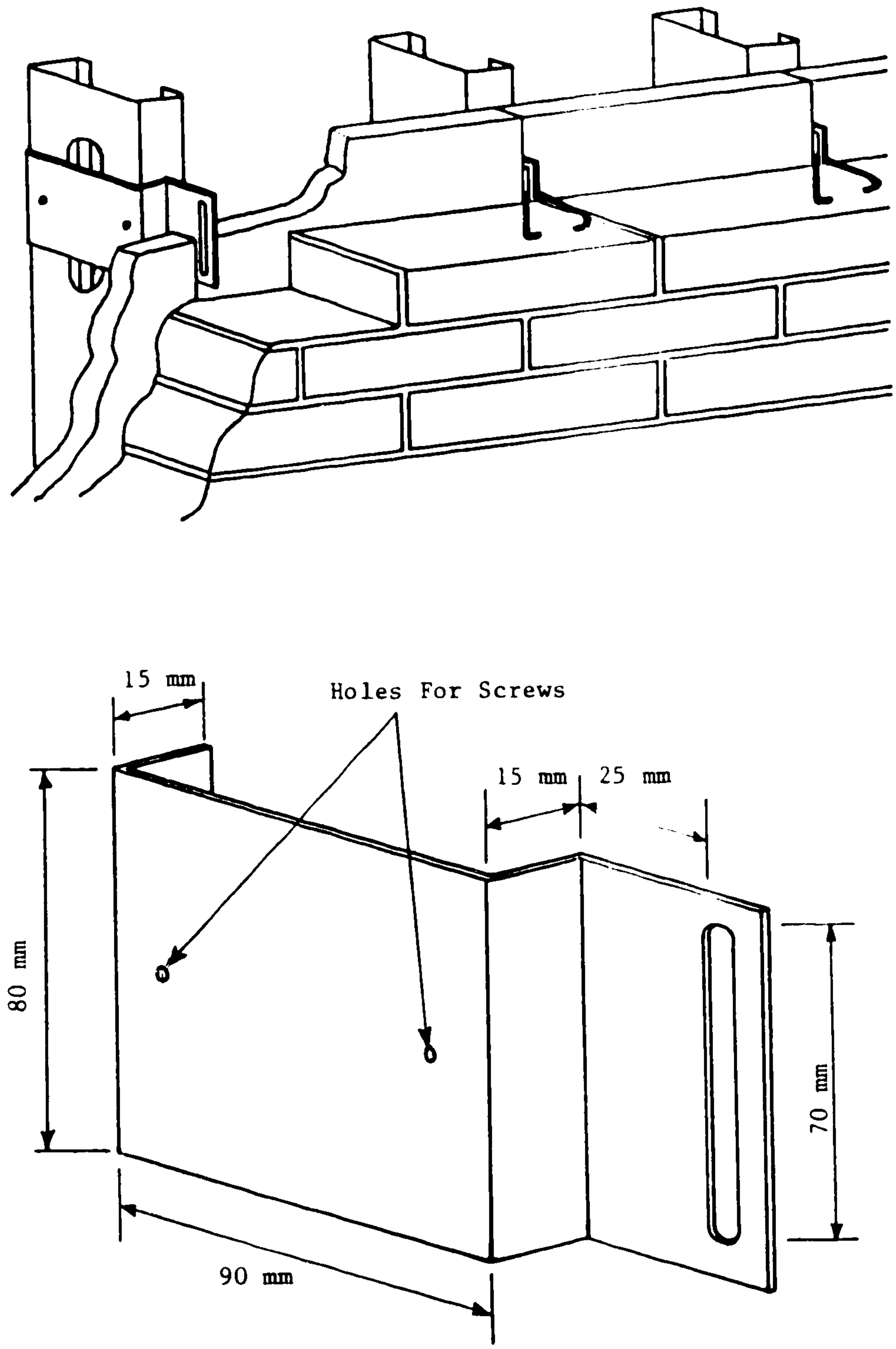


Figure 2.9 Plate Type Connector With Rod Tie Used With a Metal Stud Back-Up System

3. DEVELOPMENT OF SHEAR CONNECTOR

According to the design procedures outlined briefly in Section 2.4, the three principal factors governing the overall performance of a cavity wall system are: back-up wall strength, connector load-transfer capacity, and quality of workmanship. It follows that in order to improve on the wall system performance, we must make changes to one or more of these factors.

The lateral strength resistance of the back-up wall can be improved by increasing the size of the block, using higher quality mortar, or by implementing vertical reinforcement. However, these improvements greatly increase construction costs, often without significant strength gains. They are only beneficial up to the time when other vulnerable parts of the wall system begin to break down. That is, nothing is gained by building a back-up wall which can withstand a lateral pressure of 5.0 kPa when the connectors cannot successfully transfer a pressure of 1.0 kPa without buckling or punching through the brick veneer.

Generally, with the types of connectors available today, only small gains in strength can be achieved with minor design improvements. Similarly, one would have to extend far beyond practical limits before any increases in workmanship quality resulted in significant strength gains. Therefore, to improve the capacity of cavity walls to resist lateral loads within this scope would require strengthening of the back-up system.

Prior to searching for a solution, it is appropriate to address the problem. That is, how does a cavity wall typically fail? A laterally loaded wall system is most vulnerable when subjected to positive lateral pressure with no vertical load acting downwards on the back-up wythe. All lateral load is transferred from the brick veneer to the back-up wall by the connectors. Two modes of failure are possible. The first is that the connectors may fail by buckling or punchout/pullout of the mortar beds in which they sit. The second and more frequent failure mode is by horizontal tension failure cracks forming in the mortar joints of the block wall.

To improve on the first failure mode, a connector which would be stronger, more resistant to punchout/pullout is needed. The second failure mode can be improved by reinforcing the back-up wall or by designing a connecting system capable of resisting the shear, thus altering the mode of load transfer to the back-up wall. This improvement can be achieved by taking a novel approach and utilizing the structural attributes of the brick veneer. Traditionally, the veneer has served only to provide for appearance, weathering, and to prevent moisture from coming into contact with interior components of the wall system. By providing a connection between the brick veneer and the back-up wall capable of resisting shear, the forces acting on the wall assembly would be shared by the wall components. That is, when acted on by a positive lateral force, the brick wythe would be forced into axial compression and the block wythe into axial tension, much like as in the case of a truss. The reduced load on the back-up wall would result in lowering the overall costs of the wall system. Another benefit of using such a connector would be the reduction in veneer deflections, thus minimizing cracking and water penetration.

The prospect of developing such a wall system led to this study on the shear connector. The connector and its components can be seen in Figure 3.1 and Plate 3.1. It consists of a plate with holes and slots, cross legs, one bent-rod tie, and is installed in the concrete masonry back-up wall as follows.

Two cross rods are inserted into holes a and b and are embedded into the mortar joint of the block wall as in Figure 3.2. The plate which forms the body of the connector is extended to within 7 mm of the expected position of the inside of the brick veneer. This 7 mm is the tolerance specified under Clause 3.14 of reference 1.

The connection of the brick veneer is achieved by inserting a rod tie(Figure 3.1 - Part D) into one of several holes in the plate enabling horizontal placement of the tie in the brick mortar joint as shown in Plate 3.2. A slot is utilized to accept a wedge which in turn holds rigid insulation tight against the masonry block back-up wall.

The height, H, of the main plate of the connector is equivalent to the height of one brick and mortar joint. This permits the horizontal placement of the tie into the brick veneer mortar joint. The thickness, t, of the main plate can be adjusted to accommodate the transfer of shear forces of various magnitudes. The plate length, L, can be altered to allow for varying dimensions of the back-up wall, the insulation, and the air space. An actual wall system utilizing shear connectors is illustrated in Figure 3.3.

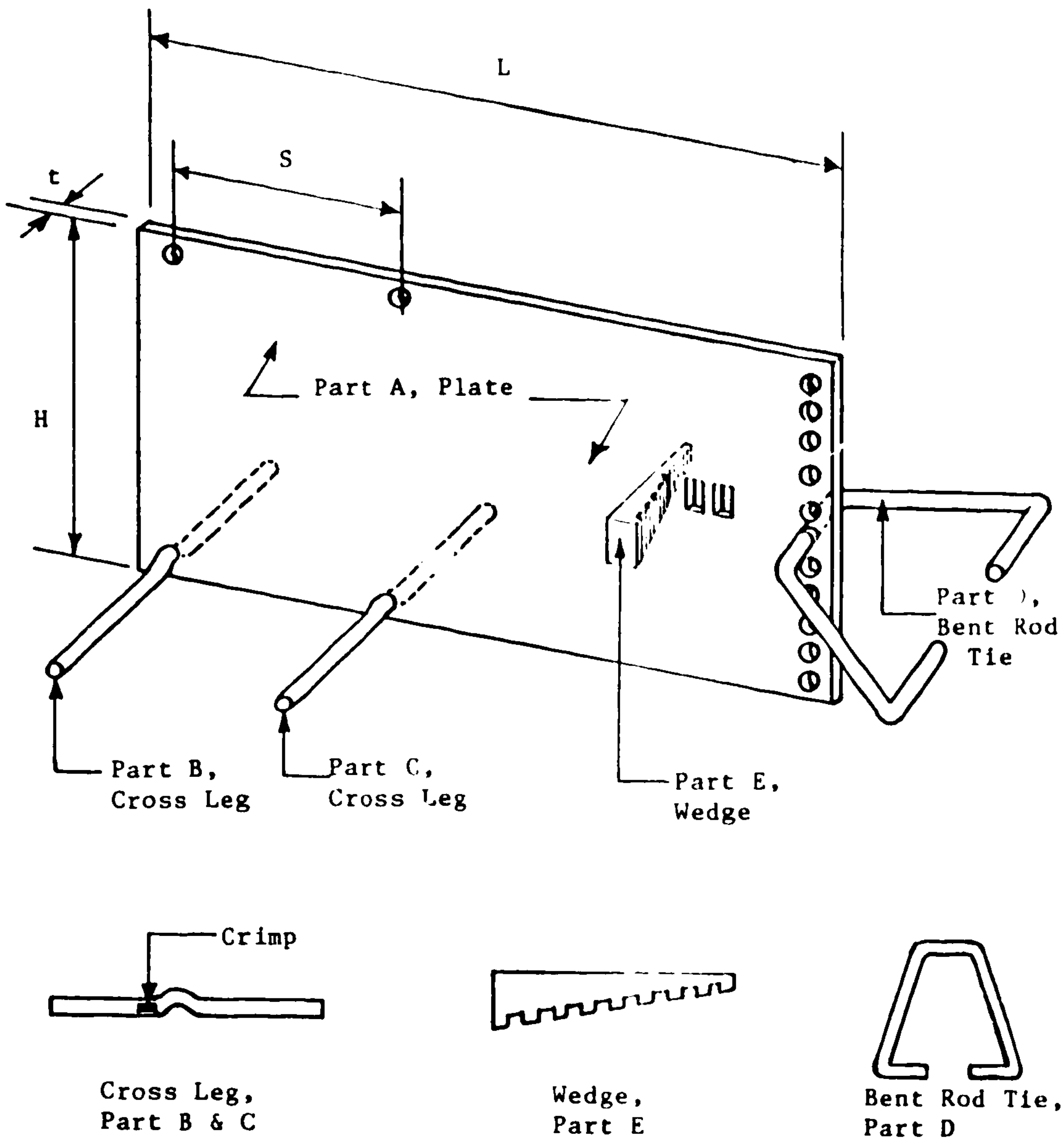


Figure 3.1 Shear Connector Prototype

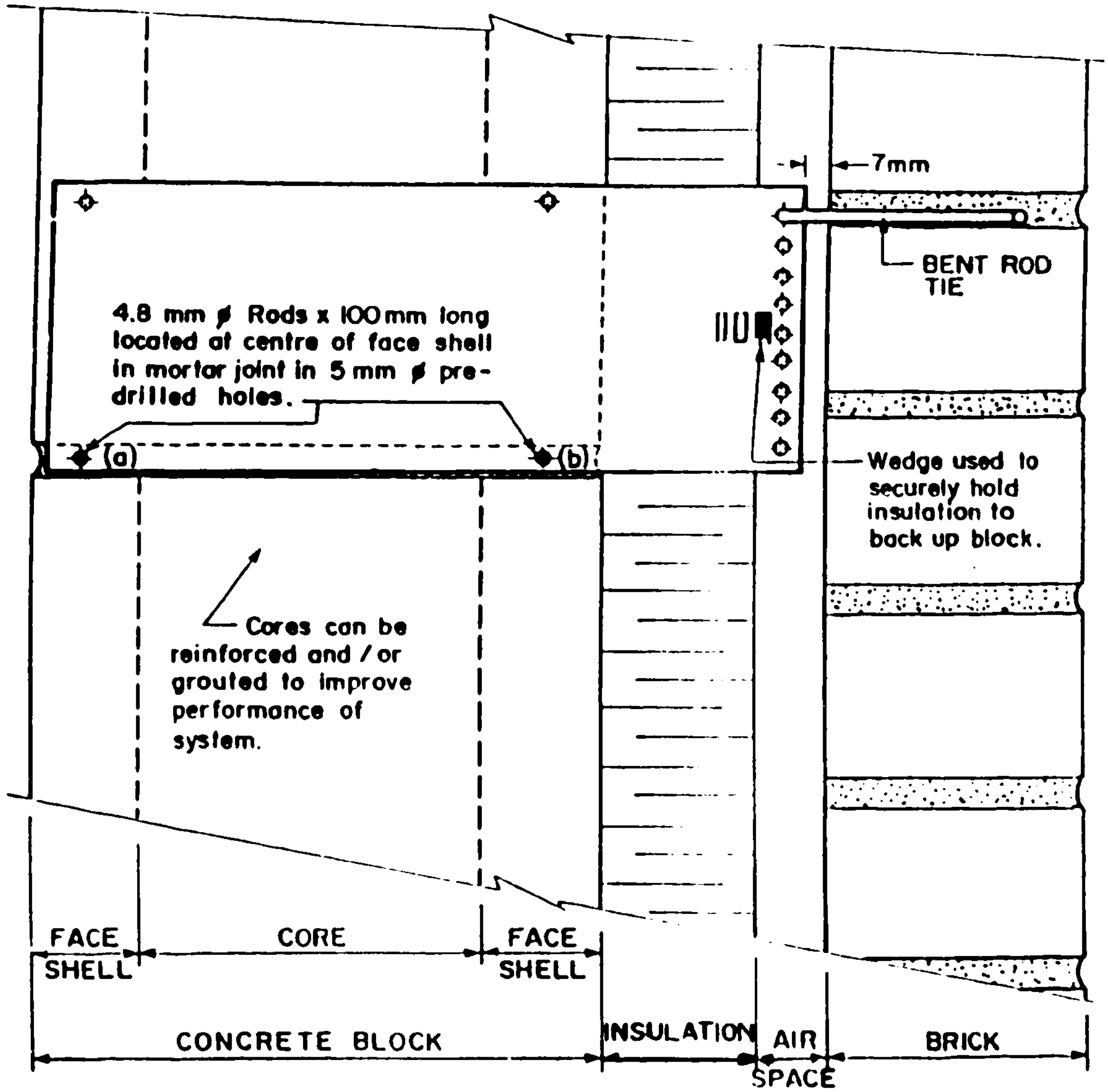


Figure 3.2 Shear Connector Application With Concrete Block Back-Up

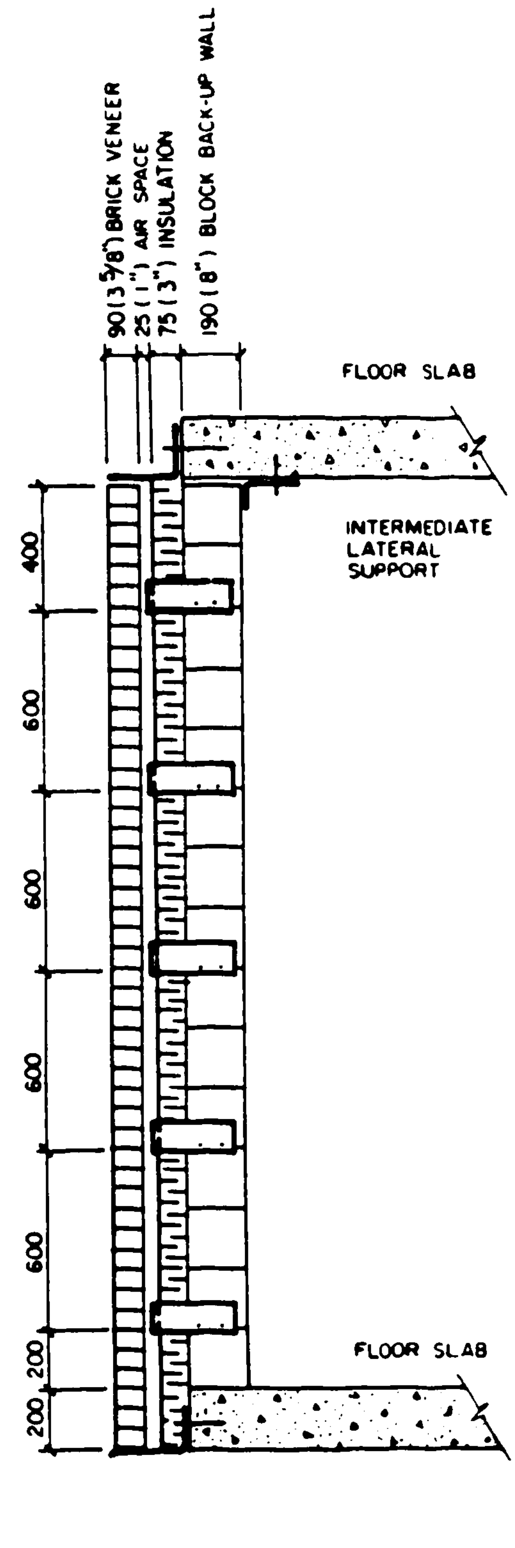


Figure 3.3 Cavity Wall System Utilizing Shear Connectors

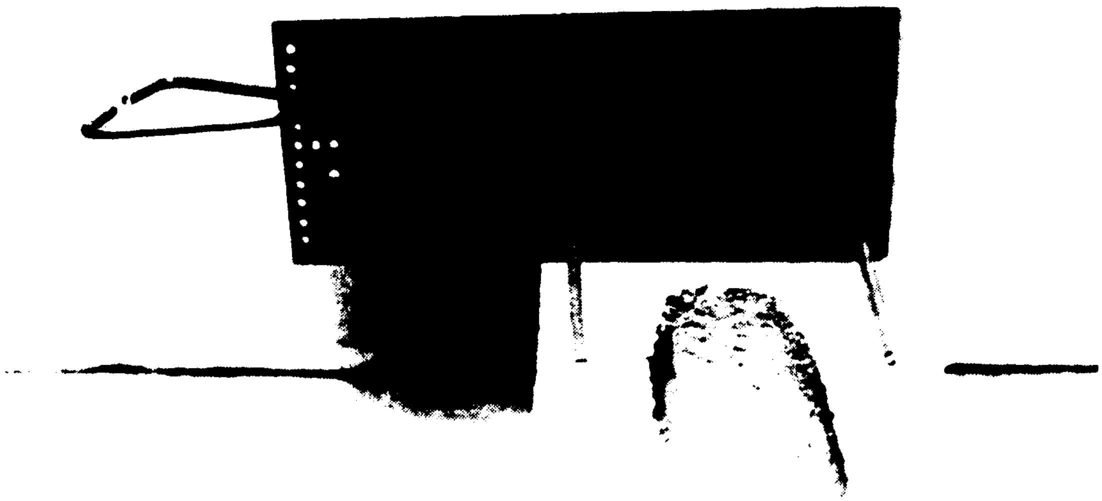


Plate 3.1 Shear Connector Prototype

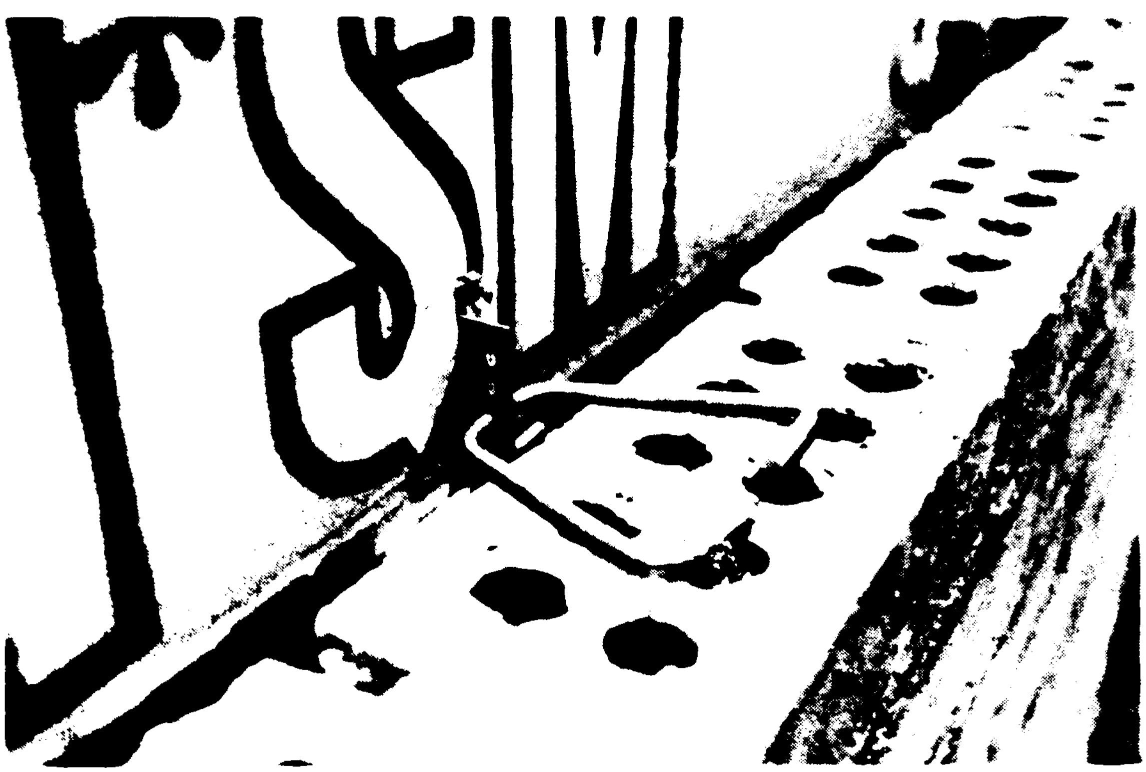


Plate 3.2 Detail of Connection of Shear Connector-Veneer Construction

4. THEORETICAL ANALYSIS OF SHEAR CONNECTOR CAVITY WALL.

4.1 Introduction

The purpose of the theoretical analysis in this study was to initiate the development of a model which would predict the elastic behaviour of masonry cavity walls which use shear-resisting connectors. A two-dimensional frame model was employed to this end. The input to the program consisted of a description of the cavity wall system and the type of load acting on it, in both global and local coordinates. The output presented the internal forces acting on the members [illegible] the resulting deflections.

4.2 Structure Description

The purpose of structure description was to define the physical characteristics and behaviour of the elements used to model the cavity wall system. Each element was composed of a member with one joint at each end. The structure description input contained material properties, joint description, and member information.

The material properties considered relevant were the elastic modulus(E), shear modulus(G), and weight density of the material. The coefficient of thermal expansion could also be incorporated into the program to fascilitate future studies regarding the thermal considerations to be made with shear connector design.

Joint description was used to allocate a specific number to each joint and to describe the ability for translational and rotational movement of each joint.

Member information consisted of member incidence, physical properties, and shape factors. Member incidence described the number of the two joints to which each member was connected. The physical properties specified were the cross-sectional area(A), and the flexural moment of inertia about the Z axis(I_Z). The shape factors were used to include the effects of shear deformation.

4.3 Model Description

The plane frame model for a general wall system shown in Figure 4.1. The solid lines represent the model, with the dotted lines symbolizing the wall system which is being simulated. The brick and block wythes are represented by a number of joints along the centerline of the wythes, connected together by members. The joints at the base of both the brick and block wythes were defined as to provide a pin at these locations. The top of the block wythe was allowed to move freely in only the X-direction. No physical constraints were assigned to the top of the brick wythe. Also in Figure 4.1 is a close-up diagram of the group of elements used to model the shear connector and its components.

The shear connector was divided into three unique elements. The first element modelled the portion of the rod tie embedded into the mortar bed of the brick veneer. It was assigned a high stiffness value because it was assumed that it does not deform during loading, thereby deflecting with the wythe. A hinge was attatched to the interior of this element to simulate the connection of the bent-rod tie with the shear connector plate. The length of the plate in the cavity was modelled by the second element. This element was assigned material properties typical of a steel plate. This cavity-portion element was attatched to the third element by means of a fixed end. This third element, the portion of the shear connector which is embedded in the block wythe, was also assigned a large stiffness.

4.4 Loading Description

The loading description required defining the type of load or load combinations, and the direction and magnitude of the applied loads. Positive lateral wind pressure was simulated by applying a distributed load acting laterally on the joints of the brick veneer, as seen in Figure 4.1. A load combination was chosen to accommodate both the lateral forces and the dead weight of the wythes which act on the plane frame model.

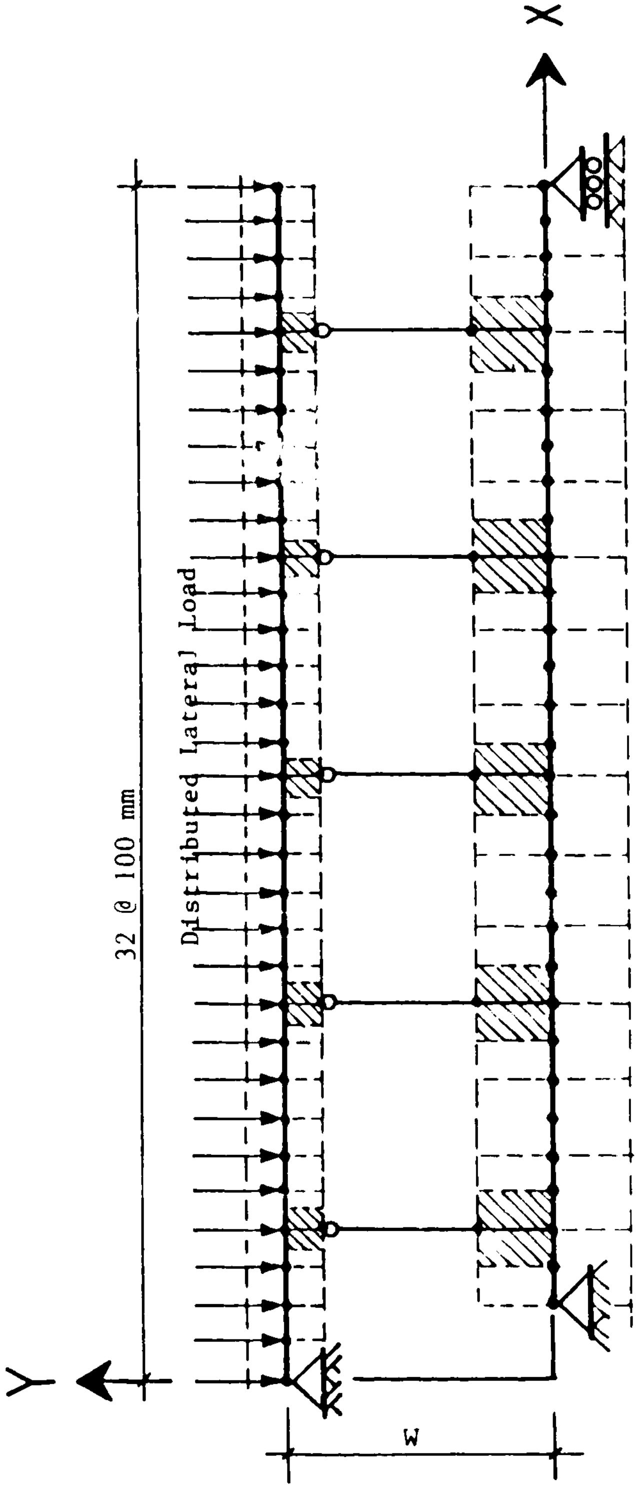
Three runs were conducted on models resembling those of specimens tested in the experimental phase of the study. The models were run with masonry component elastic moduli of 2500 MPa, 5000 MPa, and 10000 MPa. The systems were loaded with a distributed load simulating a pressure of 0.75 kPa. This pressure was chosen to enable an elastic comparison of theoretical versus actual test results.

4.5 Plane Frame Model Results

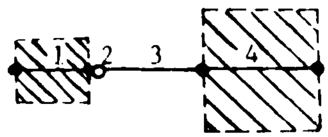
The results obtained are shown in Figures 4.2, 4.3, 4.4, and 4.5. In these figures, both plane frame model(P.F.M.) and the simple wythe model(S.W.M.) results are compared. The simple wythe model consisted of a laterally loaded, simply supported block wythe, identical to those used in the plane frame model. It serves to represent those cavity wall systems not benefitting from shear-resisting connectors.

The analysis indicates that shear connectors are effective in reducing to allowable limits, both the lateral deflections of the two wythes and the tensile and compressive forces in the block and brick wythes, respectively. The loads acting on the shear connectors were also within limits as recommended by the current design code.

The results of this part of this study are compared to those obtained from the testing of the shear connector cavity walls in section 7.4.



Shear Connector Close-Up:



1 = length of rod tie embedded in the mortar bed of brick veneer
  - large stiffness

2 = hinge assumed at the rod tie/shear connector plate junction

3 = length of shear connector in cavity
  - actual stiffness

4 = length of shear connector embedded in head joint of block back-up wythe
  - large stiffness

W = 1/2(brick + block) + cavity thickness

Figure 4.1 Plane Frame Model Description

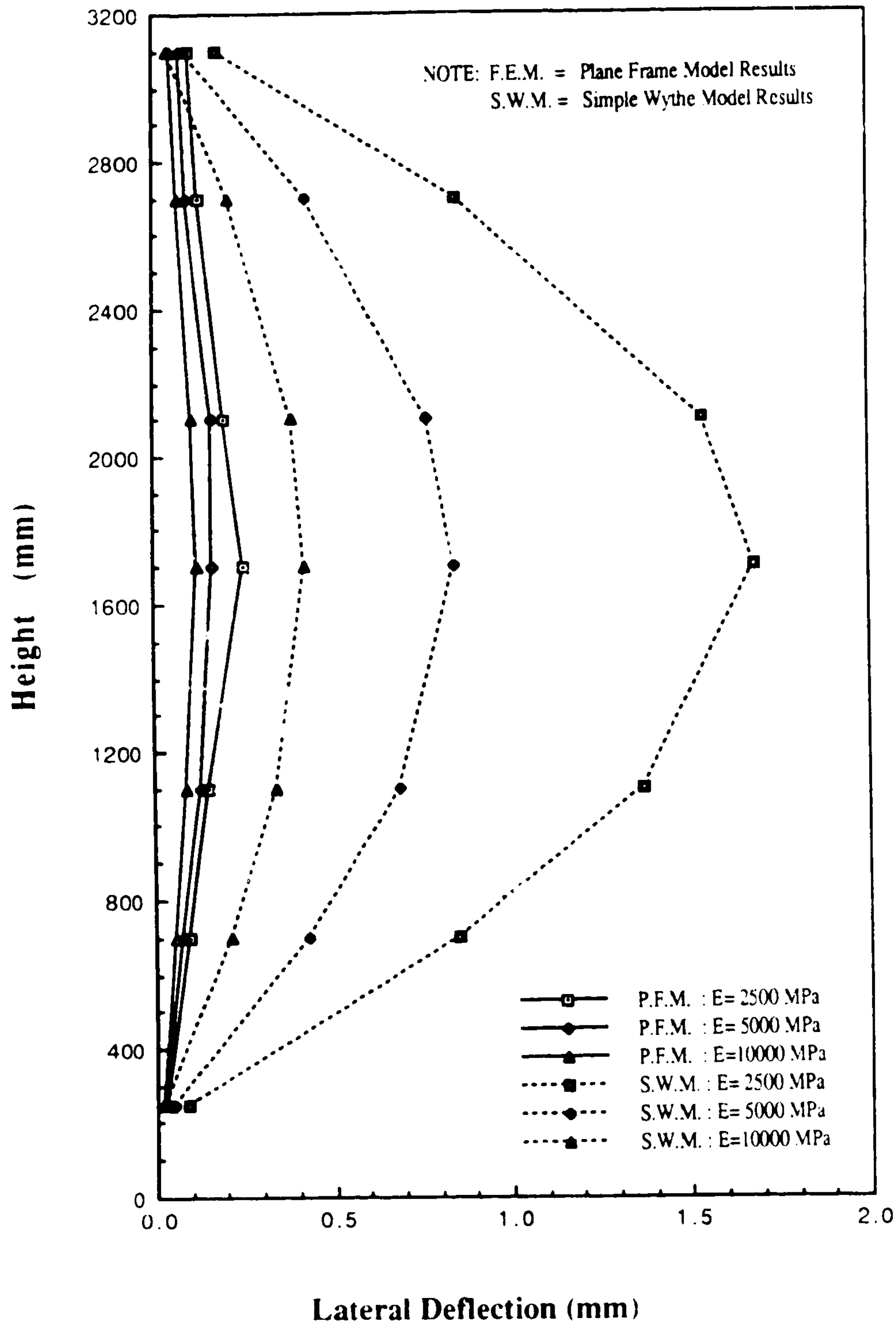


Figure 4.2 Block Wythe Deflection @ 0.75 kPa for S1W3

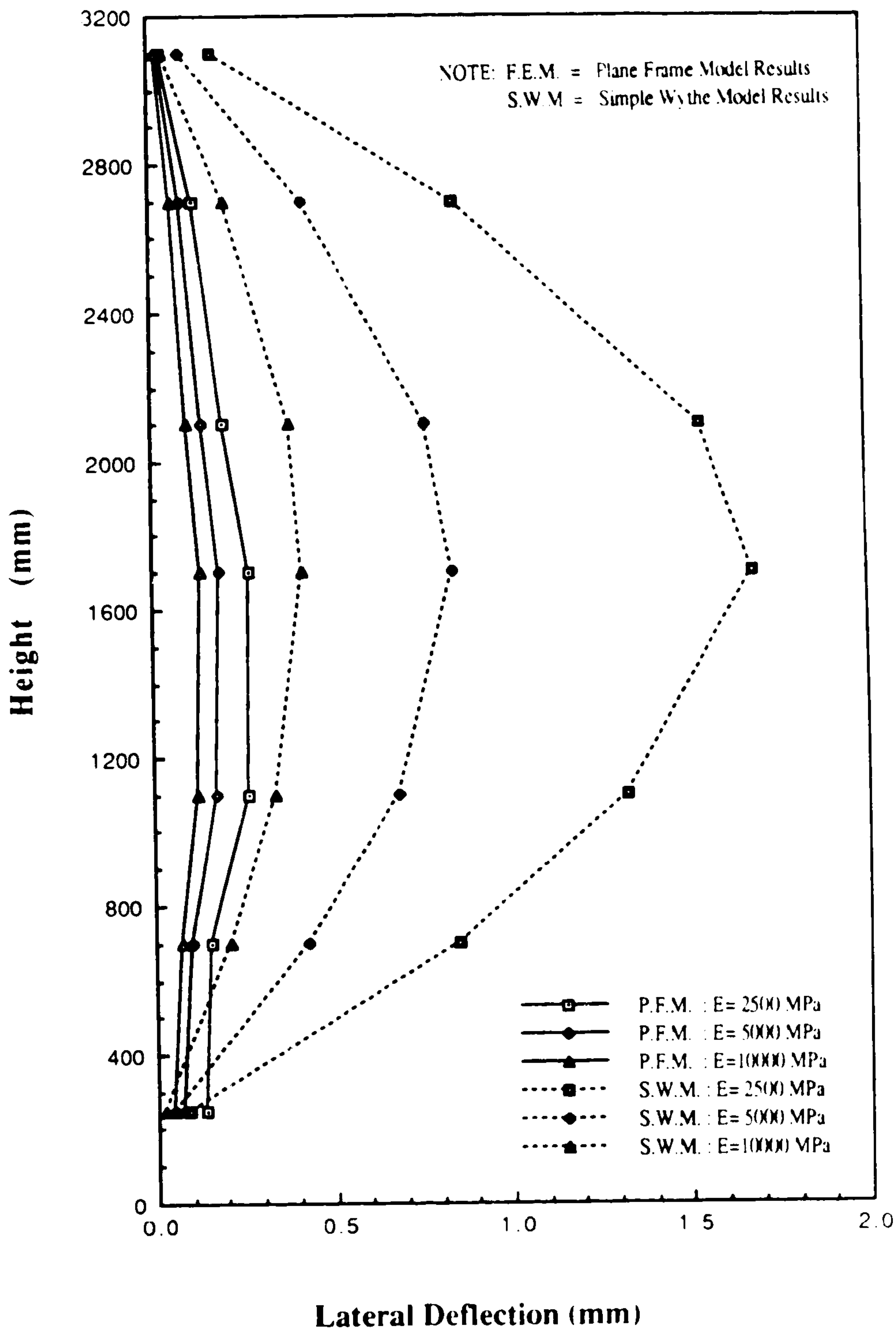


Figure 4.3 Block Wythe Deflection @ 0.75 kPa for S1W4

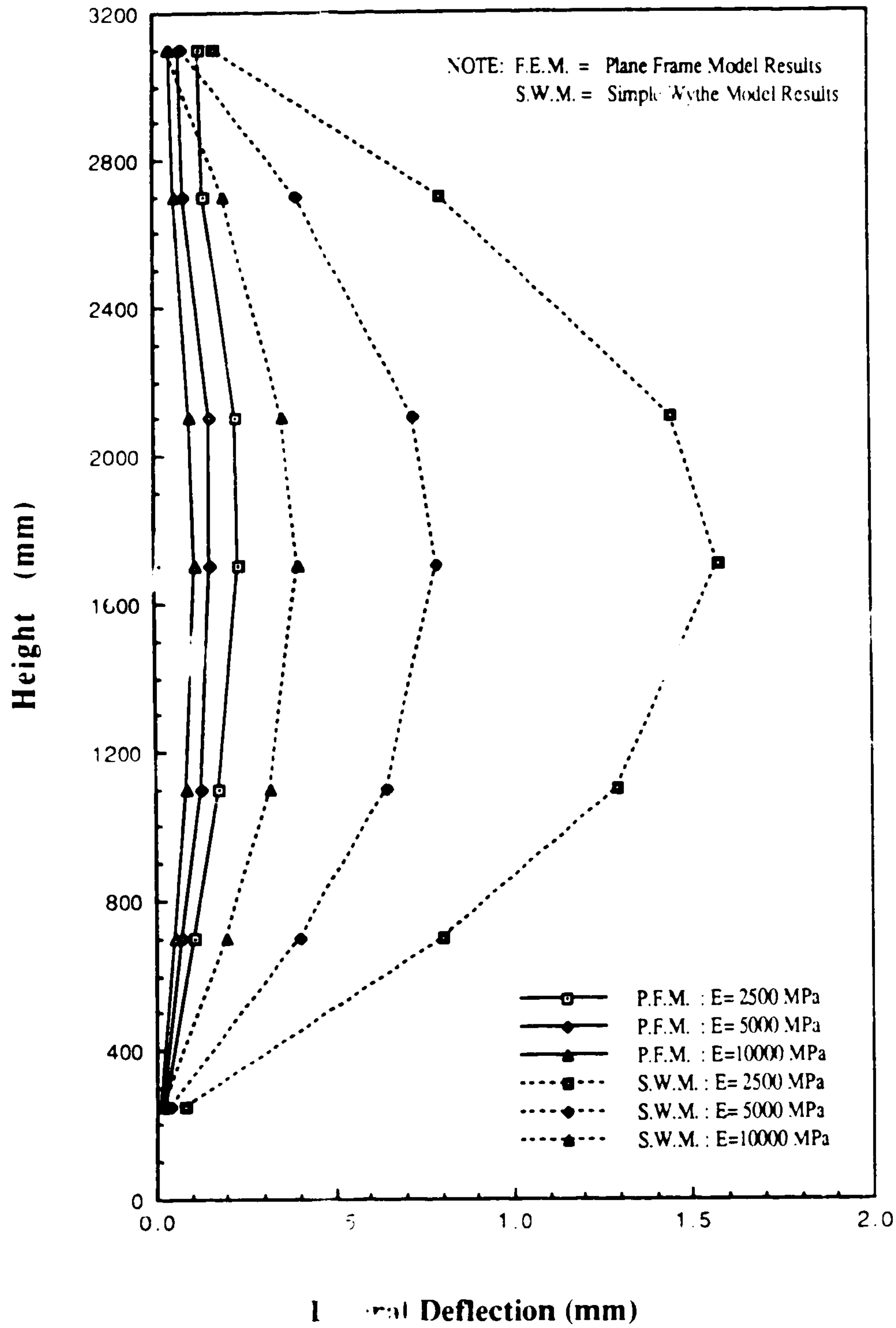


Figure 4.4 Block Wythe Deflection @ 0.75 kPa for S2W2

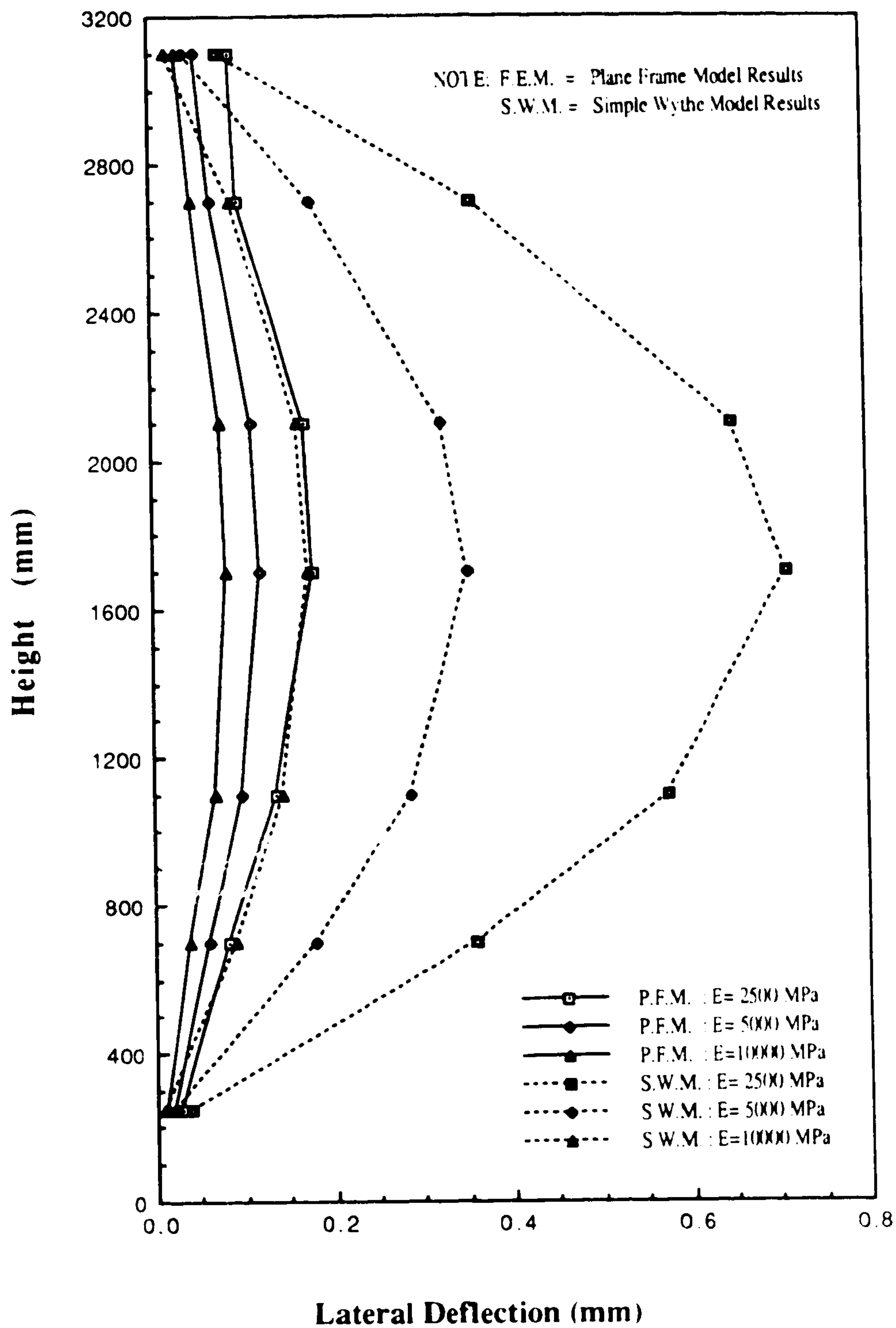


Figure 4.5 Block Wythe Deflection @ 0.75 kPa for S2W4

5. EXPERIMENTAL PROGRAM

5.1 Materials

With the exception of the shear connector prototype, materials used in the construction of the various test specimens are typical of those currently being used in masonry industry in the Edmonton, Alberta area.

5.1.1 Concrete Block Units

The back-up walls for all prisms and full sized wall specimens were constructed with hollow standard concrete block units. The 200 mm standard units had a nominal size of 200 x 200 x 400 mm, with half units having a nominal size of 200 x 200 x 200 mm. The 150 mm standard units had a nominal size of 150 x 200 x 400 mm, with half units having a nominal size of 150 x 200 x 200 mm. The units are shown schematically in Figures 5.1 and 5.2. The physical properties of the units are listed in Table 5.1.

5.1.2 Clay Brick Units

The veneer for all prism and full size wall specimens was constructed of burned clay brick units, in accordance with CSA Standard CAN3-A82.1-M1987, "Burned Clay Brick"[7]. The units had actual dimensions of 90 mm wide by 190 mm long by 57 mm high.

5.1.3 Mortar

The mortar used in the study was mixed according to the specifications given in CSA Standard A179-M1976, "Mortar and Grout for Unit Masonry"[8]. Although Type S mortar strength is more typical of construction, Type N was also used in some of the specimens in order to study the effects of mortar strength on shear connector cavity walls. Normal(Type 1) portland cement, Type S hydrated lime, and masonry sand were proportioned 1:0.5:4. The sand conformed to the grading requirements of ASTM C 144-84, "Specifications for Aggregate for Masonry Mortar"[9]. Moisture content was determined by ASTM C 566-84, "Total Moisture Content of Aggregate By Drying"[10], and found to be acceptable at 5.7%.

Three 50.8 x 50.8 x 50.8 mm mortar cubes were cast from each batch, in accordance with ASTM C 109-86, "Test Method for Compressive Strength of Hydraulic Cement Mortars"[11]. Upon casting the cubes, the molds were placed under saturated burlap for a duration of 24 hours, then stripped and left to air cure in the 30% relative humidity lab. The cubes were subjected to identical curing conditions as their companion prisms and walls. All cubes were tested at 28 days. The mortar cube 28 day compressive strengths for individual walls can be found in Table 6.1 of Chapter 6.

5.1.4 Grout

The grout consisted of Normal(Type 1) portland cement, concrete sand, and 10 mm pea gravel in proportions by weight of 1:2.5:2, in accordance with reference 8. The grout had a water/cement ratio of 1. Sieve analysis results of the concrete sand

and the pea gravel put both within the allowable limits of ASTM C 33-86, "Specifications for Concrete Aggregates"[12].

Two grout test cylinders were cast and when tested at 28 days, were found to have an average compressive strength of 21.5 MPa.

5.1.5 Reinforcement

Steel reinforcement was used only in the reinforced walls of S2W2. The vertical steel used was deformed metric 15M bars with a 300 MPa tensile yield strength. Tensile tests were not conducted on the steel as other components of the wall system were expected to fail long before the steel yielded.

5.1.6 Connector

5.1.6.1 Wire Truss Joint Reinforcement

The wire truss joint reinforcement used in the study was mill galvanized and had a wire diameter of 4.76 mm.

5.1.6.2 Shear Connector Prototype

The shear connector consisted of three components; the plate, cross-rods, and triangular tie. This connector prototype is shown in Figure 5.3. The plate was made of 14 gage galvanized sheet metal. The cross rods were mill galvanized, being 4.76 mm in diameter. For simplicity, the insulation-securing wedge, as described in Chapter 3.0, was substituted by a cross rod. The triangular tie was also mill galvanized with a 4.76 mm diameter. The length, L, of the connector plate varied with cavity width and block width.

5.2 Specimen Description

The test program was comprised of two parts. The first part consisted of a series of prism tests used to determine various material properties. The second phase of the program involved the testing of full sized cavity wall systems.

All specimens were constructed by an experienced mason. Care was taken to simulate a level of workmanship typical of a well-made wall in the field. The wythes were laid in a running bond pattern, with a 5 mm raking performed on all mortar joints. All walls cured in the lab at room temperature and a relative humidity of 30%. All specimens were cured for 28 days prior to testing.

5.2.1 Prisms

5.2.1.1 Modulus of Elasticity

Three unreinforced concrete block prisms were prepared concurrently with full-size wall specimens. The prism size was three blocks high by one and a half blocks wide. The height to thickness ratio(h/t), as specified in clause 4.3.2.2 of reference 1, was found to be 2.0 This necessitated a compressive strength correction factor of 1.0.

The elastic modulus of the brick veneer was determined in accordance with CSA Standard CAN3-A82.2-M78, "Methods of Sampling and Testing Brick"[13]. The specimens consisted of five clay brick units which were chosen at random from the pallets of brick supplied for the study.

5.2.1.2 Shear Connector

Three test specimens were constructed to evaluate the capacity of the shear connectors in accordance with reference 2. Each specimen consisted of a 800 mm x 800 mm, 190 mm wide concrete block wall segment with a shear connector placed within the mortar joints as shown in Figure 5.4 .

Testing was conducted in order to evaluate the axial and shear strength capacity of the shear connector and the shear connector/block junction.

5.2.2 Full Scale Wall Specimens

Eight full sized wall specimens were constructed to satisfy the second phase of the testing program.

Figure 5.5, Plate 5.1, and Plate 5.2 present the construction details of a typical full sized wall specimen. The 1200 mm long x 3200 mm high wall specimens were constructed in specially designed testing frames, as shown in Plate 5.3. The frames were detailed to satisfy two requirements. The first was to enable the completed wall assemblies to simulate a section of one complete storey in a concrete frame building. The second requirement was to allow transportation of the wall assemblies to and from the testing apparatus without damage. The testing frames consisted of two concrete slabs separated by four, 3.0 m long, adjustable round HSS columns. As in the case of a real structure, 12 mm shelf angles were attached to these simulated floor slabs to support the brick veneer.

The construction sequence for each wall was as follows:

(i) The first concrete block course was laid in a mortar joint on the concrete slab to ensure a smooth, uniform surface at the base of the wall.

(ii) The remaining 14 courses were laid in a running bond pattern, with the appropriate connector type and spacing. The conventional reinforcement was spaced 600 mm vertically as in Plate 5.4. The shear connectors were spaced 400 mm horizontally and 600 mm vertically as in Plate 5.5.

(iii) A 100 mm intermediate lateral support angle was mounted underneath the top slab, fitting flush against the back face of the block wall (conforming to reference 1).

(iv) Butyl flushing was applied to the from face of the block wall, extending along the surface of the shelf angle (Figure 5.6(ii)).

(v) Rigid plank styrofoam was mounted flush against the front of the block wall. The insulation was manufactured by DOW Chemical under the trademark of Styrofoam S/M. Plate 5.6 shows the insulation being mounted to a back-up wall which uses shear connectors. Insulation-securing rods built into the shear connectors, as shown in Plate 5.7, were used to fascilitate this mounting process.

(vi) At the specified cavity width away from the bl wall, the brick wall was laid, resting on a mortar joint on the shelf angle. Care was taken to lay the reinforcement ties horizontally with the brick mortar bed joints. The cavity was not cleared. Refer to Plate 3.2 of Section 3.0 for a photograph of this horizontal placement..

(vii) After 28 days of air curing in the laboratory at 30% relative humidity, the top brick expansion joint was filled with 12 mm styrofoam rope and Mono brand caulking (Figure 5.6).

Loads acting vertically downwards on the back-up wythe of a cavity wall system enhances its positive lateral load-resisting capacity. As a result, no vertical loads were applied to the block wythes in this study.

The materials used for the full sized wall specimens were those as described in Section 5.1. Care was exercised to ensure that all wire rods from all connectors were placed at midpoint in the face shell of the concrete blocks and at the center line of the brick veneer.

Two series of four walls each were to be tested. The following five variables were incorporated into this phase of the study: concrete back-up wall size, cavity width, connector type, connector pattern, and vertical reinforcing. Wall system details are summarized in Table 5.2 and Figure 5.7.

The walls are identified by their corresponding series and wall number. For the sake of brevity, a form of shorthand will be used henceforth. For instance, Wall S[illegible] 1 from Series 1 will be referred to as S1W1.

All four of the wall systems of Series 1 used 150 mm standard concrete block units for the back-up wythes. Each used 90 mm wide clay brick units and had a 75 mm cavity, except S1W4 which had a 100 mm cavity. Walls S1W1 and S1W2 compared the use of conventional reinforcement with a similar wall, S1W3, which incorporated shear connectors. Specimen S1W4 was compared with S1W3 to study the effect of cavity width increase while using shear connectors.

All Series 2 specimens used 200 mm standard concrete block units for their back-up wythes except S2W2, which used 150 mm standard units. Specimen S2W1 compared the use of conventional reinforcement with that of shear connectors as in S2W3. The effects of cavity width variation was probed by collating S2W3 and S2W4.

All but one back-up wall in the study was unreinforced. Wall S2W2 was an exception to the construction sequence in that the placement of vertical steel reinforcement and grouting was required during step (iii). Only the outermost core on each end of the wall was reinforced. Cavity wall S2W2 was reinforced in order to compare with S1W3, its unreinforced counterpart.

5.3 Testing Apparatus & Procedures

5.3.1 Prisms

5.3.1.1 Modulus of Elasticity

The concrete block prisms were tested under compression by the MTS testing machine in accordance with CSA Standard A165.1-M1977, "Concrete Masonry Units"[14]. Both the load and deflection were monitored throughout each test run. The elastic modulus, E, was taken to be the slope of the most linear position of the curve.

Test data was taken from a concurrent study conducted at the University of Alberta. This related study entitled "The Effect of Tie Type on Brick Veneer Walls"[15] used similar materials. The elastic modulus of the brick veneer was determined in accordance with reference 13. Five clay brick units were tested in compression by the MTS testing machine.

5.3.1.2 Shear Connector

Each wall segment was rigidly secured within a testing frame, as shown in Plate 5.8. Compressive axial force and shear force was applied separately to the free end of the connector by means of a hydraulic jack. Plate 5.9 presents the mechasnism used to transfer the load from the jack to the shear connector. A load cell attached to the loading apparatus measured the applied loads. Both upward and downward shear force was applied to each connectors.

5.3.2 Full Scale Wall Specimens

The test program consisted of two series of four walls each. The specimens were cured within their test frames for 28 days prior to testing. The test frames were transported by overhead crane and positioned within the testing apparatus. The specimens were lifted by placing a cargo sling under the lower supporting angle and through lifting hooks at the rear of the bottom concrete slab. This procedure was used to eliminate any damage the wall systems may have otherwise experienced during transport.

The testing apparatus consisted of four W shaped columns boxed together by four beams. At the front of the apparatus was a rigid support wall made of fluted steel decking with a 12 mm plywood shell. An air bag rested on the interior of this support wall. The test frame was moved into place with the front of the brick veneer directly adjacent to the air bag. The test frame was fixed into position by attaching the top slab to the columns via two small connecting beams. The bottom slab was secured in place by the use of a jack fixed at the base of the columns. Plate 5.10 shows a wall specimen immediately prior to testing.

Instrumentation on the wall systems consisted of 12 linear variable differential transducers(LVDT's). The transducers may be seen attatched to the specimen, which is secured within the test frame. Six transducers were connected to each wythe at various heights in order to monitor their deflections independently, as in Figure 5.9. All of the LVDT's were fastened within a wooden brace which was bolted to the two rear supporting pipes. Steel rods extended through pre-drilled holes in the block, accommodating the

LVDTs monitoring the veneer deflection. Wires attached to the LVDTs were fastened to the block wall and veneer extension rods by epoxy glue. All LVDT wires were tensioned by elastic bands.

Load was applied to the wall system by inflating the air bag, thus simulating positive air pressure. An attached pressure transducer measured the air pressure in the bag. The bag was inflated by employing the 690 kPa laboratory air supply in conjunction with a series of pressure regulators.

The data output from the LVDT's and the pressure transducer was simultaneously monitored and recorded by a computerized data acquisition system. Prior to actual full scale testing, each wall system was subjected to a pressure of 0.50 kPa. This pre-loading was performed to ensure proper seating of the specimen. This load was removed and all gages were set to zero, prior to beginning the actual test. Each specimen would experience a constant increase in load until failure was achieved. Failure was defined as being the point at which the wall system could no longer successfully sustain an increase in load.

Property	200 mm Standard Block	150 mm Standard Block
Width (mm)	190	140
Length (mm)	390	390
Height (mm)	190	140
Moisture Content (%)	10.2	21.9
Absorption (%)	14.3	20.0
Gross Area (mm^2)	74100	54600
Effective Net Solid Area (mm^2)	41500	31700
(%)	56	58
Minimum Compressive Strength (MPa)	15	15

Table 5.1 Physical Properties and Dimensions of Concrete Block Units

Name	Block Size (mm)	Connector Type	Connector Pattern	Cavity Width(mm) Total Cavity	Rigid Insulation	Air Gap	Reinforcement
S1W1	150	3.66 mm Diam. W.T.J.R.	A	75	50	25	N/A
S1W2	150	4.76 mm Diam. W.T.J.R.	A	75	50	25	N/A
S1W3	150	S.C.	A	75	50	25	N/A
S1W4	150	S.C.	A	100	75	25	N/A
S2W1	200	3.66 mm Diam. W.T.J.R.	B	75	50	25	N/A
S2W2	150	S.C.	A	75	50	25	2 Cores Reinforced
S2W3	200	S.C.	A	75	50	25	N/A
S2W4	200	S.C.	A	100	75	25	N/A

Note:
(1) Specimen name S1W1 refers to Wall 1 of Series 1.
(2) W.T.J.R. - Wire Truss Joint Reinforcement
(3) S.C. - Shear Connector
(4) N/A - No vertical reinforcement was used.
(5) S2W2 had both outer cores each reinforced with 1-15M re-bar and 20 MPa grout.

Table 5.2 Summary of Full Sized Wall Specimens

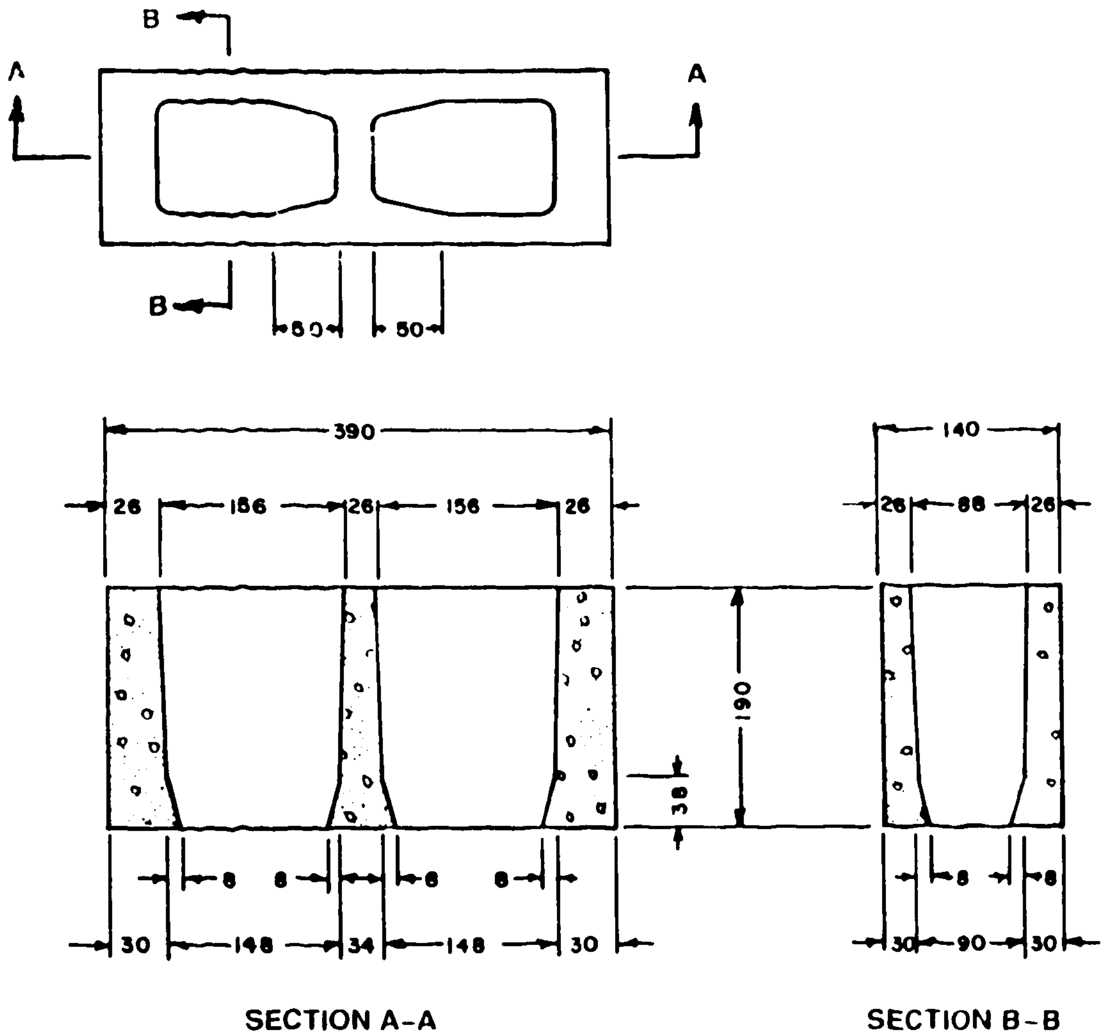


Figure 5.1 Concrete Block Unit Dimensions - 150 mm

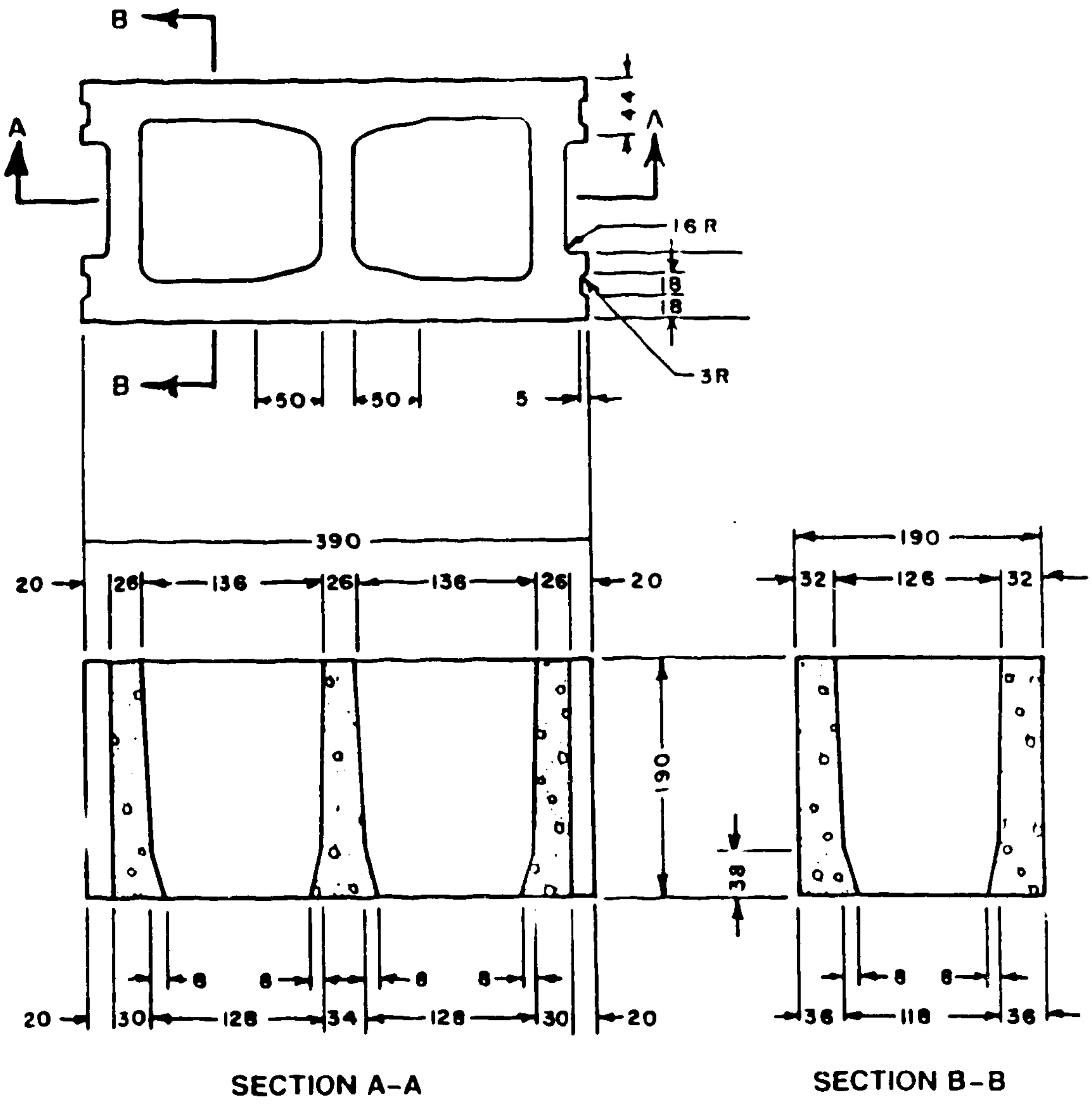
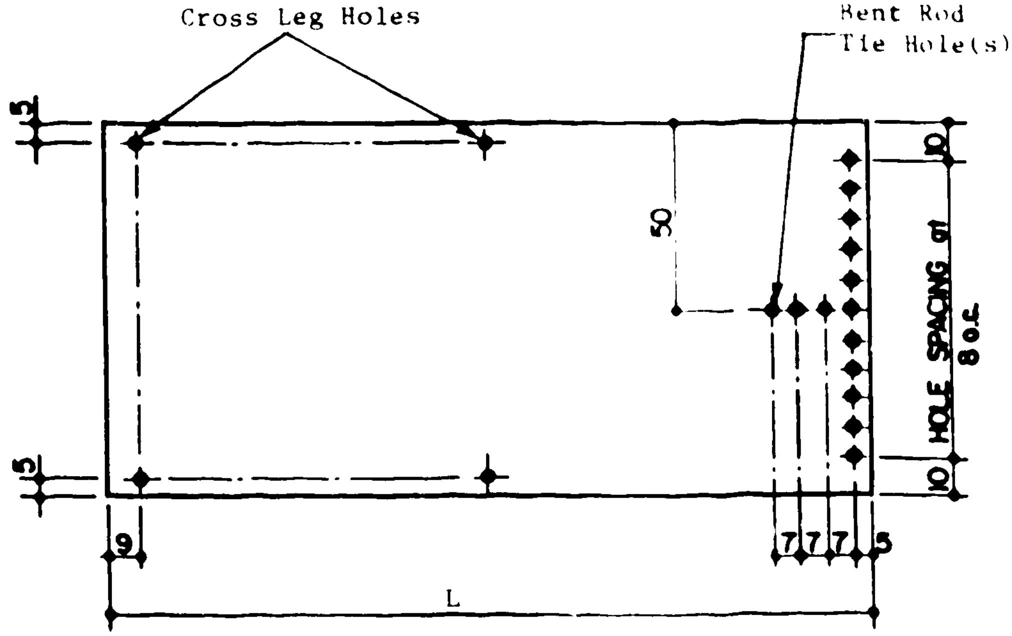


Figure 5.2 Concrete Block Unit Dimensions - 200 mm

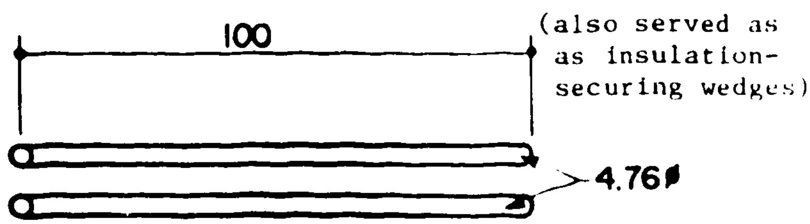
(i) Plate - 14 Gage Galvanized Sheet Metal



NOTE: L = 205 mm for 150 mm block back-up with a 75 mm cavity
L = 230 mm for 150 mm block back-up with a 100 mm cavity
L = 252 mm for 200 mm block back-up with a 75 mm cavity
L = 278 mm for 200 mm block back-up with a 100 mm cavity

All holes have a 4.77 mm diameter.

(ii) Cross Rods - 4.76 mm Diameter Galvanized Metal Rods



(iii) Triangular Tie - 4.76 mm Diameter Galvanized Metal Rod

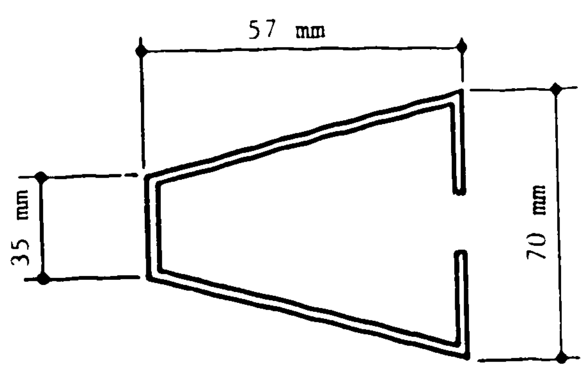


Figure 5.3 Shear Connector Components

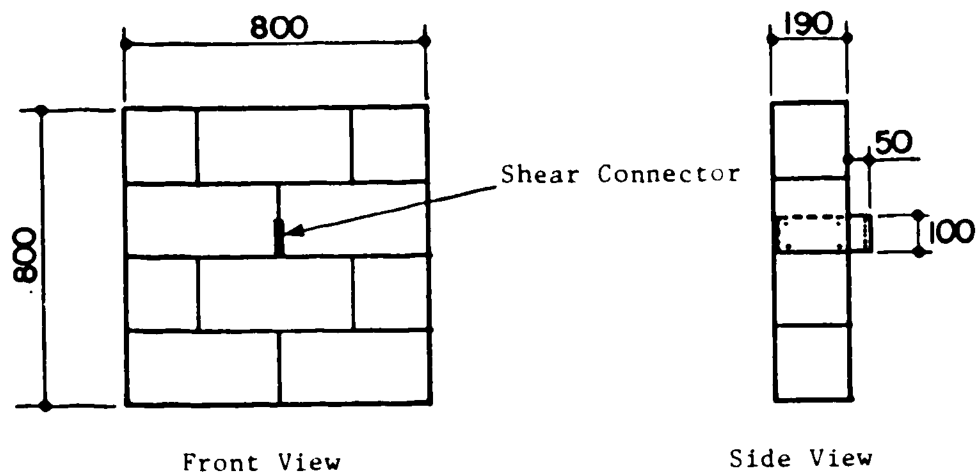


Figure 5.4 Shear Connector Prism

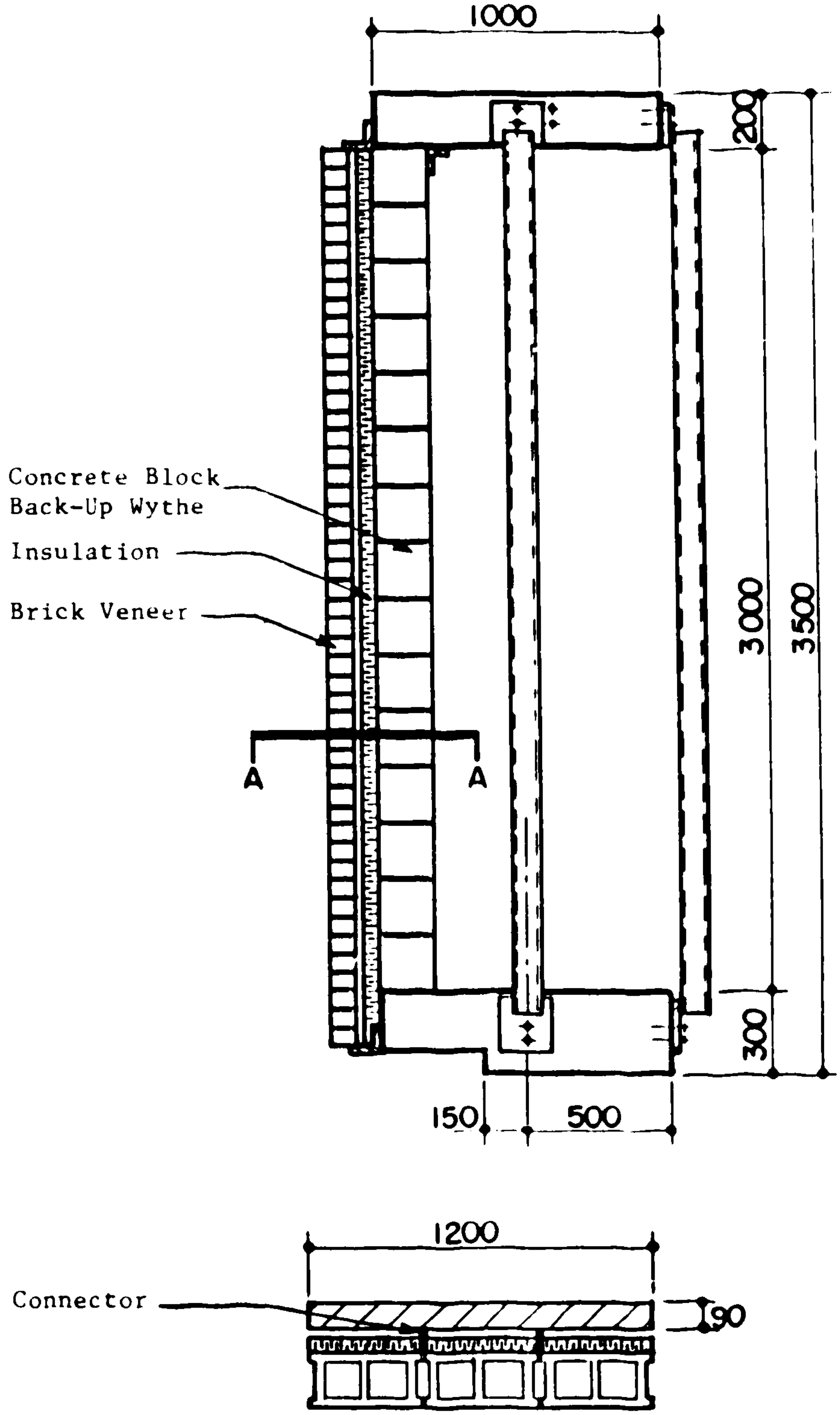
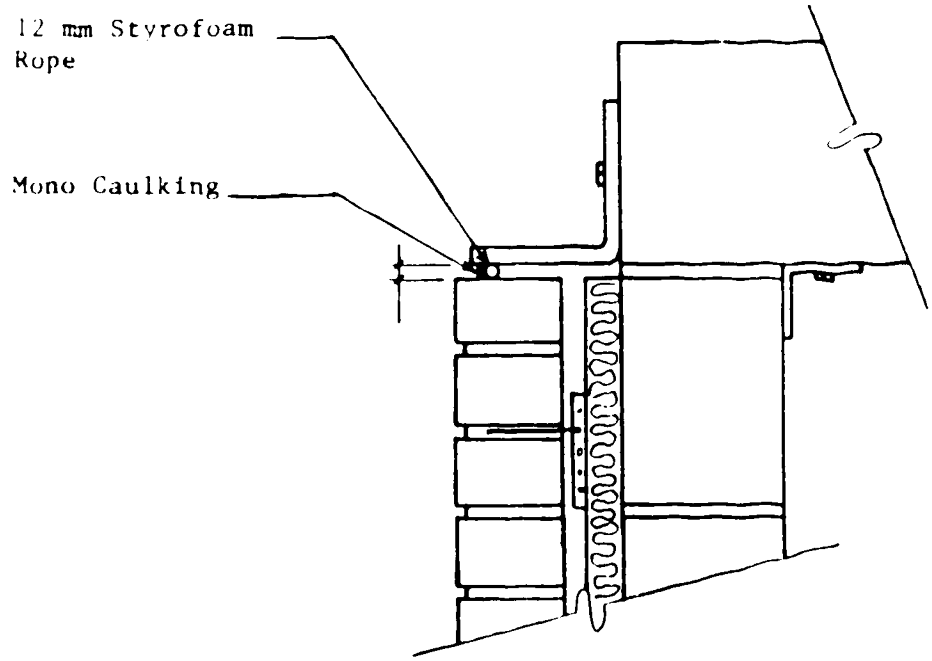
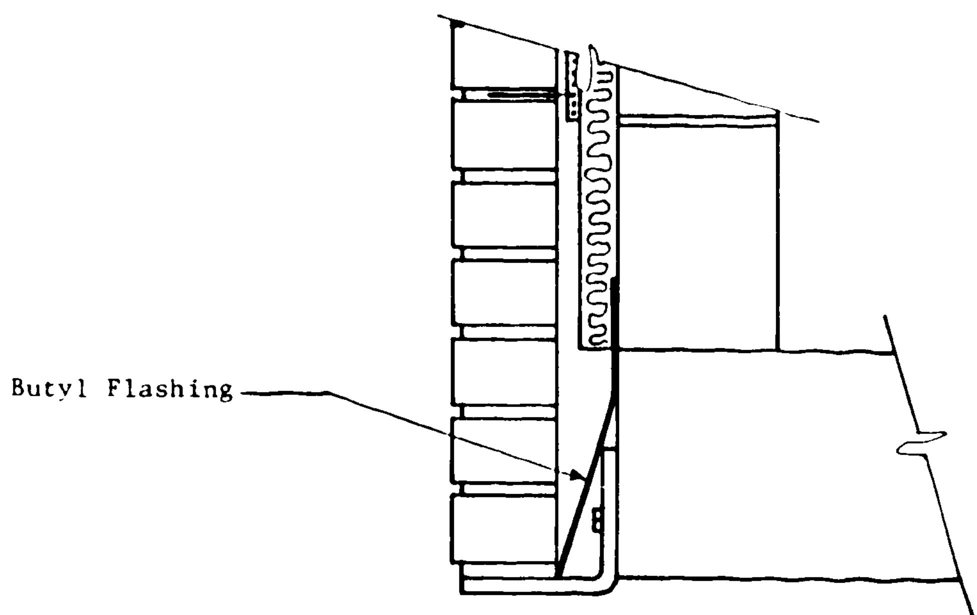


Figure 5.5 Construction Details of Full Sized Wall Specimen



(i) top slab details



(ii) bottom slab details

Figure 5.6 Full Size Wall Specimen Details

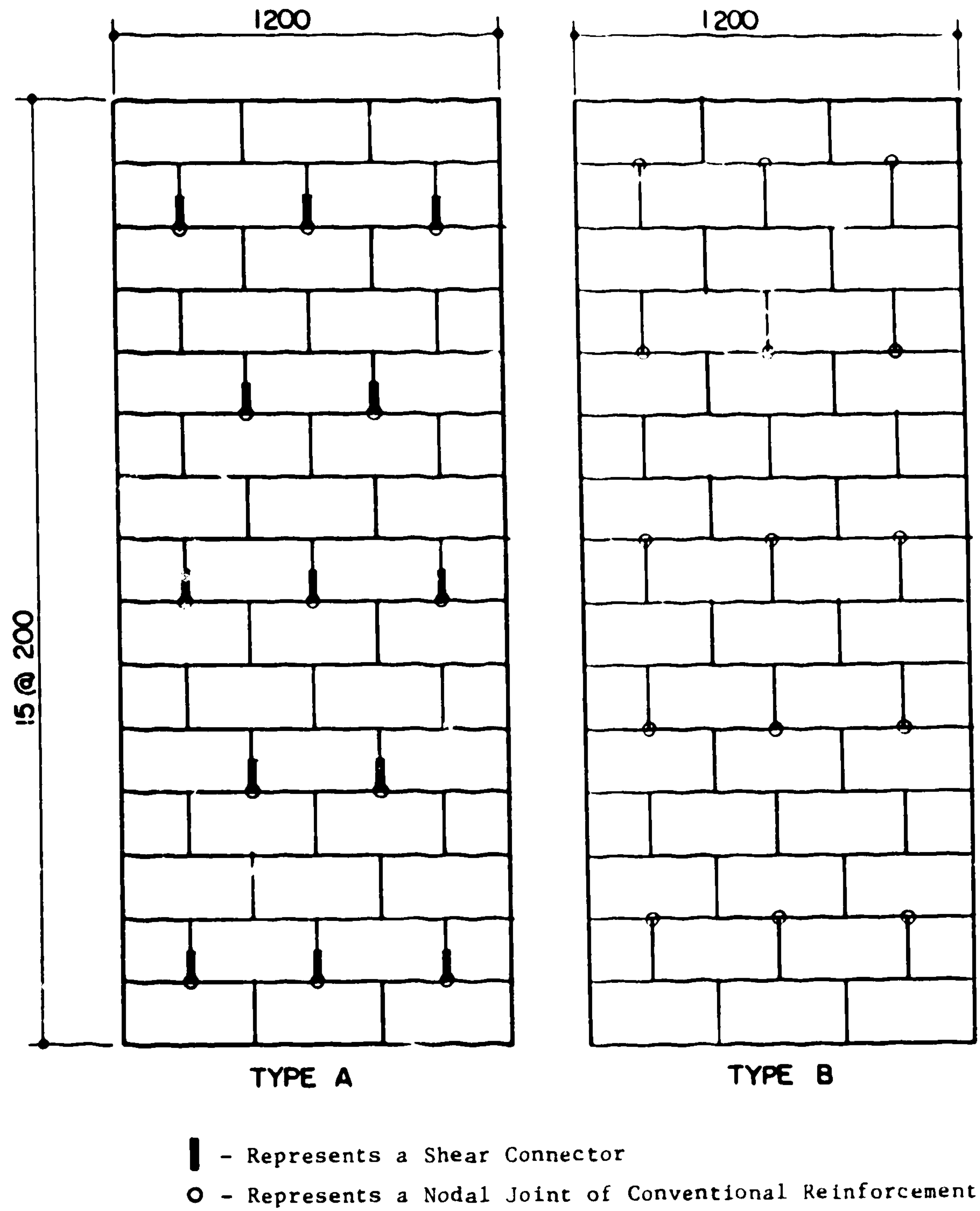


Figure 5.7 Connector Patterns

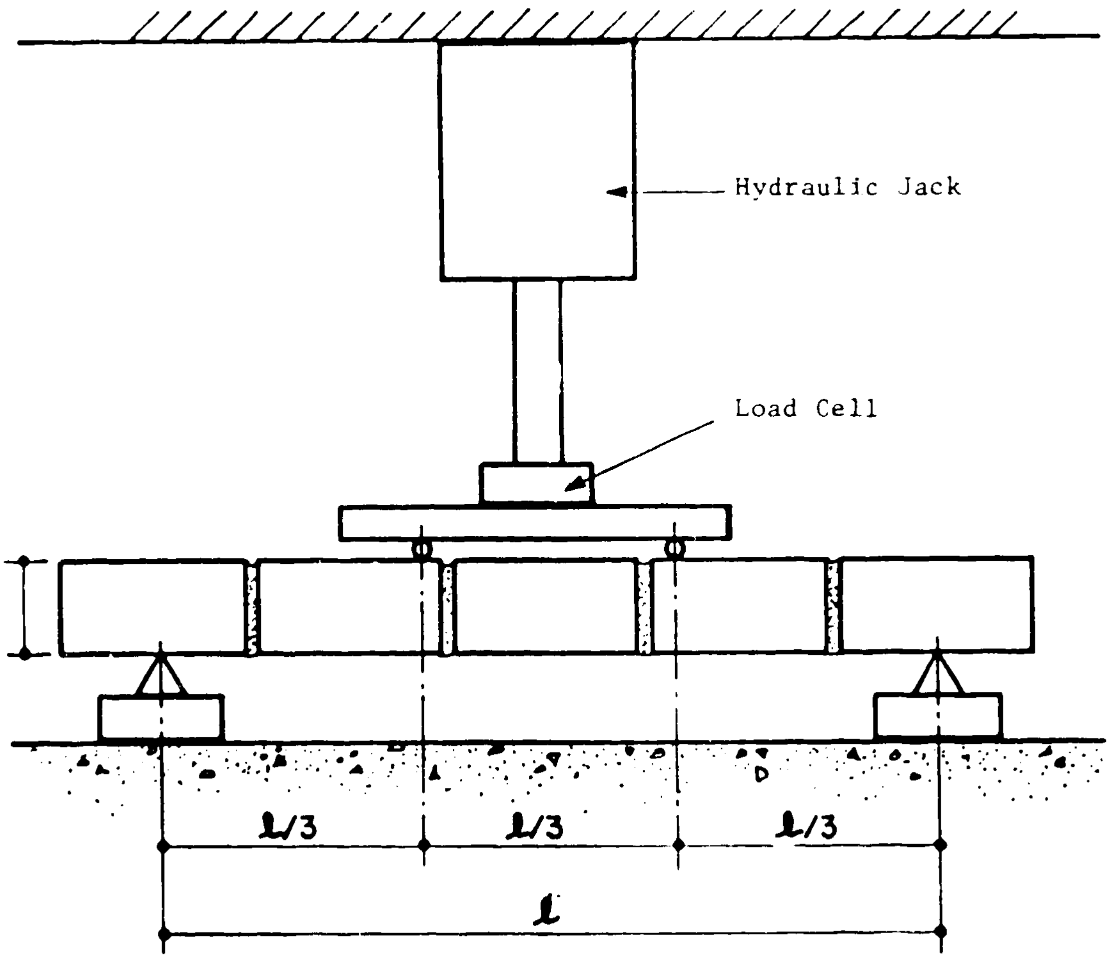


Figure 5.8 Modulus of Rupture Test Set-Up

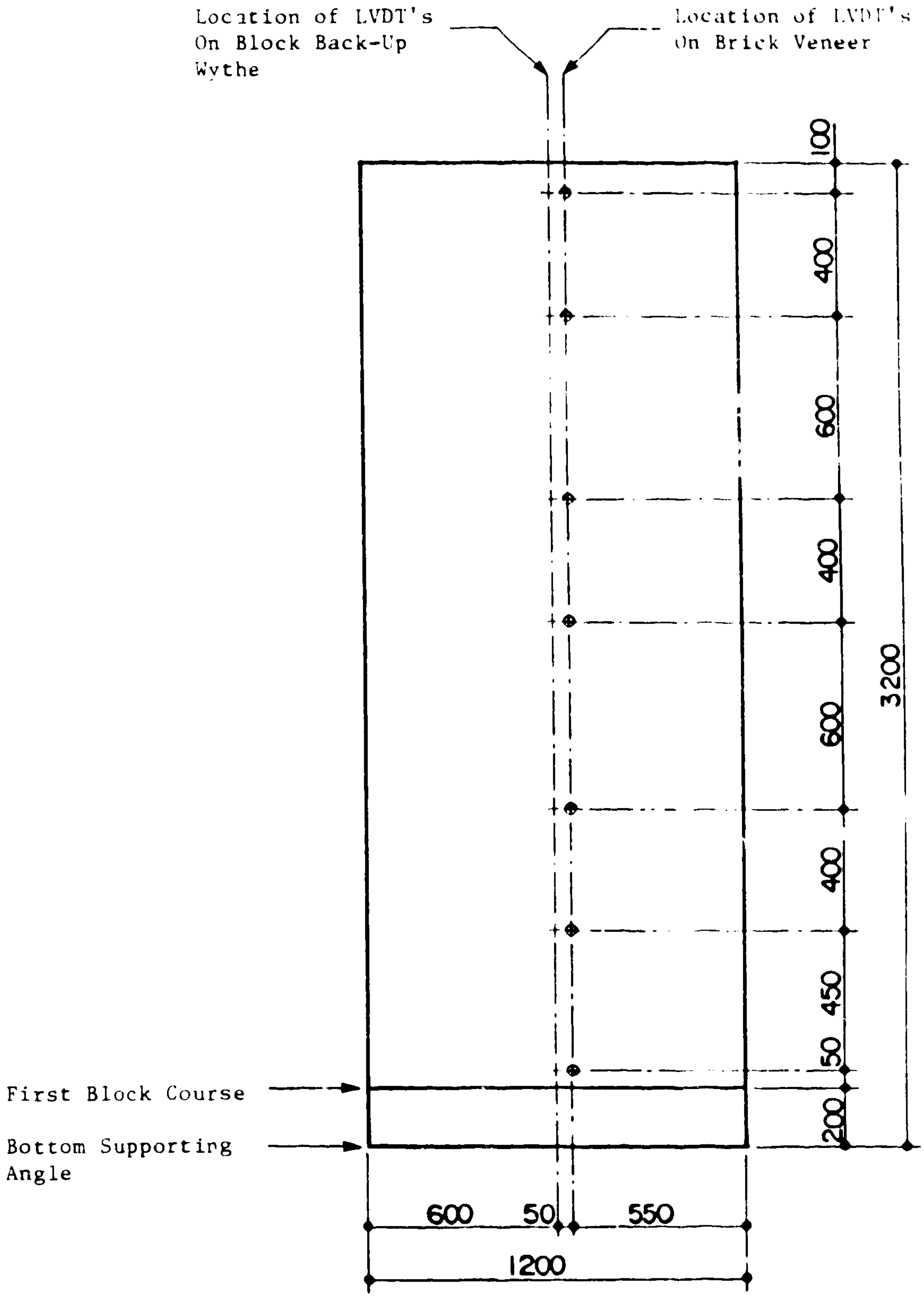


Figure 5.9 Location of the Measured Points of Deflection

Plate 5.1 Top Slab Details of Testing Frame

Plate 5.2 Bottom Slab Details of Tes[illegible]

Plate 5.3 Testing Frame

Plate 5.4 Conventional Connector Used in the Study

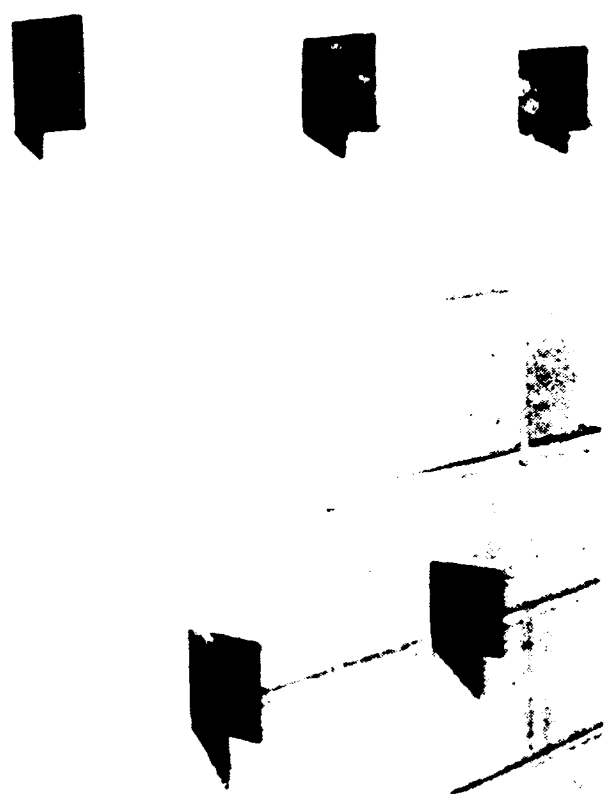


Plate 5.5 Shear Connectors Embedded in Block Wythe

Plate 5.6 Application of Insulation to Block Wythe

Plate 5.7 Insulation-Securing Rod Component of Shear Connector

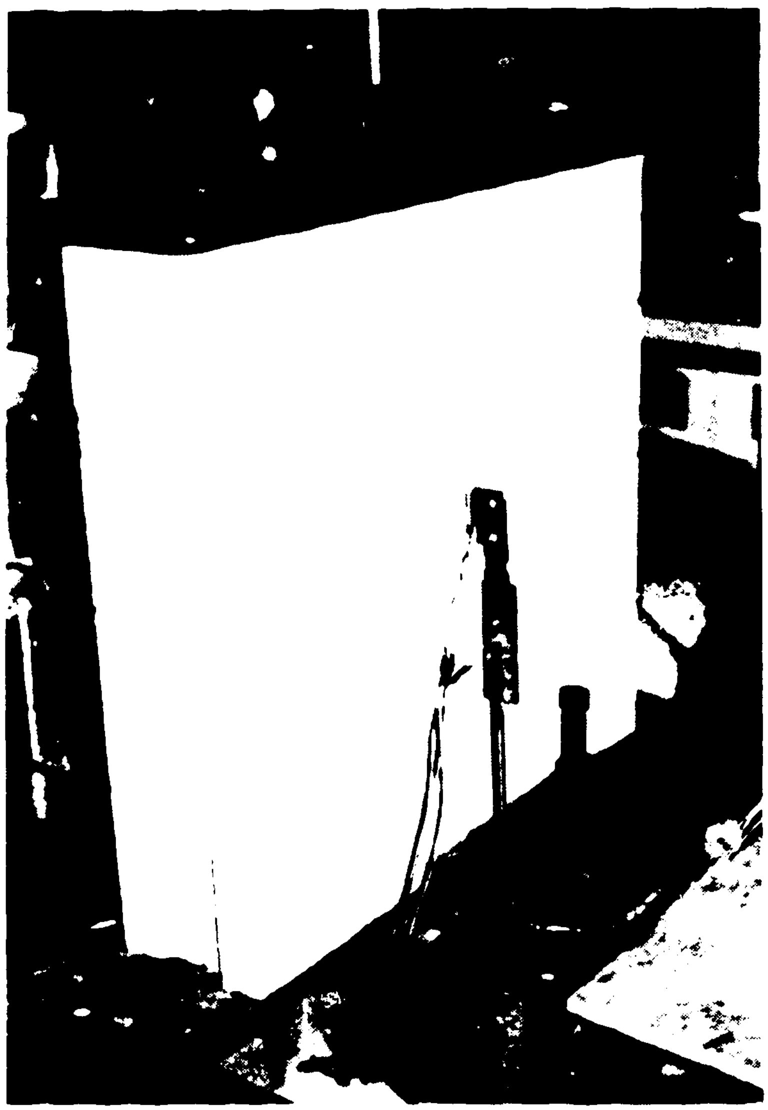


Plate 5.8 Shear Connector Prism Within the Testing Frame

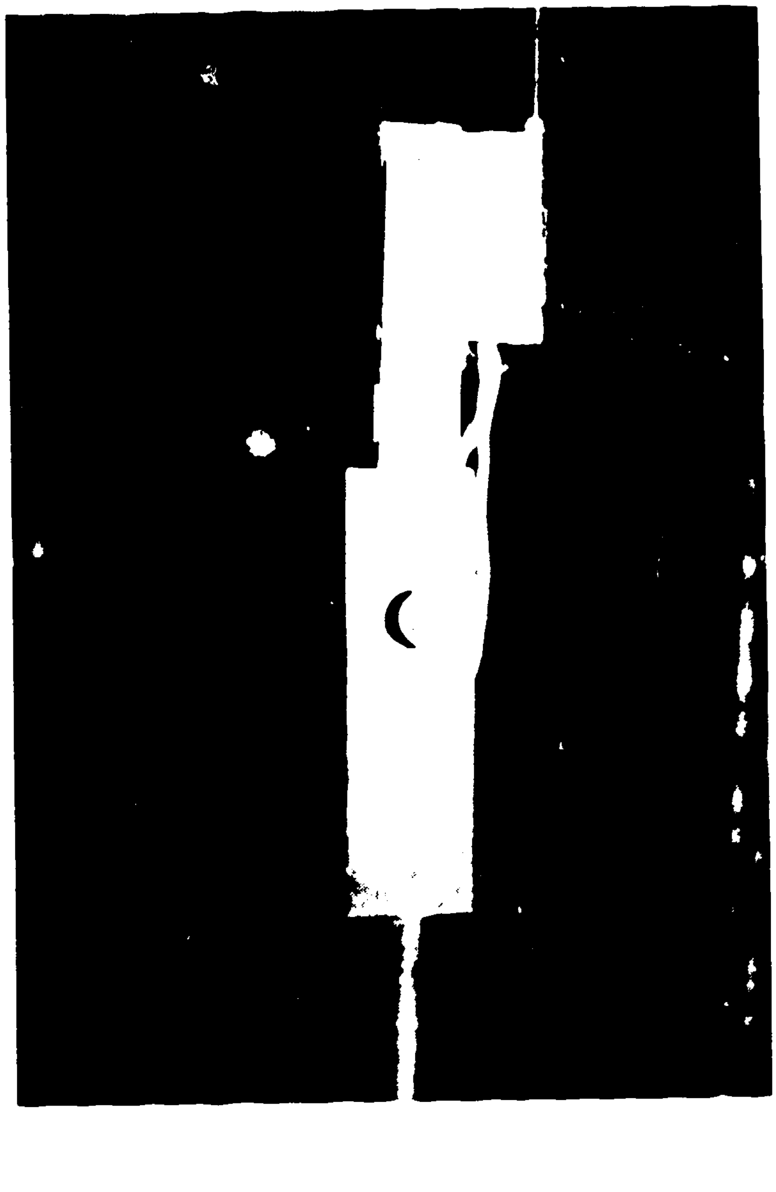


Plate 5.9 Load Transfer Mechanism Used in Shear Connector Prism Test

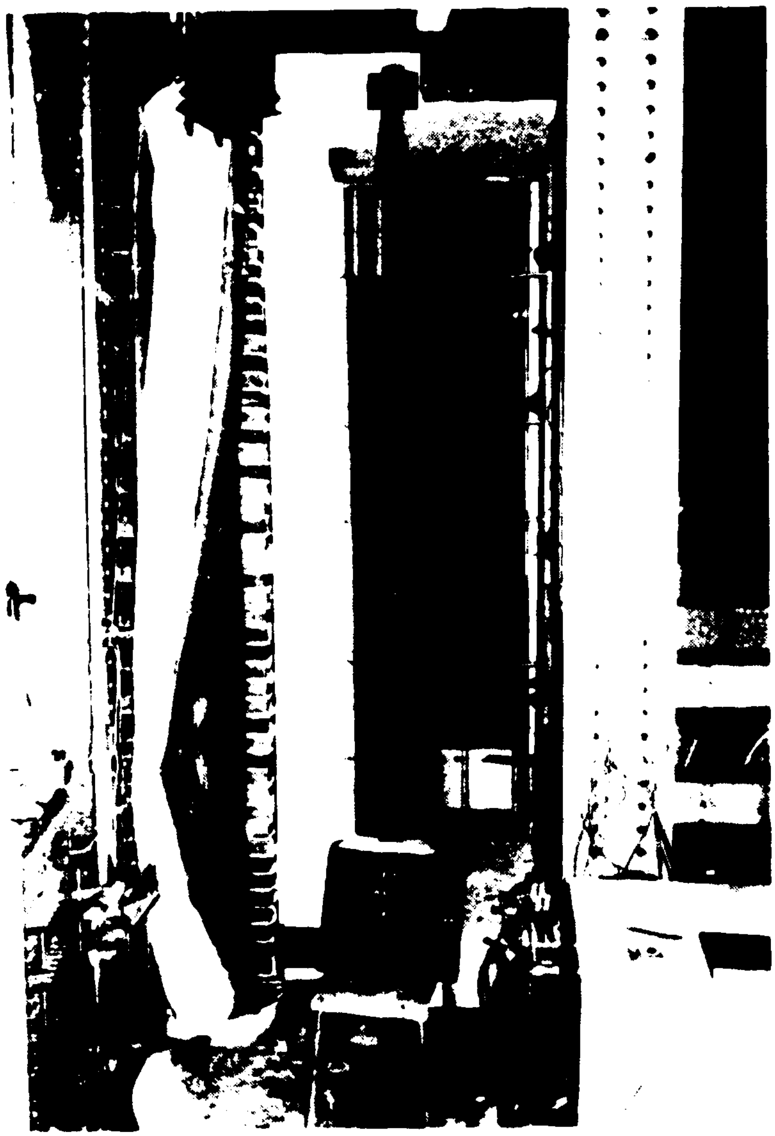


Plate 5.10 Wall Specimen Secured Within Testing Apparatus

6. EXPERIMENTAL RESULTS

6.1 Introduction

The testing program conducted on the prisms and full scale wall specimens, as described in Chapter 5, is summarized in this chapter in tabular, graphic, and photographic plate form.

6.2 Prisms

6.2.1 Modulus of Elasticity

6.2.1.1 Concrete Block Prisms

The testing of the three block prisms resulted in the stress strain curves of Figures A-1, A-2 and A-3, in Appendix A. From the slope of these plots, the elastic moduli were determined, as listed in Table 6.1. The average value of the block prism elastic modulus, E, was 3500 MPa.

6.2.1.2 Brick Units

The compressive strength tests on the individual brick units resulted in an average failure stress of 52.4 MPa, with a standard deviation of 4.6 MPa. Conforming to clause 5.3.3.3 of reference 1, the average brick unit compressive strength was 45.6 MPa. Given this unit strength, from Table 2 and Table 4 of this standard, with Type N mortar, a value of 14000 MPa was recommended to be used for the elastic modulus of the brick and mortar composite.

6.2.2 Shear Connector

The results of the testing of the three shear connector prisms, as described in section 5.2.2 are presented in Table 6.2. This table includes the holes in which the load was applied, the type of load, and the maximum load resisted by the connectors. The smallest failure loads, or limiting loads, were 2.45 kN in shear and 5.78 kN in axial compression. The average shear failure load was 3.83 kN.

The failure mode was consistently identified as the yielding of the metal plate around the hole in which the rod tie component of the connector was attached. As a result, up to six individual load cases were permitted to be applied to different holes of the same connector. Plate 6.1 presents a typical failure of a shear connector.

6.3 Full Sized Wall Specimens

6.3.1 General

The results of the full sized wall specimen testing program were used to compare back-up wall size, cavity width, connector type, connector pattern, and vertical reinforcing. The test results for each specimen were reduced to two types of diagrams. The first type were pressure versus centerline lateral deflection curves for both the brick and block wythes. The second type were deflection diagrams which displayed lateral deflections for various heights along the height of the wythes, at pressure intervals. Unless referred to in this chapter, all such diagrams may be found for each wall specimen in Appendix B.

Table 6.3 presents data on two locations for each of the pressure deflection curves: the points of yield strength and ultimate strength. For each point, the table lists the applied pressure, and the corresponding magnitude and elevation of the maximum lateral deflection which ocurred in the block wythe. According to R R Graham, Jr. et al. in the paper entitled "Glossary of Terms"[16], yield strength is defined as the stress at which a material exhibits a specified limiting deviation from the proportionality of stress to strain. Applying this term to cavity walls, yield strength shall refer to the location where the pressure-deflection curve deviates significantly. The point of ultimate strength is defined as the maximum pressure resisted by a wall specimen.

Unexpectantly high deflections at the base and at the top of the block wythe were observed in many of the specimens. At low pressures, significant deflections at the top of the block wythe could be attributed to the settling of the wall system against the intermediate lateral support attatched to the top slab. At higher pressures, excessive deflections were determined to be the result of the elastic conditions offered by this support. Large deflections experienced at the base of the block wythe ocurred as a result of a crack forming in the mortar joint at the block/concrete slab interface. Although measures were taken to avoid such damage, several of the specimens acquired such a crack during transport to the testing apparatus. Large lateral slippage of the base would take place when the lateral pressure forces would exceed those maintaining the integrity of the interface.

In general, all failures occurred by separation of the units from the mortar bed joint as a result of tension failure between the two materials. All such failures presented in this chapter shall be simply referred to as being cracks in a particular wythe, observed at a given height above the bottom supporting angle.

6.3.2 Series 1

6.3.2.1 S1W1

This wall system consisted of a 150 mm wide block back-up wall, a cavity made up of 50 mm rigid plank styrofoam insulation and a 25 mm air gap, and 90 mm wide clay brick units. The connector type implemented was 3.66 mm diameter wire truss joint reinforcement, and was spaced 600 mm vertically.

The pressure vs centerline lateral deflection diagram and the block wythe deflection diagram are given in Figures 6.1 and 6.2 respectively. The brick wythe deflection diagram is presented in Figure B-1 of Appendix B.

The wall system deflected proportionally with the load approximately up to a pressure of 0.58 kPa. At this point, the rate of deflection per load increment increased and then remained constant up until a pressure of 0.86 kPa, where a tensile failure crack in the block wythe opened at approximately midheight. Cracks opened in the brick veneer 200 mm above and below the crack at 1200 mm in the block wall. Plate 6.2 shows a cross-sectional view of the cracked region. The cracks acted as hinges so that the wythes rotated about these hinges, thus transferring a larger proportion of the load to the region of the cracks. This rotation caused the block wall to bear up on the top slab of the testing frame. The assembly was failed at 1.10 kPa. At failure, the maximum lateral deflections of 4.2 mm and 4.3 mm occurred at heights of 1100 mm and 1700 mm in the brick and block wythes, respectively. The pressure vs deflection curve remained relatively steep throughout, suggesting a somewhat brittle failure.

Unusually large deflections at the top of both wythes were observed throughout the testing of the specimen, as displayed in Figure 6.2. Small lateral slippage of the block wythe base also took place. Explanations for the above is documented in section 6.3.1.

Generally, both wythes deflected equally to a pressure of approximately 0.75 kPa, at which time the brick veneer took on a greater deflection. This suggested a deformation of the wire reinforcement. Upon disassembly of the failed wall system, Plate 6.3 exhibits how the wire reinforcement was found to be deformed.

6.3.2.2 S1W2

Cavity wall system S1W2 was identical to S1W1 except that it used 4.76 mm diameter wire truss joint reinforcement. As expected, the failure mode was also similar.

Referring to the pressure-deflection relationship of Figure 6.3, there is a discrepancy in lateral deflection between the two wythes starting at 0.1 kPa. This discrepancy remains constant between the pressures of 0.2 and 0.6 kPa. An explanation for this difference is lateral settling of the wall system.

The wall system deflected proportionally with load up to 0.68 kPa. At this point, a tension failure crack in the block wall formed at a height of 1800 mm. As with wall S1W1, cracks propagated in the brick veneer 200 mm above and below this 1800 mm height. Upon further loading, the deflection per incremental load ratio was increased. The cracks opened wider until the maximum failure pressure of 0.75 kPa was reached. The maximum lateral deflections of 1.4 mm and 1.2 mm at a height of 1700 mm occurred in the brick and block wythes, respectively. A brittle failure was observed.

As with specimen S1W1, significant lateral deflections as described in section 6.3.1 were experienced at the top of both wythes. The deflection diagrams for both wythes are included in Appendix B, Figures B-2 and B-3.

Similar to S1W1, the yielding failure of the wire reinforcement was suggested by the increase in the brick veneer deflections over those of the block wythe. This phenomenon first appeared at a pressure of 0.68 kPa. Disassembly of the wall system confirmed this failure.

6.3.2.3 S1W3

Specimen S1W3 differed from S1W1 and S1W2, because it employed shear connectors. The results of the testing are plotted in Figures 6.4, 6.5, and B-4.

Up to a pressure of 1.22 kPa, S1W3 behaved similarly to walls S1W1 and S1W2. As can be seen in Figure 6.4, this specimen exhibited elastic behaviour until the occurrence of a tension failure crack in the block wythe at a pressure of 0.6 kPa. The crack was at a height of 2200 mm as measured above the bottom angle support. Cracking in the brick veneer was detected shortly after at a height of 2250 mm. When loading was resumed, the lateral deflection to load increment ratio had increased. At pressures higher than 1.23 kPa, the behaviour of S1W3 departed from that of S1W1 and S1W2. A larger deflection with very little pressure increase was experienced, followed by an unexpected gain in resistance preceding failure. Immediately prior to failure, cracking of the brick veneer was observed at heights of 2300 mm and 1800 mm.

The wall failed at a pressure of 1.38 kPa with a maximum deflection of 15 mm at a height of 2100 mm. Both wythes deflected equally throughout the testing. Upon disassembly of the failed specimen, the shear connectors were found to be undamaged.

Referring to the block wythe deflection diagram of Figure 6.5, significant lateral deflections took place at the top and bottom of the block wythe. These deflections are accounted for in section 6.3.1.

Unlike the previous two specimens, the failure was not brittle. The specimen exhibited a loading curve plateau at a pressure of 1.23 kPa, followed by an increased gain in resistance. This suggests an increase in ductility of a wall system by the use of a shear-resisting connector.

6.3.2.4 S1W4

The configuration of specimen S1W4 only differed by a 25 mm increase in cavity width over that of S1W3. Both S1W3 and S1W4 incorporated shear connectors in their design.

Specimen S1W4 behaved elastically up to a pressure of 0.95 kPa, at which time a tension failure crack formed at a height of 1500 mm in the block back-up wall. Apparent by the change in the slope of the pressure-deflection curve of Figure 6.6, the deflection to incremental load ratio abruptly increased and remained constant up to a pressure of 1.6 kPa. Cracks in the brick veneer were then noticed at a height of 1000 mm and 1800 mm. Increased loading resulted in very large deflections up until the failure pressure of 1.75 kPa was reached. Plate 6.4 shows the tensile crack in the block wythe at failure. The maximum deflections occurred at 1700 mm (mid-height), and were 5.3 mm and 5.2 mm for the brick and block wythes, respectively.

Significant lateral deflections at the top of both wythes and a small amount of block wythe base slippage can be seen from the block wythe deflection diagram of Figure 6.7. As is the case with all of the specimens, the brick wythe deflection diagram(Figure B-5 of Appendix B), is nearly identical to that of the block wythe.

After disassembling the failed specimen, the shear connectors were found to be undamaged, still rigidly attached to both wythes.

6.3.3 Series 2

6.3.3.1 S2W1

Specimen S2W1 was comprised of a 200 mm concrete block wall, a cavity made up of 50 mm rigid plank insulation and a 25 mm air gap, and a 90 mm clay brick veneer. The connector used was a 3.66 mm diameter wire truss joint reinforcement. Test data is reduced into three plots, Figures B-6, B-7, and B-8 as found in Appendix B.

Typical elastic behaviour was observed up to a positive lateral pressure of 1.0 kPa. At this load level, a crack was observed in the block wythe at a height of 2000 mm. Further loading resulted in an increased rate of deflection per load increment. The crack widened, forming a hinge, allowing the wythes to rotate about the crack. Small lateral deflections at the top of the wythes indicate an elastic lateral support in this region(Figures B-7 and B-8). The wall system reached a pressure of 1.24 kPa, after which an increase in pressure could no longer be sustained. At this pressure, the maximum deflections were 1.37 mm at a 1700 mm height, and 1.54 mm at a 2100 mm height, for the brick and block wythes, respectively.

The failure was classified as being brittle. As with specimens S1W1 and S1W2, the reinforcement in this test was also found to be deformed in the cavity region.

6.3.3.2 S2W2

Wall specimen S2W2 differed from S1W3 only by having its outermost cores reinforced.

As indicated by the linear pressure-deflection relationship of Figure 6.8, wall system S2W2 appeared to behave elastically up to a pressure of 2.6 kPa. The rate of deflection per load increment increased gradually up to a pressure of 2.9 kPa. Although no cracking was observed, the abrupt increase in deflection as seen in Figure 6.9 suggests that a crack did form. The rate of deflection per load increment increased up until a pressure of 3.2 kPa. At this load, the middle block was punched out of the top course of the back-up wythe by the intermediate supporting angle. Plate 6.5 shows the dislodged block. The midheight lateral deflection of the brick veneer and block wythe just prior to puncning failure was 6.6 mm and 6.2 mm, respectively.

A 3200 mm long piece of supporting angle was placed along the top of the block wall, resting tightly against the intermediate lateral support as shown in Plate 6.6. The angle distributed the load from the lateral support onto the two remaining blocks on the top course, enabling the test to continue. However, as a result of the repairs and adjustments made to the wall, further deflections could only be regarded within an estimated accuracy of 10%.

Load was reapplied to the specimen. The ultimate failure pressure of S2W2 was 4.64 kPa with a maximum lateral deflection of approximately 24 mm. Prior to failure, cracks appeared in virtually every mortar bed joint in the block back-up wall. Cracking in the brick wall was hard to detect due to overlapping of the air bag. High slippage of the base of the block wythe was experienced in the post-punching failure phase of the test.

Shifting of the base of the block wythe was detected throughout the test, although the deflections did not become excessive until after the block punchout failure. This increased rate of shifting can be attributed to sudden shifting of the wall system when this failure occurred.

The wall displayed tremendous ductility, surviving far beyond the elastic range. Disassembly of the failed wall system revealed the early stages of yielding of the metal around the hole on the free end of shear connector plate. Plate 6.7 shows this yielding.

6.3.3.3 S2W3

Cavity wall system S2W3 was identical to S2W1, except that it implemented shear connectors instead of conventional reinforcement.

With the exception of a small settling deflection occurring at a pressure of 0.2 kPa, elastic behaviour was observed up to a load of 1.1 kPa(see Figure 6.10). At this pressure, a crack at a height of 2200 mm opened in the block wythe. Further loading resulted in an increased deflection per load increment ratio. At a pressure of 1.37 kPa, cracks were observed at heights of 2800 mm and 1600 mm in the brick veneer. Continuation of the test resulted in high lateral deflections, and eventually system failure at a pressure of 1.45 kPa. The maximum deflections occurred at a height of 1700 mm, being 11.9 mm and 12.8 mm for the brick veneer and block wythe, respectively.

During preloading of this wall, significant deflections were recorded. These deflections were different from the settling and shifting deflections as observed in previous tests. The deflections occured at very low pressures, and were found to progress along the entire height of the wall system(refer to Figure 6.11). This phenomenon presents serious doubt as to the validity of this specimen's deflection results. Therefore, interpretation of its results will be approached cautiously.

6.3.3.4 S2W4

This specimen served to compare the effects on varying cavity width on wall systems using shear connectors. It differed from S2W3 only by a 25 mm increase in cavity width.

Specimen S2W4 behaved elastically up to a pressure of 0.9 kPa. An increase in the lateral deflection per incremental load was then observed. This rate remained constant up to a pressure of 1.4 kPa, when a crack opened in the block back-up wall at a height of 2200 mm. The rate of deflection increased slightly until another crack formed at a height of 1600 mm. From this pressure of 1.75 kPa, the rate of deflection increased dramatically until the specimen failed at a pressure of 2.49 kPa. The maximum lateral deflection at this level was 16.2 mm at a height of 2100 mm and 15.5 mm at a height of 1700 mm, for the brick veneer and the block wythe, respectively.

Specimen S2W4 also exhibited slippage at the base of the two wythes, as can be seen in Figure 6.12. Explanations for this behaviour are discussed in 6.3.1.

The pressure-deflection relationship and the block wythe deflection diagram of this specimen were very similar to those of S2W2, prior to its punchout failure. The wall failed in a ductile manner. Plate 6.8 shows an undamaged shear connector exposed as the wall was being torn down.

Masonry Material Tests

Series	Wall No.	Mortar Cube Compressive Strength[1] (MPa)	Block Wythe Elastic Modulus (MPa)
1	1	21.7	-----
1	2	21.1	-----
1	3	14.8	-----
1	4	10.4	-----
2	1	6.5	-----
2	2	7.1	4400
2	3	7.3	2300
2	4	9.3	3800

[1] Average Compressive Strength of the mortar cubes for each wall.

Table 6.1 Summary of the Masonry Material Tes.

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

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

Table 6.2 Shear Connector Prism Test Results

Name	Yield Strength[1]			Ultimate Strength[2]		
	Lateral Pressure (kPa)	Maximum Deflection[3] (mm)	- Height[4] (mm)	Lateral Pressure (kPa)	Maximum Deflection (mm)	- Height (mm)
S1W1	1.17	3.5	1700	1.19	4.3	1700
S1W2	0.69	0.9	3100	0.75	1.2	1700
S1W3	1.21	1.7	2100	1.38	15.0	2100
S1W4	1.68	2.2	2100	1.75	5.2	1700
S2W1	1.00	0.5	1100	1.24	1.5	2100
S2W2	2.90	3.7	1700	3.20	6.2	1700
S2W3	1.37	2.8	2100	1.45	12.8	1700
S2W4	1.80	1.7	2100	2.49	15.5	1700

[1] Pressure at which the slope of the pressure vs lateral deflection curve deviates significantly.
[2] Maximum pressure resisted by the wall specimen.
[3] Maximum lateral deflection of block wythe at the given pressure.
[4] Height above supporting angle corresponding to the maximum lateral deflection.

Table 6.3 Summary of Full Sized Wall Specimen Test Results

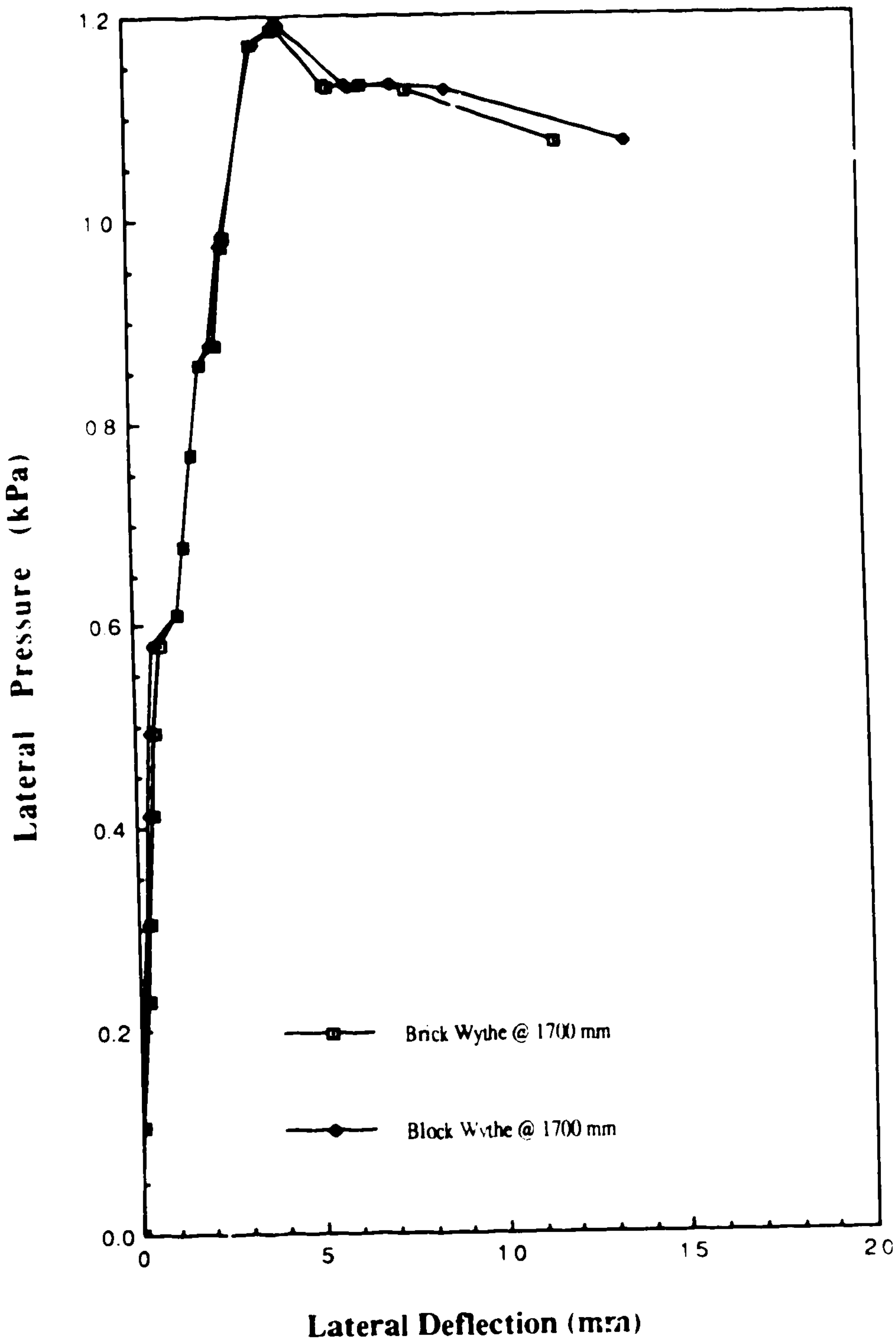


Figure 6.1 Pressure vs Centerline Lateral Deflection for S1W1

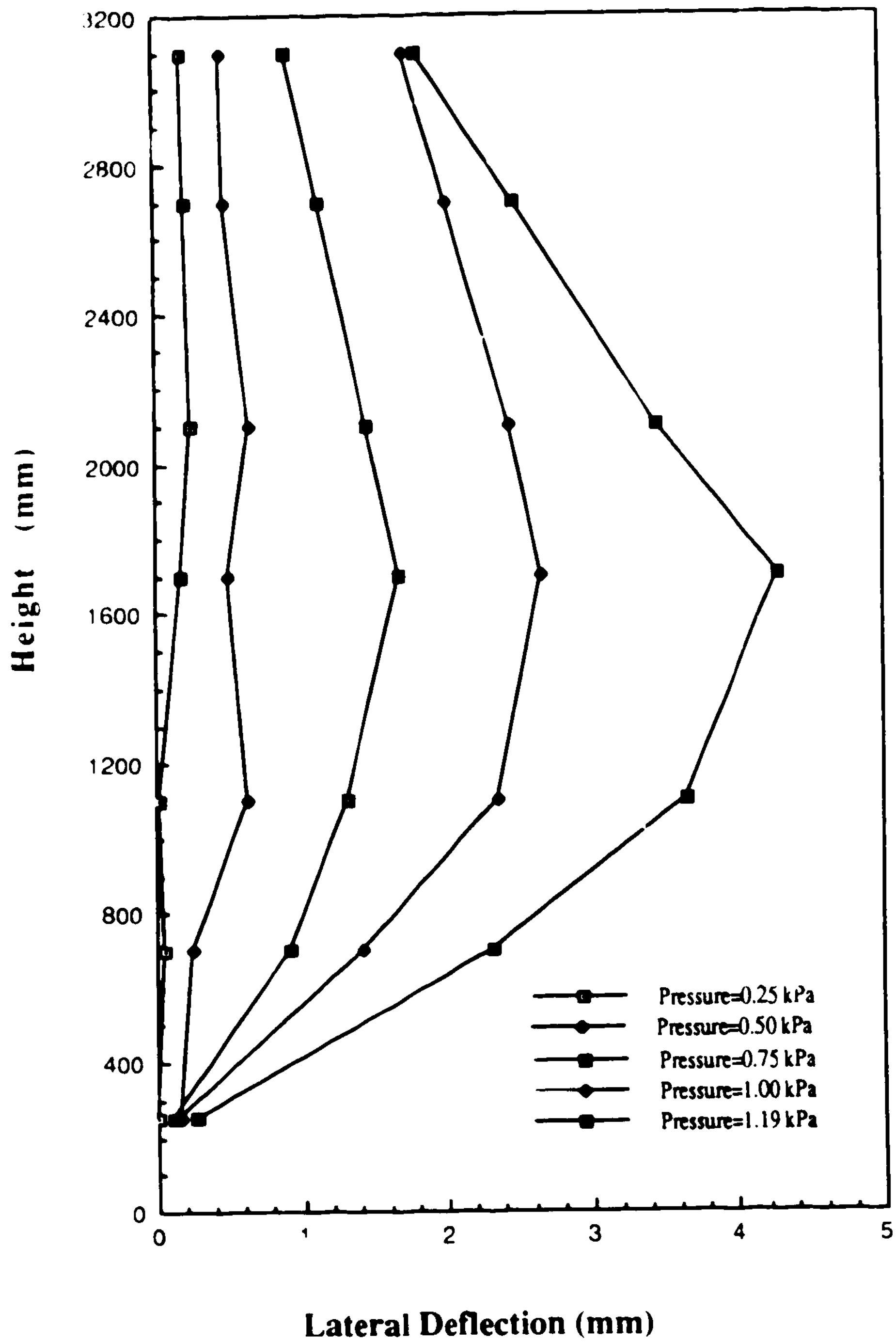


Figure 6.2 Block Wythe Deflection for S1W1

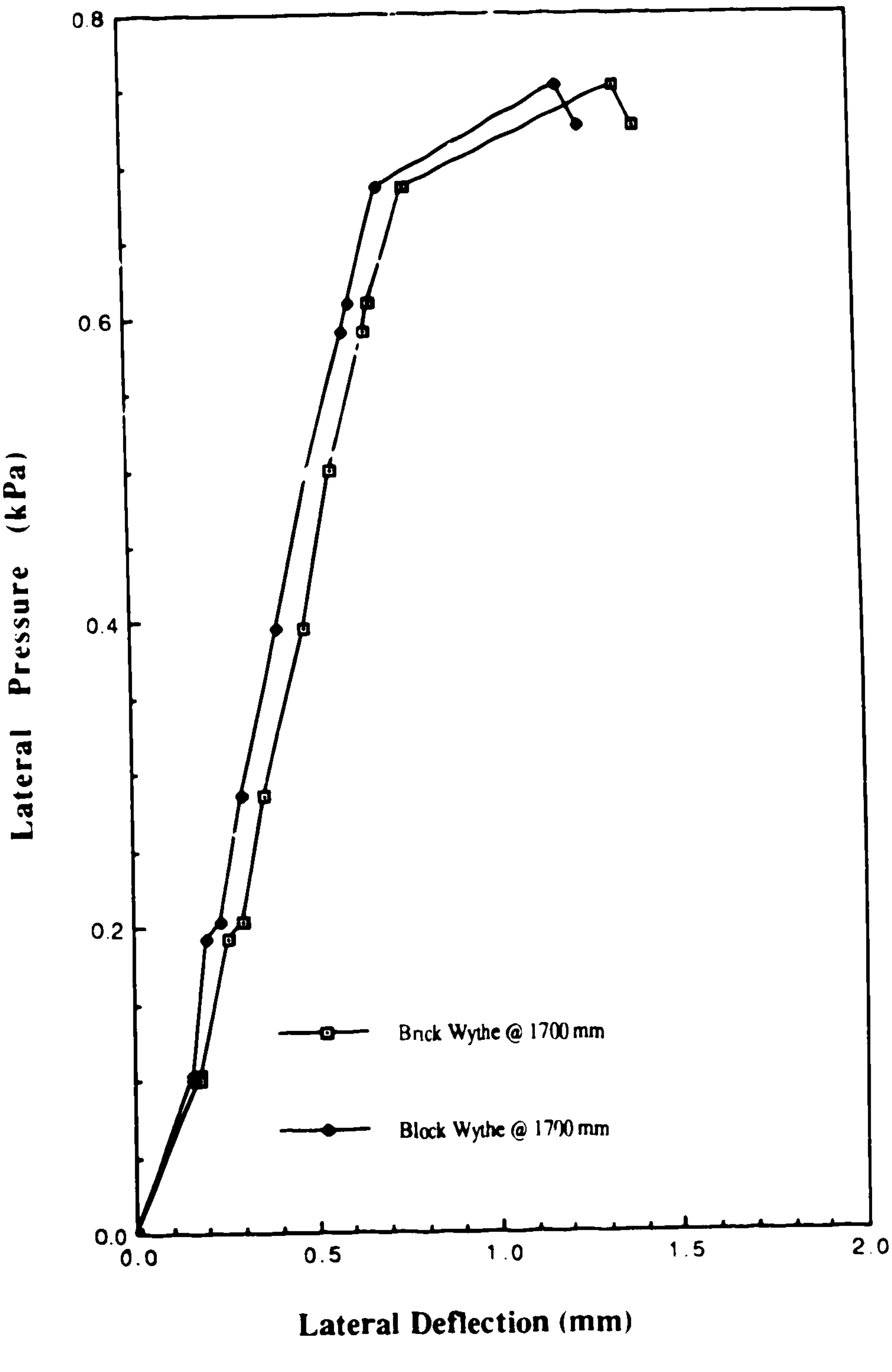


Figure 6.3 Pressure vs Centerline Lateral Deflection for S1W2

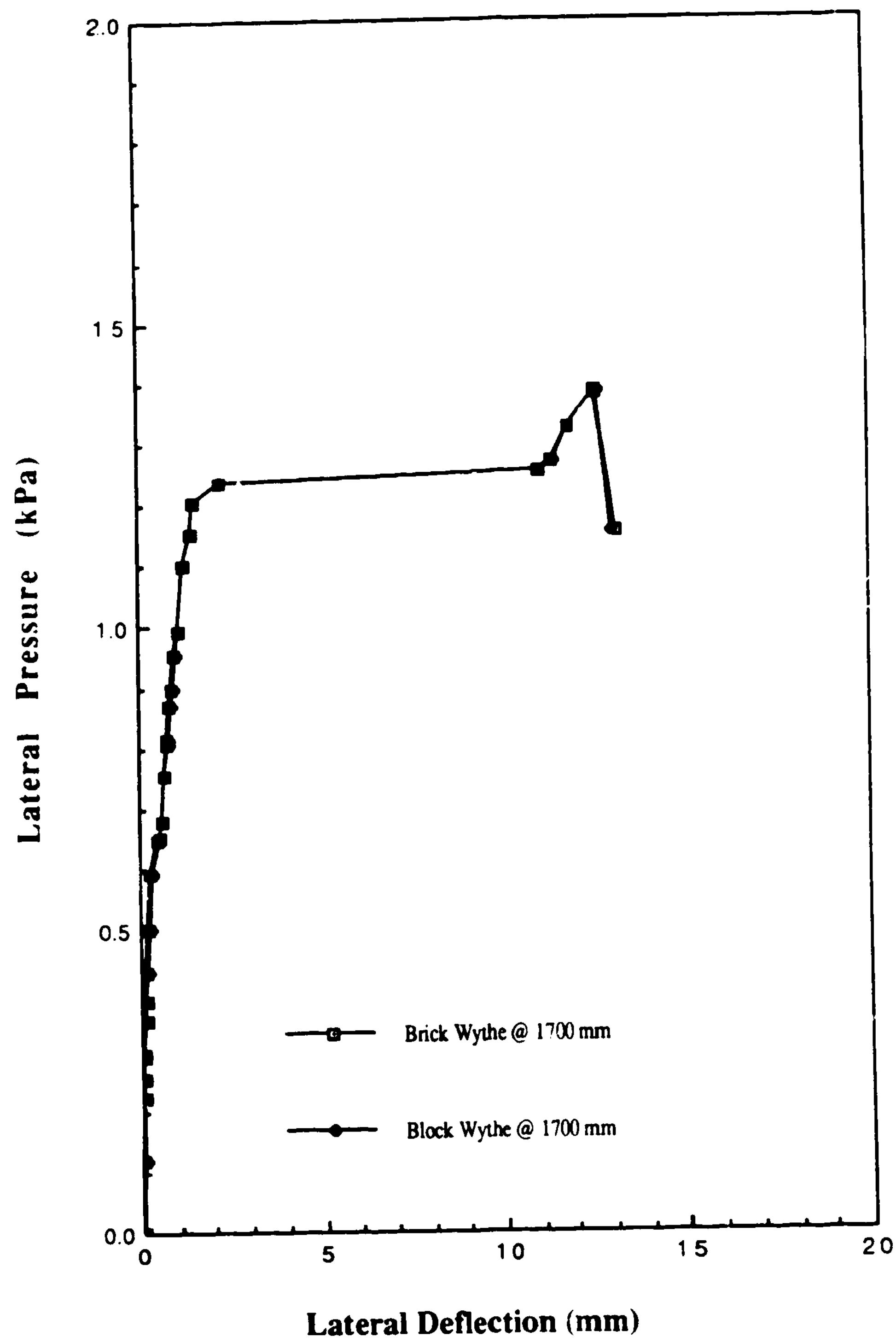


Figure 6.4 Pressure vs Centerline Lateral Deflection for S1W3

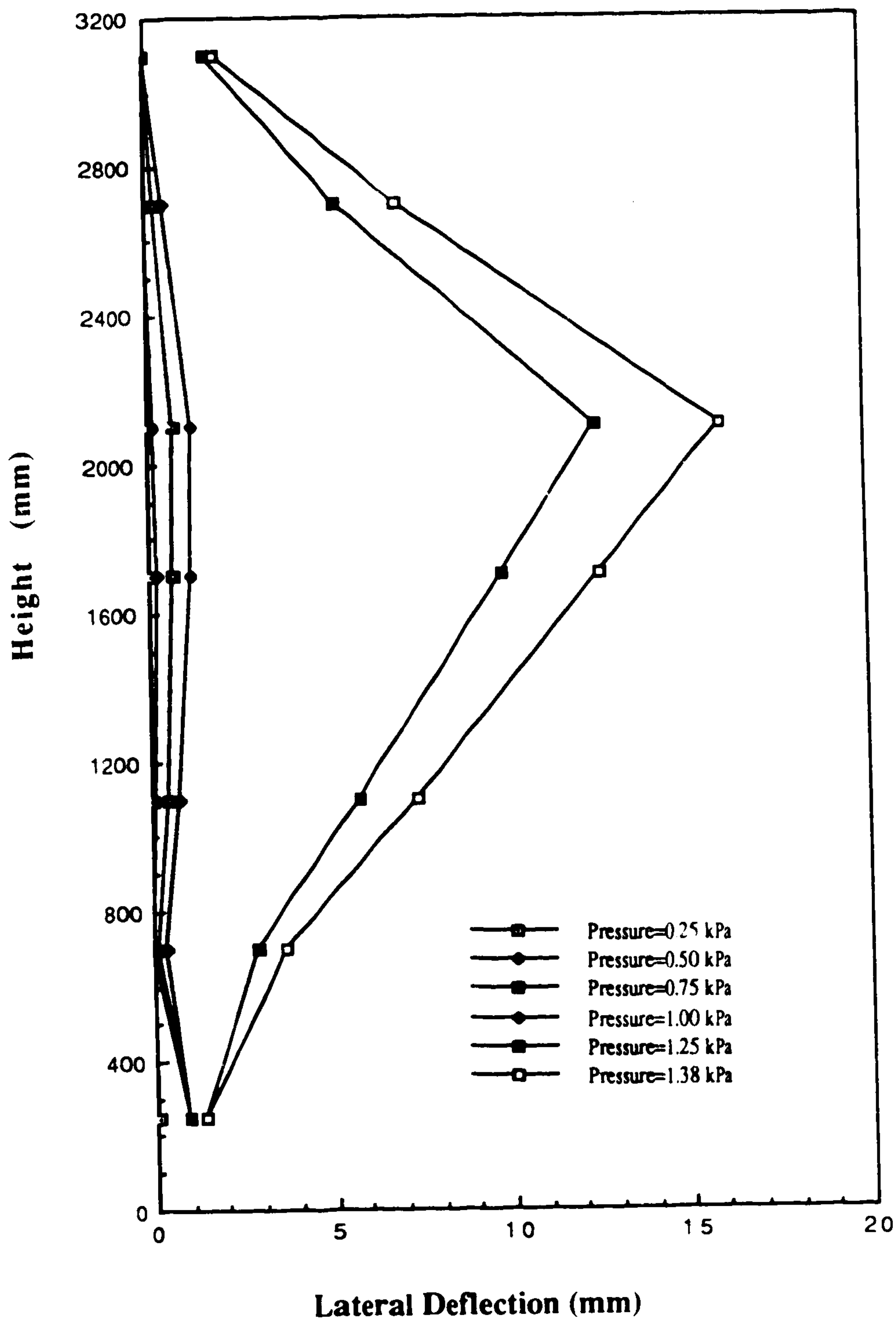


Figure 6.5 Block Wythe Deflection for S1W3

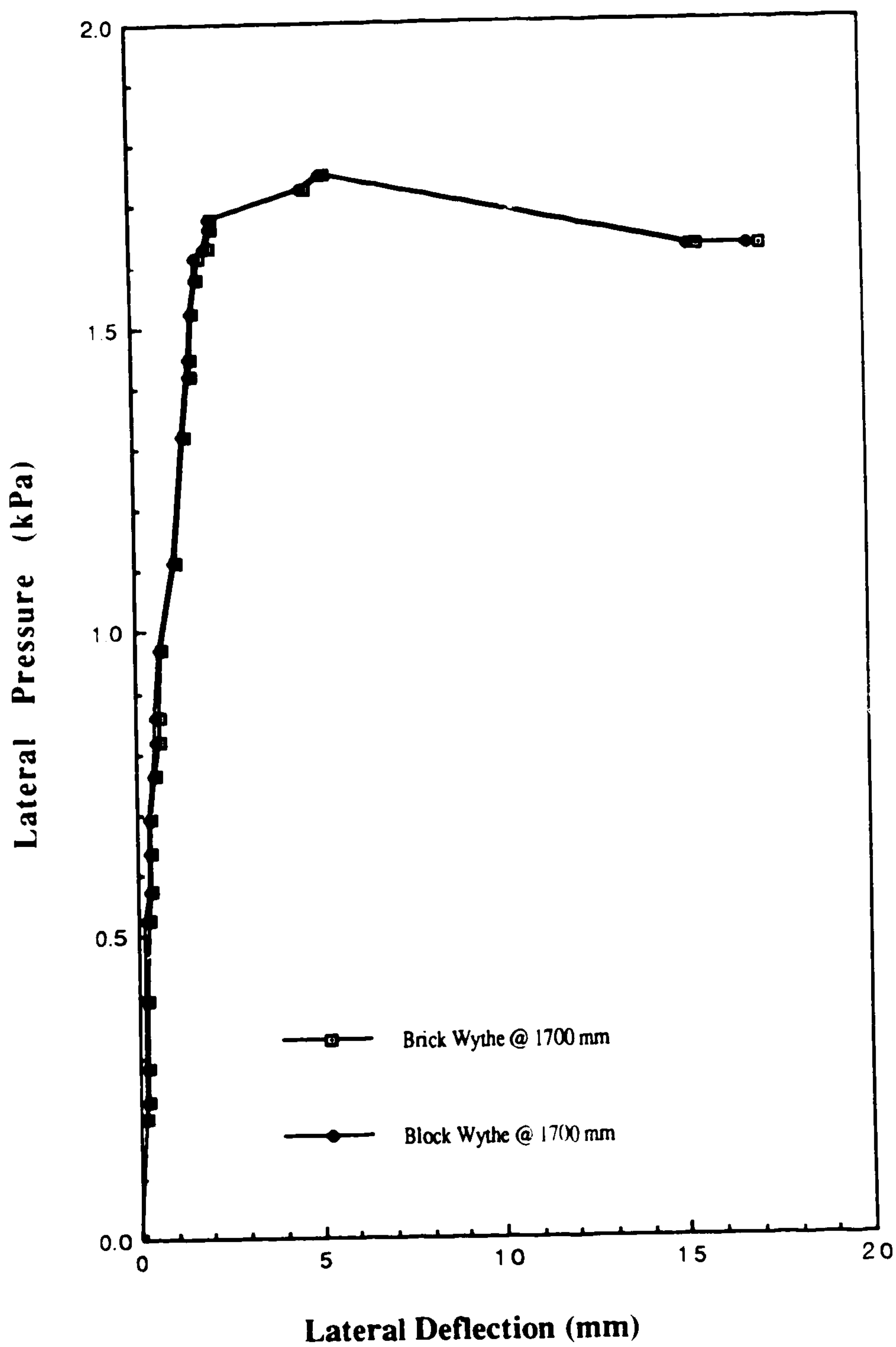


Figure 6.6 Presssure vs Centerline Lateral Deflection for S1W4

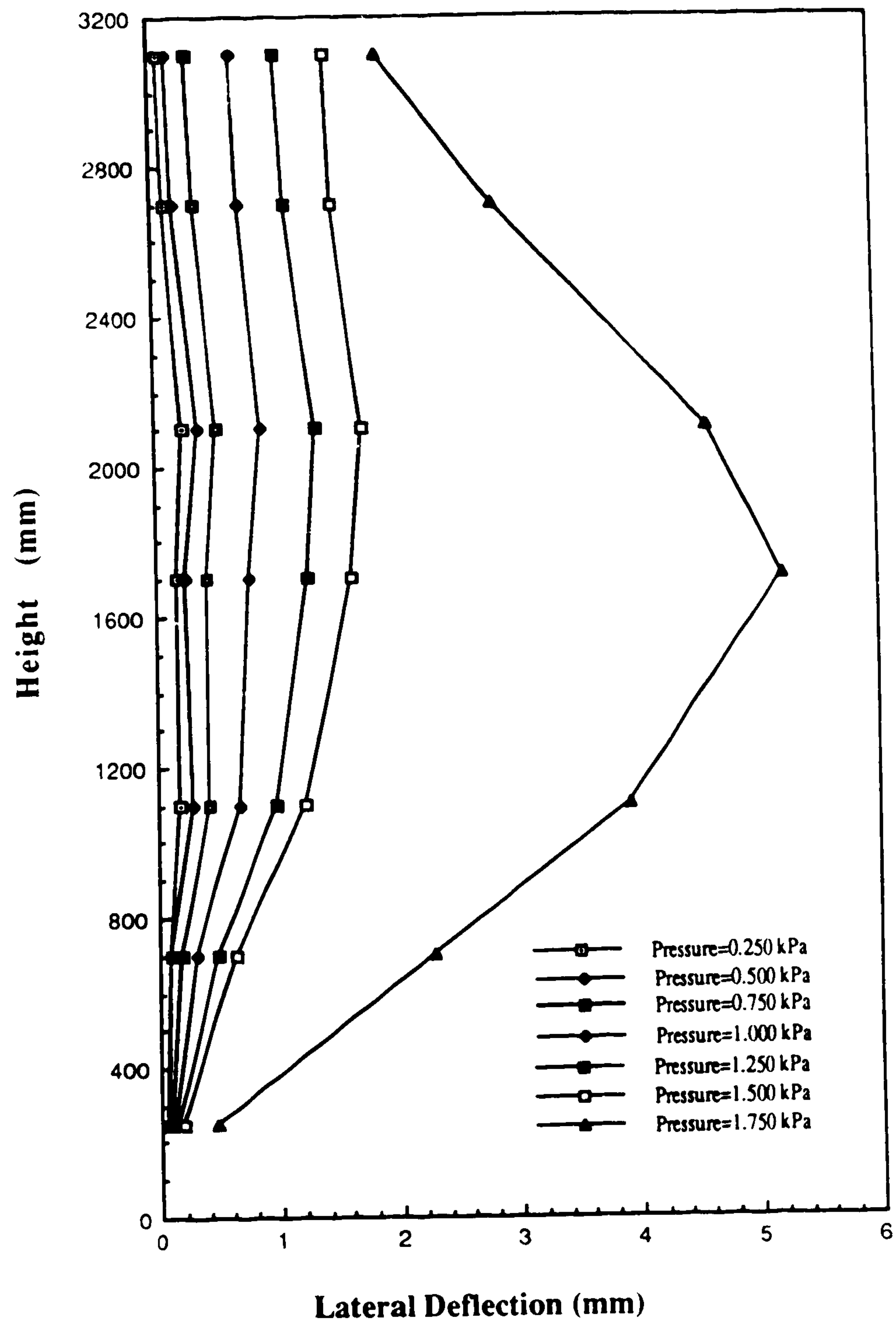


Figure 6.7 Block Wythe Deflection for S1W4

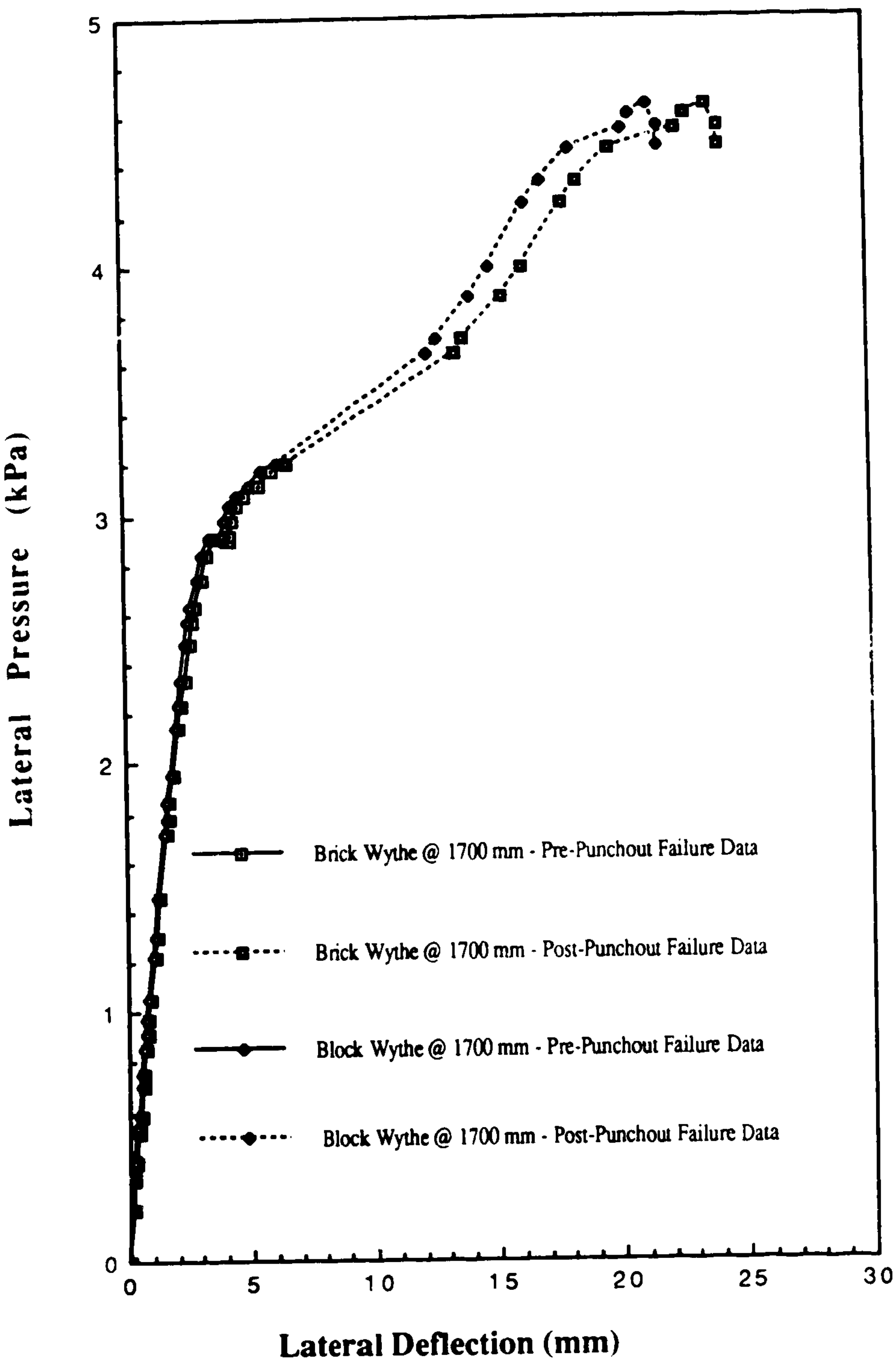


Figure 6.8 Pressure vs Centerline Lateral Deflection for S2W2

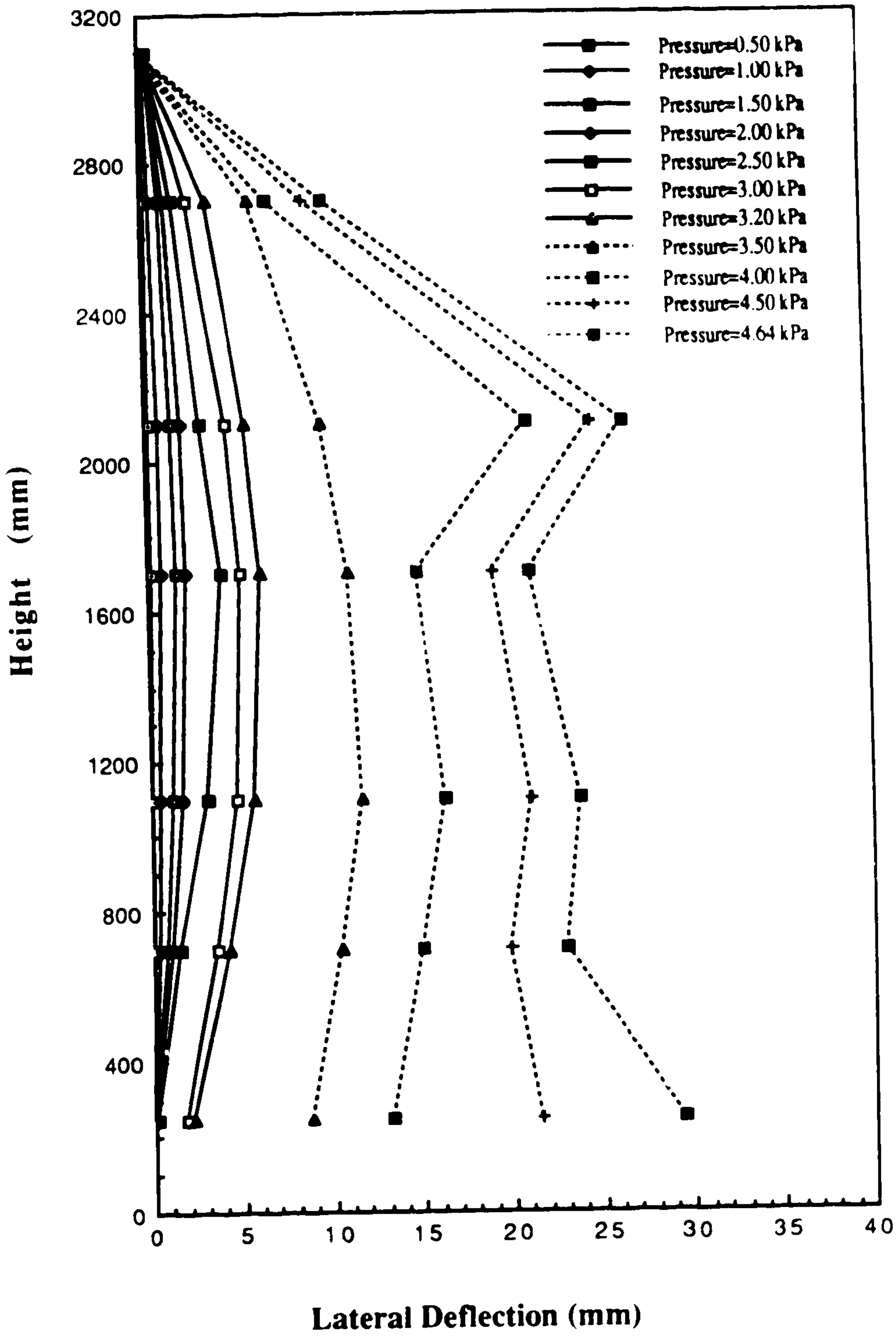


Figure 6.9 Block Wythe Deflection for S2W2

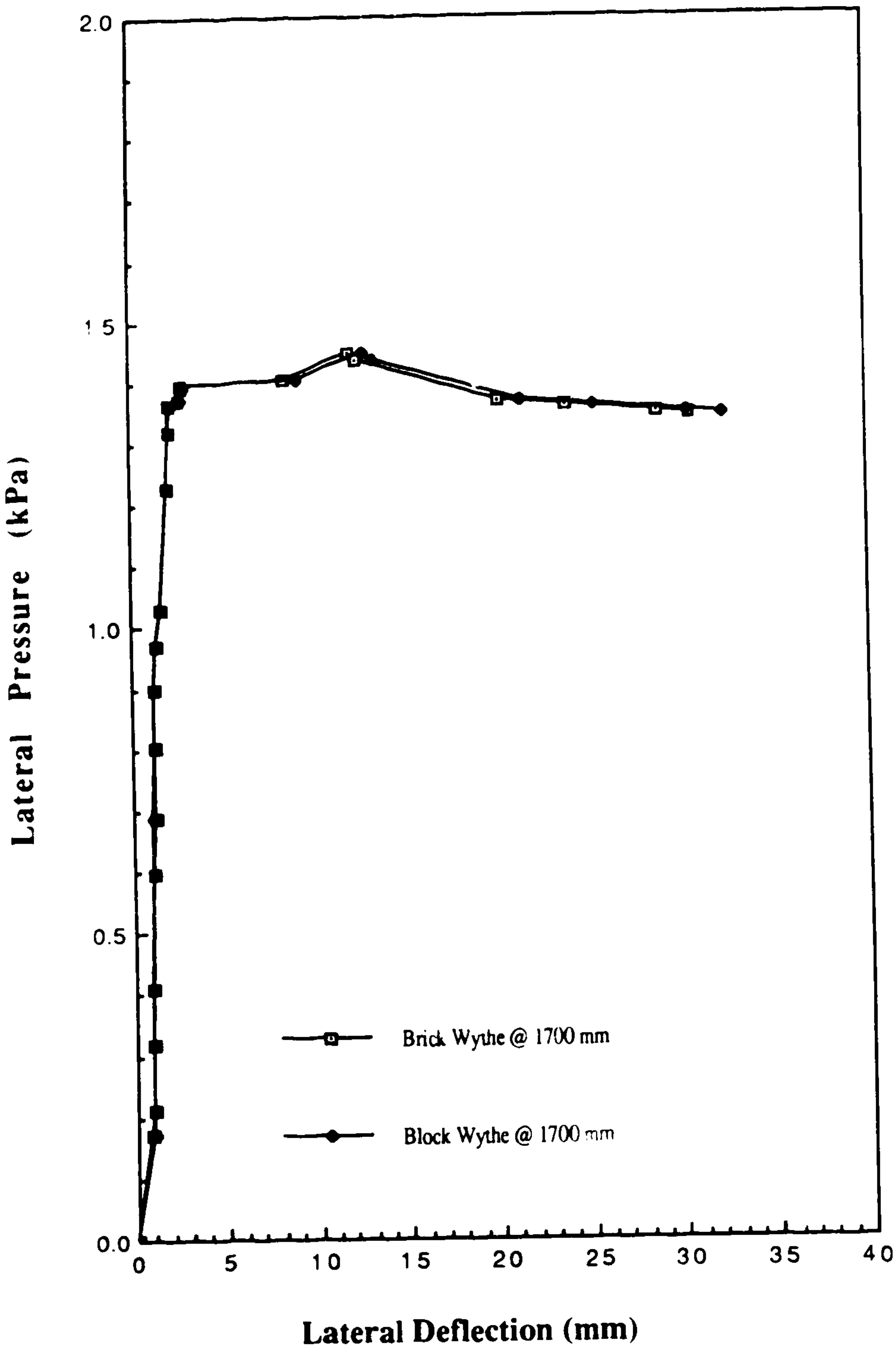


Figure 6.10 Pressure vs Centerline Lateral Deflection for S2W3

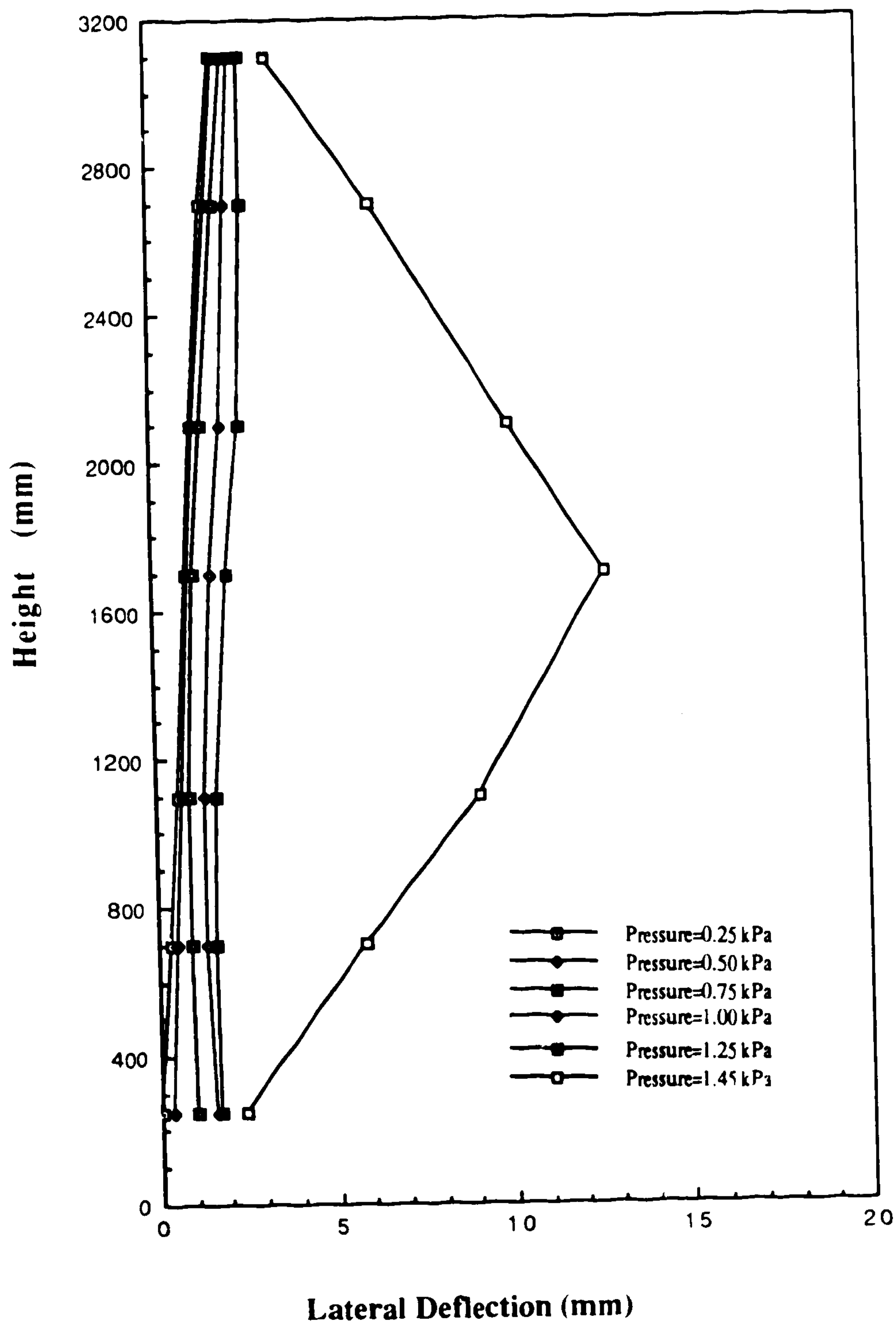


Figure 6.11 Block Wythe Deflection for S2W3

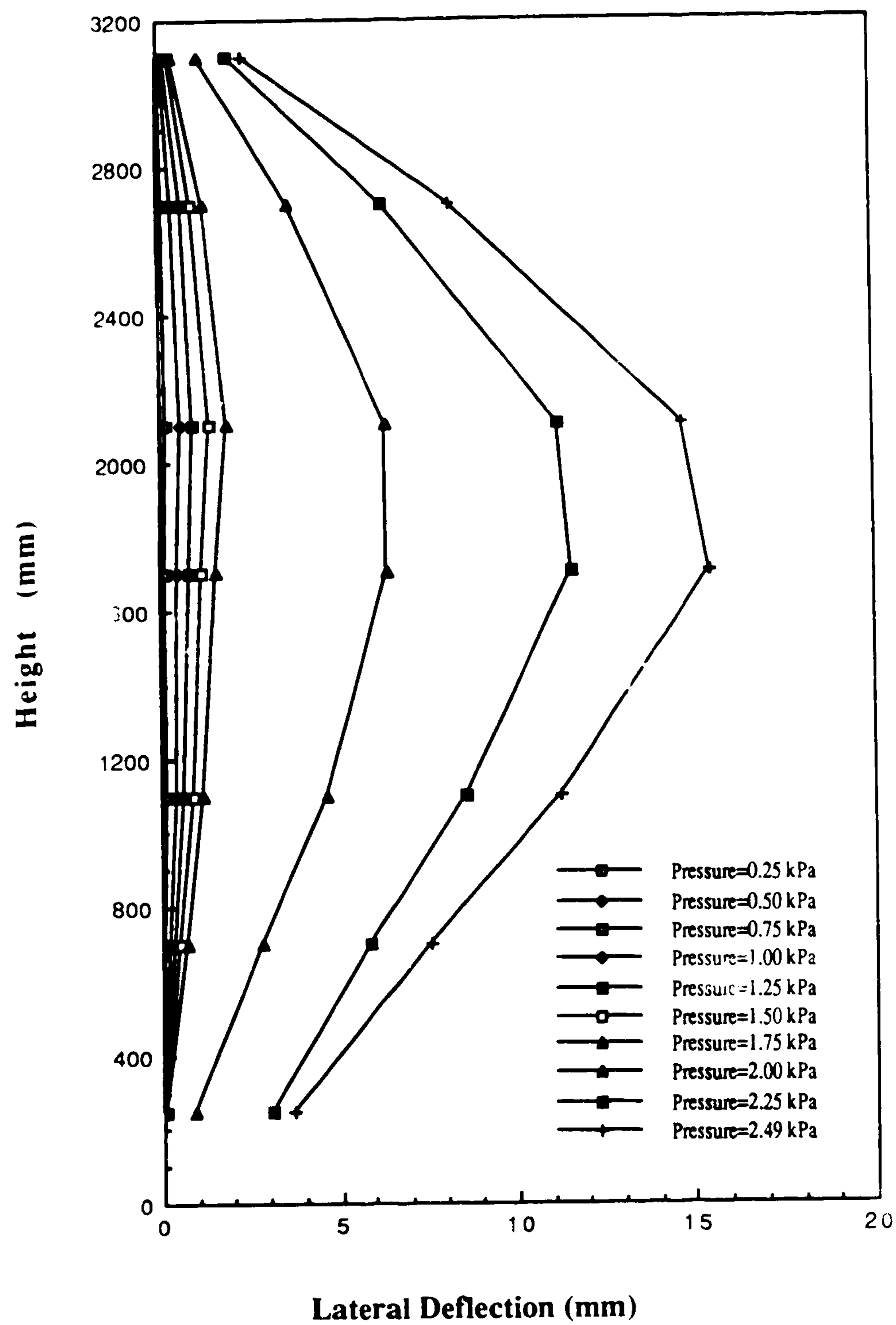


Figure 6.12 Block Wythe Deflection for S2W4

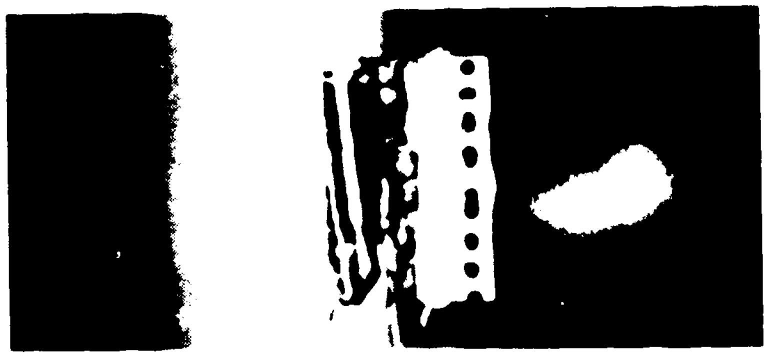


Plate 6.1 Yielding Failure of Shear Connector From Prism Tests

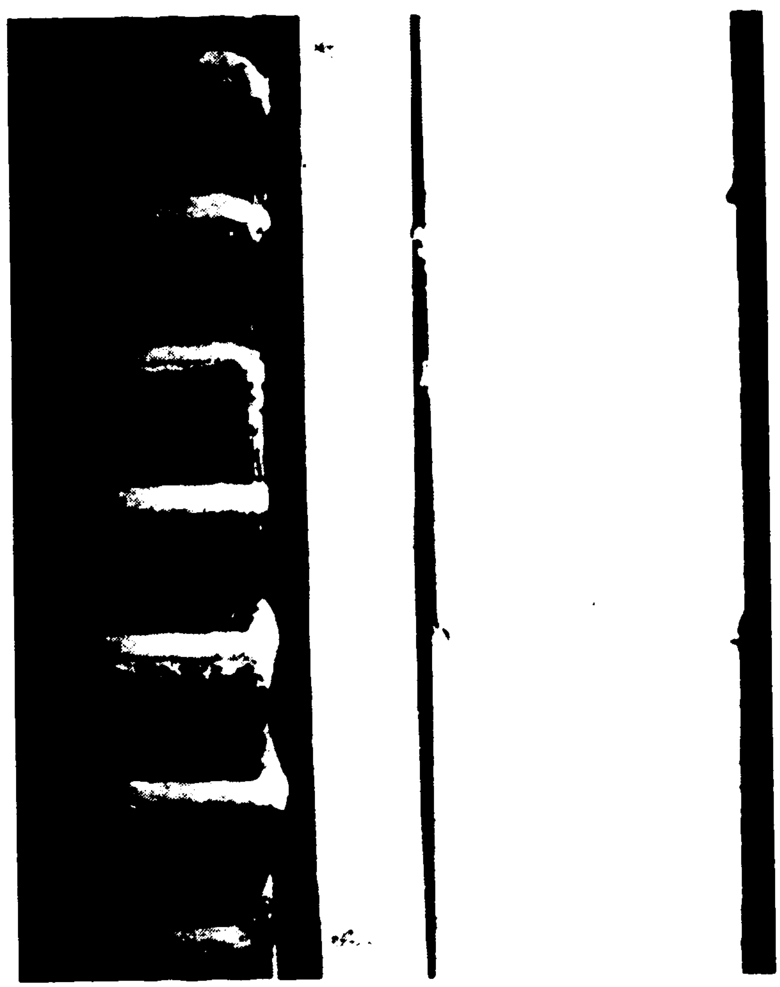


Plate 6.2 Failure Mode of S1W1

Plate 6.3 Deformed Wire Reinforcement of S1W1

Plate 6.4 Tension Crack Failure in Block Wythe of S1W4

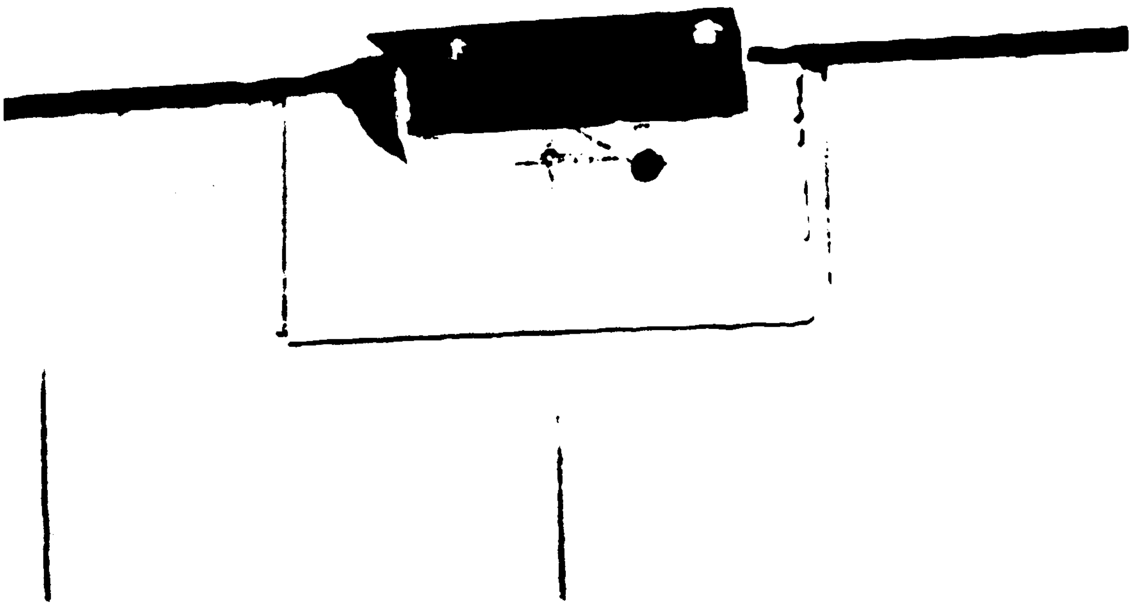


Plate 6.5 Block Punchout Failure of S2W2

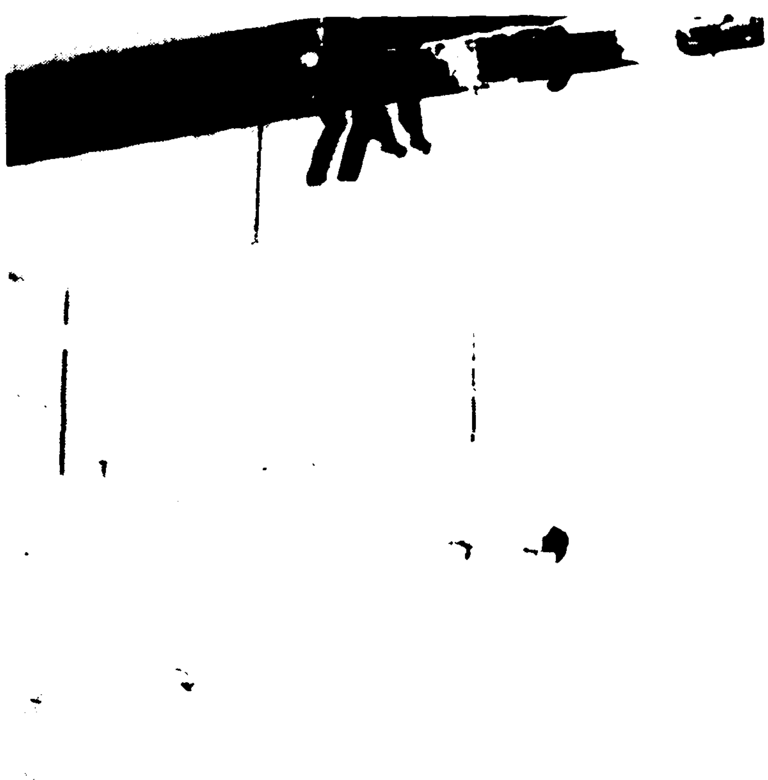


Plate 6.6 Repairs Made to S2W2 to Enable Completion of Test

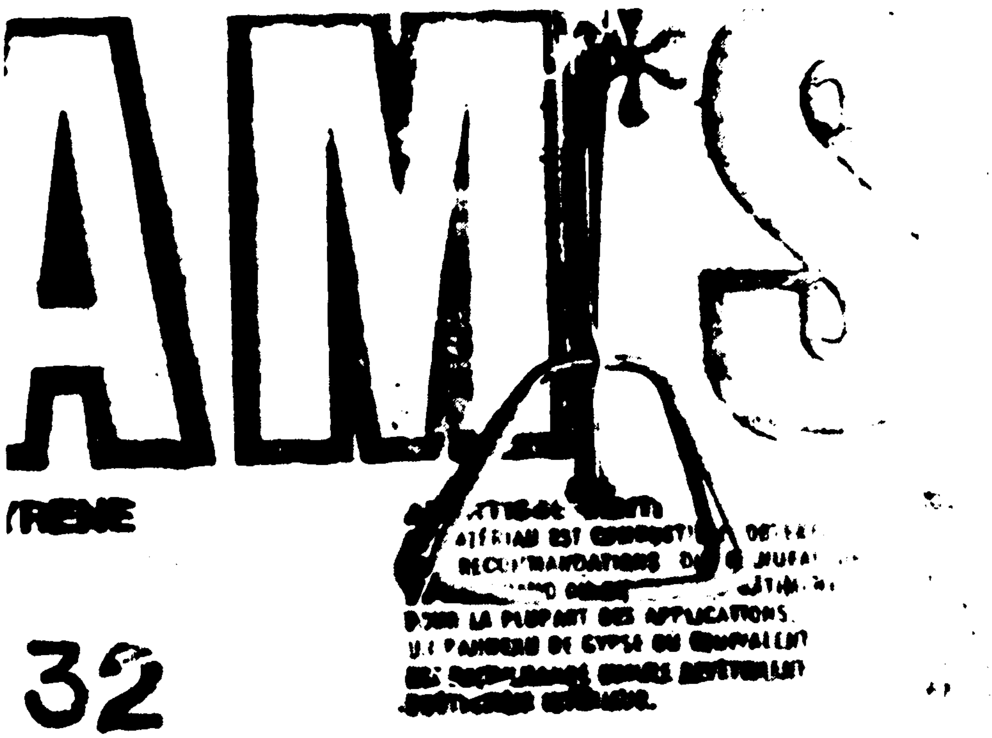


Plate 6.7 Early Stages of Yielding of Shear Connector in S2W2

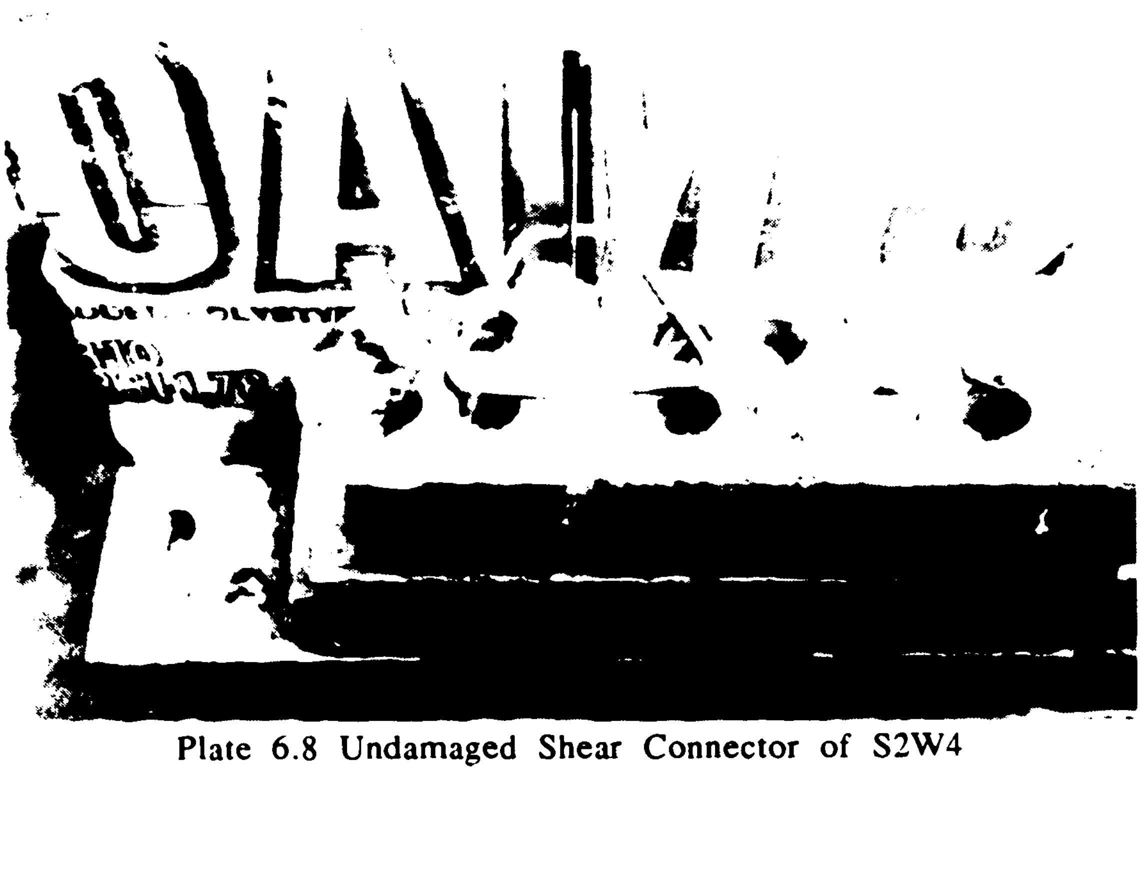


Plate 6.8 Undamaged Shear Connector of S2W4

7. ANALYSIS AND DISCUSSION

7.1 Introduction

This chapter consists of three parts. The first section briefly discusses the results from the material and prism tests, and its expected effects on the performance of the full sized wall specimens. The second section interprets the data from the full sized wall tests, and examines the effects of back-up wall size, cavity width, connector type, connector spacing, and vertical reinforcing on cavity wall system performance. The third section compares the actual full sized wall experimental values with the predicted theoretical values.

7.2 Effects of Material Properties on Full Size Wall Performance

The data compared here can be found in section 6.2. Two main variables are examined: mortar compressive strength and shear connector strength.

7.2.1 Mortar Compressive Strength

Mortar strength is a prominent factor in the overall performance of a masonry wall. However, only the unexpectedly low gain in strength of specimen S2W1 over that of S1W1 could be possibly attributed to mortar strength. That is, in all other cases, even though overall wall performance of one specimen was increased over that of another, no significant correlating increase in mortar strength(if any) was detected. This suggests that mortar strength is a relatively insensitive parameter in walls of this type, and is not a governing factor in our study.

7.2.2 Shear Connector Strength

The shear connector's performance under compressive lateral loading was quite commendable. From the study referred to as reference 4, a similar test conducted on identical conventional truss reinforcement resulted in failures of 2.25 kN and 2.80 kN. The shear connector resisted loads of twice that value. Secondly, the high shear capacity of the shear connector and the shear connector/concrete block junction also greatly surpasses any conventional type of connector. These results were supported by the undamaged shear connectors found when disassembling the full size specimens. Wall system S2W2 was an exception, because its connectors began to show some signs of yielding when subjected to high positive lateral pressures of up to 4.64 kPa.

7.3 Comparison of Full Sized Wall Tests

7.3.1 General Discussion

7.3.1.1 Deflections Due to Settling, Elastic Support Conditions, and Base Shifting

7.3.1.1.1 Deflections Due to Settling and Elastic Support Conditions

All of the wall specimens experienced deflections due to settling and elastic support conditions at the top of the wythes. The degree to which they exhibited the settling deflections was a relatively random function. It depended on material and construction consistency, and the amount of damage inflicted to the specimen upon handling and securing it inside the testing apparatus. All available precautions were taken to reduce these settlements.

The elastic support conditions at the top of the cavity wall systems were due to the upper slab detailing su[illegible] he intermediate lateral support. By complying with the design [illegible] no alterations were made so as to simulate actual walls constructed in the field.

7.3.1.1.2 Deflections Due to Base Shifting

As mentioned in the previous chapter, base shifting was suspected to be a result of a weak plane formed by a crack at the block wythe/floor slab interface. This crack was believed to have propogated during transport of the wall system to the testing apparatus. Although care was taken to avoid such disturbances, due to the cumbersome nature of the specimens, cracking was often inevitable.

Walls built in the field are not normally subjected to such conditions. In any case, since base shifting ocurred only in wall systems using the shear-resisting connector, it provides a conservative estimate of the actual lateral deflections that such walls would experience.

Perhaps in the future, for walls subjected to excessive loading such as specimen S2W2, the vertical reinforcement may be carried through and attatched to the floor slab to prevent any lateral movement.

7.3.2 Back-Up Wall Size

The effect of back-up wall size on overall performance is notable. The walls to be compared are S1W1 with S2W1, S1W3 with S2W3, and S1W4 with S2W4.

7.3.2.1 S1W1 vs S2W1

For specimen S2W1, an increase in block size from 150 mm to 200 mm resulted in an increased ultimate pressure of 1.24 kPa as compared with a pressure of 1.19 kPa for S1W1. Overall structural performance is expected to increase with block size, because the moment of inertia of the back-up wythe cross-section increases with block size. Although there was only a 6% increase in load resistance, the average maximum lateral deflection at failure decreased by three times by moving up to a larger block size.

As discussed in section 7.2.1, the unexpected small increase in load resistance may be a result of variability in mortar bond strength. Regardless, these walls both used conventional reinforcement, not the shear connector prototype, and therefore this comparison shall not be pursued any further.

7.3.2.2 S1W3 vs S2W3

As with the previous case, only a small increase in load-resisting capacity of 4% was achieved by S2W3, by increasing the block size from 150 mm to 200 mm. At specimen S1W3's failure pressure of 1 [illegible] Pa, the maximum lateral deflection of S1W3 was 15 mm, while S2W3 had a deflection of approximately 2.1 mm. This results in a decrease in deflection of more than 7 times. Even at failure, specimen S2W3 deflected roughly 30% less than S1W3 did at its failure.

7.3.2.3 S1W4 vs S2W4

Unlike the above two comparisons, the increase in block wall size resulted in a large strength gain of 40%, from 1.75 kPa, to 2.49 kPa. Dramatic reductions in lateral deflections were also a result of increasing the block size. At a pressure of 1.75 kPa, the lateral deflection of S2W4 of 1.5 mm was 3.5 times smaller than S1W4's deflection of 5.2 mm.

7.3.2.4 Comments

There is an expected significant deflection reduction and strength resistance gain with increased concrete block wall size. The increase in the moment of inertia from a 150 mm to a 200 mm standard size block is the main contributor to this reduction in deflection. The moment of inertia is increased due to an increase in the mortar bond area and the distance between the two face shells. This results in an increase in stability of the block wythe, hence a greater load-resisting capacity.

7.3.3 Cavity Width

Wall specimens S1W3 and S1W4, and S2W3 and S2W4, are compared to determine relationships resulting from increased cavity width. All walls presented used shear connectors.

7.3.3.1 S1W3 vs S1W4

Specimen S1W4 differed from S1W3 only by an increased cavity width of 25 mm. The outcome was an increased failure pressure of 1.75 kPa for S1W4, as opposed to a pressure of 1.38 kPa, resulting in an increase of 27%. The lateral deflections of S1W4 were consistently less than those of S1W3 throughout the tests. The maximum lateral deflection at failure for S1W3 was 3 times greater than that of S1W4, even though S1W4 failed at a higher load.

7.3.3.2 S2W3 vs S2W4

An increase in load resistance of 73% over that of S2W3 was experienced by S2W4 as a result of a 25 mm cavity width increase. Specimen S2W4 exhibited consistently lower lateral deflections than S2W3. At S2W3's failure pressure of 1.45 kPa, S2W3's maximum lateral deflection of 12.8 mm was approximately 11 times greater than that of S2W4's deflection.

7.3.3.3 Comments

Both of the above comparisons display the dramatic increase in load and decrease in lateral deflection, as a result of increased cavity widths of shear connector wall systems.

The improved wall performance can only be explained by the interaction of the shear connector with the other components of the wall system. Figure 7.1 shows how a shear connector cavity wall may be roughly approximated as a modified Vierendeel Truss. Such a model is constructed of a top and a bottom chord, connected rigidly at joints by web members. Unlike those of a conventional truss, members are subjected to bending, axial, and shear stresses. It was assumed that the effects of the shear connector pins and end conditions of the simplified model would cancel each other out, therefore both were omitted from the Vierendeel Truss analysis. Both wythes were considered to be identical, weightless, and share the lateral force equally. With these assumptions, the following relations were determined from section X-X of the Vierendeel Truss analysis:

$$V_3 = 0.5*(0.5 - 0.1)wL = 0.2wL$$

$$M_2 = V_3*(x/2) = 0.1wxL$$

$$V_1 = M_2/0.5t = 0.2wxL/t$$

$$V_1 = T_3 = \text{tensile force in block wythe at point 3}$$

A given lateral pressure, w, results in various moments(M_2) and forces(V_1, T_3) acting on the wall assembly. V_1 is inversely proportional to the cavity width, t. As a result, an increase in the cavity width results in a decrease in $V_1(=T_3)$, and thus a decrease in the tensile forces in the block wythe.

Large tensile forces may develop in the block wythe as a result of excessive lateral loading, or by connector failure, which requires the back-up wythe to accept all of the lateral load. If these forces exceed the tensile bond strength at the block/mortar joint interface, the system fails. By increasing the cavity width while ensuring connector integrity through proper design, the magnitude of the tensile forces in the block wythe would be reduced. This would enable such wall assemblies to withstand higher loadings. This is exactly the opposite with conventional types of connectors, because an increase in cavity width results in a higher tendency for the connector to buckle and therefore fail.

The enhanced performance due to the increase in cavity width appears more prevalent in the specimens using the 200 mm block back-up wythes. Further testing will have to be conducted to confirm this tendency.

7.3.4 Connector Type

In this section, specimens S1W1 and S1W2 are examined on the basis of different diameter conventional reinforcement. The performance of conventional reinforcement versus shear connectors are explored by comparing wall systems S1W1 with S1W3, S2W1 with S2W3, S1W1 with S1W4, and S2W1 with S2W4. In the last two comparisons, the latter wall has a cavity width of 25 mm greater than the former. However, they are compared because present code procedures do not consider an increase in cavity width an advantage. Rather, it is regarded as detrimental to wall system performance.

7.3.4.1 S1W1 vs S1W2

Specimen S1W1 withstood a much higher failure pressure of 1.22 kPa, as opposed to a pressure of 0.75 kPa as resisted by S1W2. Lateral deflections for S1W1 were also consistently equal or even less than those of S1W2.

These results were unexpected because specimen S1W2 used a higher diameter rod in its wire truss reinforcement. The thinner rod of S1W1 was expected to deform at lower loading levels, and thus fail sooner. Such a discrepancy can be attributed to the variability of materials and workmanship.

7.3.4.2 S1W1 vs S1W3

Both wall systems were identical except that S1W1 used 3.66 mm diameter conventional reinforcement, while S1W3 used shear connectors.

Specimen S1W3 failed at a lateral pressure of 1.38 kPa, while S1W1 failed at 1.16 kPa. The use of shear connectors resulted in a 20% gain in load resisting capacity. The maximum lateral deflection of S1W1 at failure was 4.5 mm, while the deflection of S1W3 at that load was 0.7 mm. By using shear connectors, the lateral deflection was decreased by six times.

7.3.4.3 S2W1 vs S2W3

As in the above case, these specimens only differed by the fact that S2W1 used conventional reinforcement and S2W3 used shear connectors.

A 17% increase in lateral load resistance was achieved by substituting shear connectors for wire truss reinforcing. However, the deflections of S2W1 were unexpectedly lower than those of S2W3, for all comparable pressures up until failure. This discrepancy in the pattern established by other walls may provide evidence for regarding the deflections of S2W3 as being too liberal. Recall the documented account of high levels of uncertainty in S2W3's deflection data.

7.3.4.4 S1W1 vs S1W4

The use of shear connectors and an increase in cavity width of specimen S1W4 set it apart from S1W1.

As expected, due to the combination of shear connectors with increased cavity width, specimen S1W4 performed much superior to S1W1. The ultimate load capacity of S1W4 was increased from 1.22 kPa to 1.75 kPa, with deflections at comparable loads being consistently reduced in half.

7.3.4.5 S2W1 vs S2W4

As in the last case, S2W4 departed from the design of S2W1 by an increase in cavity width of 25 mm and by the use of shear connectors.

The load capacity of S2W4 is virtually doubled over that of S2W1, with deflections being consistently smaller at comparable pressures.

7.3.4.6 Comments

Excluding the disparity offered by specimen S2W3, a trend was established relating connector type to system performance. By substituting shear connectors for conventional reinforcement, significant increases in load resistance was achieved, along with marked decreases in lateral deflection.

By combining the use of shear connectors with the additional factor of increased cavity width, the said trend was enhanced even further.

7.3.5 Connector Pattern

Variations in the connector pattern of the wall systems using the shear connectors were not investigated in this study. A variation in the spacing of the conventional reinforcement in specimen S2W1 was too small to be considered a relevant factor in wall system performance.

7.3.6 Vertical Reinforcement

To determine the consequences of vertical reinforcement, reinforced specimen S2W2 was compared with S1W3, its unreinforced counterpart. The two outermost cores of S2W2 were reinforced with one 15M steel reinforcing bar and 20 MPa grout.

The failure pressure for S1W3 of 1.38 kPa was increased to 3.20 kPa for S2W2, at which point it suffered a block punch-out failure. With some minor detailing modifications, specimen S2W2 could have withstood pressures of up to 4.64 kPa, as it did when its design was altered during the testing program. Although its deflections were comparable up to a pressure of 1.0 kPa, further loading resulted in S2W2 resisting deflections up to 10 times better than S1W1.

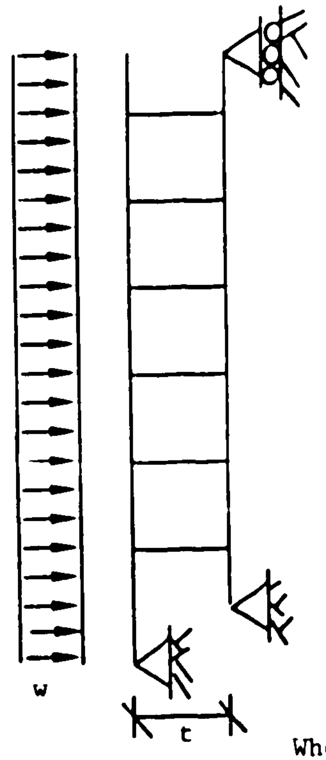
7.3.6.1 Comments

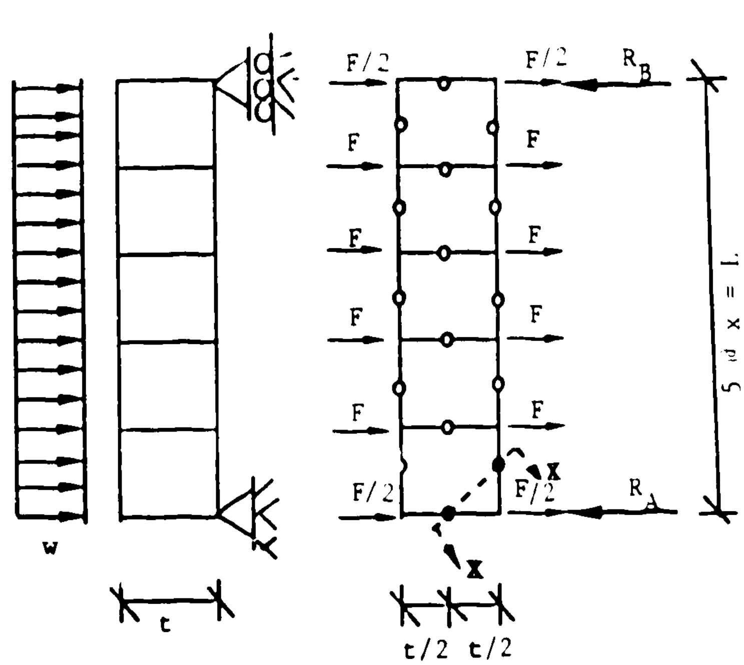
As expected, a large increase in load resistance and a large reduction in lateral deflections were observed when vertical reinforcing was implemented in a wall system using shear connectors. If the trend postulated in 7.3.3 is valid, one could expect wall system performance enhanced even further by increasing the cavity width of such a specimen.

7.4 Comparison of Experimental Results With Theoretical Predictions

The test results of specimens S1W3, S1W4, S2W2, and S2W4 were compared with values predicted by their corresponding models, as were described in chapter 4. All of the actual test data required adjusting in order to eliminate unpredictable wall system behaviour such as settlement of the specimens and base shifting. The theoretical plane frame analysis was developed to predict only elastic behaviour. As a result, the walls were compared at a pressure of 0.75 kPa, which should have been well within the elastic limits of all of the specimens. Only the lateral deflection diagrams of the block wythes were compared. Three runs of each model are presented with each adjusted test data diagram. Each theoretical run assigns a different elastic modulus to the wall system. These comparisons can be found in Figures 7.2 through 7.5.

In general, the shapes determined by the theoretical analysis were consistent with those obtained experimentally. The deflection diagram models of specimens S1W3 and S2W2 underestimate the actual deflections between 2.5 and 3 times. However, the theoretical predictions are relatively close with cavity walls S1W4 and S2W4. There does not seem to be much of an obvious correlation relating which type of wall is modelled more accurately. Further testing will have to be determined to more accurately evaluate the material properties of the cavity wall system using shear-resisting connectors. In conjunction with this testing, the theoretical models should also be refined. More positive results may be obtained by breaking down the wall system into smaller elements, thereby allowing more detailed material properties and joint information.



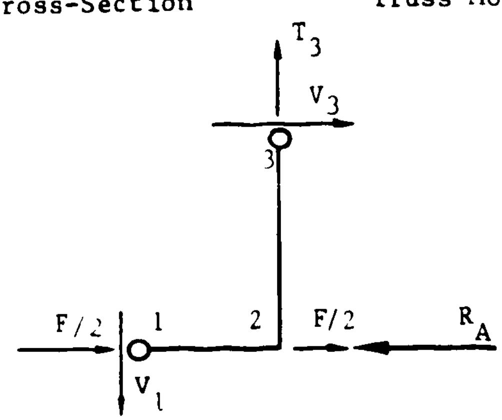


Where: 1. w = lateral pressure acting on the wall
2. t = cavity width + 1/2(brick + block)
3. F = wL/10
4. $R_A = R_B = wL/2$

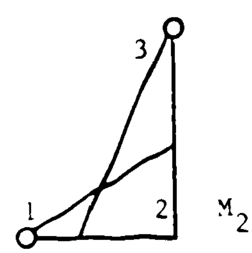
(i) Simplified Wall Cross-Section

(ii) Vierendeel Truss Model

(iii) Vierendeel Truss Analysis



Force Diagram



Moment Diagram

(iv) Section X-X of Vierendeel Truss Analysis

Figure 7.1 Distribution of Lateral Forces By Shear Connector Cavity Walls

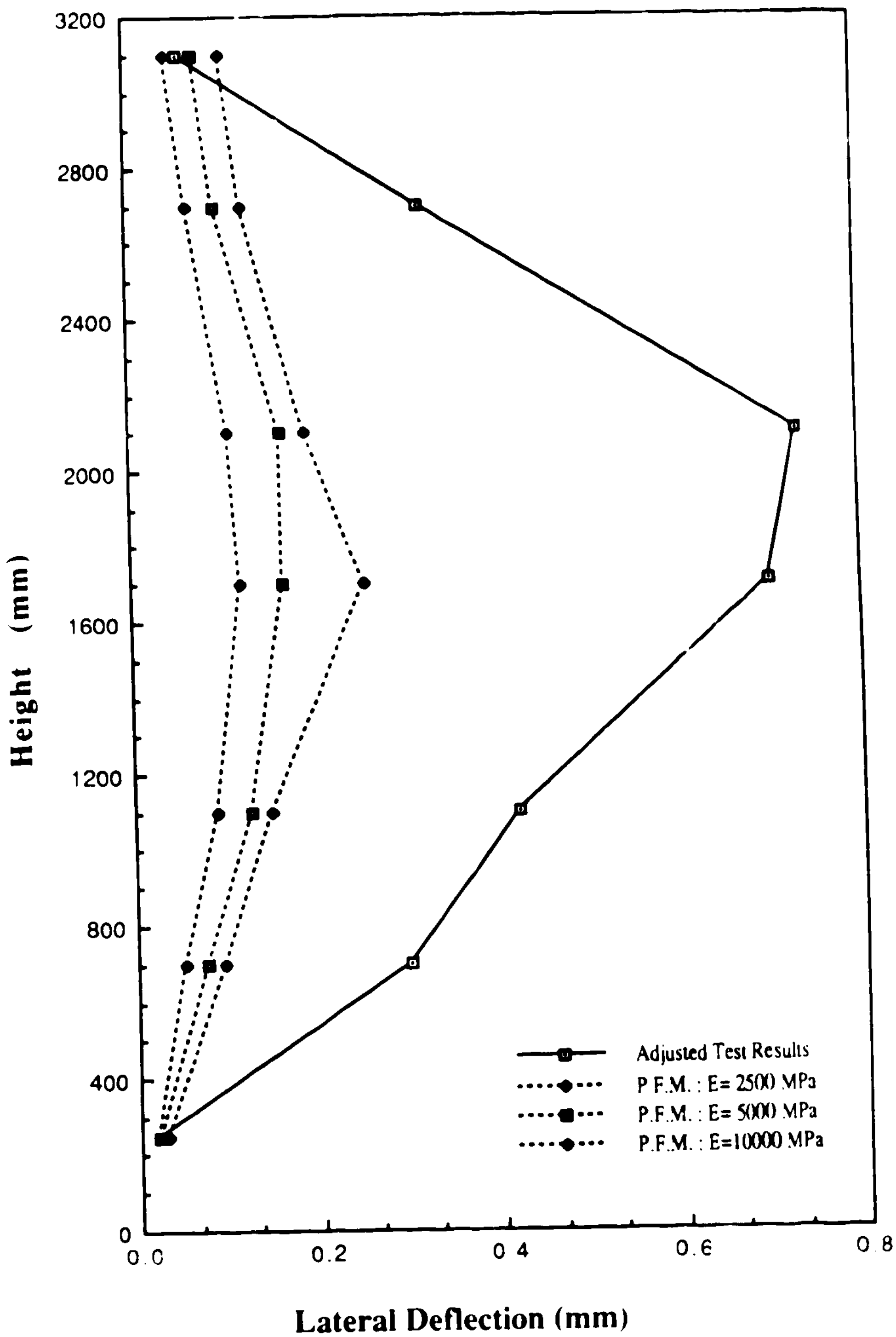


Figure 7.2 Block Wythe @ 0.75 kPa for S1W3
-Plane Frame Model vs Adjusted Test Results

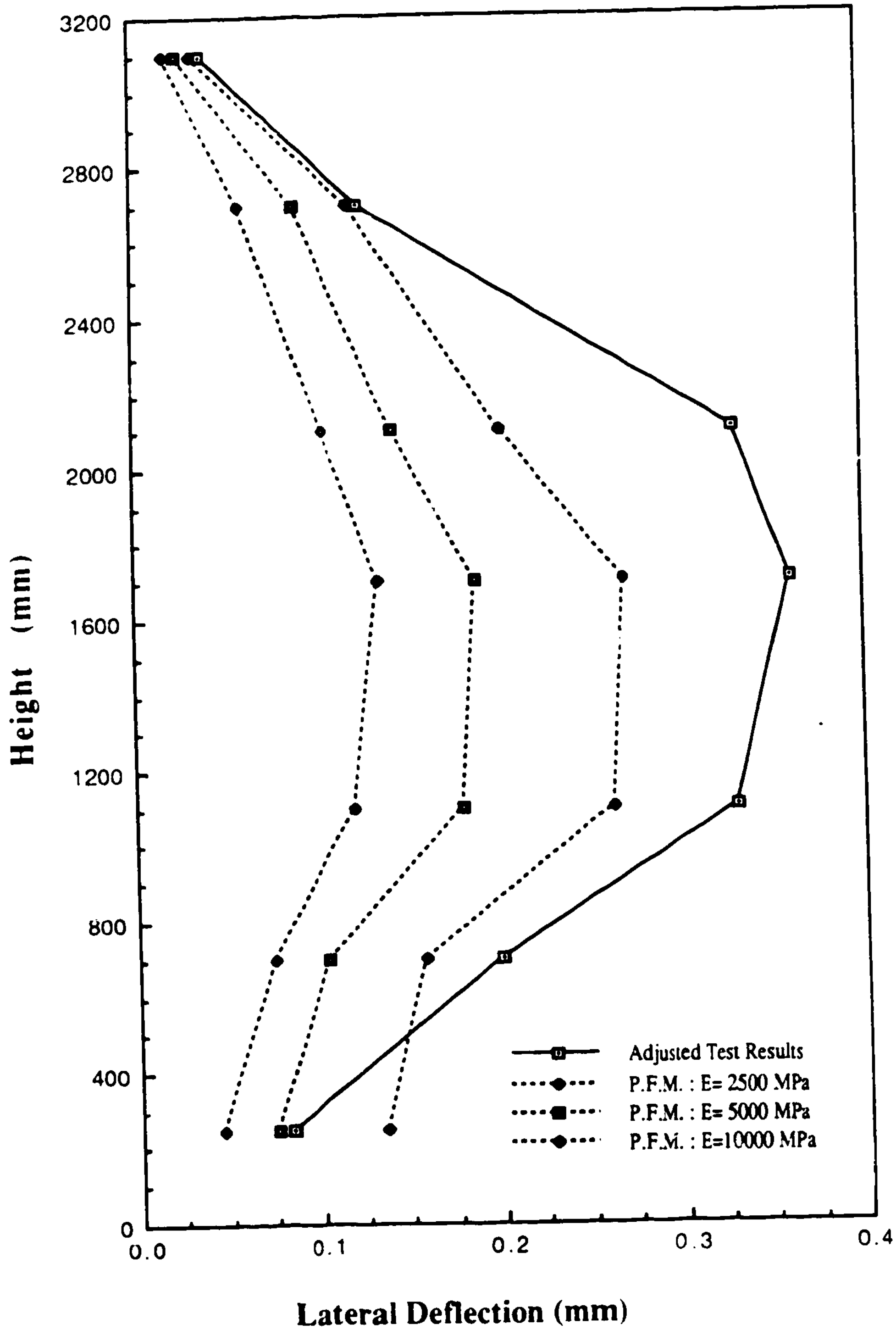


Figure 7.3 Block Wythe Deflection @ 0.75 kPa for S1W4
-Plane Frame Model vs Adjusted Test Results

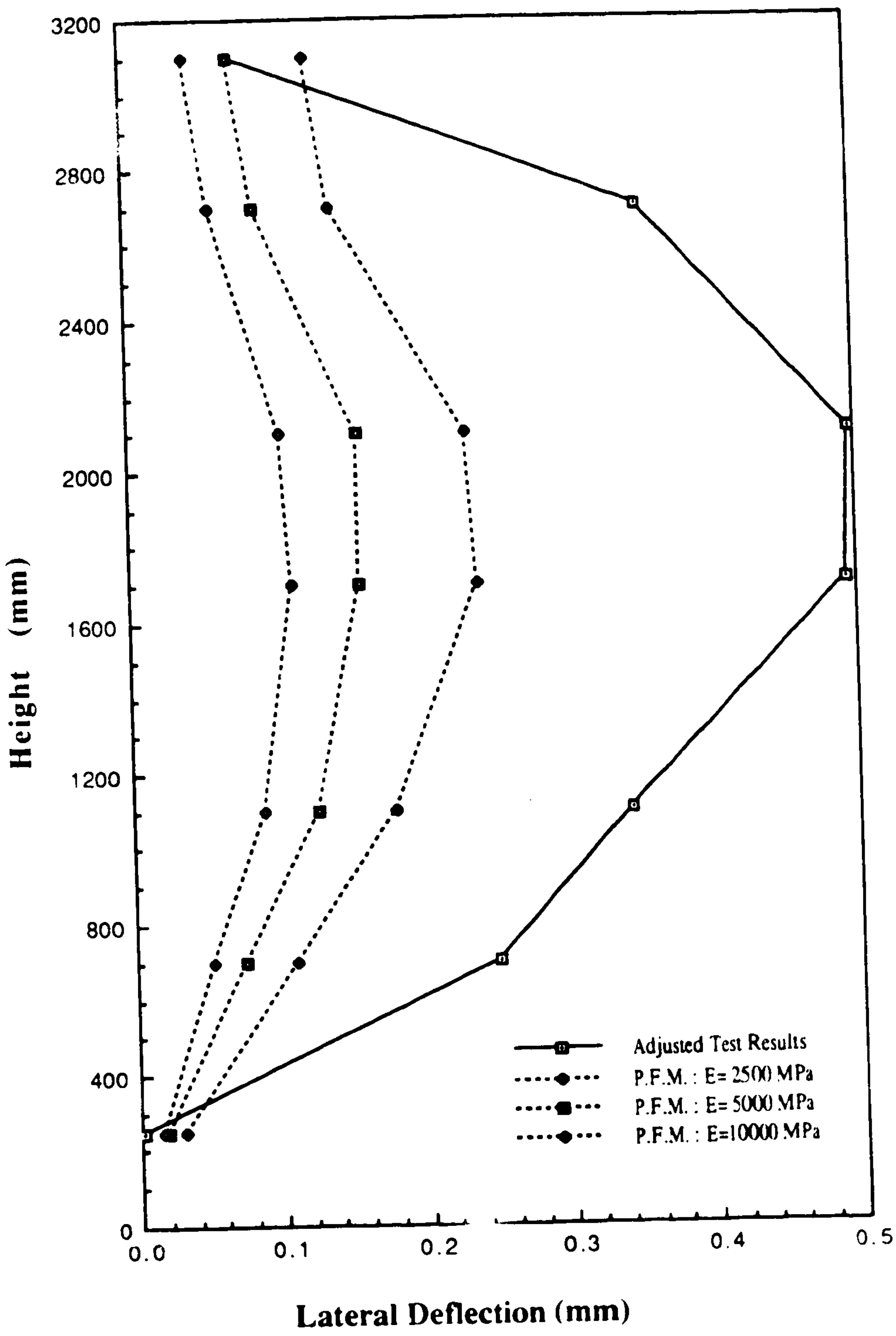


Figure 7.4 Block Wythe Deflection @ 0.75 kPa for S2W2 -Plane Frame Model vs Adjusted Test Results

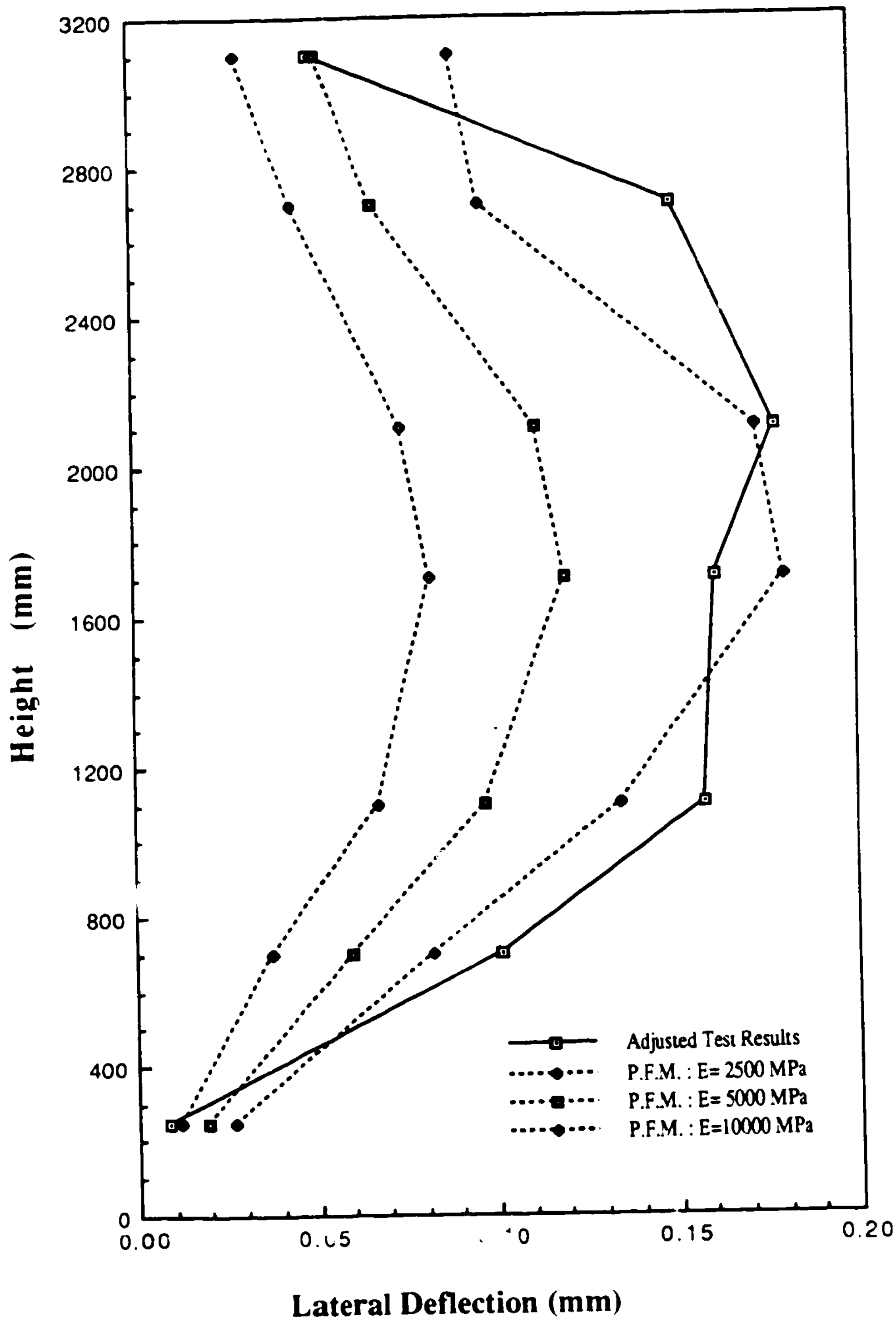


Figure 7.5 Block Wythe Deflection @ 0.75 kPa for S2W4
-Plane Frame Model vs Adjusted Test Results

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 Summary

This investigation was devoted to the study of the behaviour of masonry cavity walls, constructed with concrete masonry units, incorporating shear-resisting connectors. Eight full sized wall specimens were tested, and the results reported. Factors explored in this study included back-up wall width, connector type, cavity width, and vertical reinforcing. In addition, material property data was obtained from small specimen tests. A theoretical model was developed to predict the behaviour of such a wall assembly. The results obtained from both the limited experimental phase and the theoretical evaluation are compared and discussed in this report.

8.2 Conclusions

Based on the experimental and theoretical con ations, the following conclusions are revealed:

1. None of the shear connectors in any of the tests were stressed to their full capacity.

2. A shear connector was developed which is capable of transferring axial load and shear from the brick veneer to the block back-up wall. This resulted in a system with enhanced load-resisting capabilities.

3. The shear connectors force the two wythes to act in composite. This is evident upon inspection of the deflected shapes.

4. Experimentally determined that by replacing conventional reinforcement with shear connectors, load-resisting capacity was increased from 16% for walls of equivalent cavities, and up to 100% for walls with an increased cavity width of 33%.

5. At comparable pressures, the lateral deflection of the wall systems using shear connectors was consistently less than that of the conventional wire truss reinforcement. Reductions in crack widths and water penetration would result from such decreased deflections.

6. When using shear connectors, an increase in cavity width results in a significant increase in load resisting capacity. Dramatically reduced deflections at comparable pressures are also achieved. This enables the cavity to be increased to allow for the placement of thicker insulation.

7. When using shear connectors, increases in the block thickness of the back-up wall result in greater load carrying capacity and also lower deflections at comparable pressures.

8. When the block back-up wall of a system using shear connectors is reinforced, load carrying capacity is increased and lateral deflections at comparable pressures are decreased.

8.3 Recommendations

From the investigation, the following recommendations are presented for consideration:

1. Further studies need to be conducted to examine in detail factors affecting the design of wall systems using shear connectors. These factors include wall design, cavity width, connector spacing, and reinforcement. The interrelation of these factors and the optimization of their benefits should be studied.

2. Establish suitable design guidelines.

3. Studies to obtain optimum design for thermal considerations.

4. Further development of a model to accurately predict elastic behaviour of cavity wall systems which use shear connectors.

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APPENDICES

APPENDIX A - MATERIAL TEST RESULTS

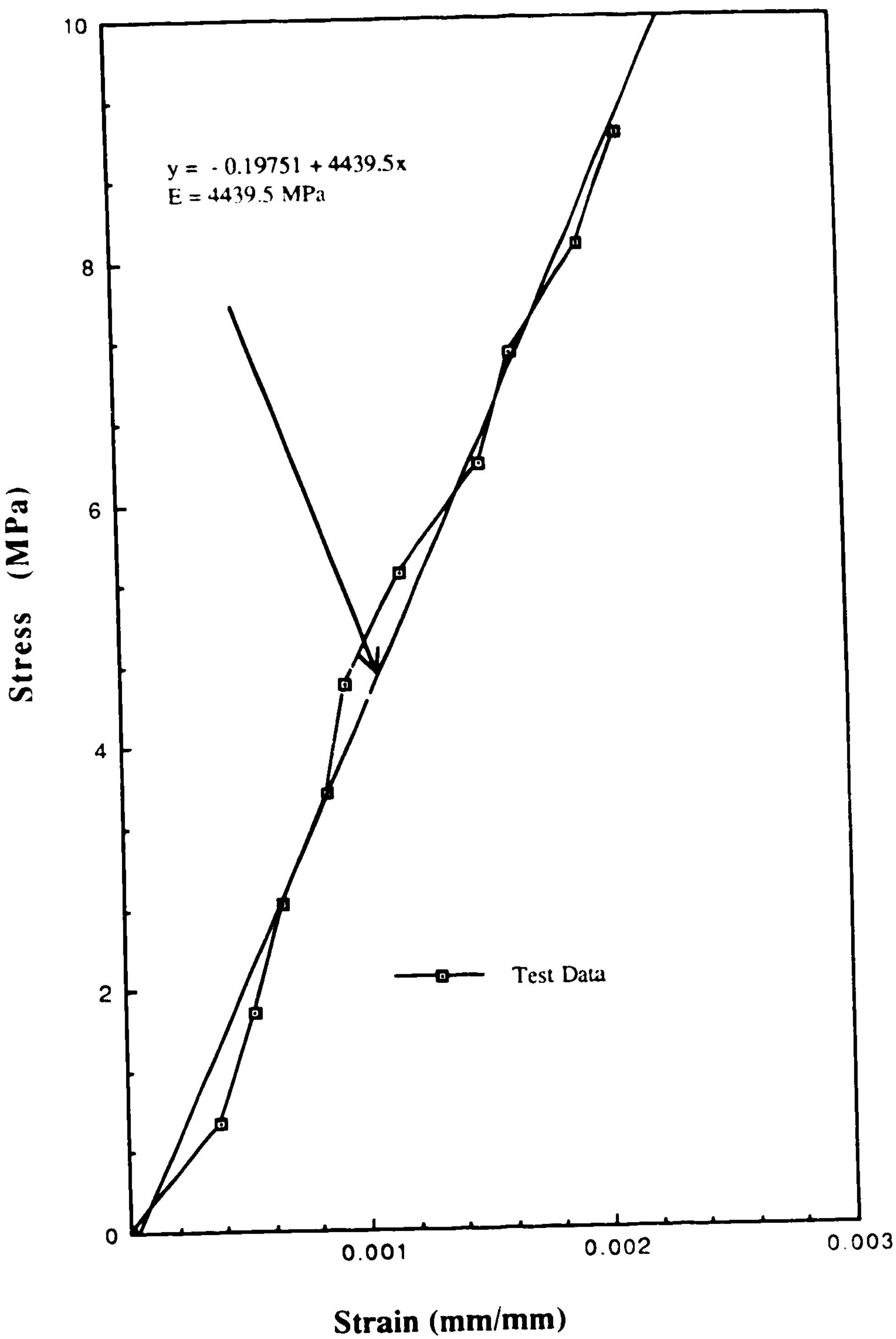

Figure A-1 Stress Strain Curve for Block Prism A

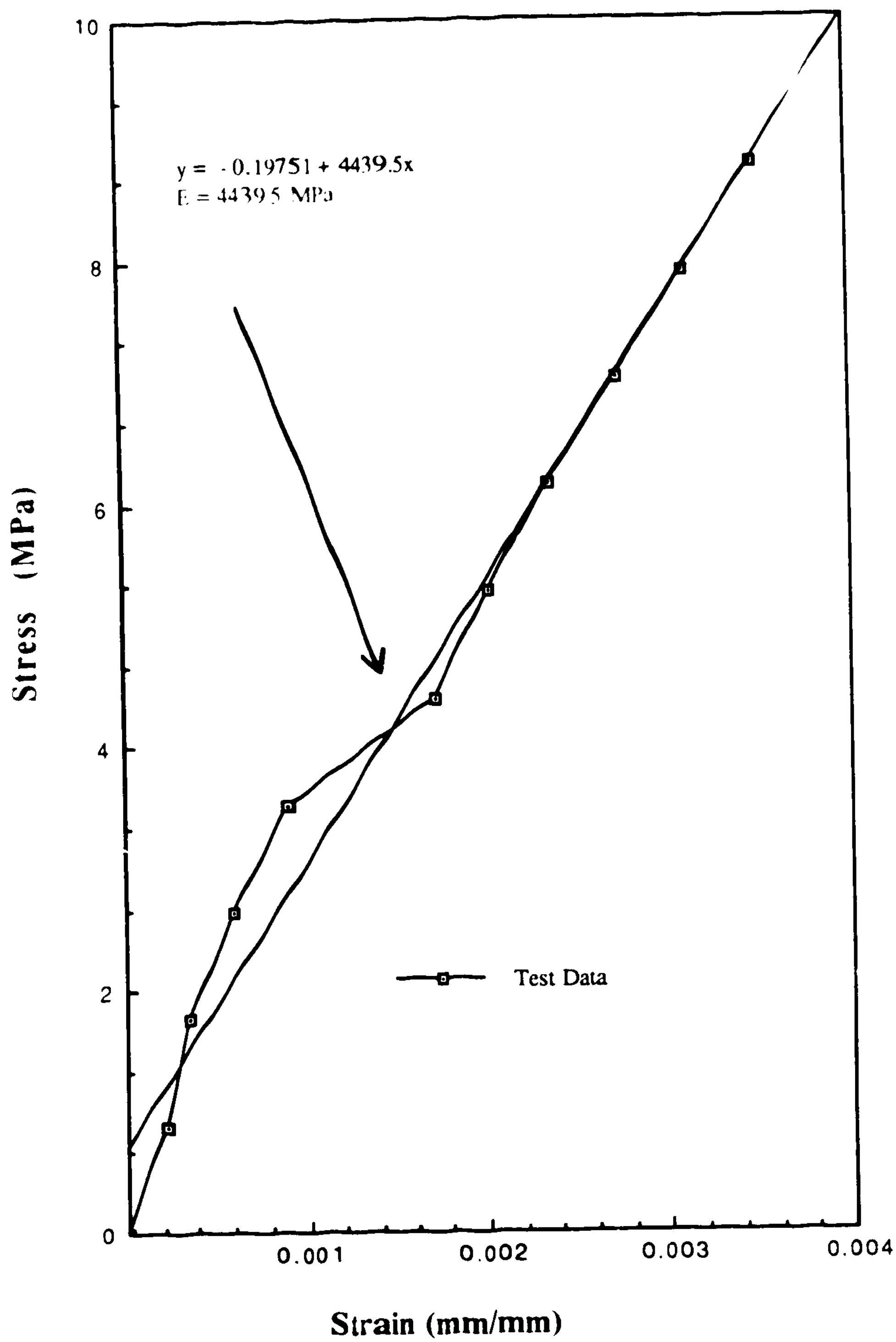

Figure A-2 Stress Strain Curve for Block Prism B

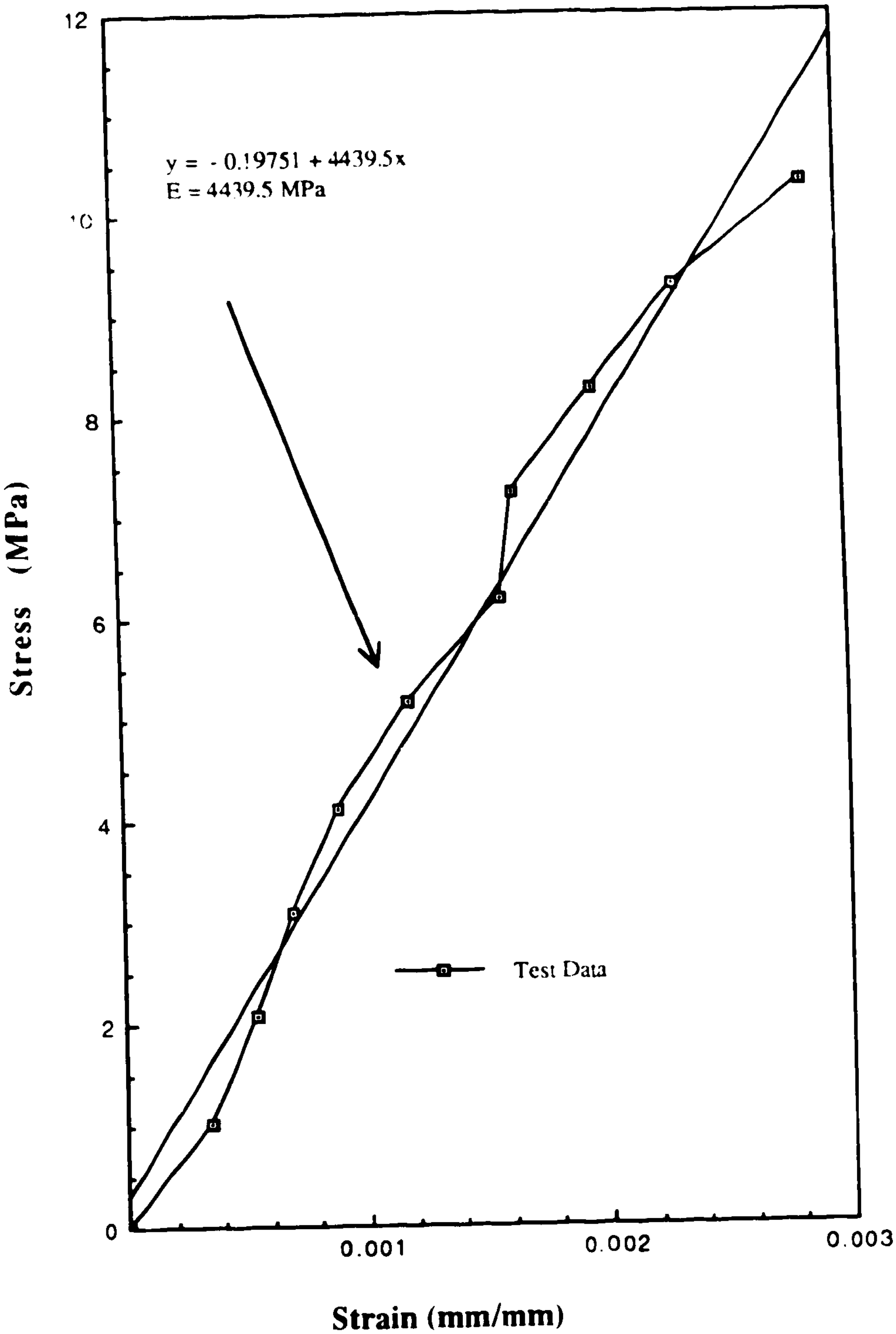

Figure A-3 Stress Strain Curve for Block Prism C

APPENDIX B - FULL SIZED WALL SPECIMEN RESULTS

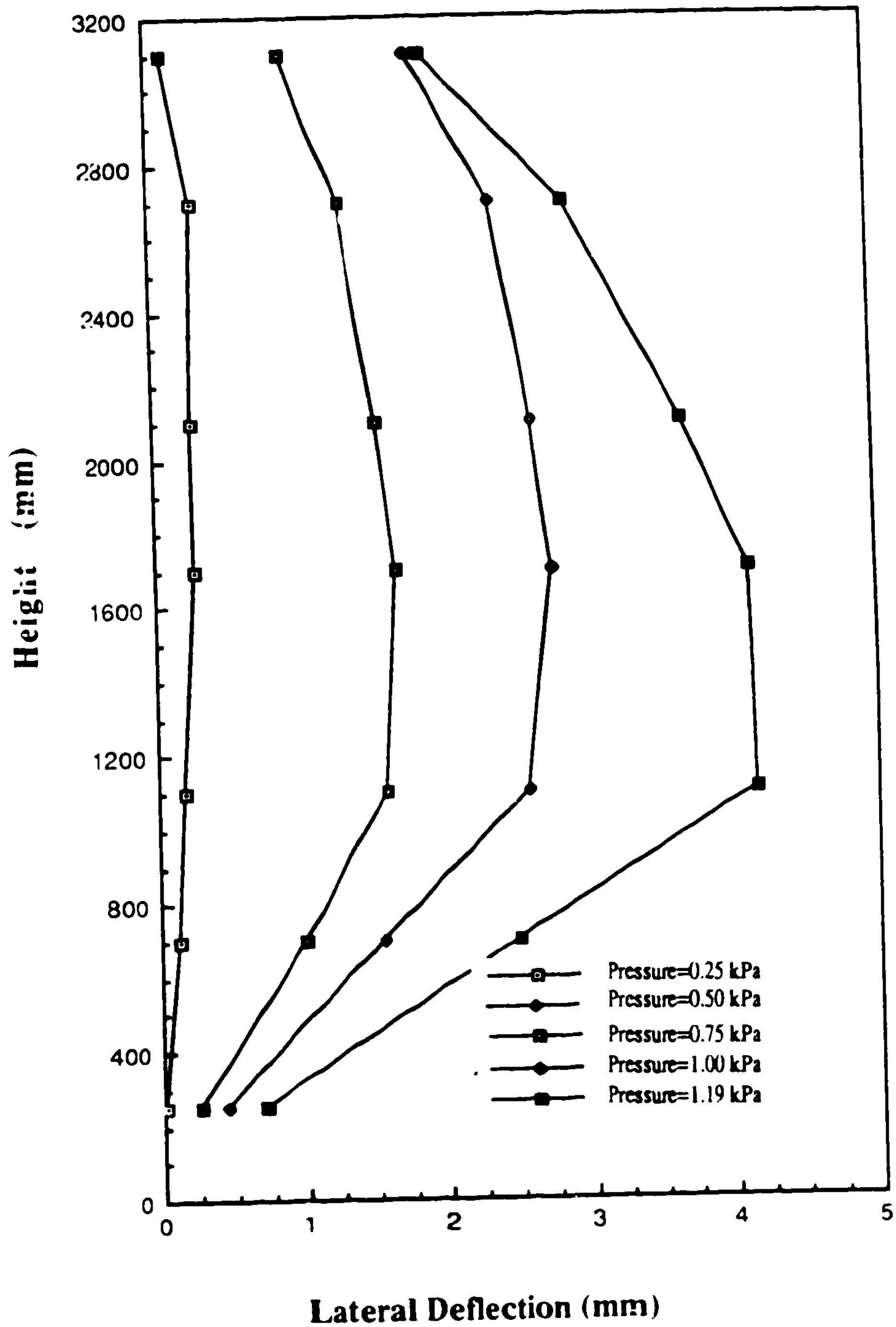

Figure B-1 Brick Wythe Deflection for S1W1

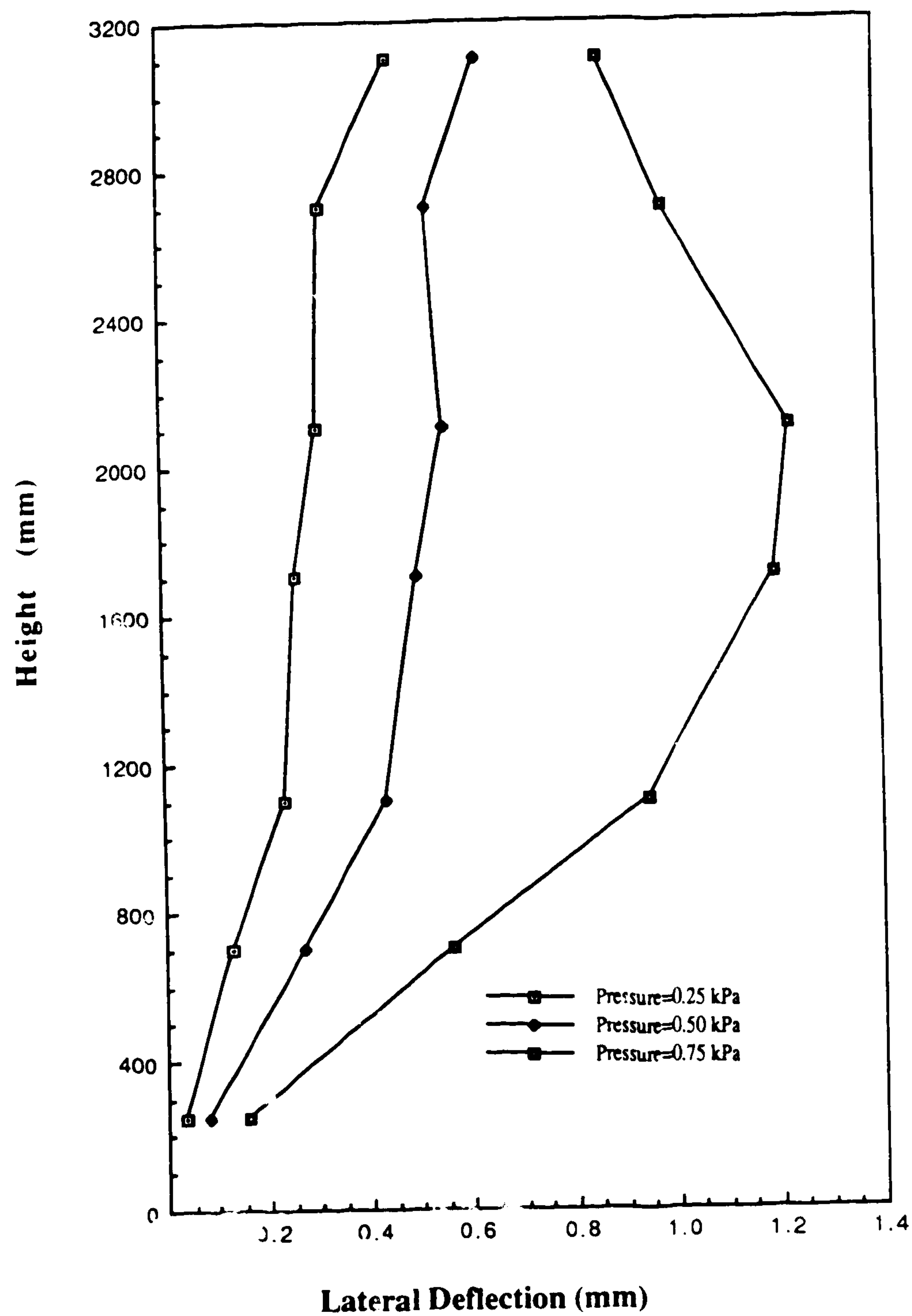

Figure B-2 Brick Wythe Deflection for SiW2

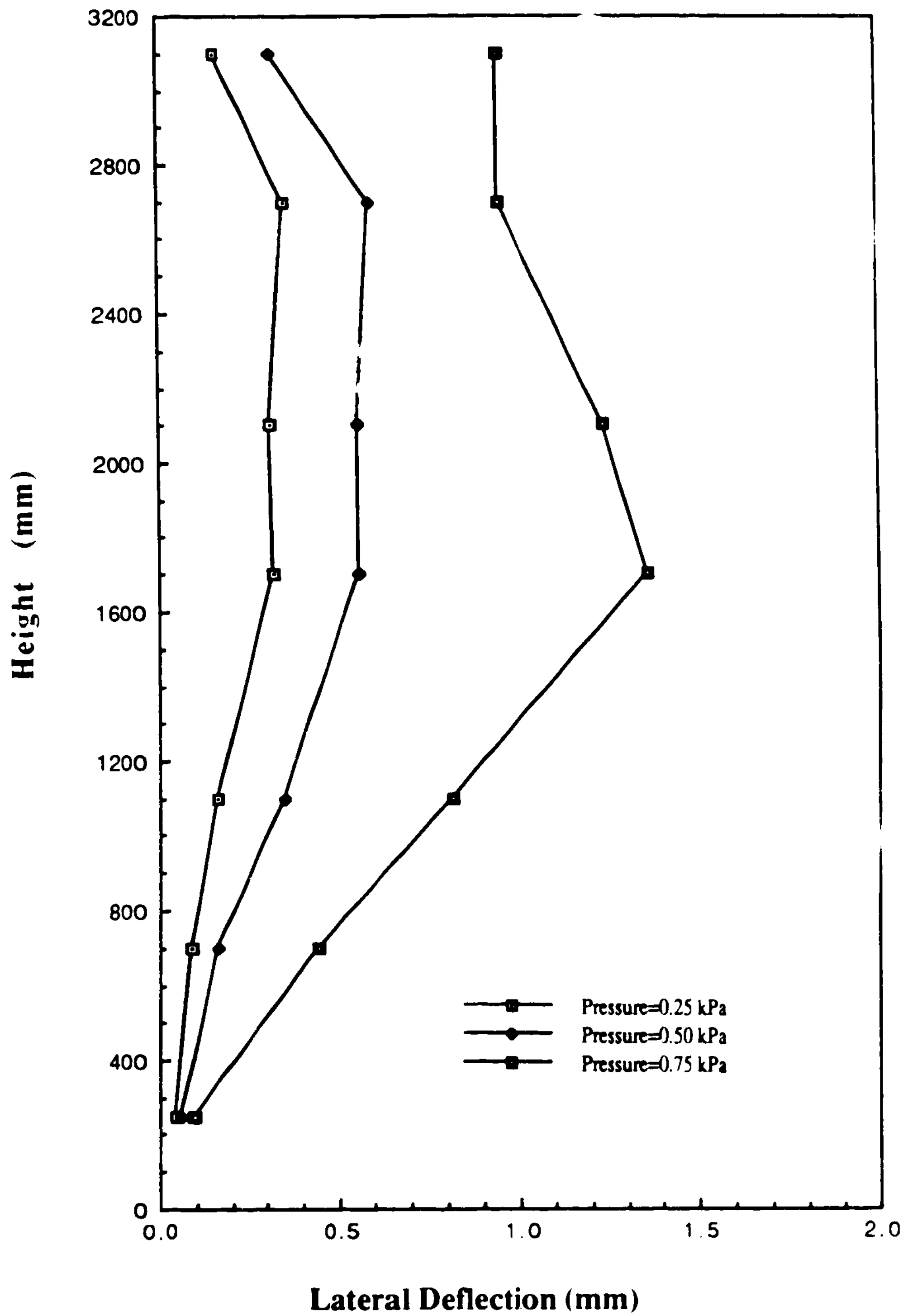

Figure B-3 Block Wythe Deflection for S1W2

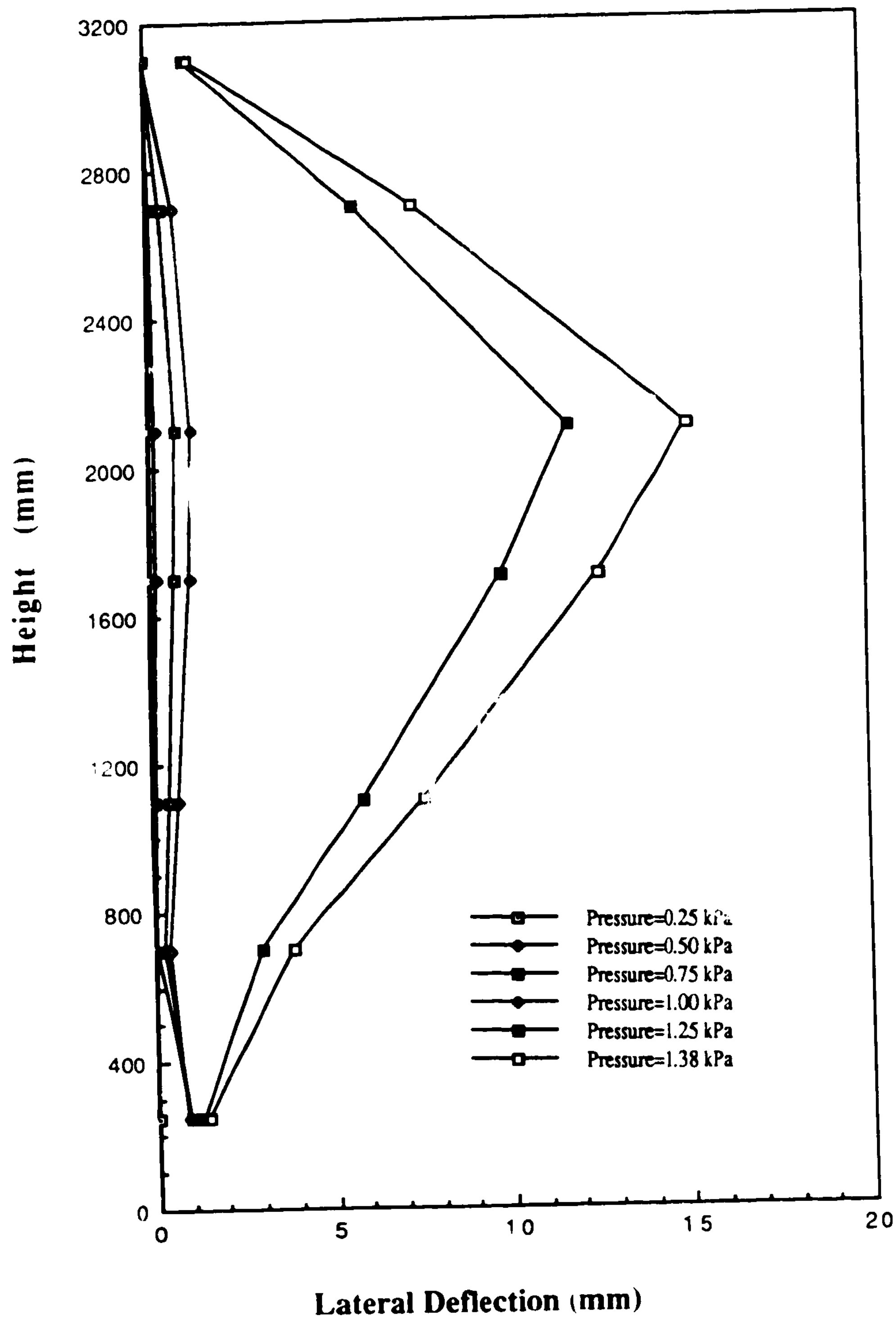

Figure B-4 Brick Wythe Deflection for S1W3

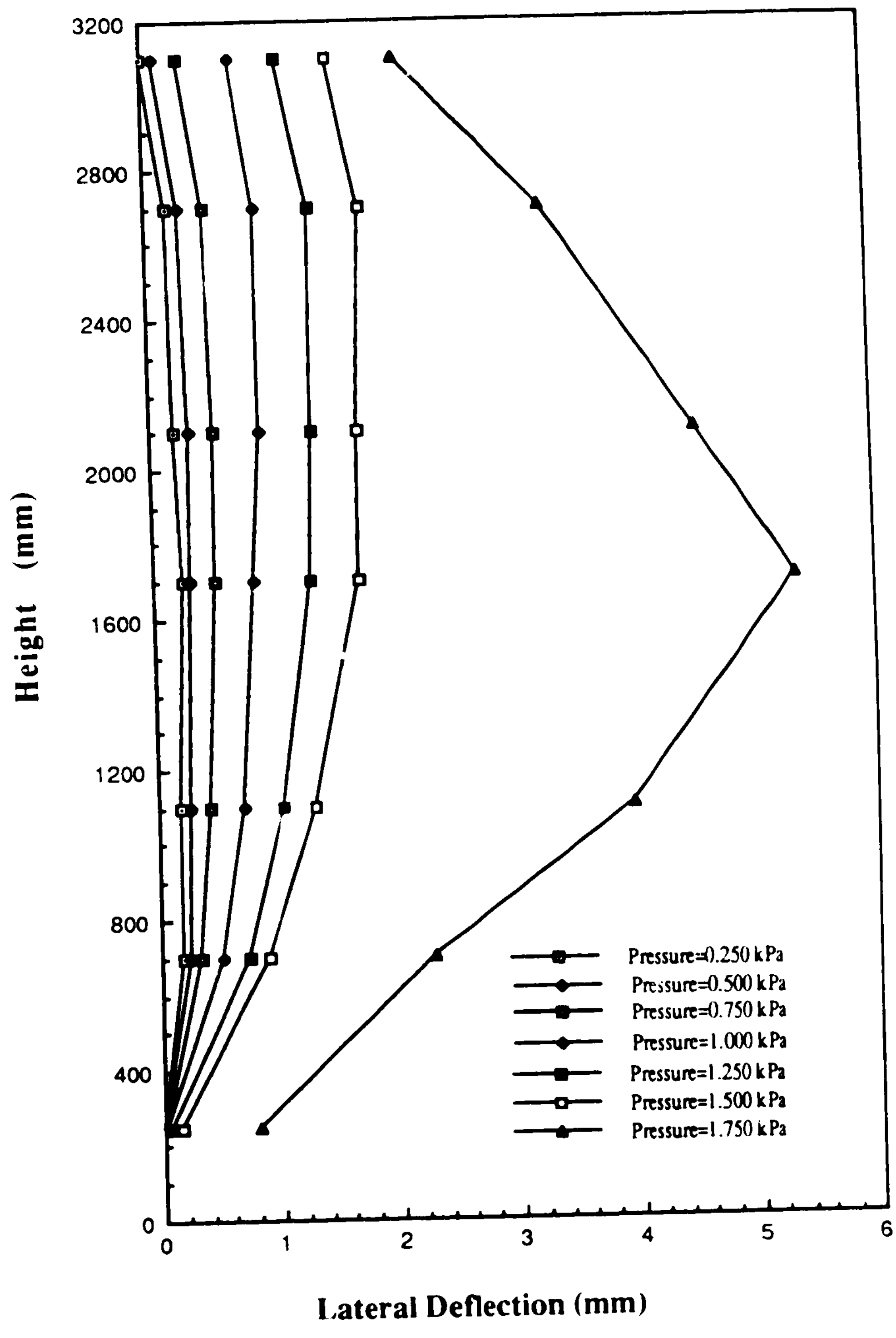

Figure B-5 Brick Wythe Deflection for S1W4

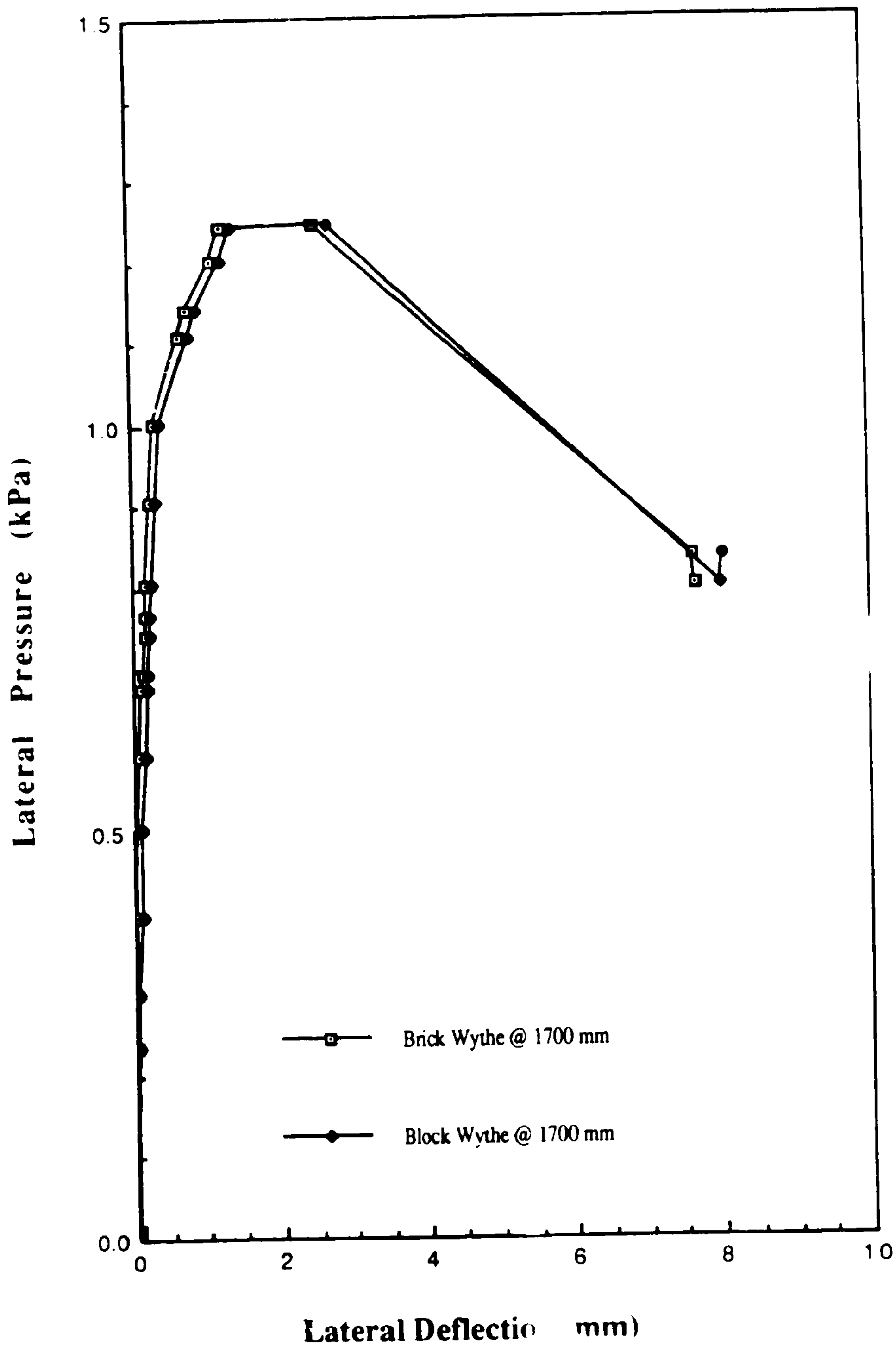

Figure B-6 Pressure vs Centerline Lateral Deflection for S2W1

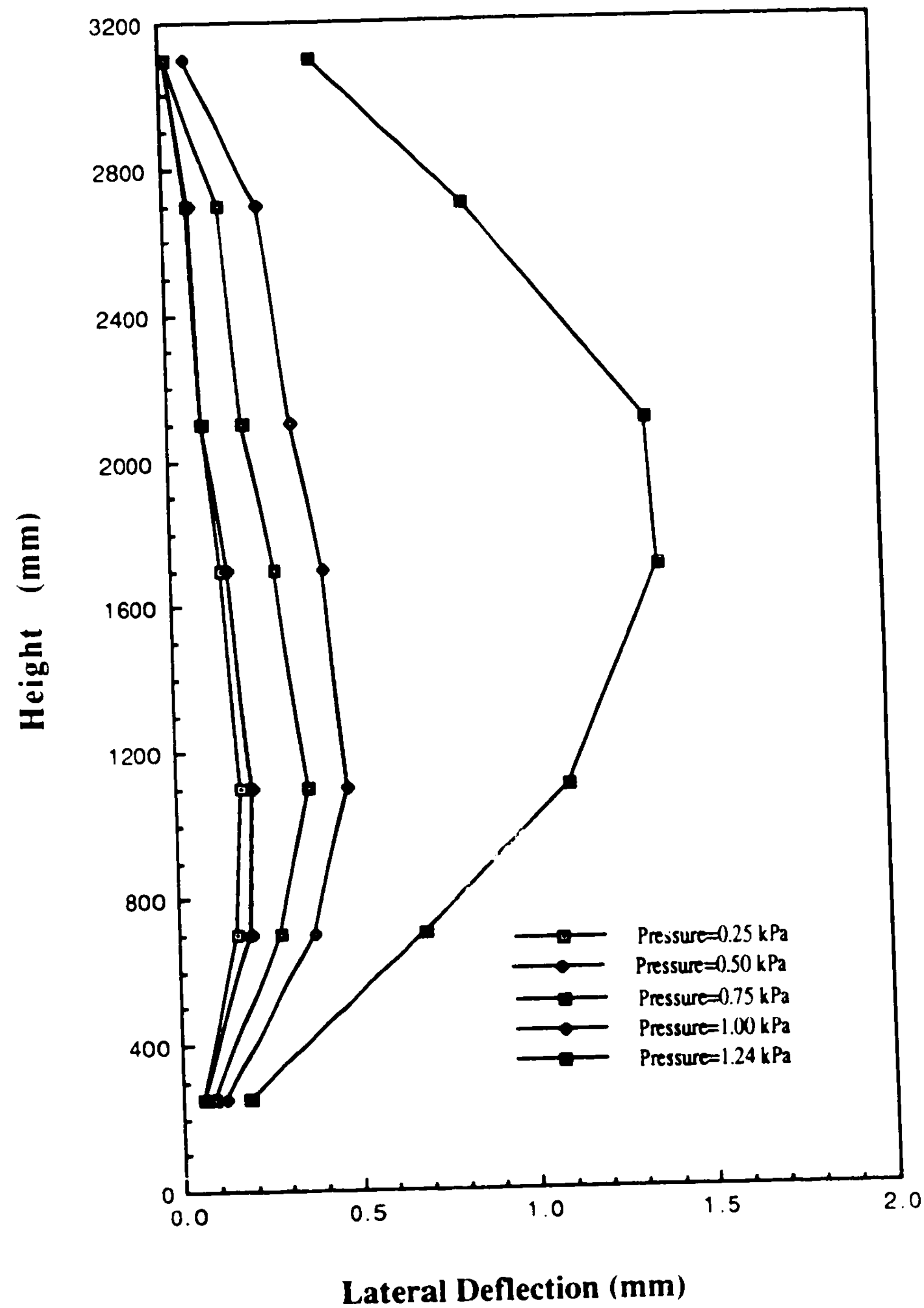

Figure B-7 Brick Wythe Deflection for S2W1

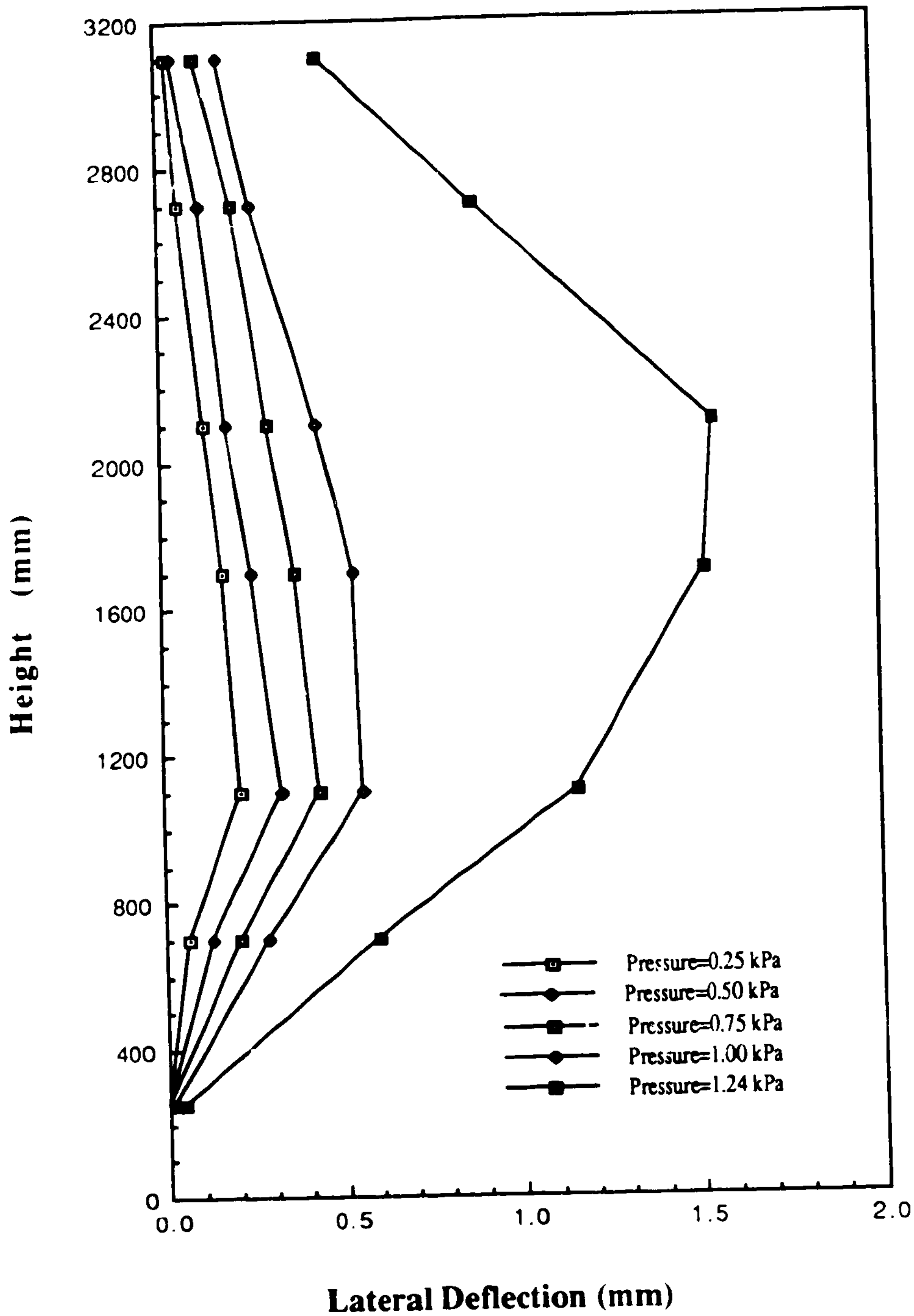

Figure B-8 Block Wythe Deflection for S2W1

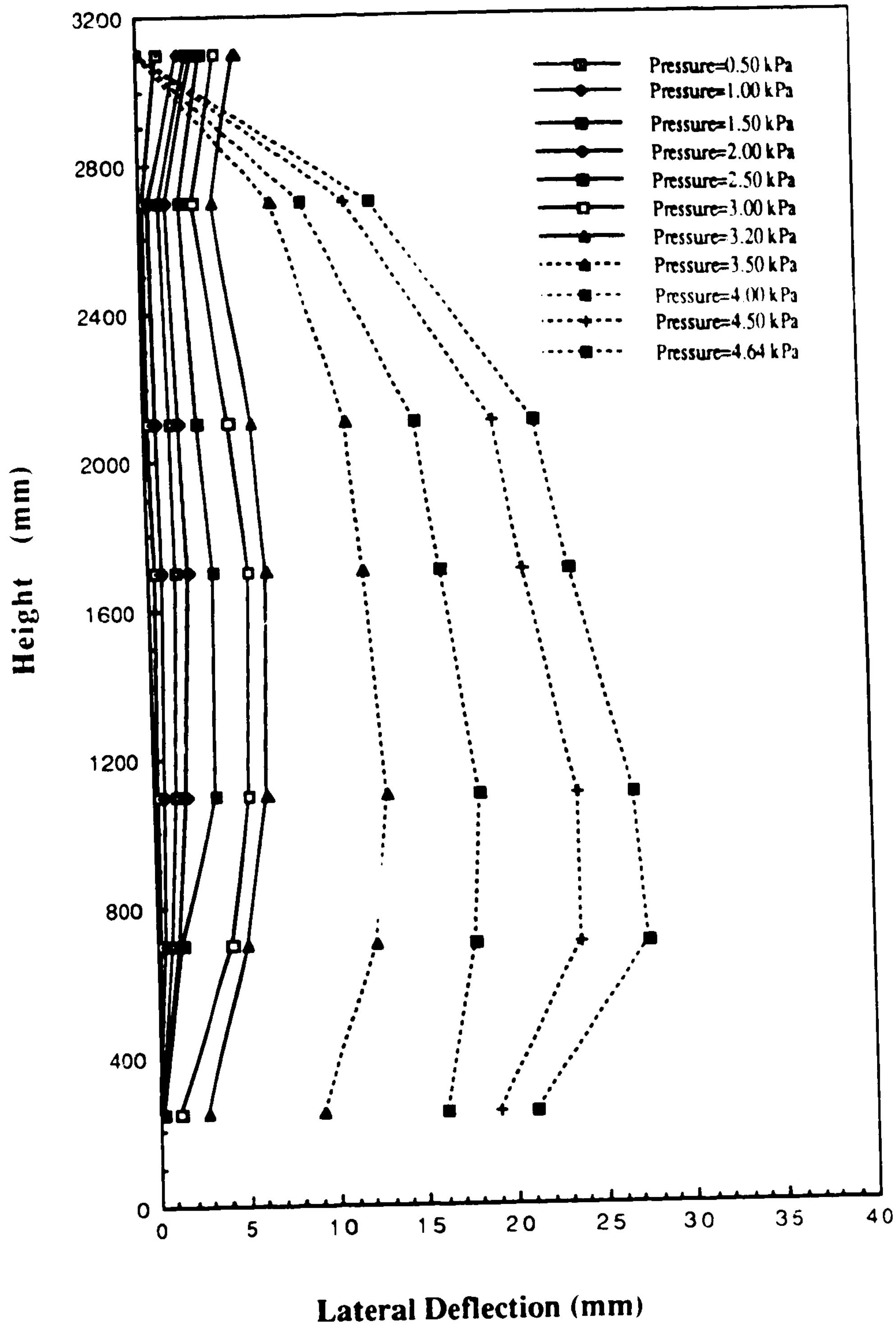

Figure B-9 Brick Wythe Deflection for S2W2

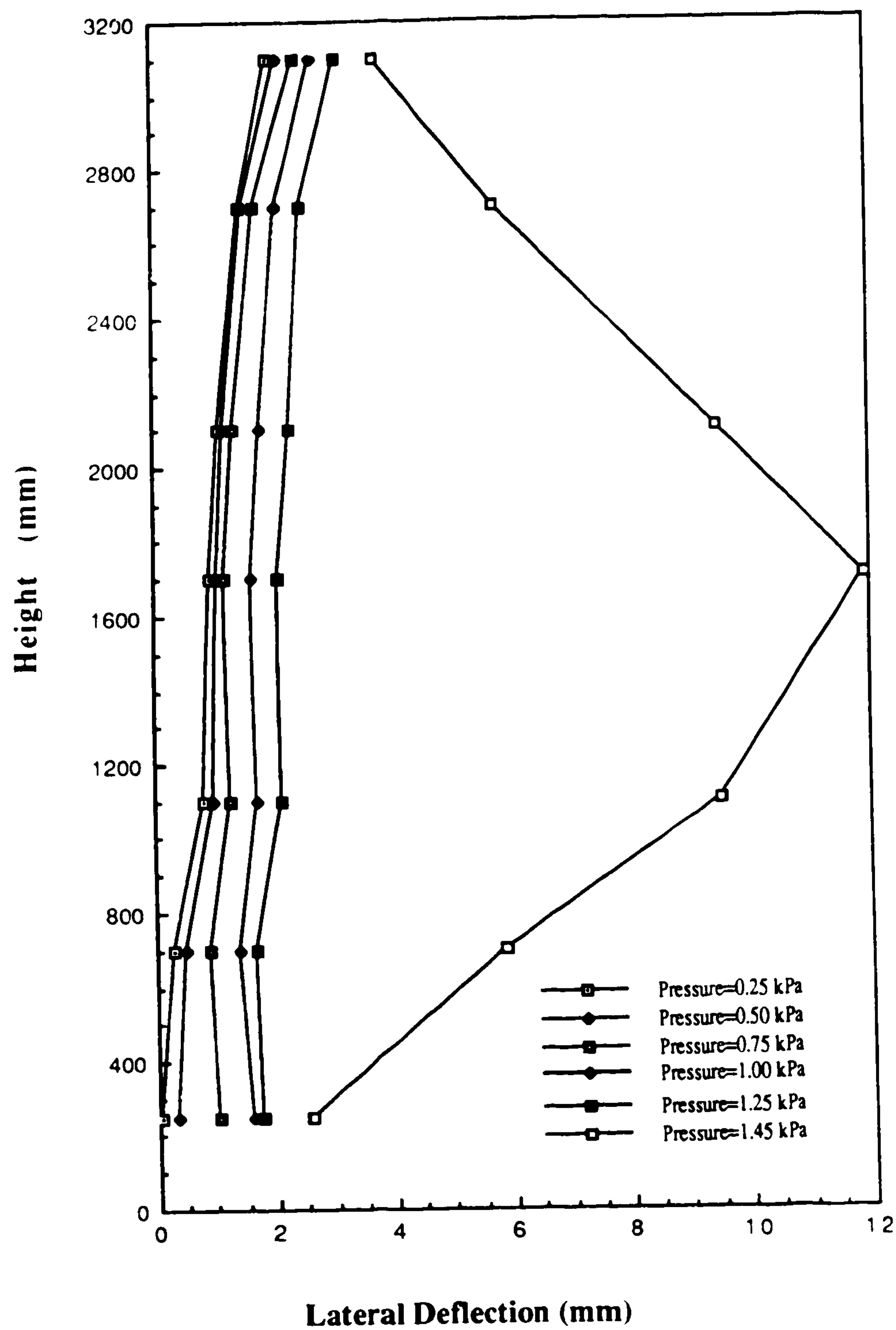

Figure B-10 Brick Wythe Deflection for S2W3

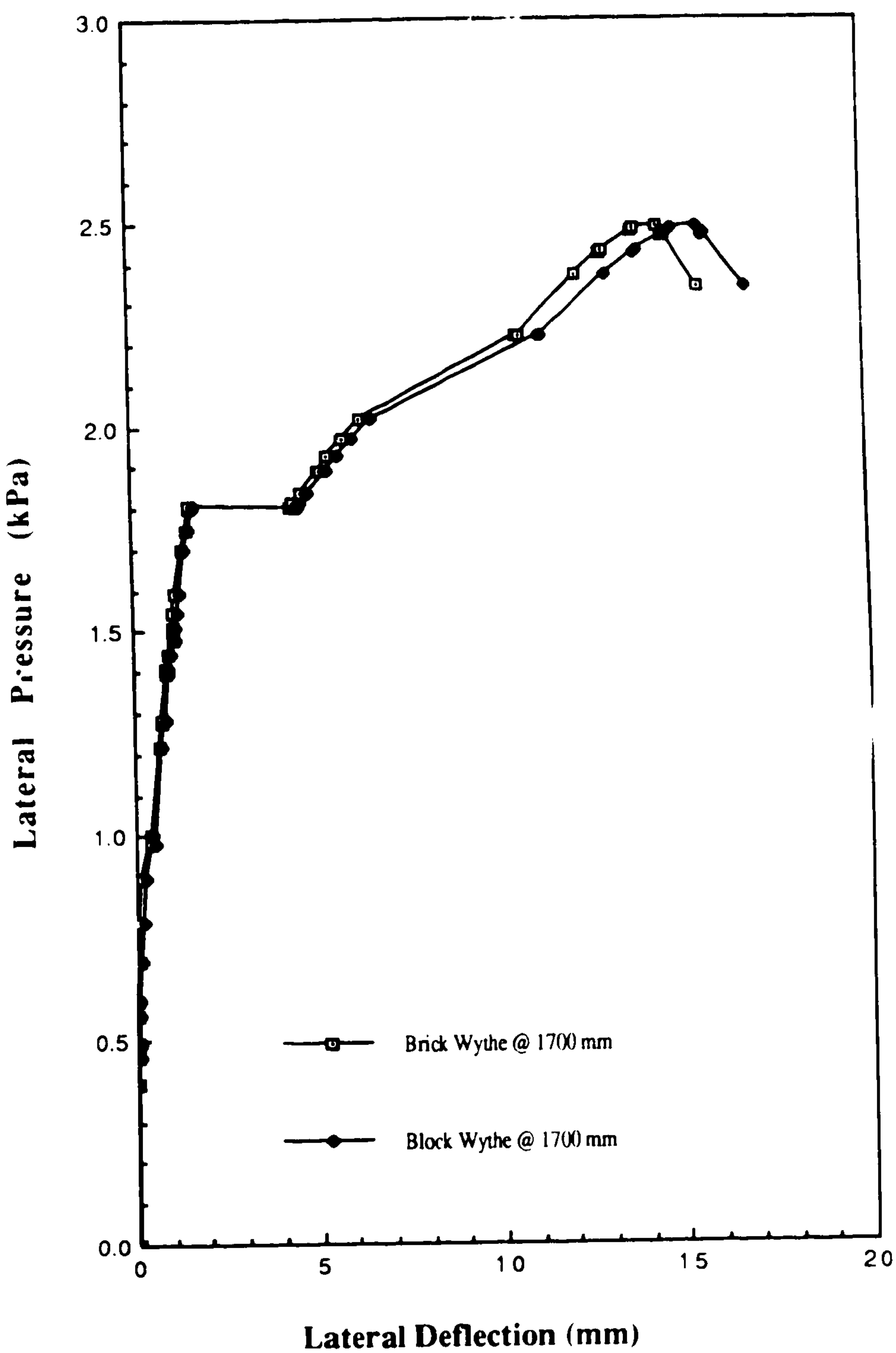

Figure B-11 Pressure vs Centerline Lateral Deflection for S2W4

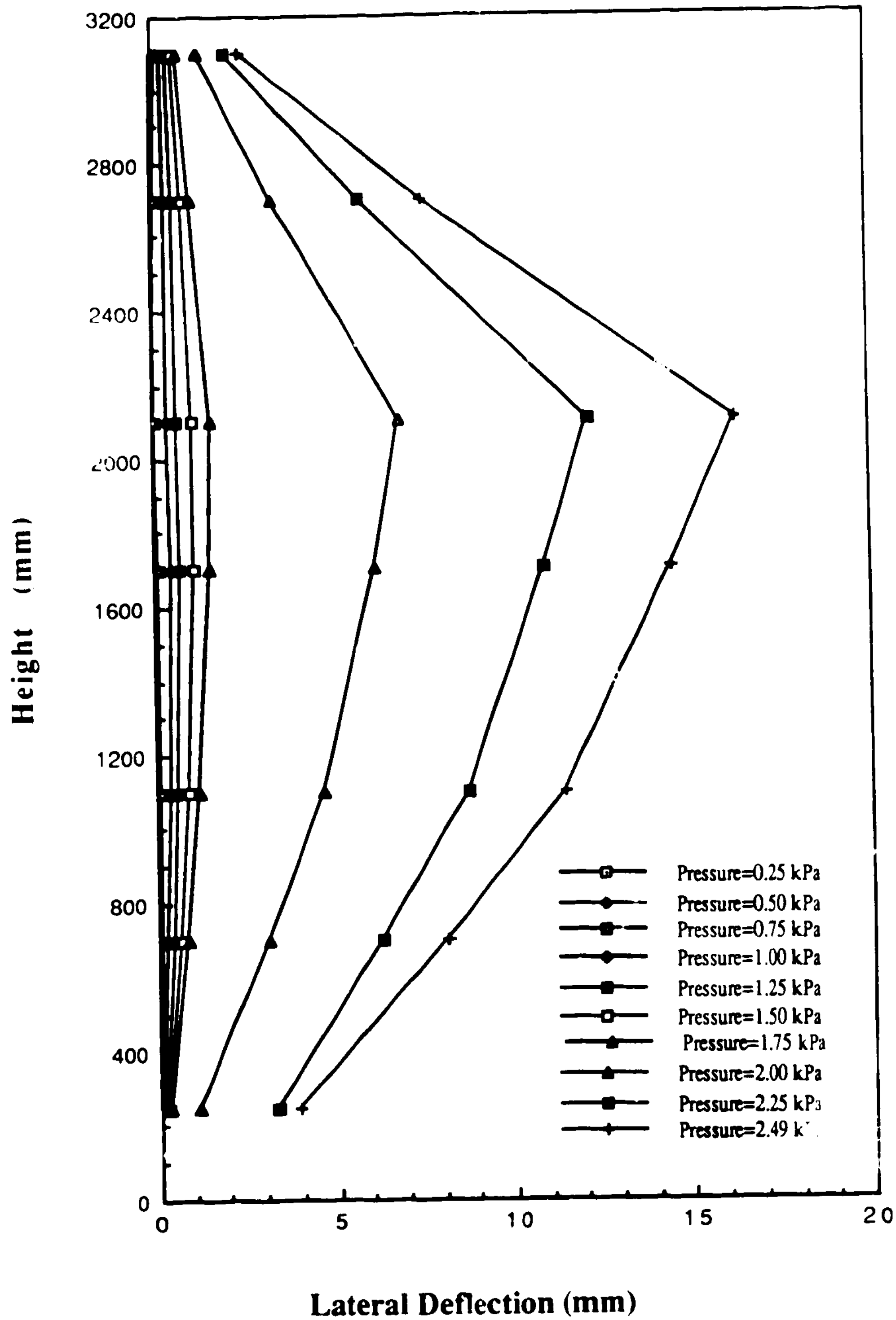

Figure B-12 Brick Wythe Deflection for S2W4