Performance of Brick Veneer Ties Under Cyclic Loadings

by

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ABSTRACT

The subject of this research program was to investigate the behaviour of brick veneer ties under reversed cyclic loading. As the first step towards the performance evaluation of the behaviour of brick ties under simulated earthquake loads, tests were performed on the embedded part of brick veneer ties. Small wall elements with one veneer tie were made to represent a localized portion of a full height brick veneer wall system.

Some building codes require single wire joint reinforcement in masonry veneer walls in higher seismic zones, the current investigation examined the effect of the horizontal wire joint reinforcement on the embedment performance of brick tie currently used in the Western Canada. Tests were also conducted on several different types of veneer ties available in North America for comparison purposes. One important parameter considered in the study was the effect of in-plane surcharge loads, which simulated the condition of brick ties located near the top and near the bottom support of a one story wall.

The experimental study was divided into three phases. Phase I served as a pilot study to help plan the subsequent tests and to test the custom designed test apparatus for the tests. In Phase II, 18 specimens were built with one type of tie having three different conditions, which were tie only, joint reinforcement not connected to the brick ties, and joint reinforcement connected to the brick ties. Other tests were conducted using the US seismic type of ties, conventional ties and to investigate the effect of construction errors. In Phase III a modification to the connection of the tie to the horizontal wire joint reinforcement was made in order to see if an improved performance would be achieved. Also the testing protocol was modified to investigate its influence on the tie behaviour.

Evaluation of test results were based on the hysteretic curves obtained from the tests and the load displacement envelopes. Discussions are based on the comparisons of different conditions. It was observed that the peak embedment strength is influenced by the tightness of the connection of tie to the joint reinforcement. An increase in embedment strength at large displacement could be achieved with wire joint reinforcement connected to the brick ties. Overall, adding wire joint reinforcement to the brick ties only slightly influenced its embedment strength under cyclic loads.

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Ari Wibowo
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Chapter 1 Introduction

1.1 Background

Exterior cladding system as an envelope to a building is frequently overlooked in terms of its structural performance. The design of an exterior wall system usually falls in the grey area of the professional responsibility between the architect and engineer. Thus, the detailed design of the wall components sometimes being neglected by both parties or becomes a secondary importance in the overall design phase of the wall systems.

In Canada and North America, the use of masonry veneer wall systems as an envelope or cladding system in residential (high-rise) and commercial buildings (low-rise) is widely acknowledged. Typical masonry veneer wall systems consisted of burned clay bricks as an exterior wythe, with the definition of wythe as one masonry unit in thickness with a minimum thickness of 75 mm. The veneer is connected to the structural backing wall by corrosion resistant metal ties embedded in the mortar joint. These metal ties are the component of the masonry veneer system that transfers all the lateral loads from the bricks to the structural backup wall. Thus the tie becomes one of the main structural components when the wall system is subjected to lateral loadings.

Previously the structural consideration of these ties is only based on empirical information and largely based only on the designer's own judgement. With more masonry veneer wall systems being built, the design criteria of wall ties become important. In Canada, CAN CSA A370-94 Connectors for Masonry provides the requirements for designing wall ties and connectors. A detailed description of the masonry wall tie can also be found in Brick Industry Association (BIA) Technical Notes (Ref. 8), which addresses all the issues regarding wall ties normally used in masonry wall system.

The wall tie, which transfers lateral loads from the brick veneer to the structural backing, is clearly one of the main components to consider in a seismically active region. Thus there are thoughts and concepts being introduced to achieve better performance of these wall ties. These concepts are then implemented in the building codes by means of establishing some requirements that have to be applied in order to achieve a reasonable performance under seismic regions. In the US, building codes require that horizontal bed joint reinforcement should be provided in the courses containing brick tie in seismic regions. With the seismic regions divided into several categories, the codes also introduced different specification according to this category. For the most severe category (i.e. the most seismically active regions), there is a requirement that the tie should have a system that will attach, enclose or engage the horizontal wire bed joint reinforcement that is embedded in the mortar joint. Considering that brick ties are a proprietary product, the concept of a connection that can attach, engage or enclose the horizontal wire reinforcement will largely depend on the manufacturer of the tie system.

While the US building codes provide seismic requirements for masonry wall ties, the Canadian masonry code, i.e., CAN CSA A370-94, does not specifically address the issue of wall ties in seismic regions in Canada. Considering that masonry is one of the most widely used wall system in Canada and the fact that some regions in Canada are seismically active; therefore it is important to assess the seismic performance of the masonry veneer wall ties. One approach that is considered to be a direct and efficient way is to investigate whether the requirements in the US building codes are necessary.

The structural design and construction of the brick veneer wall and its component is based on an empirical approach. Therefore, it is important to define the logical reasons that underlie the concept of using horizontal joint reinforcement in seismic areas. To trace the root of the concept, several discussions (Reference 4,5,8,10,26,30) and a literature study were conducted. It was hard to find such a test conducted to assess the performance of the brick veneer wall tie with the horizontal wire joint reinforcement. Even in the US itself, several opinions about the use of the horizontal joint reinforcement in brick veneer wall system can be found. With a large selection of wall ties being used in the brick veneer wall system today, it is evident that there is a possibility that the effect of the horizontal joint reinforcement is specific to the type of the tie being used.

There have been several opinions and assumptions regarding the issue of the advantages of using horizontal wire bed joint reinforcement in a brick veneer wall. Two widely accepted reasons for the use of a horizontal joint reinforcement; one is that the joint reinforcement will help to improve the embedment capacity of brick ties, while the second one is that the joint reinforcement will improve the overall integrity of the veneer assembly. There are several papers and discussions that express these opinions. The Brick Industry Association (BIA) (Reference 5) states that "Horizontal joint reinforcement should be used to add integrity to veneer constructed in locations with intermediate and higher seismic activity or when the units are laid in stack bond," also it is written that the wire should engage the brick veneer tie in seismic area and must be discontinuous at movement joints. Borchelt (Reference 4) published a paper on seismic performance of brick veneer, concluded that the joint reinforcement will provide added strength for the ties, this can be assumed as the embedment capacity, and also maintain the brick veneer integrity if cracks occur due to the lateral forces.

Another comment on joint reinforcement by Catani (Reference 10) stated that in high seismic areas, single wire joint reinforcement is required in the veneer to provide ductility and better anchorage. A previous report based on an analytical study from KPFF Consulting Engineers (Reference 26) on brick veneer over steel stud, concluded that acceptable structural performance with respect to the response to earthquake ground motion could be achieved if the ties are capable of providing adequate anchorage between the brick veneer and the steel studs. Therefore, adequate post-cracking tie anchorage will ensure adequate performance and as a result the detailing of the anchorage of the brick tie to the masonry becomes the critical part, such that this anchorage must maintain its integrity after cracking occurs. The use of mechanical anchorages such as wire or sheet metal that is mechanically attached to embedded items in the masonry bed joint will help to maintain the anchorage strength after the veneer has cracked. Also, data from manufacturers of the tie usually specify a higher pull-out or push-through ultimate strength (embedment capacity), if their tie is being used with the horizontal wire joint reinforcement. (Reference 18,19)

Because there is a lack of sufficient background information and data regarding the issue of horizontal bed joint reinforcement on the brick veneer, the need for research into the assumptions and system performance, is required. The experiments conducted by the manufacturers of ties usually only cover monotonic tests that do not represent the behaviour of the ties under reversed loading which is important for understanding the cyclic response of any structural components and systems. In North America, there have been almost no reversed cyclic loading tests conducted on this system to study the behaviour of the tie with the wire reinforcement, therefore no standard test protocols could be found. Overall, the testing of the tie with wire reinforcement under cyclic loading can be categorized as a new approach in determining its response under reversed loading. Based on all of the reasons described above, this research program was conducted to examine whether horizontal wire reinforcement will affect the embedment capacity or embedment performance of brick ties under reversed cyclic loading. This was intended to follow the trend of assessing the ductility and hysteretic properties of the main seismic load resisting members of a structure. While cyclic loading tests does not actually simulate earthquake loading of the ties, it provides a good indication of the ties performance under this condition.

1.2 Objectives

The research program was divided into three phases as described below.

The first phase of the program was a pilot test series with the following objectives:

1. To validate the function of the test frame apparatus that was designed for the experimental testing.
2. To experimentally investigate the behaviour of the veneer wall tie embedment capacity as a guide for the next test series.

The second phase of the program was the main test series with the following objectives:

1. To experimentally investigate the behaviour and performance of a veneer wall tie embedment capacity, with or without horizontal joint reinforcement (wire reinforcement). This includes the variation stated in the US Code for different category of seismic zones
2. To experimentally investigate the behaviour of different types of brick veneer wall ties embedment capacity under cyclic loading. The type of ties included in this research program was the specific seismic type of ties and conventional ties from different manufacturers.
3. To experimentally determine the effect of construction tolerances or improper placement of the ties embedment capacity with or without joint reinforcement. This was intended to simulate a real construction practice.

The third phase was added upon completion of the previous ones, and was a further investigation of the observed behaviour of the veneer wall tie with the horizontal bed joint reinforcement. The main objectives were:

1. To obtain the monotonic pull-out or tension resistance of the wall ties in order to investigate the peak strength of the embedment capacity and the backbone of the envelope curve of load against displacement.
2. To experimentally investigate the effect of number of cycles in the loading protocol.
3. To experimentally determine the adequacy of the mechanical connection between the tie and the horizontal bed joint reinforcement and provide a solution to improve its capacity and to examine the behaviour of the improved system.

1.3 Methodology

The objectives of this research program were to study an element consisting of one brick tie in a surrounding brick panel that would simulate conditions in a wall, since this program is concerned only with the embedment capacity issue of the brick tie with horizontal joint reinforcement.

To ensure embedment failure, the experimental testing used only the embedded wire portion of a two-piece adjustable tie commonly used in Western Canada. Because the nature of the brick tie as a proprietary product, all the design and construction considerations of the tie and horizontal joint reinforcement are specific to the type of tie being used. The mechanical connection of the horizontal wire reinforcement and brick tie also confirms to the manufacturer's design. Fig. 1.1 shows the embedded wire tie and mechanically connected horizontal joint reinforcement that was used in the research program. Additional tests were also conducted on different type of proprietary ties that are commonly used with horizontal wire reinforcement in the US for comparison.

The design of the brick tie commonly used is by the tributary area of the tie, which comes from the spacing requirements for each tie. It is also widely known that the stiffness of the backup system will affect the distribution and demand of the load on each tie, thus to determine only from the tributary area is not appropriate. Therefore in the North America, the current codes address this issue by setting a maximum limitation on the spacing of ties, which results in a maximum tributary area for each tie. To take account for the backup system used, the Canadian CAN CSA A370-94 also stated that for a flexible backup system the design load for the tie should be double of the load calculated by its tributary area, therefore maintaining the demand load on the tie to be at least conservative.

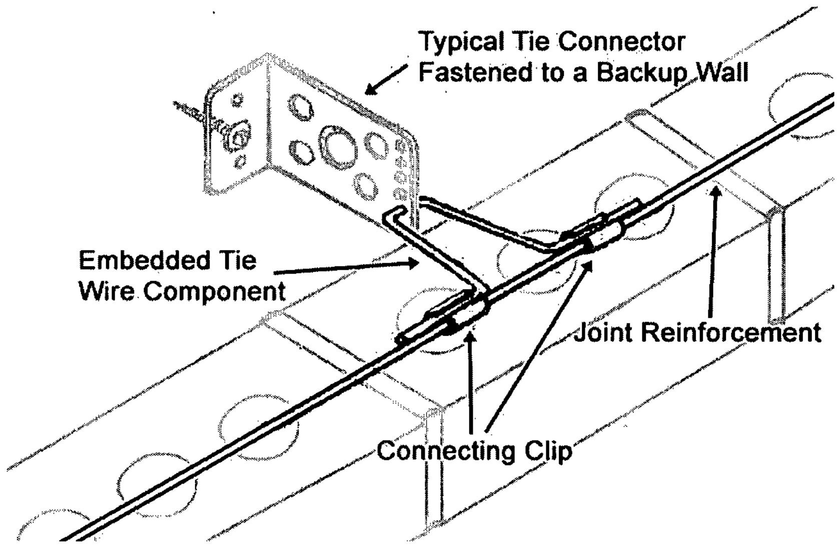


Fig. 1.1 – Two-piece adjustable tie clipped to a single wire joint reinforcement for the main study of the experimental research. (Adapted from Ref. 19)

US codes specify several spacing limitations for the brick tie related to their type, with a maximum of 450 mm (18 in.) vertically and 600 mm (24 in.) or 800 mm (32 in.) horizontally. In Canada according to CAN CSA A370-94 Connectors for Masonry, type of wall tie can be categorized to conventional (Clause 9.5) and non-conventional (Clause 10). The limit on the spacing of ties (horizontally and vertically), are governed by these tie types. Typically the maximum spacing between ties are 800 mm horizontally and 600 mm vertically (Clause 6.1.1.2 Non-conventional Ties). The spacing at the top of walls is governed by the Clause 6.1.3, the distance of the first row of ties to the top of the wall should not be greater than 300 mm for a free top of wall, and should not be greater than 400 mm for a minimal lateral support such as from a flashing material that placed between the top part of the masonry wall and the shelf angle. Figure 1.2 shows the requirements for the spacing of wall ties. Based on these requirements the size of the brick panel chosen for the experimental testing were 450 mm (18 in.) high and 800 mm (32 in.) wide with a single brick tie at the centre be according to the spacing limitations, with the width of specimen also provides adequate joint reinforcement length.

For the main study on this experimental research program, three specimens of different configuration of brick panel specimens were constructed. These are specimens with a brick tie without a horizontal joint reinforcement, brick tie with a single wire horizontal joint reinforcement that was not connected to the tie (not clipped), and brick tie that was connected to the horizontal wire joint reinforcement (clipped). As for the additional tie test, each type at least had two specimens being constructed. Experimental results were then analyzed and discussed.

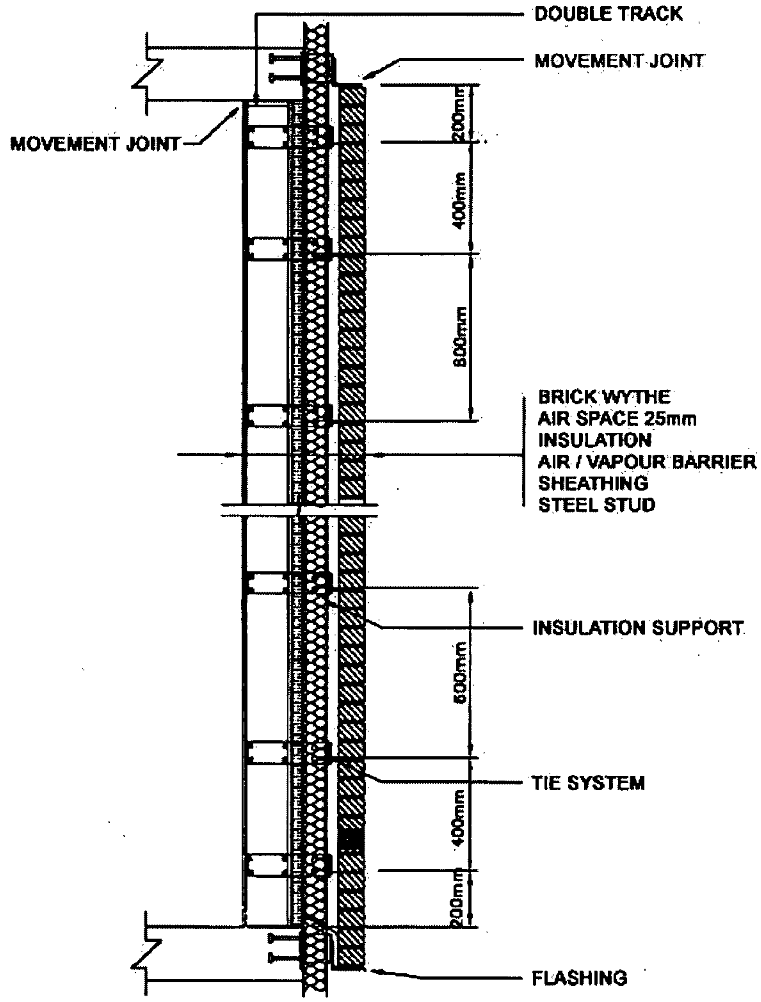


Fig. 1.2 – Tie vertical spacing requirements according to CAN CSA A370-94. (From Ref. 20)

1.4 Outline of Thesis

This thesis consists of five chapters. Chapter 2 provides a literature review on the subject, that is masonry veneer in general and then narrows down to the specific brick tie subject. In that chapter there is also information on several experimental studies that have been conducted on the brick tie and masonry veneer wall that could be relevant and useful to the experimental study that is presented here. The experimental program is described in Chapter 3. This includes explanation of the test frame apparatus, material properties, test set-up and instrumentation and the procedure used for cyclic testing. In Chapter 4 the experimental results is discussed and analyzed. The thesis concludes with Chapter 5, which presents conclusions of the experimental tests and also recommendations for the designs and possible future study of the dynamic behaviour of masonry veneer wall system.

Chapter 2 Literature Review

2.1 Structural Concept and Construction of Brick Veneer Wall

Brick veneer wall is an exterior wall system using burned clay units, which consists of one masonry unit in thickness with a minimum thickness of 75 mm; this one unit with specified thickness is called a wythe. As an exterior wall, it is securely attached or fastened to the structural backing wall, which can be steel, wood or concrete. Although it is attached to the backing wall, the veneer wall should not exert a common reaction with the backing wall under lateral, out-of-plane loading. This will restrict the wall system not to behave as a composite system with the structural backing wall.

The brick veneer is designed to resist only lateral loads such as those imposed on the structures by winds or earthquakes. Although there is a vertical load applied on the brick from the self-weight, the overall wall system is not designed to be a vertical load path for the structure (non load-bearing); therefore it is intended only to act as a layer. The brick veneer rests on a shelf angles as a vertical support, which is attached to the main structural component or to the secondary support framing.

From a structural point of view, a brick veneer wall system is composed of brick veneer that act as the outer wythe, connectors, cavity or air space, backup wall or the interior wythe, and the vertical supports for the wall. The clay brick typically used is 90 mm in thickness with standard dimensions of 190 mm in length and 57 mm in height. The bricks are typically running bond and the height of the wall depends on the spacing of the vertical bearing support (maximum is 3.6 m according to CAN CSA S304.1). The veneer is too slender to stand by itself without a lateral support and because it is intended to resist lateral loads, connectors are needed to attach it to the structural backing and to transfer all the lateral loads. These connectors are horizontal metal ties and are spaced with a regular interval. The horizontal ties must transfer all horizontal tensile or compressive forces due to wind or earthquake from the veneer to the backing wall, so it is a very important part in term of the structural behaviour of the veneer wall. Therefore it requires sufficient strength and stiffness to perform the function. These connectors are essentially tying the veneer to the building and span across the cavity or air space between the veneer and the structural backing. The cavity or air space is normally 25 mm and can also provide space for an insulation material. The backup wall or the interior wall will receive all the lateral loads that being transferred by the ties; but the backup wall will not act in composite with the veneer, therefore no shear transfer action will occur. The most common backup wall used for a low-rise is wood stud framing, and for a high-rise construction is steel stud framing. The rigidity of the backup wall will influence the serviceability of the veneer. The more flexible the backup wall is, the more lateral loads that the veneer has to carry. The ratio of rigidity between the veneers and the backup wall will actually be the governing value to the serviceability limit of the system. Figure 2.1 is a typical brick or masonry veneer wall with a backup wall.

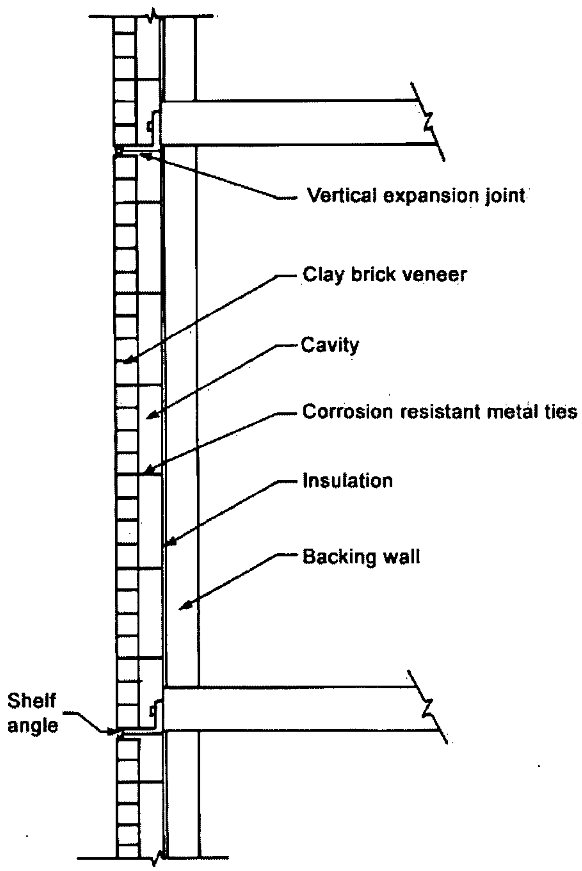


Fig. 2.1 – Typical masonry veneer attached to a backup wall. (From Ref. 27)

When a brick veneer wall system is subjected to out of plane lateral loads from wind or earthquake, from the design concept, these forces should be transferred to the structural backing wall. The ties will directly transfer these forces to the backing wall, if the backing wall is rigid (such as a concrete block), all ties that are engaged will give the same contribution in term of tie forces. In case of a flexible backing wall, the brick veneer, which is more rigid, will eventually resist most of the loads, until it cracks. The distribution of tie forces when the veneer is uncracked and after first crack occurred is considerably different. According to the Brick Industry Association (Ref. 7) and an analytical investigation developed by KPFF Consulting Engineer (Ref. 26) on brick veneer steel stud walls system, portion of the load that will be carry by the veneer and the steel stud is relative to their flexural stiffness, span length and the stiffness of the ties itself. The brick veneer is stiffer than the steel stud, thus it carries most of the lateral load. The ties that carry most of the loads are the ones nearest the top and bottom support of the steel stud, as the deflections on the brick wythe are transmitted to the steel studs through the metal ties. In this condition (uncracked) the brick veneer behaves as a one-way beam. As the load is increasing, the brick veneer continues to deflect and this will cause cracking because the flexural tensile stress developed in the veneer exceed the modulus of rupture of the brick veneer and most likely the crack will be near the centre where the largest moment concentration is.

After crack occurs, the brick veneer will act as a two separately independent one-way beam, which span between the tie nearest the crack and the respective tie at the bottom or the top of the veneer. Figure 2.2 will show the distribution of forces on the tie before cracking and after first crack occurs. The stiffness of the ties will also contribute to the distribution of the tie forces. Another important factor is the metal or steel stud stiffness, as the more rigid it is, the more it will resist the load and the distribution of tie forces will be more uniform. This subject of brick veneer on steel stud backing wall system has long been an interesting topic to investigate for its unique behaviour.

Acceptable structural performance will be achieved if the ties can provide adequate anchorage between the brick veneer and the backing wall. Acceptable behaviour of the tie can be explained by the post-cracking tie anchorage to the brick veneer and the backing wall. If the tie could maintain its anchorage after cracks occur, an adequate system performance will be achieved. In a masonry veneer wall, several ties provided as a support for the wall. Thus the brick veneer wall system with metal ties anchor can be considered as a very redundant system. If the ties are sufficiently flexible, the distribution of loads to the adjacent ties can be accomplished and the ability of the ties to dissipate energy as they deform are several factors that have to be taken into account if they were designed to resist wind or earthquake loadings.

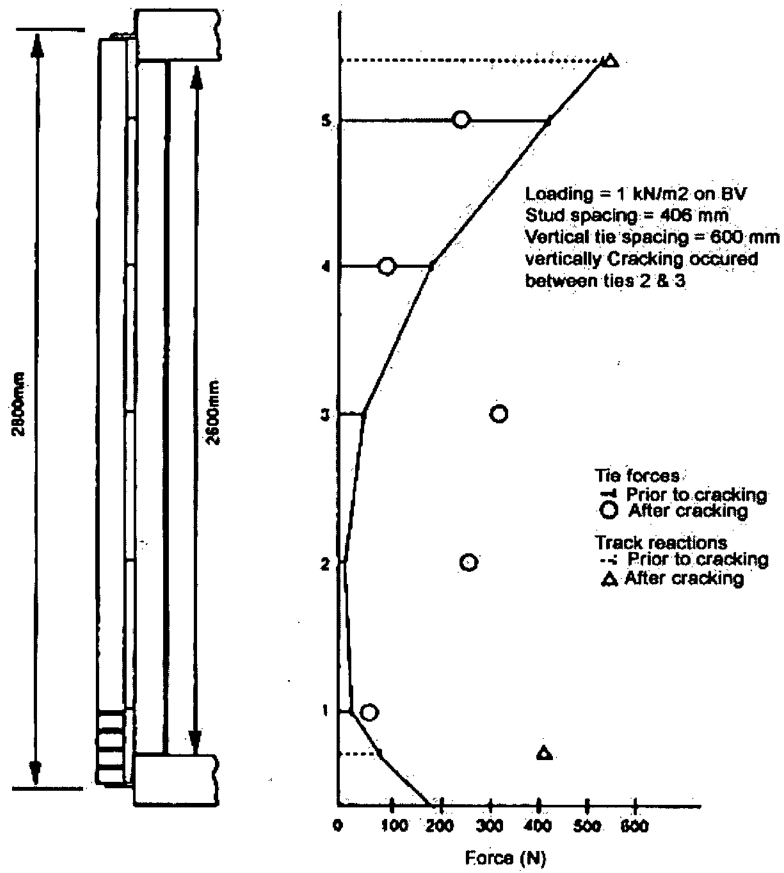


Fig. 2.2 – Distribution of tie forces prior to cracking (uncracked) and after cracking (post-cracked). (From Ref. 23)

As explained before the detailing of the anchorage (that is the embedment) of these ties to the brick becomes a critical point, such that this anchorage must maintain its integrity after the veneer has cracked. From this judgement, there is an approach on the design of the system by applying mechanical anchorages such as wire reinforcement that is mechanically attached to embedded items in the masonry bed joint, which will in turn maintain their strength after the brick veneer has cracked (Ref. 26). Brick Industry Association also stated in their technical notes that this wire (horizontal joint reinforcement) will add integrity to the system after cracking. Another comments from Dur-O-Wall Inc. and Fero Corp, which are the manufacturer of these metal ties (in North

America), suggested that the horizontal wire would provide more anchorage (increasing push-through and pull-out resistance of the ties from the bed joint) and improve ductility. It is therefore of great importance to actually predict the advantages or improvements that this horizontal wire reinforcement will give to the metal ties, especially under severe loading condition that will ensure the veneer to crack such as from an earthquake load. Figure 2.3 shows the brick veneer system with the horizontal wire joint reinforcement.

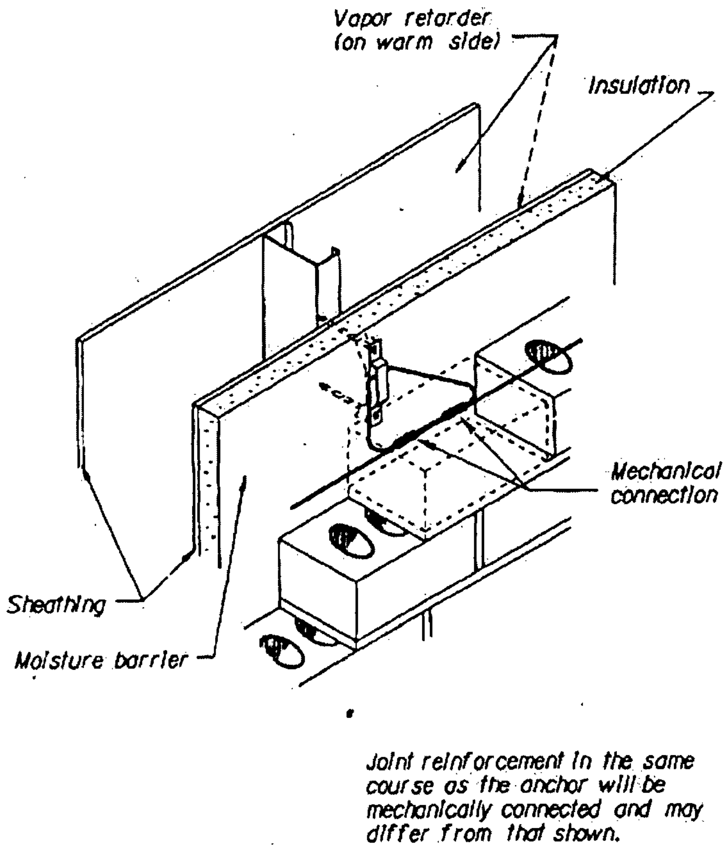


Fig. 2.3 – Masonry Veneer wall system, with attached horizontal wire joint reinforcement. (From Ref. 32)

2.2 Seismic Requirements of Brick Veneer Wall System

An earthquake is a natural phenomenon, which causes the ground to moves both horizontally and vertically. The response of this ground motion would create an inertia force in the brick veneer, which will have to be resisted by the overall system. The vertical forces can be neglected because of the reverse strength that the brick veneer has from its self-weight, whereas the horizontal forces (lateral loads) are the one that require special considerations in design and construction.

In a brick veneer wall system the inertia force generated by the mass of the brick will contribute as an out-of-plane loading to the system. In other words the metal ties that act as a connector between the bricks and the backing walls must resist these loads, in order to keep the brick veneer attached to the structural backing wall, as the term anchored veneer relates to. Thus the performance of the ties will be the most significant part in considering the seismic requirements of a brick veneer wall system.

In the United States, seismic requirements for veneer is govern by the Uniform Building Code (UBC) and the Masonry Standards Joint Committee's ACI 530/ASCE 5/TMS 402 Building Code Requirements for Masonry Structures (MSJC Code). Both of the codes specify the connections of the masonry veneer to the structural backing wall, depending on the type of the backing wall and on the expected seismic loads. Masonry veneers located in an area with the lowest seismic risk have no special requirements, and as the seismic risk increases the requirements for the attachment of the masonry do too.

1997 UBC covers anchored veneer in section 1403.6, which specifies tie size, spacing and physical properties for masonry. According to the Seismic Zones map (Zone 0 with almost zero possibility of earthquake and Zone 4 with a high probability of a strong earthquake), in Seismic Zones 3 and 4 the UBC has the following requirements: "In Seismic Zones 3 and 4, wall ties shall have a lip or hook on the extended leg that will engage or enclose a horizontal joint reinforcement wire having a diameter of 0.148 inch (No. 9 B.W. gage) or equivalent. The joint reinforcement shall be continuous with butt splices between ties permitted."

While in the MSJC code 1999, section 6.2.2.10 has similar requirements for masonry in higher seismic areas. However, the MSJC Code is using a different system to categorize the level of seismic risk in an area, which is called a Seismic Performance Category (SPC). Although it is resemblances to the UBC Seismic Zone category, it does not really match up exactly with the UBC Zoning map. The SPC go from A through E. In SPC C, there must be isolation between the anchored veneers and the structure on the sides and top, so as not to transfer the seismic forces resisted by the structure to the veneers.

In SPC D, there are additional requirements on the support of the veneer, anchor spacing, and a requirement to provide continuous, single-wire joint reinforcement of minimum wire size W1.7 with a maximum spacing of 18 in. (457 mm) on centre vertically as minimum reinforcement. And for the SPC E the requirements for SPC C and D also apply with an addition of providing vertical expansion joints at all returns and corners and also to mechanically attach anchors with clips or hooks to the joint reinforcement with the size and spacing required.

While in the US there are codes that provide a stringent policy of the anchored veneer, in Canada there is only one code that governs all the requirements for masonry that is the CSA Standard S304.1 Engineered Masonry Design – Limit States Design. In this code Section 13-Unit Masonry Veneer covers the requirements for masonry veneer including the thickness, tie spacing, tie design load and the structural backing. However there are no special requirements can be found for masonry veneer under seismic areas. This is one of the drawbacks in the Canadian masonry codes; and therefore with this research and experimental study, the necessity to address these seismic requirements for veneer wall system in the code could be accomplished.

2.3 Wall Ties Structural Requirements and Performances

Based on the design concept of brick veneer wall attached to a structural backing, the veneer should only be imposed by axial force from its own weight and the backup wall should be designed to resist the full lateral load. Thus, theoretically the only strength requirement is for the ties, which should be able to transmit wind or seismic loads to the backup walls and the structure. In order to meet this requirement, the brick veneer must have a sufficient flexural strength to span at least between ties so there is no potential for rotation and instability of veneer sections. Figure 2.4 shows the potential instability after cracking occurs. Therefore for stability purposes, the potential crack spacing should be at least twice the spacing of the ties; this requirement will provide the limits on maximum vertical spacing of ties. Most building codes incorporates this requirement to the design by limiting the allowable maximum spacing (either vertical or horizontal) for the wall ties.

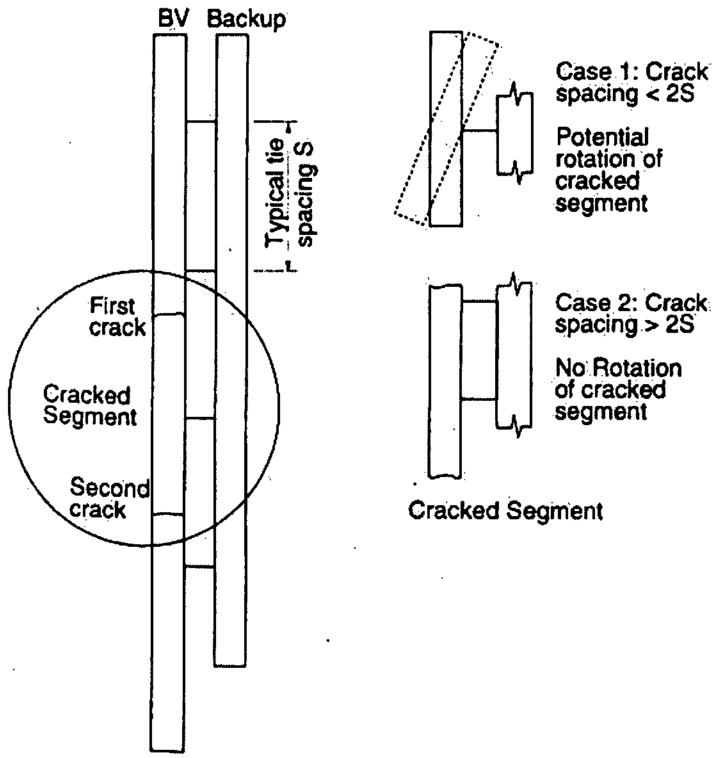


Fig. 2.4 – Potential instability of cracked veneer for large tie spacing. (From Ref. 16)

As stated in the previous discussion, the key component of an acceptable behaviour in terms of strength of the system is the wall ties that provide a connection from the brick veneer to the structural backing. The main function of the wall ties in a brick veneer wall system is to transfer force between the brick and backup wall without excessive relative movement. The wall ties should also permit an in-plane (vertical) movement to accommodate differential material movements between the brick veneer and the structural backup. The current approach to allow unrestrained vertical movement in the directions parallel to the plane of the wall is by designing the ties to be sufficiently flexible, so the differential displacement at the ends of the tie, will not cause a coupling action between the wythes. This means that the ties will be made from relatively thin plates or rods that have low flexural stiffness. Another method is by using adjustable ties, which have adequate axial strength and stiffness, but allow free differential vertical movement along the plane of the wall. Figure 2.5 below shown the design concept of this method.

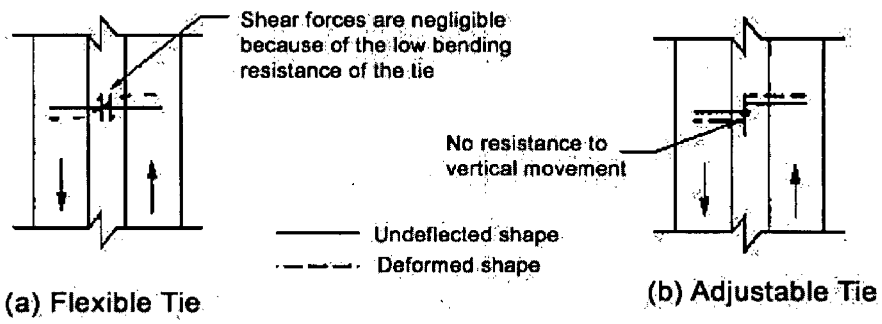


Fig. 2.5 – Flexible Tie and Adjustable Tie to allow relative movement of wythes. (From Ref. 16)

Wall ties are such a proprietary products, thus the manufacturer must provide the information regarding their performance characteristics. In Canada the code for this ties can be found in CSA A370-94 Connector for Masonry. The code categorized wall ties into conventional ties, which includes corrugated strip ties, Z-wire and rectangular wire, continuous welded ties/reinforcement (ladder and truss joint reinforcement), and non-conventional ties which includes the new seismic type wall ties. The use of adjustable two parts ties will help to avoid the interference of a projecting tie during the construction and will easily adjust to match the mortar bed joints. Figure 2.6 shows the common conventional ties most widely used in practice.

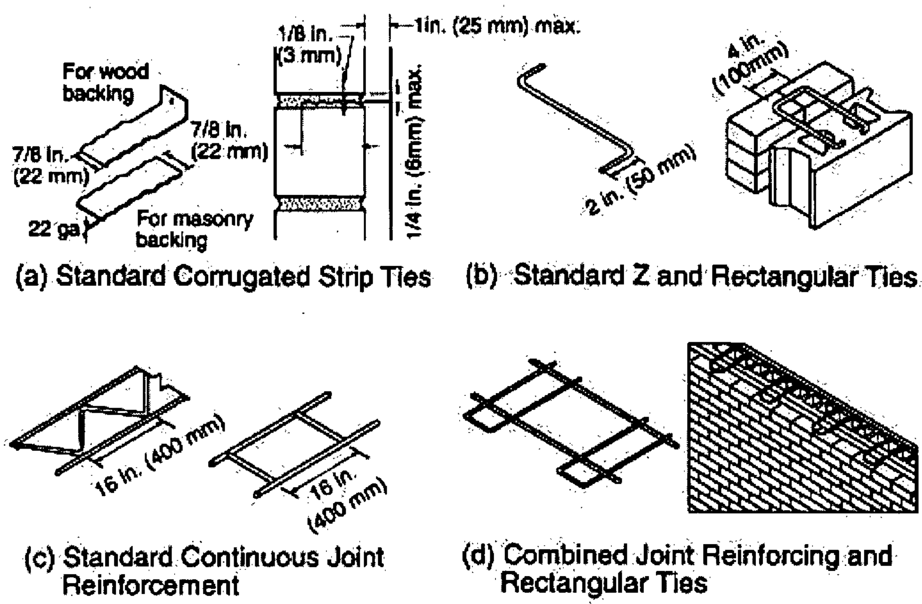


Fig. 2.6 – Standard Non-adjustable Conventional Ties. (From Ref. 16)

R.G. Drysdale, A. A. Hamid and Lawrie R. Baker in their book Masonry Structures Behaviour and Design explained that most of the non-conventional ties are adjustable type and can be categorized from the adjustability mechanism of the ties, i.e.:

1. Pintle, the vertical adjustability achieved by using pintle, which is made by a bent wire, and this pintle go through an opening or so called an eye which restrain the pintle from horizontal movement. This opening or eye is located in the second unit that is embedded in the backing wall (for concrete block). Refer to Figure 2.7 (a).
2. Slot, usually consists of triangular wire tie that fits in into a vertical slot, the length of the slot will limits the range of adjustment. Figure 2.7 (b) shows the second unit (on the interior wythe) as a backing plate used to limit tie movement perpendicular to the wall, and it is mounted over exterior sheathing.
3. Fastener adjustment, the adjustability comes from the fastener itself, which is connected to the backing wall. Figure 2.7 (c) shows a wire tie with a slot in the tie itself to allow adjustment on the tie relative to the fastener.
4. Fixed position, actually is a non-adjustable tie, but it can be attached after the desired location has been determined. Figure 2.7 (d) shows a self-drilling tie.

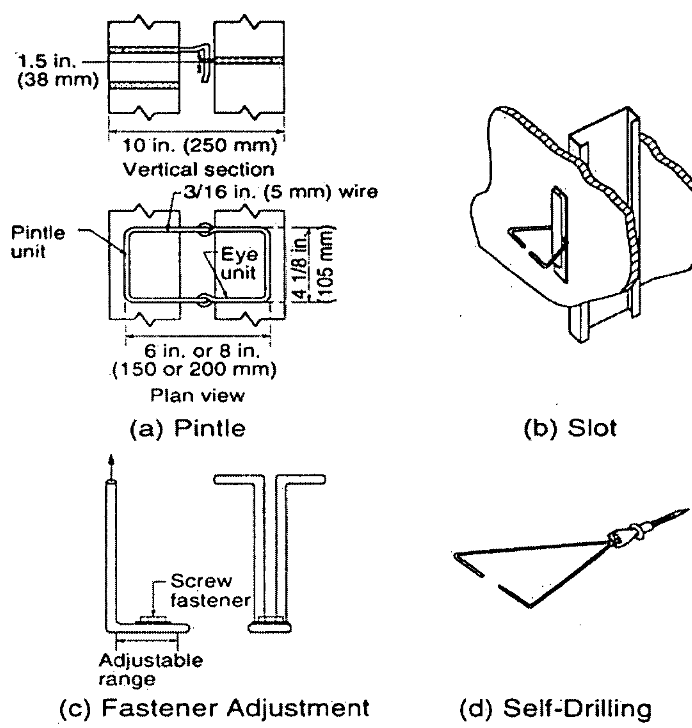
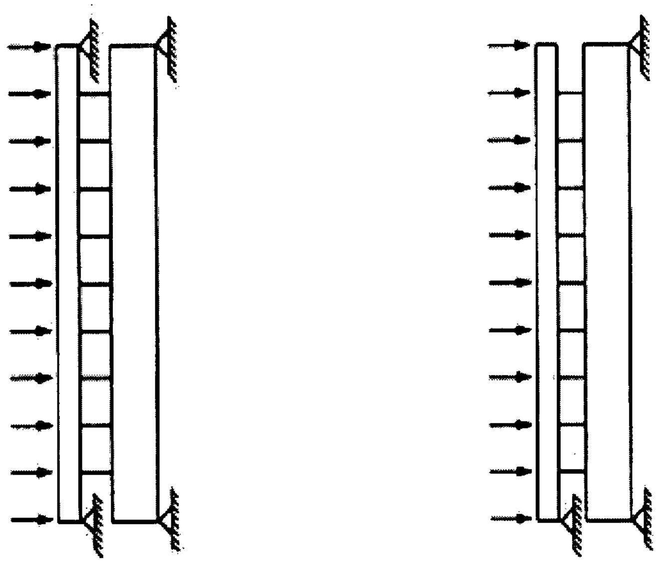


Fig. 2.7 –Non-conventional adjustable ties. (From Ref. 16)

In designing the tie loads, traditionally the demand load has been approximated using the tributary area around the tie; this results in prescriptive limits on tie spacing. This method basically assumed that all ties resist equal forces, which only possible if both the veneer and the backing wall are rigid. However in a real wall system, the wall will deflect, and the rigidities of the wythes (the veneer and the backup wall) also the tie stiffness and location determined the force in each tie.

The stiffness of the tie affects the distribution of tie forces. Analysis based on the equal deflection of the wythes implies that the ties are infinitely rigid, or the deformations of the tie are very small compared to the deflection of the wall system. For some type of ties, such as the more stiff ties (Z-tie, ladder tie, etc.), this condition can easily be satisfactory. But for the more flexible ties such as the corrugated strip tie and some adjustable ties, they may allow unrestrained movement due to mechanical play.

The support condition of the wall system also affects the distribution of forces on the tie along with tie stiffness. In Figure 2.8, two support conditions are usually found in a real wall system with: (a) wythes are supported independently at the top and bottom and (b) no independent support for the top of wall. For case (a), very rigid ties will have a load sharing proportional to the stiffness of the wythes whereas for very flexible ties there will be very less load transfer between the wythes.



(a) Independent support at top **(b) Tie support near top**

Fig. 2.8 – Top support condition for exterior wythe. (From Ref. 16)

In case (b), where the veneer only supported by ties near the top, the distribution of tie forces to transfer lateral load is affected by tie stiffness, the relative stiffness of the backup wall, and the ratio of stiffness between the wythes. With the tie acts as the support for the flexural behaviour of the un-cracked veneer heights, the maximum tie force will occurs at the top tie or at the tie nearest a crack in the veneer. However, the stiffness criteria of ties should be based on practical considerations such as the acceptable movement of the veneer and fatigue effects on other wall components.

Another important factor that affects the stiffness of the ties are the range of adjustability. This applies to the adjustable type of tie, which the tie itself consists of two parts. Research on a variety of adjustable ties designed to transfer loads from the veneer to the backup walls shows a wide range in strength in stiffness. In order to limit these values Brick Industry Association suggested a minimum stiffness for veneer ties defined by a displacement of 1.2 mm (0.05") at a load of 445 N (100 lb). There is also a requirement for maximum mechanical play of 1.2 mm (0.05"), to limit the allowance of mechanical play between the joining parts of the tie for manufacturing tolerances. These requirements are also introduce in the CAN. CSA A370-94.

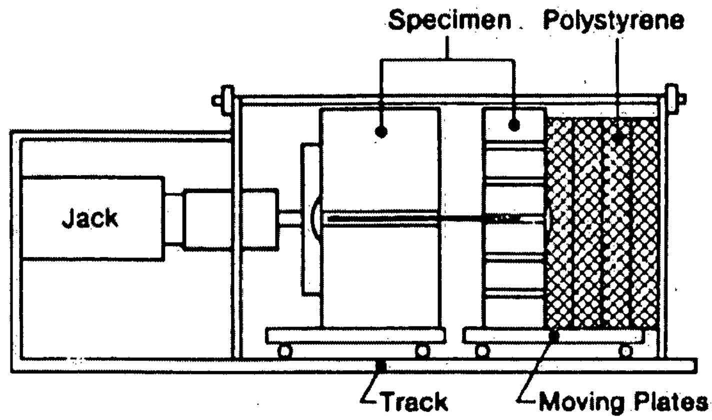
The performance of these ties has been determined under both tensile and compressive loading. Modes of failure for the common type of ties were, in compression, the failures were predominantly buckling of the tie, although for the stiffer ties, push-through of the mortar joint does occur. And in tension modes of failure were pull-out tie from the mortar bed joint, this mode of failure was influenced by the amount of embedment in the mortar joint.

2.4 Review of Brick Veneer Wall Ties Research

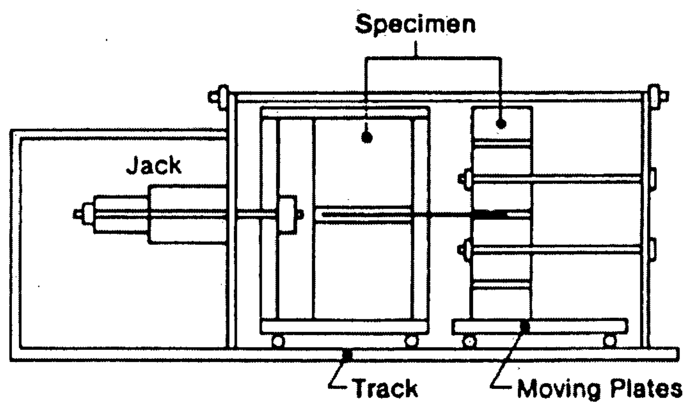
Through the 1980's and 1990's, many experimental studies were conducted to determine the behaviour of brick veneer wall system. Parameters of interest included are type of backup wall, these are mostly steel stud walls, type of ties, mounting surface of the ties (drywall etc.), the effect of adjustability of ties, location of attachment of the tie to steel stud, and the type of fasteners used to attach ties. There were some analytical study conducted to achieve a good design procedures, but mostly were dealing with wind forces and not specifically to seismic loading.

Clayford T. Grimm (Ref. 21) presented a detailed review of the metal ties and anchors for brick walls. In his paper, he clearly explained the needs to assess a rational analysis design of these ties, since at that time it was still being argued who is responsible to design the anchorage for the brick wall. He used an extensive bibliography on works related to bricks and anchorages, and summarized them. There are formulas to estimate the pull-out strength of the ties for different type of ties, which based on all the references. Through the 1980's there were quite a large number of research concerning the design of brick veneer wall on steel stud wall system, this include experimental test on the system and an analytical modeling, that was done by J.O Arumala (PhD dissertation 1982, Experimental Mechanics 1988). He included a phase where he tested different type of metal ties for the effective axial stiffness and modeled them as a linear spring. There was a growing need to obtain the behaviour of the wall ties, as it became more important. As this topic was discussed in the paper by Glenn R. Bell and Werner H. Gumpertz (1985), that such research is needed to obtain the tie embedment strength and the tie strength and stiffness.

With Brick veneer and steel stud wall system become a more popular subject, greater understanding of the wall tie has led to changes in building codes in North America. M. A. Hatzinikolas, J. Longworth and J. Warwaruk from The Canadian Masonry Research Institute (formerly Alberta Masonry Institute) had conducted a research specifically on wall ties in cavity and veneer masonry walls (Ref. 24). The objective of their test is to evaluate the ties behaviour including the effect of insulation/ sheathing in a large cavity and veneer wall. Their test was a monotonic compression and tension test to a wall tie, in which the tie was embedded in six course laid clay brick masonry in height and one brick in width with a type S mortar. The schematic of this test for concrete block backup wall is shown in Figure. 2.9. The ties were the corrugated strip tie and the adjustable rod T type tie. Parameters included in the test were the cavity width or air space between the brick and the backup wall and also the thickness of the insulation, which was placed to provide restraint to buckling or bending of the corrugated strip tie.



Compression Test



Tension Test

Fig. 2.9 – Schematic representations of compression and tension testing system used in the CMRI experimental study. (From Ref. 24)

The failure modes observed in the test could be categorized by the type of tie and the type of the backup wall. For the concrete block backup and the adjustable tie type, the compression failure were the push-out of the tie through the mortar joint (the bricks failed in flexure), while the tension failure modes mostly governed by an excessive flexure of the tie rather than pull-out of tie. With the steel stud backup, the failure in compression was the buckling of tie and the stud flange. And in tension the fasteners pull-out was the dominant failure mode. Corrugated strip ties were failed (in buckling) at a very low load.

Conclusion made from the test includes the items listed below:

- The Insulation is not effective to reduce the effective length of the rod tie for the buckling resistance and bending strength.
- The adjustable T type rod will carry more load than the corrugated strip tie.
- With larger cavity width or air space, the unsupported effective length of ties becomes longer thus there is a greater possibility of a buckling failure mode of ties in compression
- For tension, fasteners pullout governs the failure modes.

B. Pitoni, R.G. Drysdale, E. A. Gazzola, and Ahmed A. Hamid (Ref. 28) conducted a test to obtain the capacity of several types of wall ties in a cavity wall. The test was also to investigate factors that will influence the capacity of several types of tie systems in cavity wall or veneer wall such as the cavity width and anchorage condition. The test was a monotonically loaded tension and compression of ties.

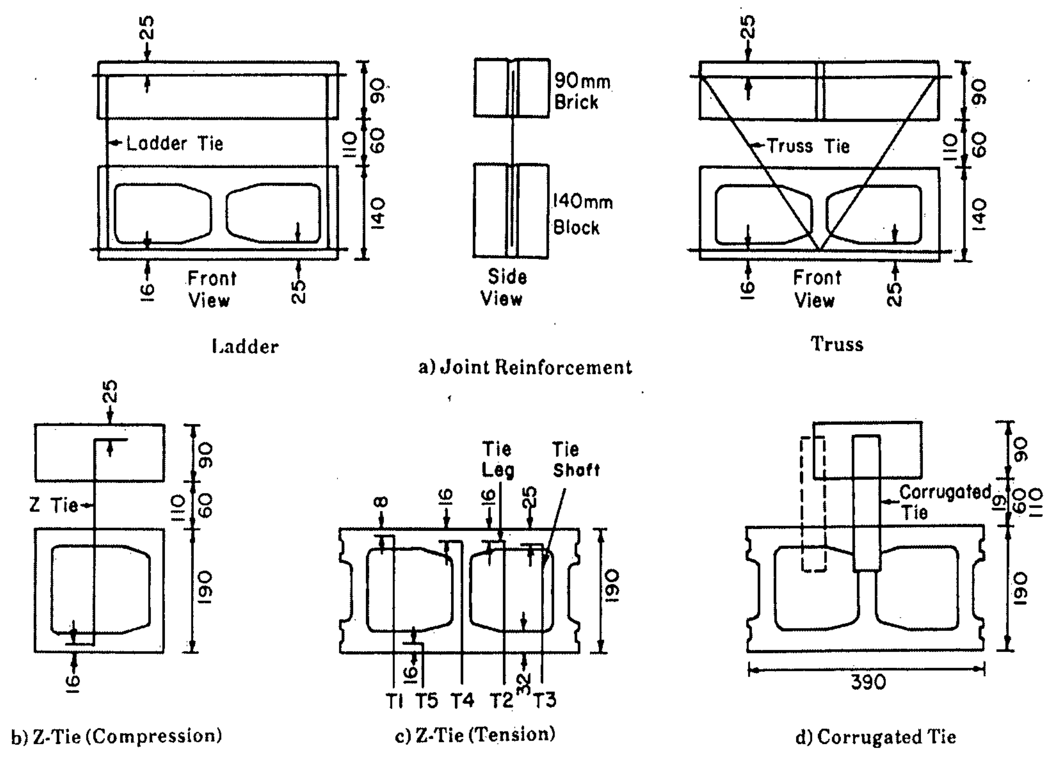


Fig. 2.10 – Varied positions of ties in the test specimens to account for the anchorage condition. (From Ref. 28)

There were large number of test being conducted (110 tests) with the specimen size of two bricks course and two concrete blocks course (it was called a bricks or blocks couplet) with the tie embedded in the type S mortar bed joint. The types of tie being used in the test were the Z-tie, corrugated strip tie and the continuous wire joint reinforcement that includes the truss type wire reinforcement and the ladder type wire reinforcement. To consider the anchorage condition as a parameter in the test, the position of ties in the test specimens was varied, as can be seen in Figure 2.10.

Failure modes observed for compression were mainly governed by the buckling of ties for the continuous wire type and the corrugated strip tie, for the Z-tie type, the push-out through the mortar bed governed. In tension, pull-out from the mortar bed joint was the dominant failure mode for all type of ties. The test positively confirmed that cavity width significantly affect the performance of the ties in a buckling failure mode. In a push-out failure mode, the position of embedment of the tie in the mortar joint was a significant factor affecting the strength. Therefore they concluded that if an axial compressive stress were applied to the brick, the clamping force would give a significant benefit to the strength capacity of the tie.

W.M. Mc Ginley, J. Warwaruk, J. Longworth and M. Hatzinikolas (Ref. 27) conducted a full study of the interaction between masonry veneer and steel stud backup wall. The test consisted of several tests and finalized in full wall test panels. The smaller tests were the component test for the system, which includes connection of tie to the steel stud, steel stud connection with the stud track and interaction between steel stud and gyproc as insulation material.

The research also provided a design approach of a masonry veneer wall system with limit state design, which then further evaluated by the experimental results. The limit states design approach of the masonry veneer wall identified five limit states for out of plane loading, which are:

1. Crack formation in one or more of the veneer mortar joints by flexure.
2. Failure of tie system that connects the veneer to the backing wall.
3. Flexural failure of the backing wall.
4. Failure of the supports of the backing wall (local failure), under the concentrated reaction load.

5. Excessive deflection of the wall system.

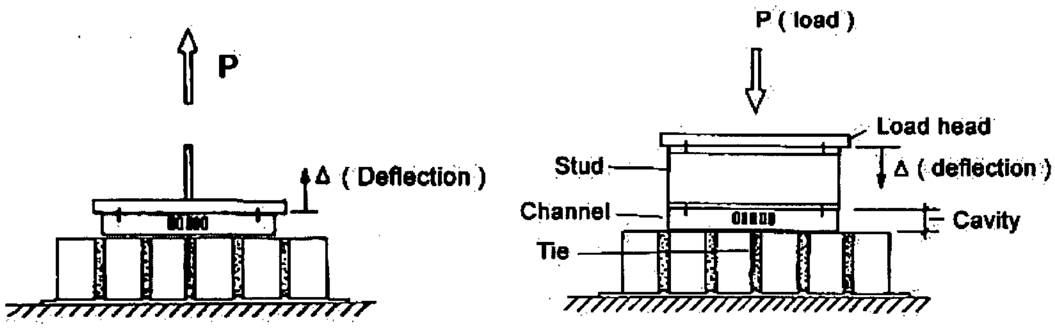
There was also an analytical model in three dimensions and two dimensions, developed to predict the behaviour of the wall system. The predictions were compared to the results obtained from the full wall test.

R.G Drysdale, Andrew Kluge and G.T. Suter (Ref. 17) conducted several physical tests on brick veneer steel stud wall system. Included in the study were the tests of tie connection to the steel stud. Tests were conducted monotonically with parameters including the type of tie, steel stud thickness, mounting surface of ties, adjustability of ties, location of the attachment of tie to the steel stud and type of fasteners. Among the conclusions of the test was, with a larger cavity width or air space the unsupported effective length of ties becomes longer, thus greater possibility of ties fail in compression by buckling.

One of the parameter that affected the stiffness of the wall ties was the adjustability of the ties, as explained before. This was the subject of the study by M.J. Wilson and R.G. Drysdale (Ref. 33), with their study on the influence of the adjustability on the behaviour of Brick Veneer/Steel Stud (BV/SS) wall ties. The objective of the test was to determine the influence of the adjustable range on the tensile behaviour characteristics of the wall tie systems, with an evaluation of the ultimate load carrying capacity of the ties and stiffness values over a serviceability performance range. The tests were a monotonic tension test between the wall ties and the steel stud with an exterior sheathing attached.

The conclusions for the experimental study was the position of the tie within the adjustable range could dramatically influence its stiffness and strength. At the extreme of their adjustability, some ties have very low capacities and would not be adequate to resist wind forces generated at top ties or at ties near veneer crack locations. The tensile behaviour characteristics of adjustable BV/SS wall ties vary considerably under the influence of factors such as type of attachment, eccentric loading within the tie, bearing area on support material, degree of mechanical play, type of exterior sheathing material and eccentricity of load transfer to the stud.

Knowledge gain from previous research and study push the innovation of veneer wall ties even further. J.L Dawe, C.K. Seah and N.A. Valsangkar (Ref. 15) conducted several tests on a new tie system (later known as the Dur-o-wal tie system) for the BV/SS wall. The tests were to determine the strength of new tie system components both individually and as a whole system. The entire system were subjected to compressive, tensile and shear forces monotonically. The screws were subjected to pull out and shear test. Ties embedded in the veneer were tested under tension and compression. Fig. 2.11 described the configuration of the test.



(a) Tension test. **(b) Compression test.**

Fig. 2.11 – Test configuration for the new tie system loaded monotonically. (From Ref. 15)

The results of the tests in compression was cracking of the mortar joint (push through failure) and in tension, a pull-out action of the flat bar (the new tie), because of its high stiffness. The weakest link of the system was found to be the pull out strength of the screws, as it was proven with the overall system test. They concluded that the performance of the new tie system in compression is governed by capacity of stud, and the tension capacity of the tie system is governed by the flexural strength of stud flange and pull out strength of screws.

Further study on the pullout of tie from brick veneer was conducted by E.F.P Burnett and M.A. Postma (Ref. 9). The study had several objectives, which were:

1. Experimentally assessed the performance of the Helifix HRT60 tie when pulled out from brickwork compared to three other commonly used proprietary systems
2. Documented the response of brick ties at relatively large displacements in order to assess their potential for use in, for example, seismic areas.
3. Examined the influence and magnitude of likely clamping forces during pullout.
4. Compared Canadian, American and British testing method for pullout resistance of ties from brick couplets.

The tests were conducted with a monotonic pull-out in tension with a pre-conditioning phase to address the mechanical free play by cyclic loadings. The specimens' size used were brick couplets, which is a single wythe, stack bonded, two-unit prisms, two brick high. Parameters included in the test were type of tie (Helifix HRT60, Z-tie, corrugated dovetail tie, straight dovetail tie), and compressive clamping or surcharge forces (in-plane constraint) from 0 to 33 kPa.

The conclusions for their test were:

- The presence and variation of the in-plane restraint has a significant effect on the performance of the ties during pullout testing.
- If coupled test are conducted without any in-plane constraint, the tension pullout force is not a measure of the strength of the connection, but it's more indicative of the bond between brick and the mortar.
- When the tie is pulled out of the brick couplet, a bursting force is exerted on the brick couplet and this force can split the interface between the mortar and the brick.
- In reality there are at least two contributions to in-plane constraints:
  Surcharge from dead load.
  The interfacial tensile resistance of all the joints involved or engaged.
- The Helifix tie has low pullout force and maximum capacity, but with a large ductility ratio.
- Helifix tie has a very ductile post maximum behaviour, with significant benefit from initial clamping effect and that little if any bursting forces are generated during pullout. These are useful in seismic areas.
- The pullout response of the ties is not necessarily brittle.

The first reversed cyclic loading test for veneer wall ties was conducted by G. Simundic, A. W. Page and T.L Neville (Ref. 29) with their research on behaviour of wall ties under cyclic loading. The tests were conducted by Australian standard code and using Australian veneer wall tie. The objective of the test was to assess the performance of both the ties and their attachments under cyclic loading.

Several monotonic tests in compression and tension were also conducted for comparison of the load displacement envelope. The cyclic loading was applied using force control; this involved specifying the loading rate (kN/min), maximum load to be applied, load amplitude (kN) and number of load cycles. For each tie test four load cycles were applied for every specified maximum load.

The size of the specimens was a brick couplet – two bricks high with ties embedded in the mortar joint and attached to a small section of backup systems. Parameters included in the test were type of ties (for concrete, wood, steel backup) for cavity or veneer wall systems, type of backup wall (concrete, wood, steel), start of cyclic test (compression or tension first) and prescribed relative displacement of the tie ends. The conclusions from this experimental study were:

- Strength and stiffness generally governed the failure mode of the ties, with failed mortar joint at high loads.
- Several ties (Australian veneer type) typically failed by buckling or by movement of the mortar joint for the case with high stiffness.
- For most of the tests, the cyclic tests give significantly high stiffness before failure, in the face fixed ties the compression stiffness is much higher than in tension.

The most recent study of cyclic loadings on veneer wall ties was conducted by Y.H. Choi and J. M. LaFave (Ref. 11). They conducted an experimental study of brick-tie-wood subassemblies. The study was actually part of a bigger scale test of brick veneer attached to wood framing with corrugated sheet metal ties. Their objective was to capture the performance of the entire subassembly rather than just the performance of the tie only. This subassembly represented a localized portion of a full height brick veneer wall system.

Tests were conducted in monotonic tension and compression, and reversed cyclic loadings. The cyclic loadings were displacement controlled, with a total of 24 cycles of loadings. The displacement was applied at a rate of 1 cycle per minute. The loadings were stopped when the final failure of the specimens occurred or when the subsequent applied load had no further meaning.

Specimen's size was consisted of a brick couplet with two standard bricks, one wood stud and one tie. The only type of tie being tested in the experimental program was the corrugated sheet metal tie type. Parameters included in the study were initial displacement or offset and type of fasteners (nails or screws connection).

The experimental study concluded that the general failure modes were pull-out of nail for the nail-connected specimens, and pull-out of tie for screw-connected specimens due to loss of bond strength between mortar and the brick. With compression all specimens failed by buckling of the tie, irrespective of other parameters.

Chapter 3 Description of Experimental Program

3.1 Introduction

As described in Chapter 1, the test specimens for Phase I, Phase II and Phase III was to simulate local effect of one tie and study the embedment capacity. The wall tie was located in the middle of the panel. This wall tie was the point of loading for the brick panel and is connected to the hydraulic actuator, which provided the reversed cyclic load.

The boundary conditions chosen for the test are described in the next section (i.e. Section 3.2), while a detailed description of the brick panel test specimen is given in Section 3.3, and properties of mortars used to construct the specimens are given in Section 3.4. Section 3.5 and Section 3.6, describe the test frame apparatus and the test set-up, which includes the instrumentation used in the test. The last section (Section 3.7) describes the cyclic testing procedure used in this experimental study. The discussions of test results are then summarized in Chapter 4.

3.2 Boundary Conditions

The boundary conditions were developed to model the behaviour of a masonry veneer wall. In order to simulate a local embedment failure of the panel, the tributary area concept of tie loads was used to determine the size of the specimens, as described in the methodology in Chapter 1.

The brick panel specimen was used to simulate an element of a full veneer wall panel such as that it will behave as close as possible to the real wall. Brick veneer walls are supported by shelf angles for their self-weight on each floor for each storey in a building. In seismic regions, movement joints along the top (horizontally) and sides (vertically) isolate the veneer. These conditions will force each brick veneer wall to act as an independent panel. The movement joints and support condition determine the size of each panel. Thus the most common analytical model for a brick veneer panel with metal ties connector is by taking a vertical strip of a brick veneer wall with the height corresponds to the storey height and model it as a continuous beam, while the metal ties that connect the wall to the structural backup will be modelled as lateral supports as shown in Figure 3.1.

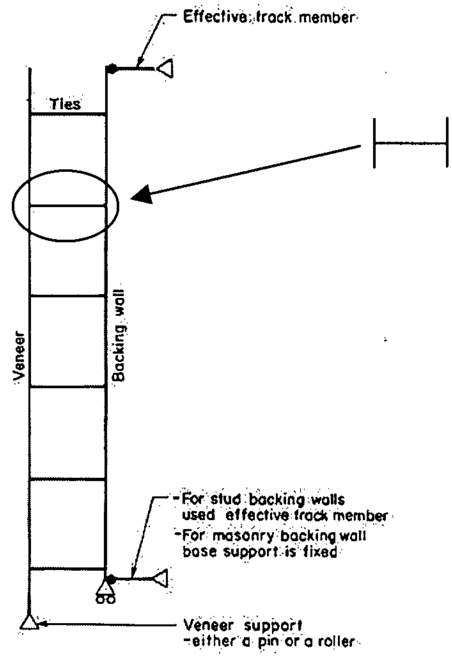


Fig. 3.1 – Analytical model of brick veneer connected to a structural backup. (from Ref. 27)

An element consisting of one brick tie within surrounding bricks then can be simulated as a panel having fixity against rotation along its four edges. Movement joints will actually allow some deformation or expansion of the veneer wall in such a way, therefore the fixity along the panel edges was not continuous until the full length of the panel. All four corners of the brick panel specimens were left unrestrained to allow the possible deformations in a real wall. The lengths of the bar, which provide the fixity along the edges, were 815 mm (24 in.) for the top and bottom edges and 205 mm (8 in.) for both sides of the panel specimen. As the force was being applied to the tie, fixed four edges of the panel would provide the reactions necessary (see Fig. 3.2).

To consider wall ties related to their location on a masonry veneer wall, the effect of a surcharge load should be taken into account. For a brick panel specimen with wall tie located on the uppermost part of a veneer wall (the highest tie location), top part of the panel was left unrestrained and free to deflect due to the fact that in a real masonry veneer wall the lateral support condition at the top was almost free from any restraining effect (low frictional resistance from a flashing material usually used as a movement joint between top of the masonry and supporting self angle above it). In the case of wall tie located on the bottom row of a brick veneer wall, there will be an applied surcharge load on it while maintaining a condition of restraint against rotational deformation. To allow surcharge load be applied on the panel, vertical movement of the brick panel specimen should not be restrained. Thus the final boundary condition for the bottom row of tie would be a free vertical movement with rotational restraint, as is shown schematically in Figure 3.3.

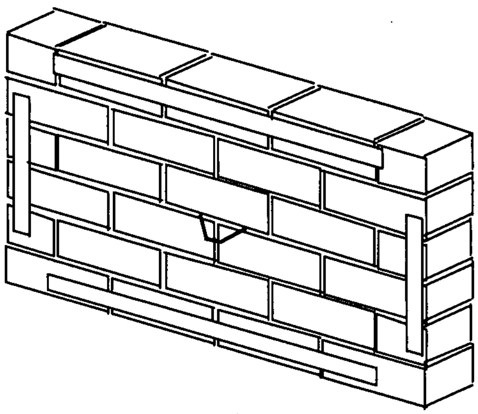


Fig. 3.2 – Fixed four edges against rotation of a brick veneer panel specimen.

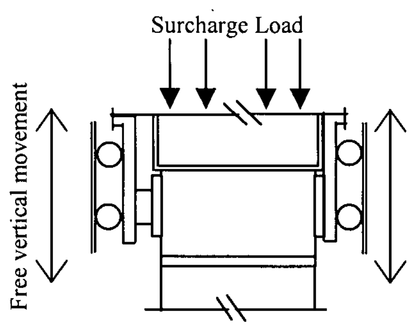


Fig. 3.3 –Schematic diagram of edge restrain along top part of specimen with a surcharge load.

A wall tie's main function is to transfer lateral loads from the brick veneer wall panel to the structural backing. In a real brick veneer wall, there are many ties supporting the veneer panel. Out-of plane loading from wind or earthquake will deform the brick veneer panel. With the veneer panel resisting the load, each tie will then carry a portion of the load. These ties will resist axial load perpendicular to veneer panel (tension and compression). To simulate loading condition for a veneer panel with wall ties, a loading guide was designed to verify the applied load to the tie to always be perpendicular to the brick panel (out-of-plane loading). The loading guide restricted the degree of freedom of the applied load only to axial direction along the longitudinal tie axis. A mechanical device that clamped or attached to the tie was designed with almost negligible free play between the tie and the device itself. The detail of this mechanism will be described more thoroughly in the next following section.

3.3 Test Frame and Loading Apparatus

Based on the boundary conditions explained in the previous section, a test frame was built to simulate the conditions. The test frame was developed to correctly provide the deformation and fixity for each edges of the brick panel. A loading apparatus was designed as a part of the system, and was developed to apply a one directional loading (out-of-plane) to the brick panels. The loading system will restrict the hydraulic actuator to apply only an axial load (longitudinal axis of the tie) to the metal tie that is embedded in the mortar bed joint. The test frame is shown from front view in Fig. 3.4, and Figure 3.5 shows the frame from a side elevation view.

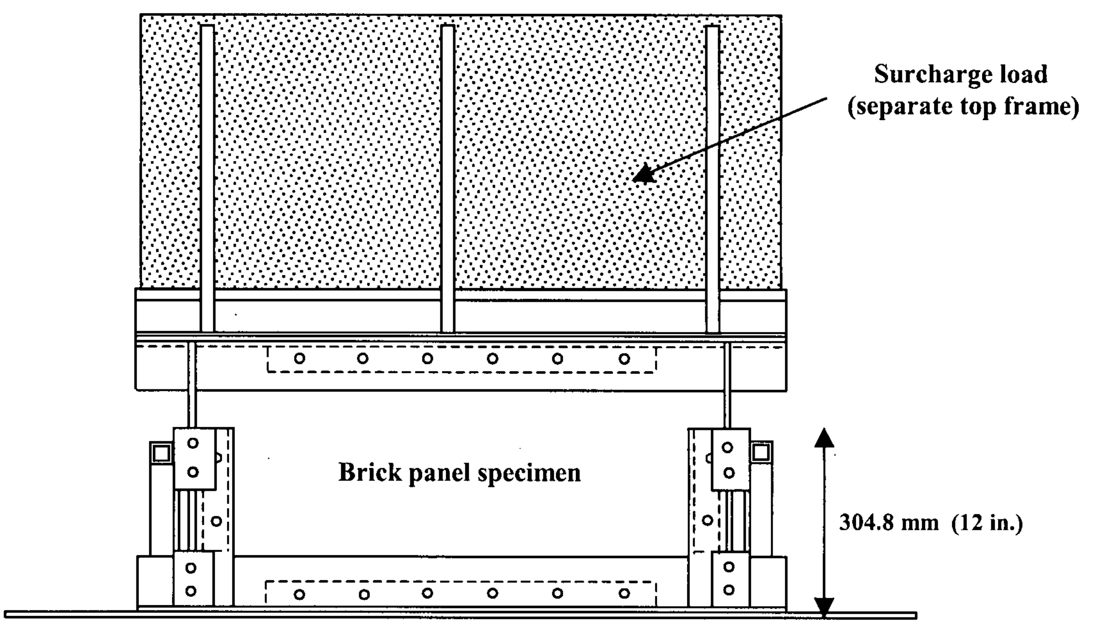


Fig. 3.4 – Front view of the test apparatus.

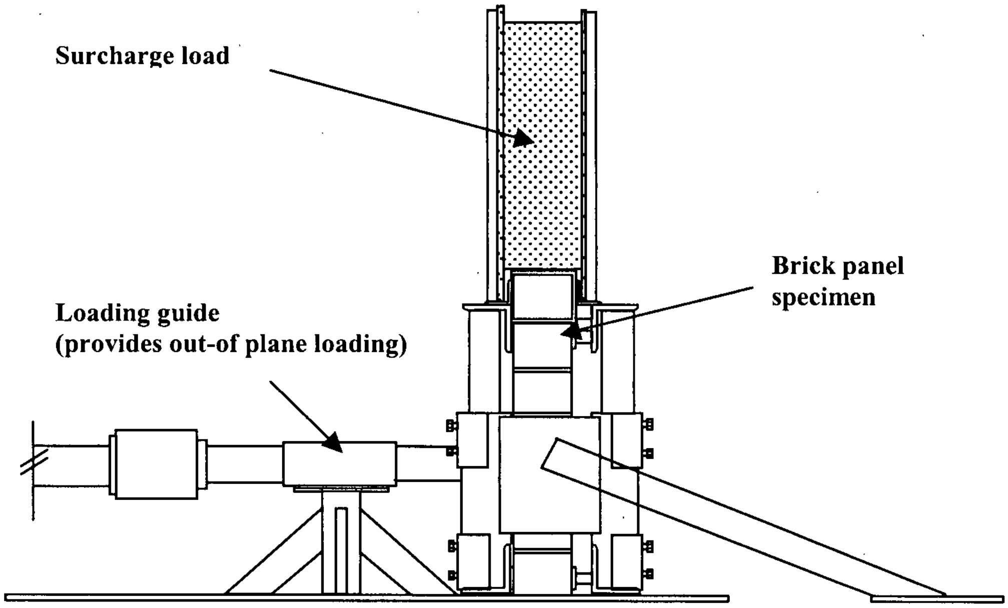


Fig. 3.5 – Side elevation view of the test apparatus.

The test frame consisted of two parts, the U-shaped frame welded to a base plate that bolted to the strong floor in the structures laboratory, and a top frame with four steel guidance bars. The U-shaped frame were built from two angles (two U-shaped single frame) that will hold the edges of the panel, thus it will provide the required reactions for the forces along the panel edges and transfer them to the base plate. The U-frame was made slightly larger than the width of the brick panels to provide access for setting up the panel inside the frame and for an easy removal of the specimen after the test. In Figure 3.6 a photograph of the installation of the specimen inside the U-shaped frame is shown. Steel bars running along the angles as shown previously in Figure 3.2 provides fixity at the edges of the brick panel. The length of the bars were not the same length of the brick panels, as was explained in the boundary conditions requirement. Therefore all four corners of the brick panel were not laid up against the steel bars that provided the fixity. To control the fixity provided by the steel bars, on all the angles that made one side of the U-shaped frame, there were several ½" bolts (6 on the top and bottom angles and 2 on the side angles) that could be driven through the angles and pushing the loose steel bars. Thus the bolts would push the loose steel bars, which in turn will press the brick panel and provides the fixity. Figure 3.7 shows this fixity system. Lateral supports of the U-shaped frame were provided by diagonal steel bracing that welded to the two sides of the frame and to a separate steel plate, which is bolted down to the strong floor.

The separate top part of the test frame was designed to provide a fixity on the top edge of the panel while also maintaining an axial load on the brick panel to simulate the self-weight of the brick. The boundary condition required to fix the four edges but not to fully restraint the deformations while providing a surcharge load, lead to the design of the four steel guidance bars. Figure 3.8 shows this top part section, which consists of an HSS beam, angles, four steel guidance bars and a holder for the axial load. Figure 3.9 shows the function of the four steel guidance bars as legs, which provides no restrained effect for the axial deformation, while maintaining the fixity in the lateral rotation. The fixity in the lateral rotation of these bars was designed similar to the fixity of the edges of the brick panels. When the top part frame is connected to the U-shaped frame (with the test specimen inside), these four steel guidance bars are inserted between the U frame angle and two smaller angles, which are welded to the U frame angle itself.

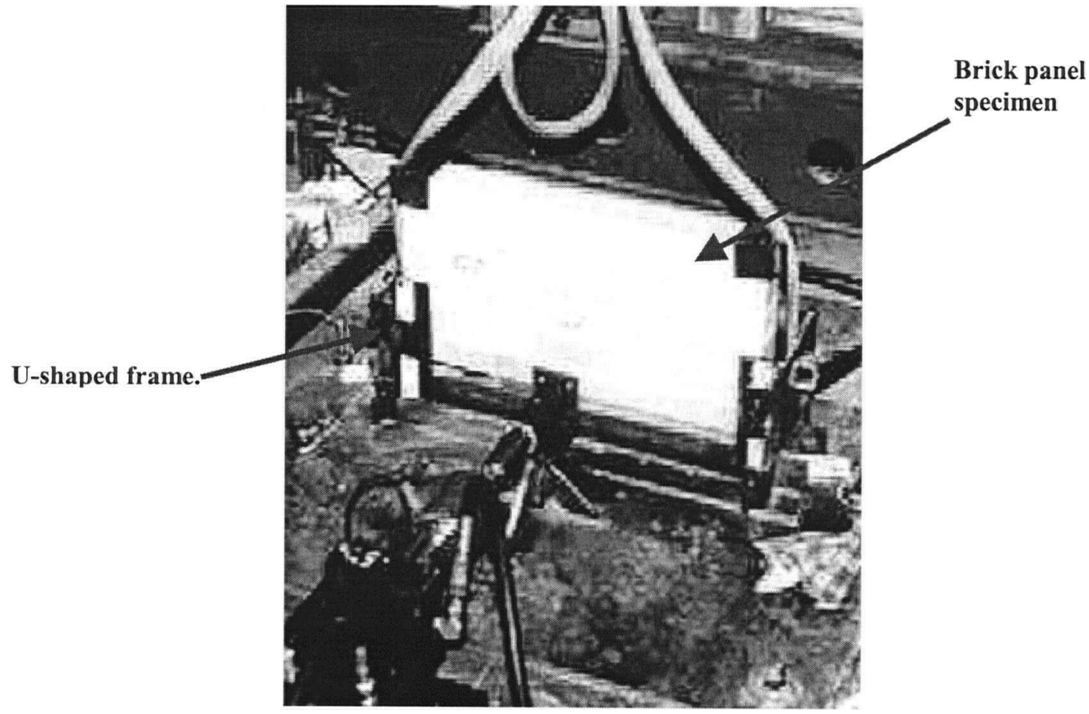


Fig. 3.6 – Photograph of installation of a specimen inside the U-shaped frame.

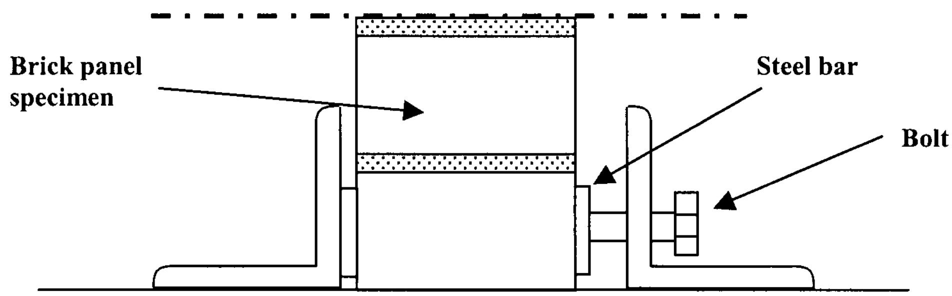


Fig. 3.7 –Bolt to control the fixity of the steel bars, which clamped the panel.

This system shown in Figure 3.10, consists of another separate small steel plates (equal length as the small angles) in pair with the small angles, and it is use to provide the clamping effect by pressing it to the guidance steel bar. The level of fixity provided by the steel plates could be adjusted by means of turning the ½" steel bolts that driven through the small angles. Both the small steel plates and the angles that built the U-frame had Teflon bearings (limited in length) glued to it. These Teflon bearings provided a smooth surface for the guidance steel bars; therefore there should not be any restraining effects axially (vertical movement was allowed) for the brick panel.

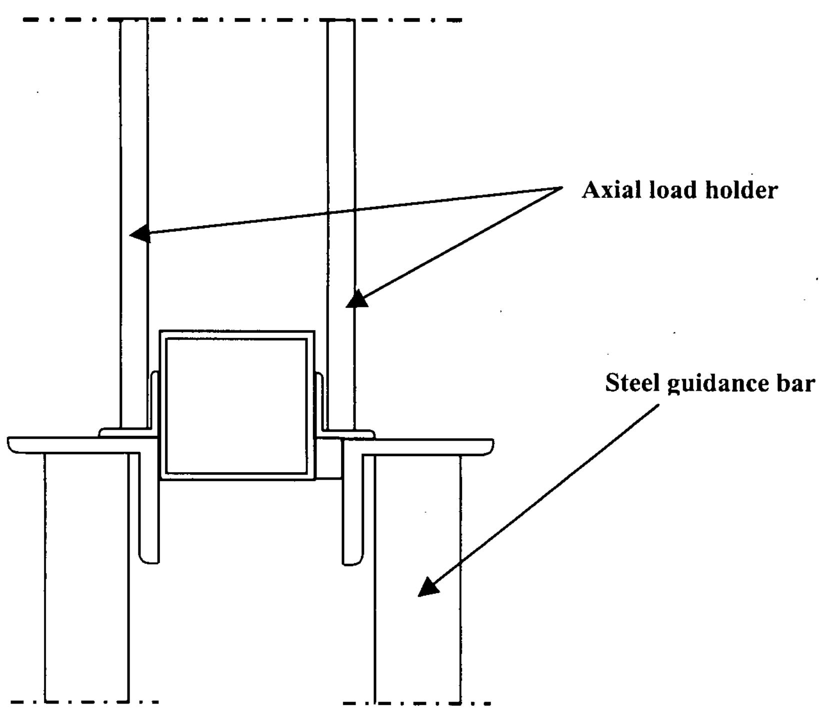


Fig. 3.8 – Separate top frame section of the apparatus to apply surcharge load.

The surcharge load itself comes from the metal leads that are held by the load holder. This load holder consists of six small steel bars welded on the top, which will hold all the metal leads, that are put on top of the frame. The mass of the leads then provides the calculated surcharge load needed for the brick panel. The axial surcharge load system is shown in a photograph of tested specimen in Figure 3.11.

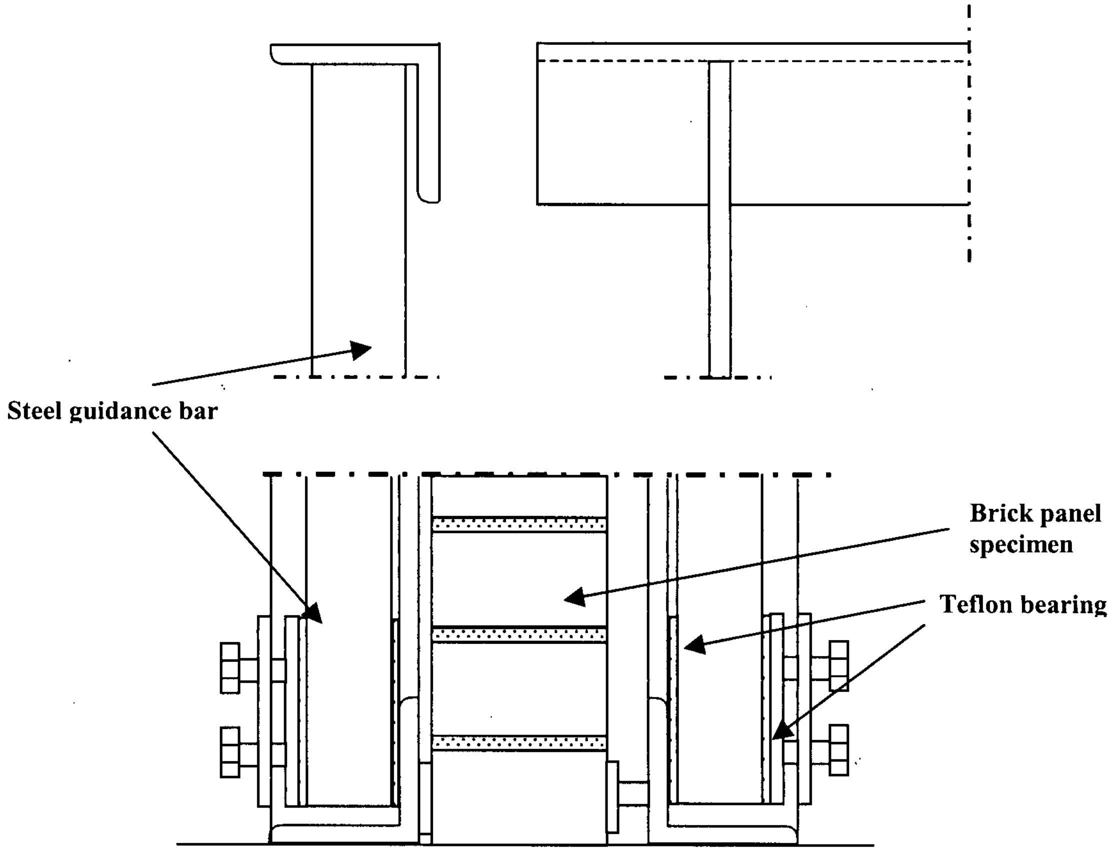


Fig. 3.9 – Steel guidance bars provide the degree of freedom required.

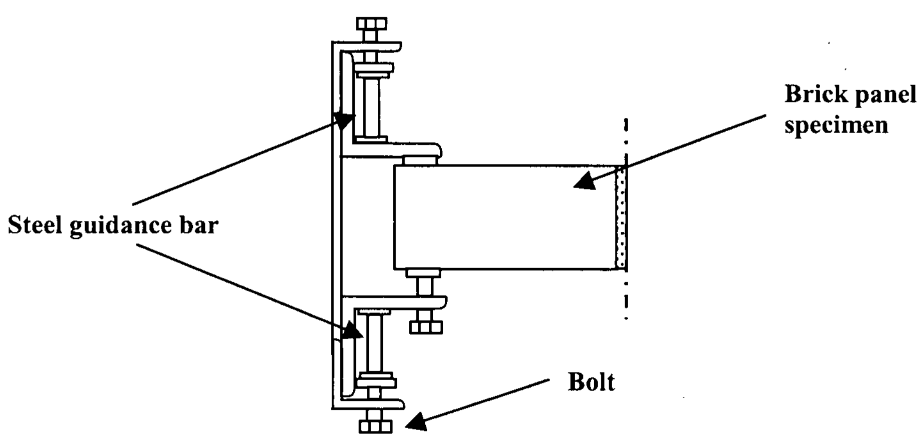


Fig. 3.10 – The bolts system that provide the fixity for the guidance bars.

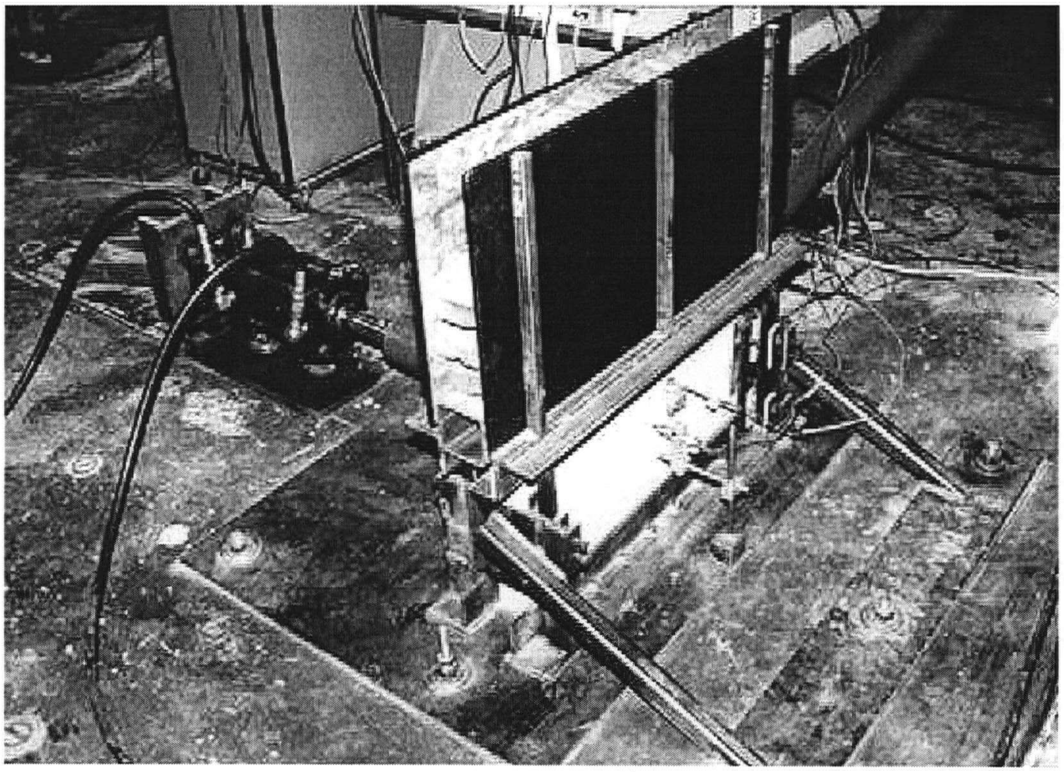


Fig. 3.11 – Photograph of a specimen with applied surcharge load.

The weight of leads for the axial surcharge load on the brick panel total is 3.3 kN (740 lbs), this is to simulate a brick veneer wall with a height of 2.8 m (9 ft). The method and calculation of this axial surcharge load is described in detail in Appendix C. Thus considering the area of the brick panel specimen which being compressed by the axial load (including the three courses of bricks on top of the wall tie and the weight of the top frame section), the total surcharge load is 60 kPa. While if the condition is for a top row of ties without the top frame section, the total three brick courses will give a surcharge of 4.2 kPa.

The loading apparatus consisted of one hydraulic actuator, a sliding track system as a loading guide, and a clamping device. The hydraulic actuator is bolted to a steel reaction frame. The sliding track system consisted of a steel rod that is inserted into a steel tube case with Teflon bearing inside; this steel tube case will guide the direction of the applied force from the actuator, therefore only an axial force (out-of plane direction to the panel) will be applied. A clamping device is connected to one end of the steel rod, which is then use to hold or clamp a portion of the tie that sticks out from the bed joint of the brick panel specimen. This clamping device was made to be easy to replace or to detach from the whole loading apparatus; this is to accommodate the different types of clamping devices that match the wall ties that are going to be tested. The loading apparatus is shown in Figure 3.12. The hydraulic actuator has a capacity of 225 kN (50 kips) and has a stroke of 150 mm (6 in.).

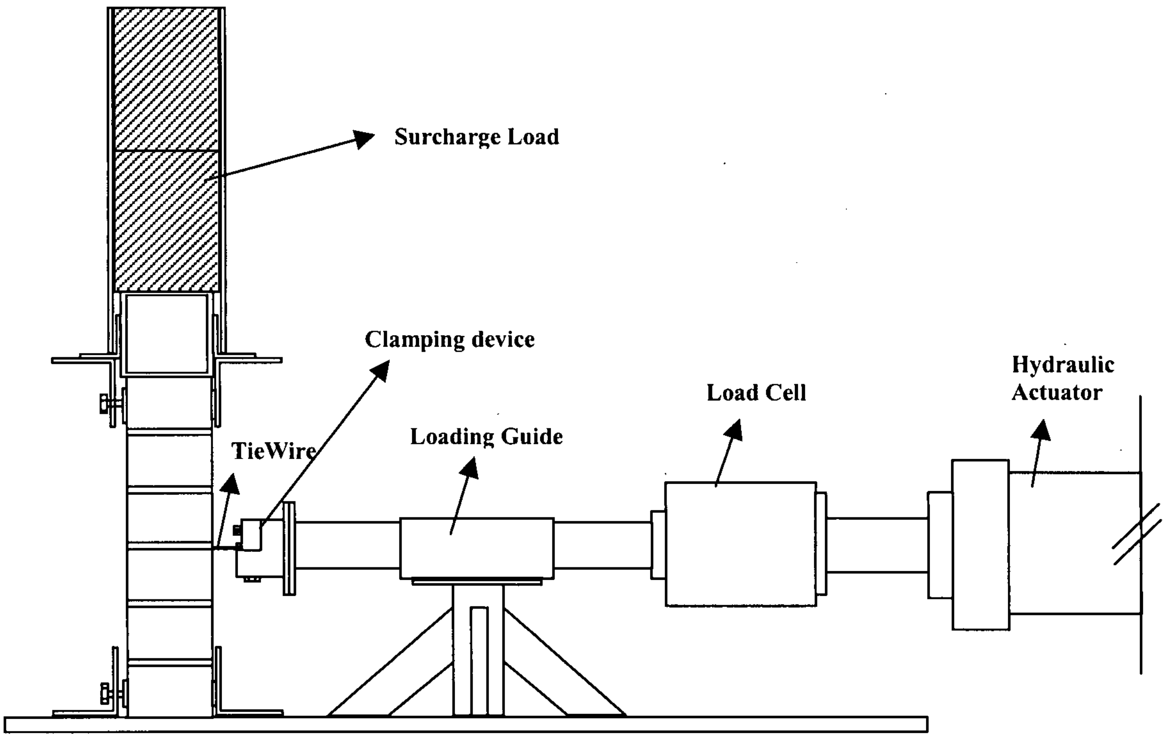


Fig. 3.12 – Test frame loading apparatus (includes hydraulic actuator, loading guide and clamping device).

3.4 Test Specimen

The brick panel test specimens were constructed according to the boundary conditions and spacing requirements described earlier in Chapter 3.2. The design load of the specimens was carried out in accordance with Chapter 13 of the Canadian Standard Engineered Masonry Design CAN/CSA-S304.1. A summary of the design calculations is presented in Appendix B.

Typical brick panel specimens size is shown in Fig. 3.13, which are 450 mm high (18 in.) and 800 mm (32 in.) wide with a single brick tie at the centre. These test specimens were constructed by an experienced mason using extruded clay bricks 90 mm (3.5 in) wide, 63 mm (2.5 in) high, and 190 mm (7.5 in.) long with a 12 mm (½ in.) thick mortar joint for both phase of test. The mortar joint was flushed on the tie side and tooled on the other side. For Phase I specimens, type N mortar was used and for Phase II and Phase III, type S mortar was used. Both were cement-lime mortar that was from a single batch of a premixed wet mortar. Figure 3.14 (a) shows the construction of Phase I specimens, (b) shows Phase II specimens construction, (c) shows the construction of the Phase III specimens and (d) shows the curing condition of specimens covered by plastic sheets.

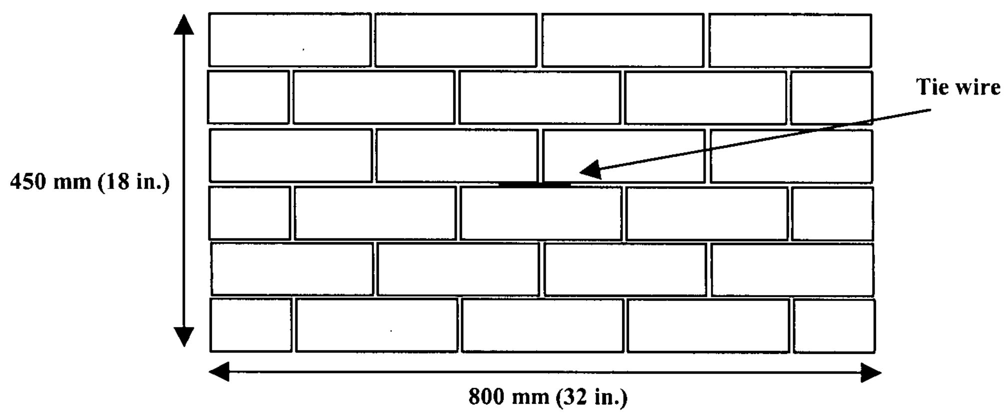
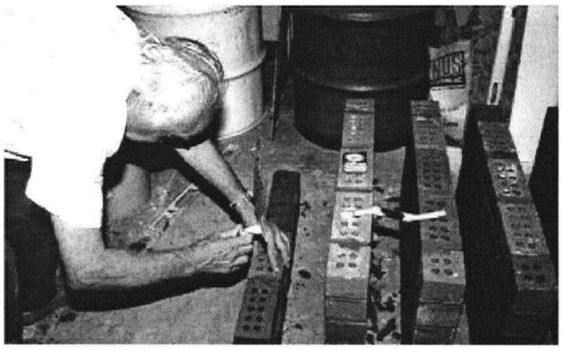
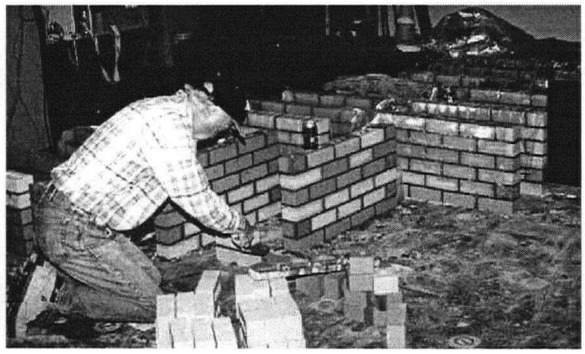
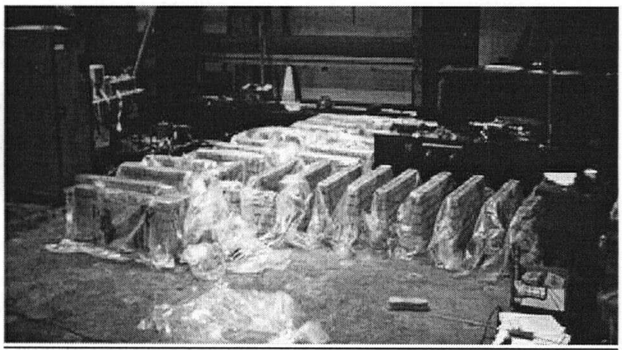


Fig. 3.13 – Typical brick panel specimen size and tie location.

The tie that was used for the main study was manufactured by FERO corporation with the embedded wire portion called the V-Tie consists of a V shape wire with legs. This tie was made from 4.76 mm (0.19") diameter wire conforming to CSA standard G30.3. The mechanical connection to engage or enclose the wire reinforcement consisted of a clip that was manufactured from 16 gauge (1.61 mm thick) sheet metal conforming to ASTM A570. Figure 3.15 shows the geometry of the tie and the clip. The clip was inserted (one for each leg of the tie) between the V-Tie leg and the horizontal wire joint reinforcement, which is made from a 9 gauge wire.

(a) **(b)**

(c) **(d)**

Fig. 3.14 – Construction of test specimens: (a) Phase I, (b) Phase II and (c) Phase III and (d) curing condition of specimens covered by plastic sheets inside the structures laboratory.

Specimens for Phase I test were constructed in a different lab. While specimens for Phase II test were constructed inside the structural lab and Phase III were outside the structure lab (see Fig. 3.14). For Phase II specimens the temperature inside the structure laboratory at the time the specimens were built was 19 °C (66 °F) while outside temperature recorded was 7 °C (44 °F). These specimens were then covered by plastic sheets to maintain the humidity (see Fig. 3.15). At 28 days, all plastic sheets were removed and the specimens were allowed to cure at laboratory room temperature, which were measured and recorded around 22 °C (71 °F) until the day of testing for each specimen. As. The temperature was measured and recorded, which was around 15 °C (59 °F). Plastic sheets were used to cover the specimens with a tarp on top to cover the specimens from rain. At 28 days the cover was removed and specimens were left to cure with the surrounding temperature until the day of the test.

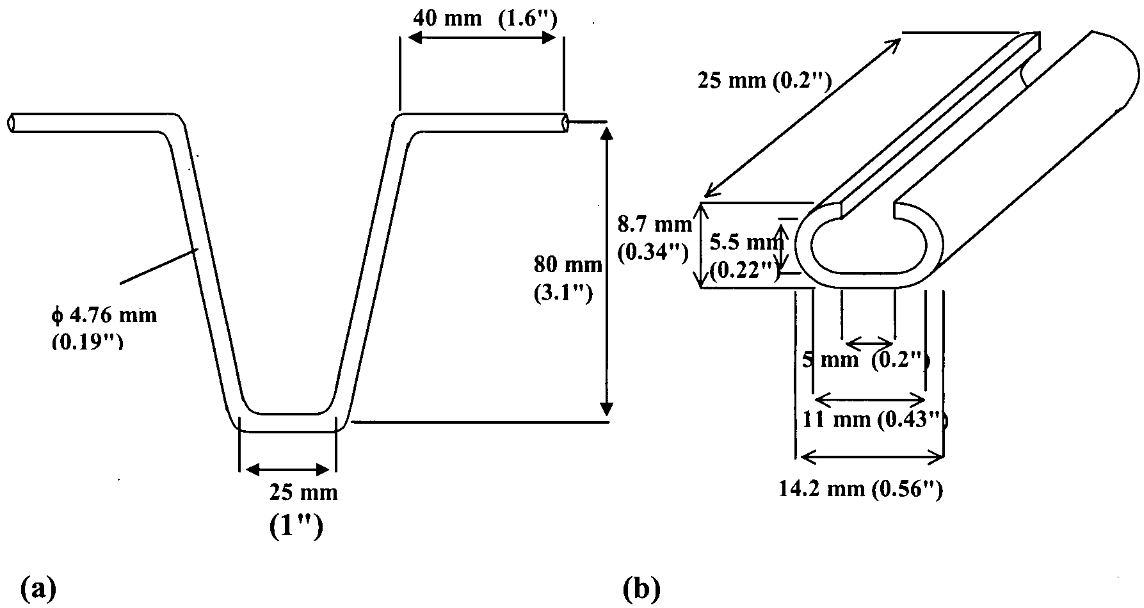


Fig. 3.15 – Geometry of: (a) the embedded wire portion of a two pieces adjustable tie, and (b) the tie clip. (From Ref. 19)

Table 3.1 shows all the specimens with their properties that were built for the experimental program. All ties were embedded at the centre course and for the V-Tie system, the leg of the tie was located at the centreline of the bricks and for the one that use a single wire horizontal joint reinforcement, the placement of the horizontal reinforcement was as close as possible to the centreline of the bricks, as shown in Fig. 3.16 The increased length of the V-Tie chosen for the second and third phase of the experimental testing was mainly because it was found that with a 60 mm length of V-Tie, there was not enough free space for the tie to deform before the clamping device collided with the brick panel. Therefore the situation was corrected by using a longer length of V-Tie (80 mm).

Table 3.1 – Properties of the brick panel specimens for the experimental program.

No.	Specimen Name	Mortar Type	Tie manufacturer / type	Horizontal Bed Joint Reinforcement	Test Phase
1.	PL1	N	Fero V-Tie (60 mm)	None	I
2.	PL2	N	Fero V-Tie (60 mm)	None	I
3.	PL3	N	Fero V-Tie (60 mm)	None	I
4.	PL4	N	Fero V-Tie (60 mm)	Not clipped	I
5.	PL5	N	Fero V-Tie (60 mm)	Not clipped	I
6.	PL6	N	Fero V-Tie (60 mm)	Clipped	I
7.	T1	S	Fero V-Tie (80 mm)	None	II
8.	T2	S	Fero V-Tie (80 mm)	None	II
9.	T3	S	Fero V-Tie (80 mm)	None	II
10.	T4	S	Fero V-Tie (80 mm)	None	II
11.	T5	S	Fero V-Tie (80 mm)	None	II
12.	T6	S	Fero V-Tie (80 mm)	None	II
13.	T7	S	Fero V-Tie (80 mm)	None	III
14.	T8	S	Fero V-Tie (80 mm)	None	III
15.	T9	S	Fero V-Tie (80 mm)	None	III
16.	T10	S	Fero V-Tie (80 mm)	None	III
17.	T11	S	Fero V-Tie (80 mm)	None	III
18.	T12	S	Fero V-Tie (80 mm)	None	III
19.	T13	S	Fero V-Tie (80 mm)	None	III
20.	TW1	S	Fero V-Tie (80 mm)	Not clipped	II
21.	TW2	S	Fero V-Tie (80 mm)	Not clipped	II
22.	TW3	S	Fero V-Tie (80 mm)	Not clipped	II
23.	TW4	S	Fero V-Tie (80 mm)	Not clipped	II
24.	TW5	S	Fero V-Tie (80 mm)	Not clipped	II
25.	TW6	S	Fero V-Tie (80 mm)	Not clipped	II
26.	TWC1	S	Fero V-Tie (80 mm)	Clipped	II
27.	TWC2	S	Fero V-Tie (80 mm)	Clipped	II
28.	TWC3	S	Fero V-Tie (80 mm)	Clipped	II
29.	TWC4	S	Fero V-Tie (80 mm)	Clipped	II
30.	TWC5	S	Fero V-Tie (80 mm)	Clipped	II
31.	TWC6	S	Fero V-Tie (80 mm)	Clipped	II
32.	TWC7	S	Fero V-Tie (80 mm)	Clipped	III
33.	TWC8	S	Fero V-Tie (80 mm)	Clipped	III
34.	TWC9	S	Fero V-Tie (80 mm)	Clipped	III
35.	TWC10	S	Fero V-Tie (80 mm)	Clipped	III
36.	TWC11	S	Fero V-Tie (80 mm)	Clipped	III
37.	TWC12	S	Fero V-Tie (80 mm)	Clipped	III
38.	TWS1	S	Fero V-Tie (80 mm)	Clipped - Modified	III
39.	TWS2	S	Fero V-Tie (80 mm)	Clipped - Modified	III
40.	TWS3	S	Fero V-Tie (80 mm)	Clipped - Modified	III
41.	TWS4	S	Fero V-Tie (80 mm)	Clipped - Modified	III
42.	TWS5	S	Fero V-Tie (80 mm)	Clipped - Modified	III
43.	TWS6	S	Fero V-Tie (80 mm)	Clipped - Modified	III
44.	OT*	S	Fero V-Tie (80 mm)	None	II
45.	OTWC*	S	Fero V-Tie (80 mm)	Clipped	II
46.	TT1	S	Triangular Tie	None	II
47.	TT2	S	Triangular Tie	None	II
48.	S1	S	Dur-o-Wall SMP 11 plate	Attached	II
49.	S2	S	Dur-o-Wall SMP 11 plate	Attached	II
50.	F1	S	Halfen Fleming Anchor	Attached	II
51.	F2	S	Halfen Fleming Anchor	Attached	II
52.	C1	S	Corrugated Strip Tie	None	II
53.	C2	S	Corrugated Strip Tie	None	II

* Off-Centre / Construction Tolerance specimen

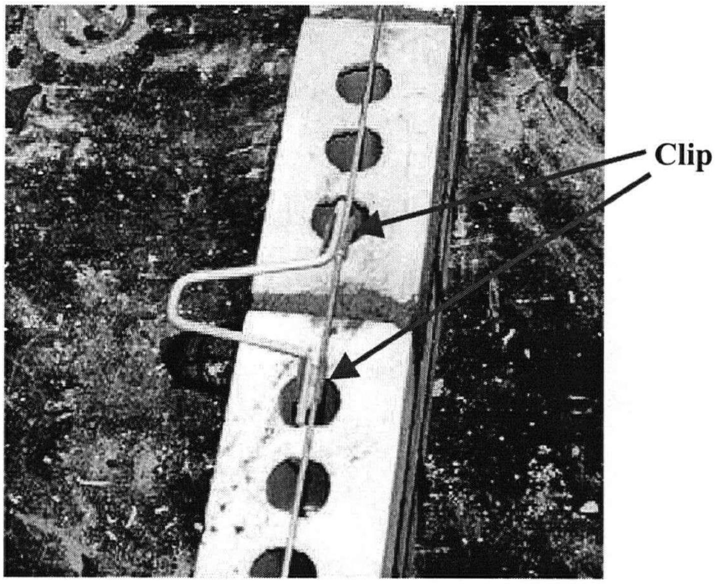
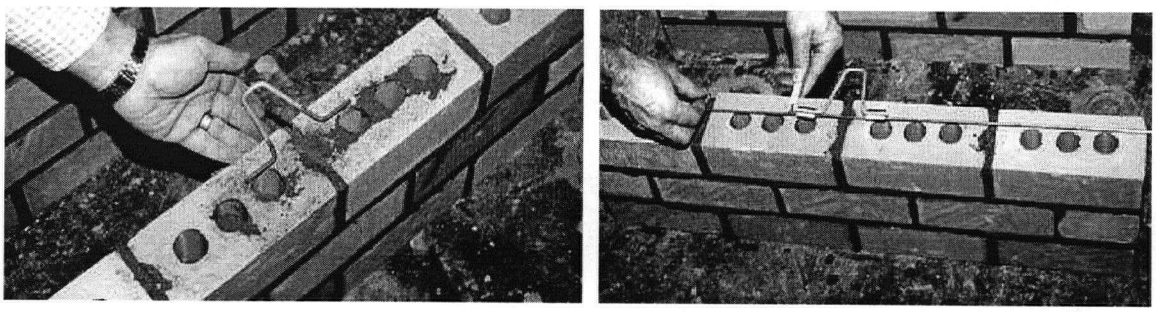


Fig. 3.16 – Placement of the tie with horizontal wire joint reinforcement in the bed joint.

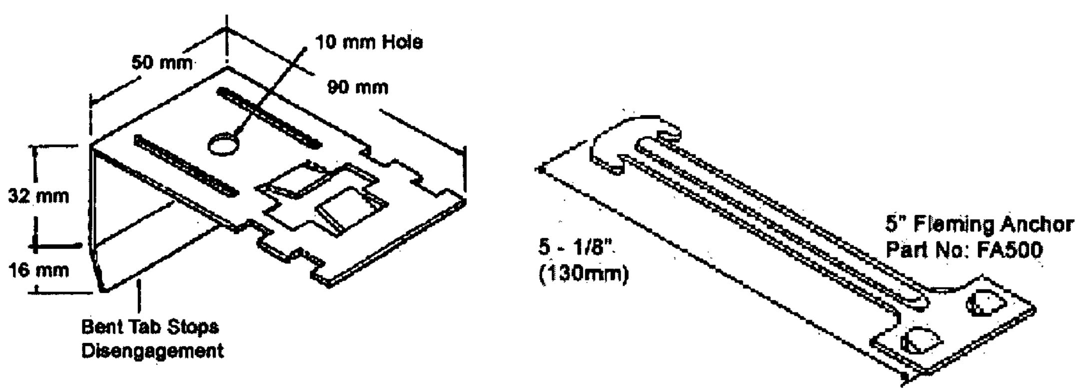
The off-centre specimens were built to take account of the misplaced or error that may occur in a real construction practice. While the embedment tolerance for non conventional connectors is not specified in CAN CSA A370-94 Connectors for Masonry, the conventional wire tie has a requirement of an embedment tolerance of ± 13 mm. Therefore the embedment location chosen for these specimens were 19 mm (3/4") from the centreline of the brick. For the tie only specimen (OT), the leg of the V-Tie was off 19 mm from the centreline, while for the tie with horizontal wire joint reinforcement clipped to the tie, the horizontal wire reinforcement was placed off centre as much as 19 mm. Figure 3.17 shows the location of these off centre ties.



(a) **(b)**

Fig. 3.17 – Off centre tie location for the construction tolerance specimens: (a) tie only, (b) tie with clipped wire joint reinforcement.

The Dur-o-Wall SMP 11 plate was manufactured from 11 gauge (3.05 mm thick) stainless steel sheet metal. The plate was modified to accommodate the clamping device by flattening the plate so as the angle shape becomes flat and to give a secure connection between the clamping device and the plate, a 3/8" pinned hole was drilled. The horizontal wire joint reinforcement was placed at the centreline of the brick unit. While the other US specific seismic tie type, the Fleming Anchor, was manufactured from 14 gage (1.9mm thick) galvanized steel strip. The specimens for this tie type (F1 and F2) were constructed as suggested from the manufacturer catalogue, that is the tie head and the galvanized horizontal wire joint reinforcement should be in the middle 1/3 of the brick width. Corrugated strip tie specimens were built according to the CAN CSA A370-94 specifications, with the embedded depth of 64 mm (2.5") (the minimum is 50 mm), and the unsupported length of tie between the brick panel and the designed clamping device was 25 mm (1"). Figure 3.18 shows the US specific seismic tie system.



(a) **(b)**

Fig. 3.18 – US specific seismic tie system: (a) Dur-o-Wall, (b) Fleming Anchor. (From Ref. 18,22)

For the Phase III test, which was a further investigation of the observed behaviour of the embedment capacity of the brick ties, modification was made on the clip, which was observed as the critical connection between tie and the horizontal wire bed joint reinforcement. As can be seen in the Fig. 3.15, the original lateral tie clip was designed to be installed by first placing the continuous horizontal bed joint reinforcement into the slot of the clip and then sliding the clip onto a leg of the V-Tie. Once it is installed the clip will provide the mechanical connection between the tie and the horizontal joint reinforcement while the gap between the two components theoretically will be filled with mortar when the next brick course is laid. While it is true that the gap was actually filled with mortar for the next course, there is a question concerning the gap between the tie and the wire joint reinforcement, that this gap will eventually provide a slack movement or a play between the two components, and while there is also an opening to slide the clip, it is considered that at some point of loading the two components will detached itself with the clip failed to provide the mechanical connection anymore.

Therefore to prevent this from occurring, a modification had to be made to the clip. The idea was to make the clip provide a securely tight connection between the leg of the V-Tie and the horizontal wire joint reinforcement. This was done by inserting the clip to the legs of the tie together with the horizontal wire joint reinforcement, clamped the clip so it tightly held the two components together and then welded the opening to close the connection. It was realized that the modification made to the clip was almost impossible to do on a real construction practice and it was only an idealized condition to actually examine the effect of a securely tight connection. Figure 3.19 is a photograph showing the difference between the original clips and the modified clips. One other thing that should be noted in the construction of the specimens, the masons were filing up the cores with mortar to make the placement of the clips on the brick unit easier, this was also implying that the possibility of the clips to drop to the cores was almost none.

Several specimens were built to further examine the embedment resistance of brick ties with horizontal wire joint reinforcement. The parameters that were investigated in Phase III besides the effect of mechanical connection were the monotonic envelope and the effect of loading history (number of cycles) to the load displacement relationship. Originally there were 22 specimens built for Phase III test, but because of an accident while the specimens were being cured (some specimens were disturbed and cracked), there were only 19 specimens left to be tested. Table 3.2 lists the remaining specimens with their properties along with their condition after they were disturbed. Repairs (to bend back the tie to its original form) were conducted to make the remaining specimens to perform at least as they were initially, but some were not able to perform as well as it was designed for (this is described on the Appendix A of the detailed explanation of the test for each specimen).

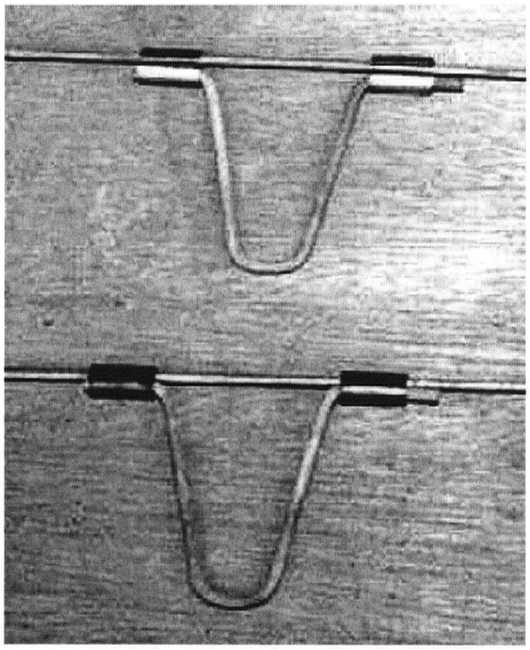


Fig. 3.19 – Photograph of the two different clips used in Phase III tests, above is the original clips and the modified one below.

Table 3.2 – Damage to the Phase III brick panel specimens.

No.	Specimen Name	Horizontal Bed Joint Reinforcement	Condition
1.	T7	None	Not disturbed
2.	T8	None	No apparent damage
3.	T9	None	Not disturbed
4.	T10	None	Crack at top corner
5.	T11	None	Not disturbed
6.	T12	None	Cracks and tie bent
7.	T13	None	Crack s and tie bent
8.	TWC7	Clipped	No apparent damage
9.	TWC8	Clipped	Not disturbed
10.	TWC9	Clipped	Tie bent
11.	TWC10	Clipped	Not disturbed
12.	TWC11	Clipped	Some cracks
13.	TWC12	Clipped	Tie bent
14.	TWS1	Clipped - Modified	Not disturbed
15.	TWS2	Clipped - Modified	Tie bent
16.	TWS3	Clipped - Modified	No apparent damage
17.	TWS4	Clipped - Modified	Not disturbed
18.	TWS5	Clipped - Modified	Cracks and tie bent
19.	TWS6	Clipped - Modified	Cracks and tie bent

3.5 Mortar type & properties

Mortar used in Phase I was a type N mortar and for Phase II and Phase III specimens was type S. Both mortars at the time of construction came from a single batch of premixed wet mortar and were cement-lime type mortar.

Mortar strength properties that mostly contribute to the performance of a wall are compressive strength and flexural bond strength with the brick unit. In order to evaluate these properties, two tests were conducted. Tests on compressive mortar strength were performed according to CSA A179-94 Mortar and Grout for Unit Masonry, while tests on flexural bond strength of the masonry assemblages were performed according to ASTM C1072 – 99 Standard Test Method for Measurement of Masonry Flexural Bond Strength.

3.5.1 Compressive Strength

For Phase I test, a total of 6 mortar cubes were made. These cubes were 50 mm by 50 mm in size (2 in. by 2in.). The mortar cubes were cured inside a room with a relative humidity greater than 90% for 28 days and three of them were tested while the rest were cured inside the laboratory room temperature where the brick panel specimens were stored. These three remaining cubes were then tested at the last day of the test for the specimen PL6 in order to examine the relation of the mortar compressive strength to time and curing condition. Results of the compressive strength of the mortar for six cubes from the Phase I specimens are listed in Table 3.3. All value of the value exceed the specified compressive strength property for type N mortar (Table 5 of CAN CSA A179-94 Mortar and Grout for Unit Masonry).

Table 3.3 – Compressive strength of mortar cubes for Phase I specimens.

No.	Age (days)	Mortar Type	Compressive Strength MPa (psi)	Notes
1.	28	N	9.86 (5920)	Laboratory cured
2.	28	N	10.89 (6390)	Laboratory cured
3.	28	N	10.03 (5820)	Laboratory cured
4.	147	N	10.91 (6550)	Room temperature
5.	147	N	10.14 (5950)	Room temperature
6.	147	N	11.55 (6700)	Room temperature

It took two days to construct the brick panel specimens for Phase II test, in which there were a total of 15 mortar cubes made with the code specified size of 50 mm by 50 mm (2 in. by 2 in.). All mortar cubes were cured under the same condition as the brick specimens inside the laboratory room temperature and covered with plastic sheets to keep them moist. The curing condition was chosen in order to measure the actual compressive strength of the mortar used to form the brick panels. The tests were divided into three parts, with the first part to evaluate the compressive strength at the early part of test, the second part to evaluate the middle part of the test, and finally the third part for evaluating the last part of the test. These three parts of test was done considering the large number of brick specimens to be tested, with which the age of the specimen might be a factor that change the behaviour of the specimens. Results of the tests are listed in Table 3.4. From each part of tests, the average, standard deviation and coefficient of variation (COV) are calculated and presented in Table 3.5. All of the values from these tests exceed the specified requirements of type S mortar according to the CAN CSA A179-94 Mortar and Grout for Unit Masonry.

Table 3.4 – Compressive strength of mortar cubes for Phase II specimens.

No.	Age (days)	Mortar Type	Compressive Strength MPa (psi)
1.	44	S	9.6 (1398)
2.	44	S	10.3 (1495)
3.	44	S	7.9 (1139)
4.	44	S	11.0 (1591)
5.	69	S	11.9 (1732)
6.	69	S	11.1 (1604)
7	69	S	11.3 (1640)
8	69	S	12.3 (1791)
9	69	S	10.6 (1539)
10	69	S	9.4 (1367)
11	114	S	8.9 (1294)
12	114	S	10.8 (1562)
13	114	S	11.2 (1620)
14	114	S	9.7 (1403)
15	114	S	11.9 (1728)

Table 3.5 – Average, standard deviation and coefficient of variation of the compressive strength of cubes for the Phase II specimens.

Age (days)	Mortar Type	Average Compressive Strength MPa (psi)	Standard Deviation MPa (psi)	COV (%)
44	S	9.7 (1406)	1.3 (194)	13
69	S	11.1 (1612)	1.0 (150)	9
114	S	10.5 (1522)	1.2 (173)	11

Nine mortar cubes were cast for Phase III test with 50 mm by 50 mm (2 in. by 2 in.) size as specified in the code. All cubes were cured alongside the brick panel specimens, which were cured outside the structure laboratory. They were then tested at 56 days. Table 3.6 shows the results of the compressive strength tests. The values obtained from the tests exceeded the specified requirements of a type S mortar according to the CAN CSA A179-94 Mortar and Grout for Unit Masonry.

Table 3.6 – Compressive strength of mortar cubes for the Phase III specimens.

No.	Age (days)	Mortar Type	Compressive Strength MPa (psi)
1.	56	S	9.0 (1303)
2.	56	S	8.9 (1290)
3.	56	S	8.8 (1277)
4.	56	S	9.7 (1406)
5.	56	S	10.4 (1510)
6.	56	S	10.8 (1561)
7	56	S	11.7 (1703)
8	56	S	10.2 (1484)
9	56	S	9.0 (1303)

The average compressive strength is 9.8 MPa (1427 psi) with a standard deviation of 1.02 MPa (149 psi) and a coefficient of variation of 10.41%.

3.5.2 Bond Wrench Test

ASTM C 1072-99 commonly known as the bond wrench test was the standard test chosen for evaluating the flexural bond strength of the masonry assemblages in the experimental study. The test allows individual mortar joints to be tested by applying an eccentric load to a single joint in a masonry prism. Therefore more data is collected from each prism and the data is more representative since each joint in a specimen is tested.

The basic test apparatus is shown in Fig. 3.20 according to the specification standard. For the one that used in this research project, the apparatus was originally designed for concrete block masonry prism but with some minor modification, it could be used for the brick clay masonry as well. The vertical downward load for the eccentric arm was applied using a single acting hydraulic cylinder that was connected to a hand operated hydraulic pump. The cylinder would press a bar that was connected by a chain to the lever arm, which in turn provided the necessary bending moment to rupture the joint that was being tested. A pressure dial gauge was connected to the pump and recorded the pressure needed to fail the joint. This cylinder was calibrated earlier to a tinius-olsen machine for an equivalent load, thus the relation between pressure and load could be identified. Fig. 3.21 shows a picture of the bond wrench test machine that was used in the experimental study.

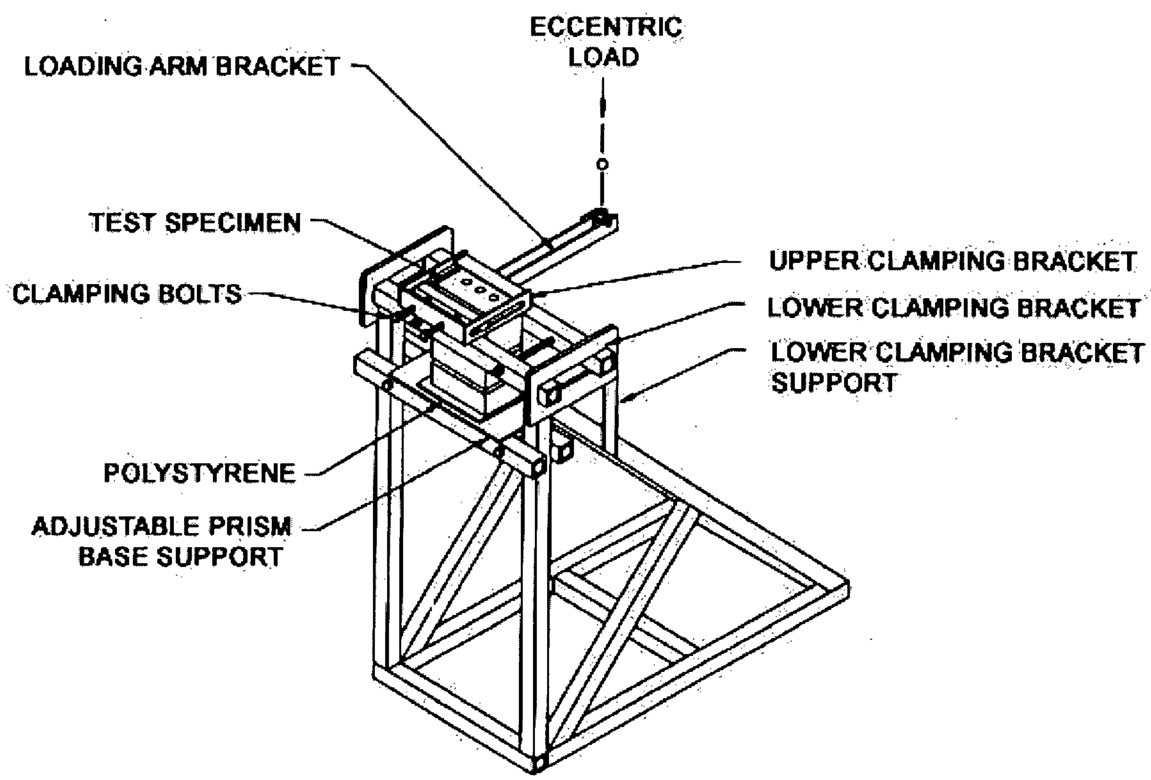


Fig. 3.20 – Test apparatus for the bond wrench test method according to the ASTM C1072-99. (From Ref. 1)

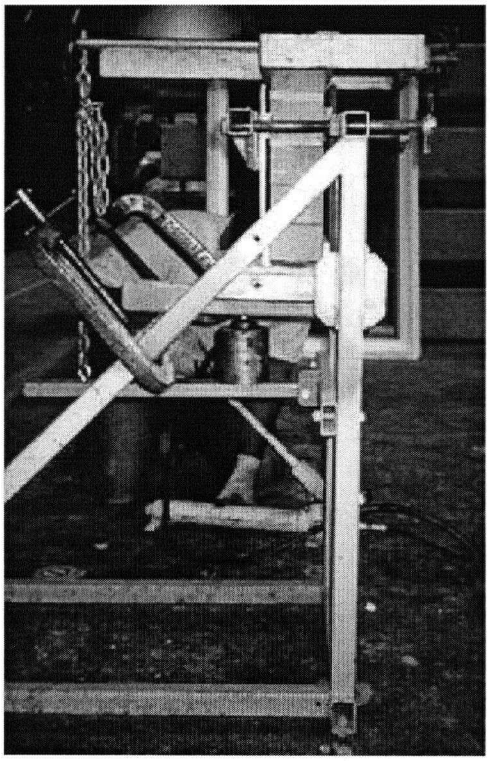


Fig. 3.21 – Modified bond wrench apparatus for the experimental study.

A total of 6 prism specimens consisted of 5 stack bonded bricks (4 mortar joints) were built in the first Phase, 11 prism specimens of 6 stack bonded bricks (5 mortar joints) were constructed in the second Phase and 5 prism specimens consisted of 5 stack bonded bricks (4 mortar joints) were constructed in Phase III along with the brick panel specimens. The joints were tooled on one face of the prism specimens in order to match the joints on the brick panel specimens. The prism specimens for Phase I test (the pilot series) were cured inside the laboratory condition alongside the Phase I brick panel specimens without being covered. While for the Phase II prism specimens, all were covered with plastic sheets and cured until 28 days when all the plastic sheets were removed. This was also the case with the Phase III prism specimens, where all prism specimens were covered with plastic sheets until 28 days. After the plastic sheets were removed, the prisms were left to cure with the outside temperature until the day of the test.

The prism specimen was then put inside the support frame with the tooled joints face the bolts on the loading lever arm so as to applied flexural tension to the joints. The prism specimen was placed vertically with one single brick projected over the lower clamping bracket with the mortar joint that was going to be tested. The upper clamping bracket then placed on top of the prism and the bolts were tightened using a torque. Hydraulic cylinder put into place and then the loading sequence begin. The loading was applied with a uniform rate at approximately 2 minutes duration.

Tests performed on the Phase I prism specimens were unsuccessful, the bond was weak and there were no significant data could be recorded. This condition was later examined and evaluated. Observations showed that the failure was caused by several reasons including curing conditions on the laboratory and also the handling of specimen during the test. Careful attentions were taken for the Phase II prism specimens.

Tests for the Phase II prism specimens (only 6 were successfully tested) were conducted at 44 days and 87 days to account for the age factor. Table 3.7 below shows the average value of bond wrench test for the Phase II prism specimens along with the standard deviation and coefficient of variation

Table 3.7 – Results from the flexural bond wrench tests for the Phase II prism specimens.

Prism No.	Age (days)	Average Flexural Bond Strength Mpa (psi)	Standard Deviation Mpa (psi)	COV %
1.	44	1.18 (171)	0.48 (70)	41
2.	44	1.29 (187)	0.38 (55)	30
3.	44	1.03 (150)	0.47 (68)	46
4.	87	1.15 (166)	0.29 (45)	26
5.	87	0.78 (113)	0.22 (32)	28
6.	87	1.27 (183)	0.41 (59)	32

The results from all the prisms exceeded the specified requirement for type S mortar bond strength as specified in the CAN CSA A179-94, it is also an indication that good bond between brick units and mortar was achieved.

The bond wrench test for Phase III prism specimens were conducted at 74 days for all the specimens. The results are listed on Table 3.8, which consisted of mean value, standard deviation and coefficient of variation for the prism specimens.

Table 3.8 – Results from the flexural bond wrench tests for the Phase III prism specimens.

Prism No.	Age (days)	Average Flexural Bond Strength Mpa (psi)	Standard Deviation Mpa (psi)	COV %
1.	74	1.31 (190)	0.15 (21)	11
2.	74	0.93 (134)	0.37 (53)	40
3.	74	0.54 (79)	0.26 (38)	49
4.*	74	0.59 (85)	----	----
5.*	74	0.97 (140)	----	----

* Only one value could be obtained from the prism specimen.

The results from the tests indicated values that exceed the specified requirement for type S mortar bond strength according to CAN CSA A179-94 Mortar and Grout for Unit Masonry.

3.6 Instrumentation

Instrumentation used in the experimental program includes one load cell and five LVDT displacement transducers. The load cell measured the magnitude of the applied point load to the brick tie. Prior to using the actuators and load cell for the test, both were calibrated. Since the load applied will be through the loading guide, the friction in the loading guide should be low; this was confirmed by testing the hydraulic actuator without connecting it to the tie and recorded the measured load, which was considerably low.

The stroke of the hydraulic actuator was measured by LVDT that was built in with the actuator. Another LVDT was used to measure the displacement of the brick tie in the out-of plane direction. This LVDT was actually measuring the clamping device movement, which will justified the displacement of the tie, the purpose of this LVDT was to confirmed that there was not any free play between the clamping device and the tie. The movement of the brick at the location of the tie were determined by another LVDT, as this was intended to obtain the relative displacement of the veneer tie itself. The vertical displacements across the critical mortar joint on the two sides of the panel were also measured by two other LVDT. Figure 3.22 shows the locations on the brick panel specimen that were measured by the LVDT.

The electronic signals from the load cell and the five LVDT displacement transducers were converted from analog to digital data, and recorded by a personal computer equipped with a data acquisition program. During the loading, approximately four data points were recorded every second.

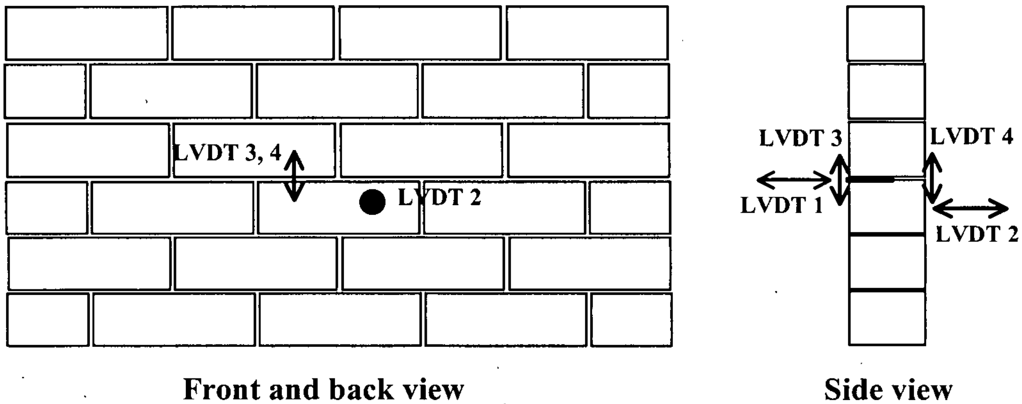


Fig. 3.22 – Location of displacement transducers at the brick panel specimen.

3.7 Cyclic Testing Procedure

The applied load was controlled by regulating the hydraulic pressure in the actuator, which was controlled using an "MTS" controller. All specimens for the Phase I and Phase II were tested using a displacement-controlled reversed-cyclic loading protocol, which is specifying a target displacement in each loading stage and then the hydraulic actuator move to the specified target displacement. A cycle of loading involved: applying tension to the tie with the specified amount of displacement, unloading, in which the hydraulic actuators moved back to the initial position (displacement equal to zero), and then applying compression to the specified displacement and finally unloading the tie again.

The loading protocol used in Phase I and Phase II consists of three cycles of loading for each loading state that is for each target displacement level, the load will be applied three times in tension and compression, with 1 mm displacement as the initial target displacement. For the next loading stage, the target displacement level was increased at ± 1 mm until it has reached excessive displacement. Observations from the Phase I test concluded that some minor modification have to be made to the loading guide for the hydraulic actuators to apply a displacement with more than 10 mm in tension and compression. For the 2nd Phase of test, the small increments went up to 12 mm, and then increased directly to 15 mm. Figure 3.23 shows the loading protocol used in the experimental program. The loading protocol was defined in terms of the actuator stroke, although it was stated in the instrumentation section that the main displacement of interest is the movement of the tie relative to the surrounding brick

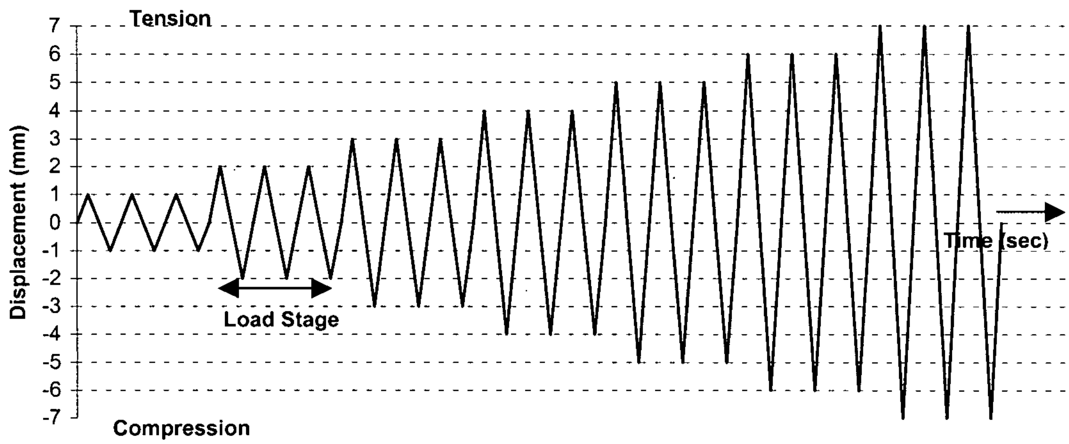


Fig.3.23 – Loading protocol used for Phase I and Phase II of the experimental study.

In Phase III test, which was intended to further evaluate the brick veneer tie mechanically connected to a continuous horizontal wire joint reinforcement; one of the parameter being investigated was the loading history or loading protocol that was applied to the specimens. This was decided to validate the load displacement envelope curves from the previous test when compared to different loading cycles. It was also to investigate the different failure modes or behaviour of the system that may be discovered with different loading history and fewer cycles of reversed cyclic loading. Several monotonic tests conducted to predict the maximum embedment strength of the system under tension and to compare to the available data from the manufacturer of the tie. The monotonic envelope curve is useful to show the initial or backbone envelope curve without any degradation from repeated reverse loadings.

The subsequent loading history consists of two parts. First part was determined on a force demand thus the loading cycles was a force-controlled reversed cyclic loading protocol. This was based on the evaluation of the veneer wall system under seismic loading, which in the first few cycles of the earthquake loading will experienced a high demand of force from the inertia force of the wall. Force-controlled loadings means that for each loading stage there is a target force level that is applied to the specimen with the force level that is applied to the specimen with the movement of the hydraulic actuator until it reached the target force level. In this experimental study each of the loading stage consisted of three cycles that is the load will be applied three times in tension and compression, in the specified target force level. The initial force level was decided to be 1.5 kN (337 lbs) and then increased by ± 0.5 kN (112 lbs) for the next loading stage, this was continued until the displacement level reach 2 mm displacement. The rationale behind the 2 mm displacement limit was from the observation of the previous tests (phase I and II tests), in which the maximum resistance of the embedment strength of the system was almost always reached between 1 mm and 2 mm displacement in the low surcharge case. This was then the basis of the investigation on the case where the first few cycles of loadings from the earthquake were actually happening below the maximum capacity of the tie system.

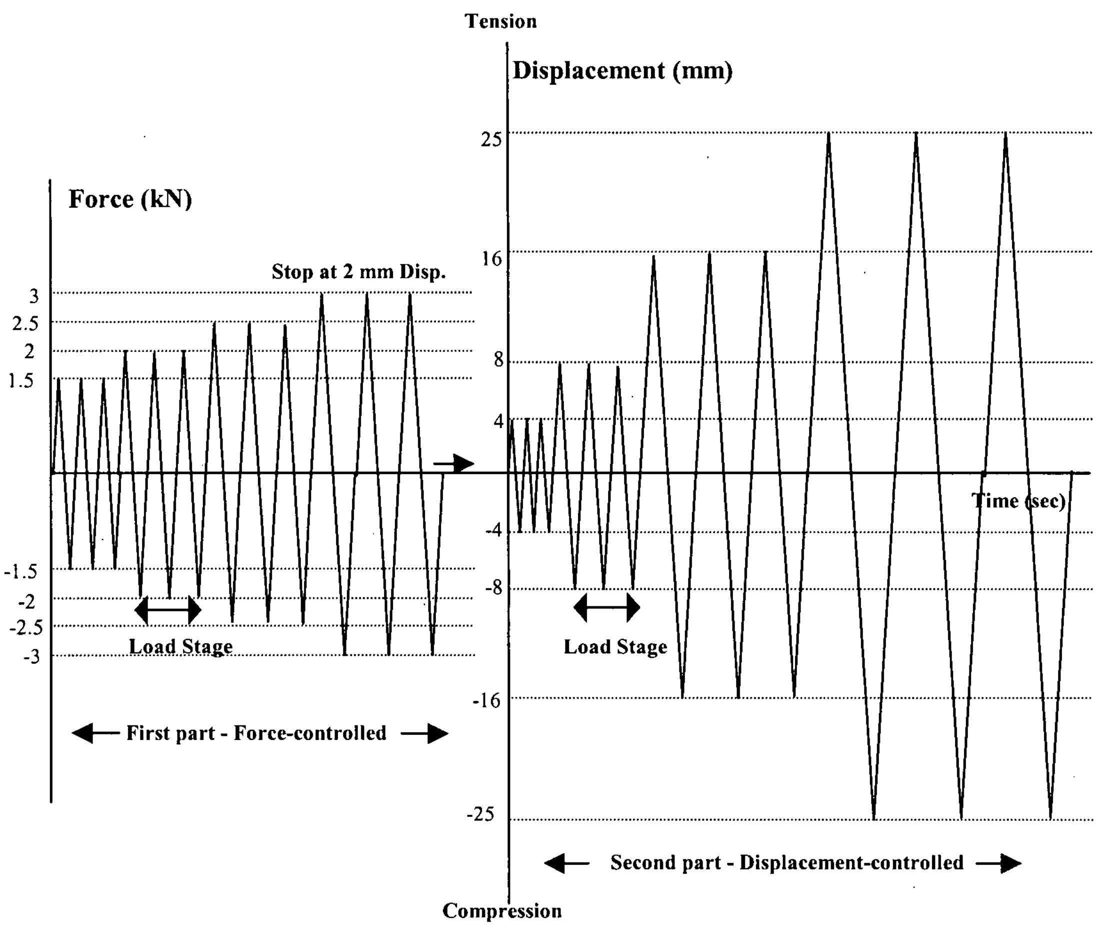


Fig. 3.24 – Loading protocol for the Phase III tests.

The second part was switched back to a displacement-controlled reversed-cyclic loading protocol but with a different target displacement level for each loading stage. It was intended to apply less number of loading cycles to the specimen, in order to examine the different behaviour of the system that might be significant. Each loading stage still consisted of three cycles of loading for each target displacement level and the load will be applied three times in tension and compression. The initial displacement was 4 mm and then increased to 8 mm and finally 16 mm for the final stage displacement. The later specimens added one more loading stage that was 25 mm displacement for the final stage of loading. All the displacements were defined in terms of the actuator stroke. Fig 3.24 shows the complete loading history for the subsequent loading protocol which includes the first part, which is force-controlled reversed-cyclic loading and the second part that is the displacement-controlled reversed-cyclic loading.

Chapter 4 Discussion of Test Results

4.1 Introduction

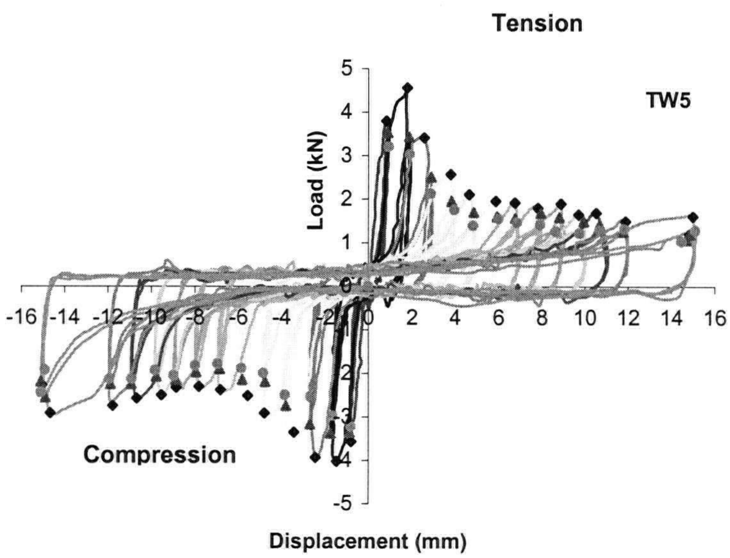
The experimental results from Phase I, Phase II and Phase III tests are summarized and discussed in this chapter. The discussions start from the Phase I and Phase II tests and continue on to Phase III test, which was a further investigation of the behaviour of the system. The Phase III discussions are also linked back to the results from the earlier Phase of test. A detailed explanation of the test result for each specimen can be found in Appendix A. This chapter is followed by Chapter 5, which are conclusions and design recommendations for the experimental study.

4.2 Summary of Experimental Results

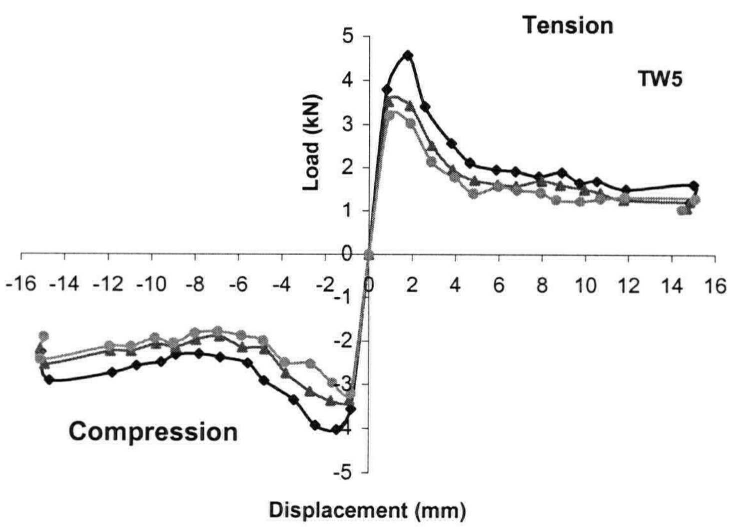
The Phase I tests was done to study the test apparatus and to determine the typical behaviour of the ties, as a guideline for the next phase of tests. The reversed cyclic loading was done relatively slow to measure the displacements of the ties, the brick panels and also crack width on the mortar bed joint in which the tie embedded. A total of six tests for the six pilot test specimens were conducted. From these six tests, the first test (PL1 specimen) result was discarded due to limited number of data obtained. After observations on the Phase I tests, it was decided that some minor modifications would be made to the testing apparatus. The first one was to modify the clamping device so it will accommodate more space between the tie and the brick panel. The second was shortening the length of the loading guide so as to give more stroke to the actuator to achieve more than 10 mm displacement in both compression and tension.

4.2.1 Load-Displacement Relationship Curves

Typical measured relationship between the applied force on a tie and the displacement of the tie relative to the brick is shown in Figure 4.1(a). At first stages of the loading, typically larger loads were required to displace the tie a few millimetres while the mortar in the tie location was still relatively undamaged. After several cycles of loading, the mortar became significantly damaged and thus smaller loads were required to displace the tie to larger target displacement levels. This behaviour can be evaluated by examining the hysteresis curves, which then became very "pinched" in the later stages of loading after the mortar was damaged. This "pinched" effect was due to the reason that a very little load was required to move the tie through the middle part of the cycle (mortar damaged), while near the ends of the cycle the resistance begin to pick up sharply showing that some embedment strength still left in the mortar at the target displacement level.



(a)



(b)

Fig. 4.1 – Typical brick veneer tie load-displacement curves: (a) complete hysteresis curves, (b) corresponding envelopes.

To compare the different load-displacement relationships from each specimen, the envelope curves of every load-displacement relationships were formed. This was done by identifying the peak load in each cycle of loading to a specified target displacement, as shown by the markers in Figure 4.1(a). Then these peak loads were used to plot the envelope curve, which is shown in Figure 4.1(b). There are three envelope curves corresponding to the three cycles of loading for each load-displacement relationship.

4.2.2 Load-Displacement Envelope Curves

The typical envelope curve from the three main type of specimens, that is the tie only, tie with horizontal joint reinforcement and tie with horizontal wire reinforcement clipped, is shown in Figure 4.2(a), (b) and (c) respectively.

Table 4.1 – Summary of test results from Phase I and Phase II specimens.

				Summary of Results							
				Tension				Compression			
				1st Cycle		3rd Cycle		1st Cycle		3rd Cycle	
Specimen Name	Horizontal Bed Joint Reinf.	Surcharge (kPa)	Age (days)	Peak Load (kN)	Displ. at Peak Load (mm)	Load at 5 mm (kN)	Load at 10 mm (kN)	Peak Load (kN)	Displ. at Peak Load (mm)	Load at 5 mm (kN)	Load at 10 mm (kN)
PL2	none	60	140	3.17	2.09	1.11	....	-4.84	-4.98	-3.09	....
PL3	none	13	145	2.29	1.59	1.65	1.81	-3.17	-2.39	-1.81	....
PL4	wire (no clip)	60	144	2.01	5.70	1.65	....	-4.31	-2.27	-2.40	....
PL5	wire (no clip)	13	146	1.50	2.03	1.12	0.45	-2.42	-3.23	-1.36	....
PL6	wire - clipped	60	147	2.97	4.06	1.60	1.20	-4.85	-4.61	-2.50	....
T1	none	4.2	47	3.41	0.92	0.46	0.35	-4.62	-1.56	-1.24	-1.31
T2	none	4.2	54	3.12	0.55	0.52	0.44	-3.45	-1.00	-1.31	-1.18
T4	none	60	58	3.68	1.71	1.41	1.01	-2.95	-1.98	-1.92	-1.71
T5	none	60	64	3.68	1.66	1.69	1.49	-3.05	-0.68	-1.73	-1.18
T6	none	4.2	75	2.87	1.50	0.82	0.54	-2.90	-1.12	-0.87	-0.59
TW1**	wire (no clip)	4.2	51	1.21	6.81	0.66	0.79	-1.98	-5.93	-1.39	-1.11
TW2	wire (no clip)	4.2	56	2.32	0.65	0.92	0.33	-2.68	-0.95	-0.94	-1.00
TW3*	wire (no clip)	4.2	72	0.81	3.68	0.48	0.48	-2.40	-2.81	-1.45	-1.01
TW4	wire (no clip)	60	62	3.03	1.46	1.48	0.86	-4.08	-4.72	-2.35	-2.50
TW5	wire (no clip)	60	65	4.56	1.77	2.07	1.66	-4.03	-1.47	-2.81	-2.52
TW6	wire (no clip)	4.2	75	3.57	1.57	0.95	0.79	-3.87	-0.76	-1.71	-1.68
TWC1	wire - clipped	4.2	51	2.80	3.15	1.34	0.86	-2.74	-3.21	-1.28	-0.71
TWC2	wire - clipped	4.2	57	2.03	6.85	1.51	0.65	-2.68	-0.89	-1.07	-0.90
TWC3	wire - clipped	4.2	72	2.55	1.40	1.42	0.90	-2.82	-1.72	-1.38	-0.87
TWC4	wire - clipped	60	63	4.22	3.64	3.45	2.01	-3.89	-1.53	-2.39	-1.87
TWC5	wire - clipped	60	70	3.99	5.78	3.24	2.97	-5.33	-2.74	-3.48	-1.91
TWC6	wire - clipped	60	78	3.64	3.31	2.53	0.87	-5.31	-3.32	-4.01	-1.84

All specimens were built using type S cement lime mortar, except for pilot series, which were type N.

* Loading problem, data included in table but not in figures.

** Specimen pre-cracked, data included in table but not in figures.

Table 4.1 lists the maximum loads and the displacements for Phase I and Phase II specimens during the first cycle, as well as the loads at 5 mm and 10 mm displacements during the third cycle of loading which were noted in the figures also.

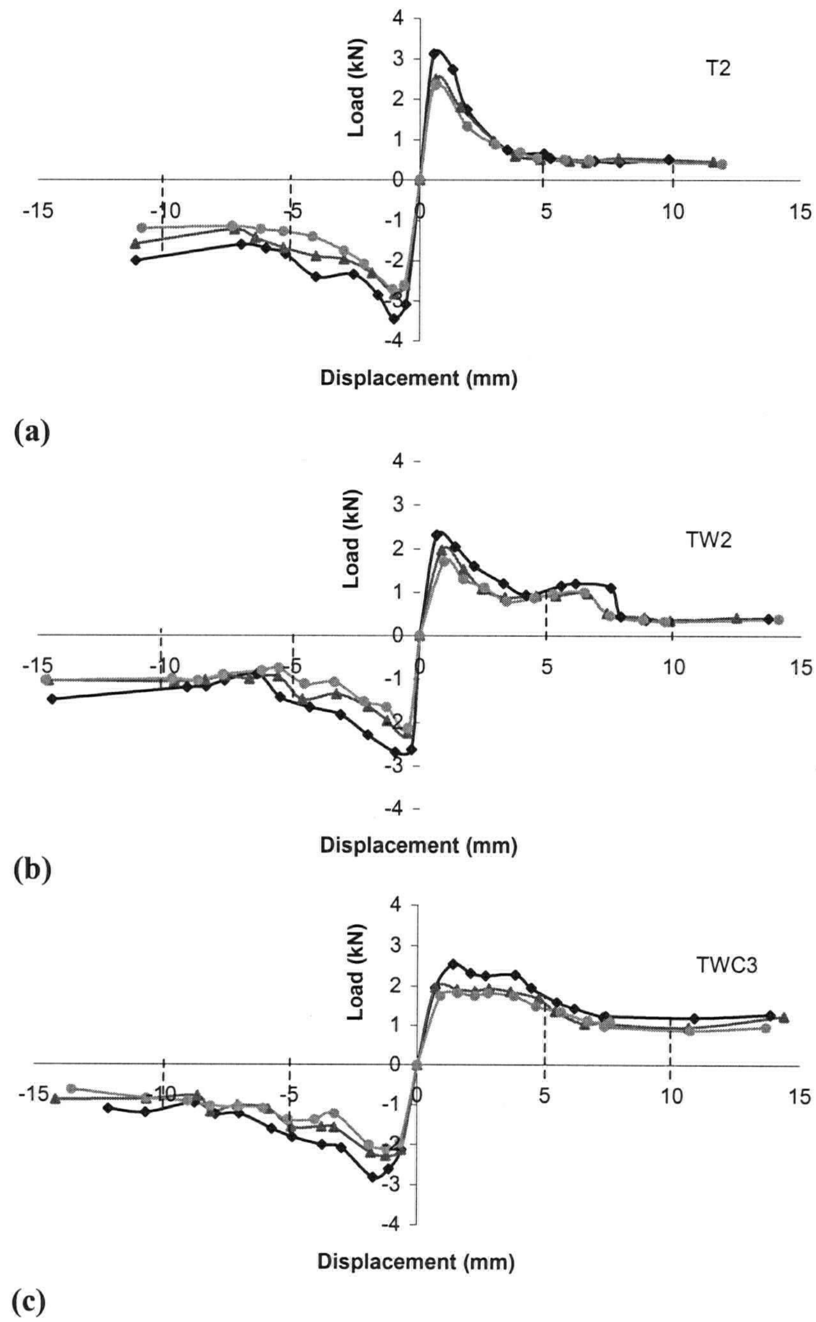


Fig. 4.2 – Typical envelope curves for the three different types of specimens: (a) tie only, (b) tie with horizontal joint reinforcement unclipped and (c) tie with horizontal joint reinforcement clipped.

The envelope curve shows the degradation of the embedment strength of each type of ties as it is being loaded in tension and compression.

The maximum loads listed in Table 4.1 are the peak embedment resistance of the wall tie system. While embedment failure is a possible failure mechanism of a tie system, it is only one link in a series of other potential mechanisms such as metal failure and/or buckling of the tie and fastener failure. Based on monotonic tests conducted by manufacturer of these ties, which include other failure mechanisms, the typical failure loads are of a similar magnitude to the maximum loads listed in Table 4.1 (Reference 19). Therefore embedment strength is an important factor in capacity of a brick tie subjected to reversed cyclic loading

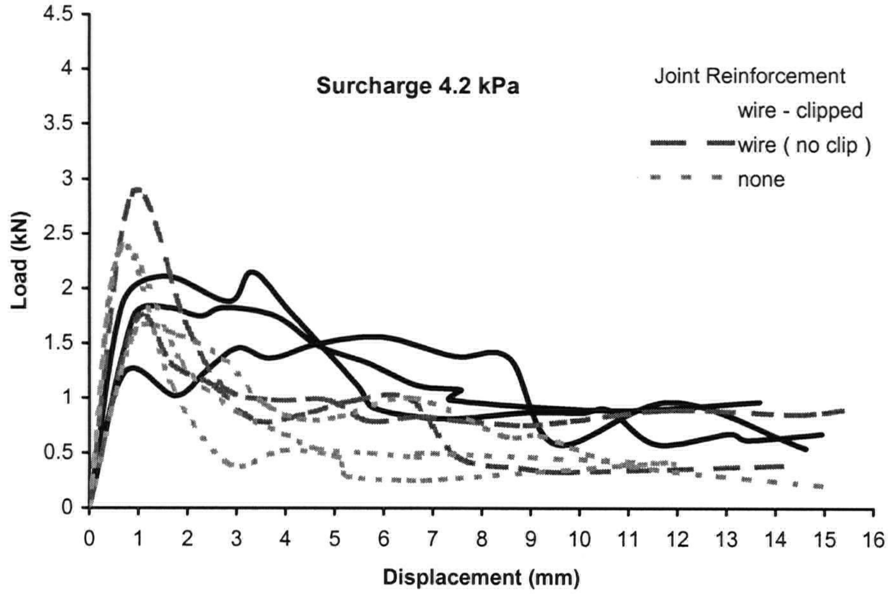
4.3 Comparison of Load-Displacement Envelope Curves

Performance of the brick veneer tie under the reversed cyclic loads is evaluated by comparing the envelope curves of the main three types of specimens. In order to take account of the damage caused by previous cycle of loading, the third cycle loading envelope will be the main interest of study. Therefore the comparison will be the envelopes from the third cycle of loading for all tests. To facilitate comparison, the envelopes are plotted to the same scale.

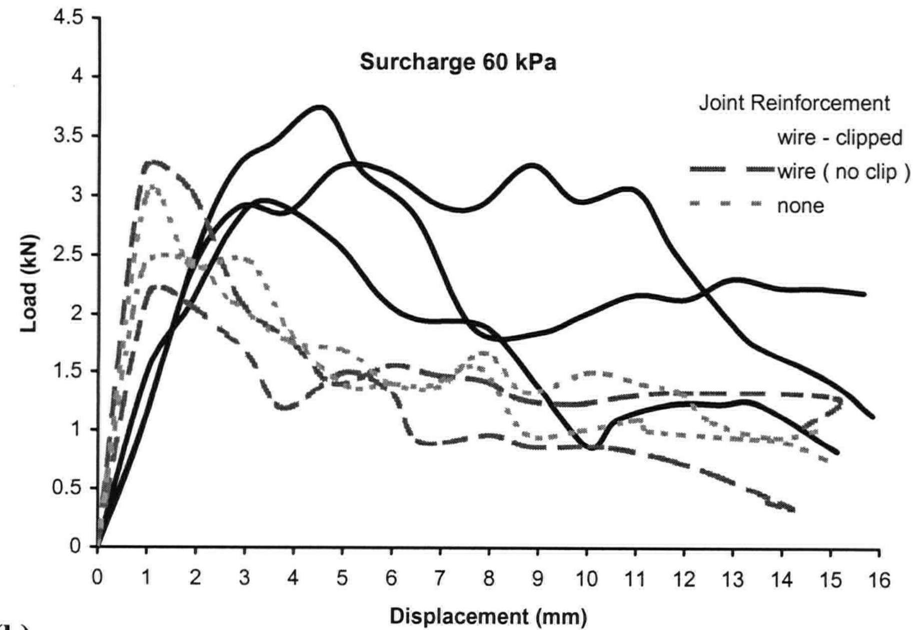
4.3.1 Tension Envelope Curves

Figure 4.3 shows the comparison for tension, with part (a) is for low surcharge (represents top row of ties) and (b) is for high surcharge (bottom row of ties). The tension envelope for the tie only specimens and tie with horizontal joint reinforcement with no clips shows almost no difference, compare the long dashes and short dashes on the figure. This is an indication that by adding a horizontal joint reinforcement without the clips had only little effect to just using a tie only.

When the horizontal joint reinforcement was clipped to the brick tie, the solid line on the figure, the effect was quite significant. For both surcharges, the clipped horizontal joint reinforcement increases the force required to pull-out the tie at higher displacement levels, at least an increase about a factor of two in load was achieved. It should be noted also with the lower surcharge case, the peak loads for the tie with clipped horizontal joint reinforcement was reduced. Observed from Figure 4.3 also that for the high surcharge case, the slope of the envelope (which is different than the effective stiffness) is significantly reduced when the horizontal joint reinforcement is clipped to the tie.



(a)



(b)

Fig. 4.3 – Influence of joint reinforcement on the third cycle tension envelopes: (a) low surcharge, (b) high surcharge.

4.3.2 Compression Envelope Curves

Figure 4.4 below shows the compression envelope curves for the three different types of specimens, part (a) is for the low surcharge case and part (b) for the high surcharge. For the low surcharge case, there is no visible trend in the envelopes from the specimens with

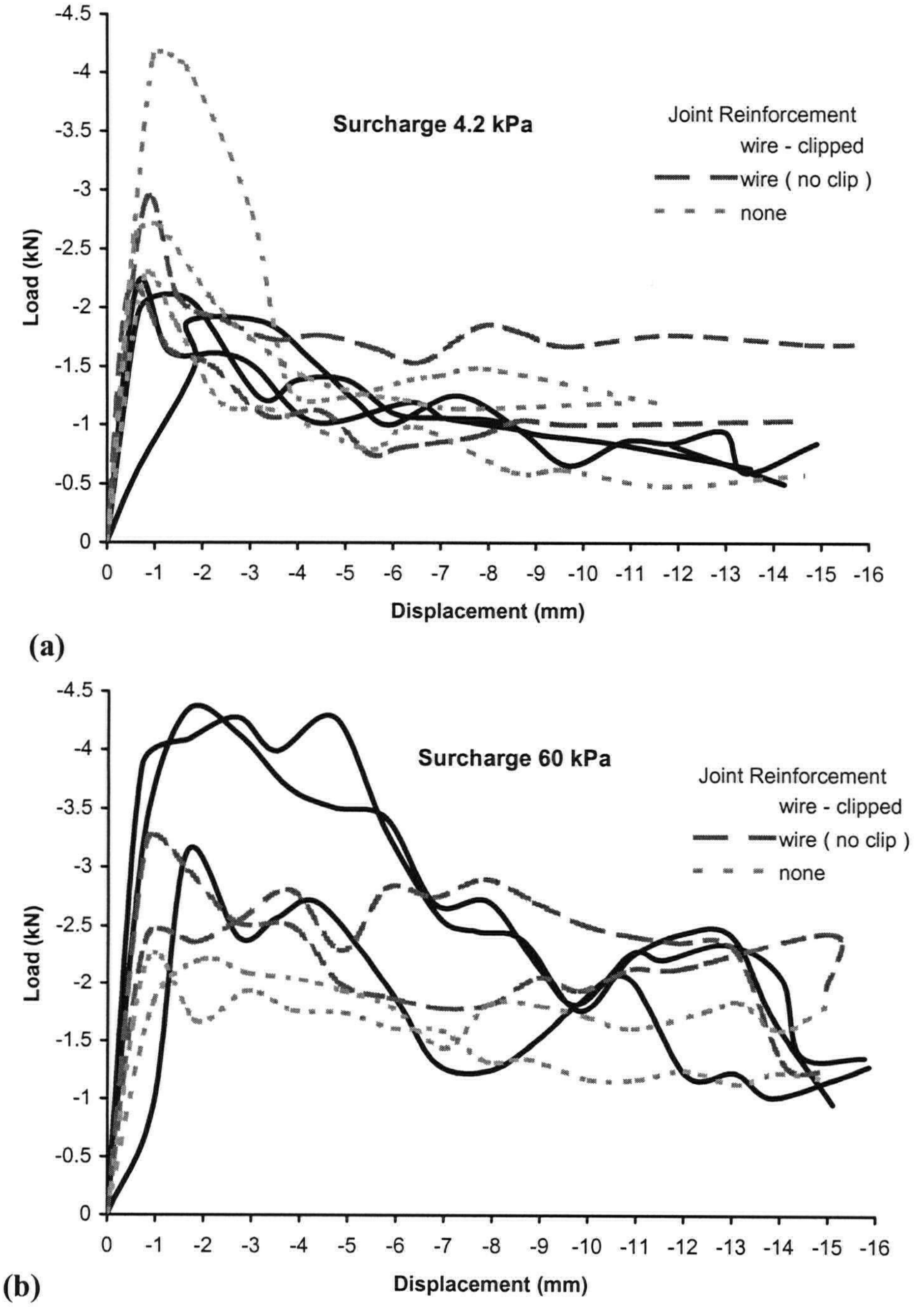


Fig. 4.4 – Influence of joint reinforcement on the third cycle compression envelopes: (a) low surcharge, (b) high surcharge.

different joint reinforcement, except notably for the peak loads. All the lines on the figure show almost no difference between them. For the high surcharge case, part (b), there appears to be somewhat of a trend, that is the envelopes for the specimens without the horizontal joint reinforcement are located near the bottom on the figure, while the envelopes for the specimens with clipped horizontal joint reinforcement are located near the top.

4.4 Observation of Failure Modes

Visual observations were made on the specimens during the cyclic loading tests and when the bed joint where the tie was located, was exposed. The later was done by carefully opening up the mortar bed joint of the damaged specimens. The observations were based on three main types of specimens that were the main study of the experimental research program.

4.4.1 Tie without Horizontal Joint Reinforcement

This type consisted of six specimens, with five successfully tested and one specimen fail to be tested (specimen T3) due to a loading problem of the hydraulic actuator. From the observations on the pull-out resistance of tie in tension, the failure mode was a pull-out of tie from mortar bed joint. This failure mode is a combination of local crushing of mortar on the area of the tie legs and the deformation of the tie itself. The deformation of the tie wire was the bending at the tie-bend location (the curve that formed the joint of the V-shaped body and legs of tie). The resistance of this bend possibly was a factor that influenced the area of mortar bed that crushed by the leg of the tie, since the external pull-out of the mortar bed joint occurred mostly in this location.

On the compression side, push-through of tie from the mortar bed joint was the dominant failure mode. The case was similar to the tension side, where the leg of the tie crushed the mortar around it and push-out the mortar bed joint with some deformation on the tie-bend location. In high surcharge, more deformation of the tie was evident; this was probably due to the fact that the mortar bed joint was harder to be damaged because of the surcharge load effect. Figure 4.5 shows a typical failure of tie only specimens after visual observations by opening up the mortar joint location of tie.

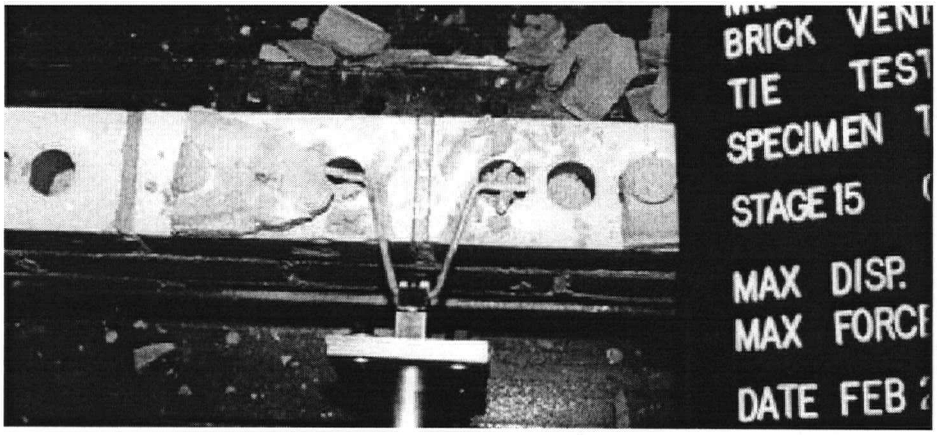


Fig. 4.5 – Photograph of a failed specimen with tie only, showing area of mortar crushed and the deformation of the tie wire.

4.4.2 Tie with Horizontal Joint Reinforcement No-Clips

There were six specimens constructed, and all of them were tested with one specimen (TW1) was pre-cracked before being tested and one (TW3) was damaged due to loading problem of the hydraulic actuator, but still manage to be tested. Observations on the specimens indicated that, in tension, a very similar case happened as the one with the tie only specimens. The horizontal joint reinforcement did not really affect the strength of the embedment of the tie, and the mode of failure of this type of specimens in tension was the same as the previous type without the joint reinforcement.

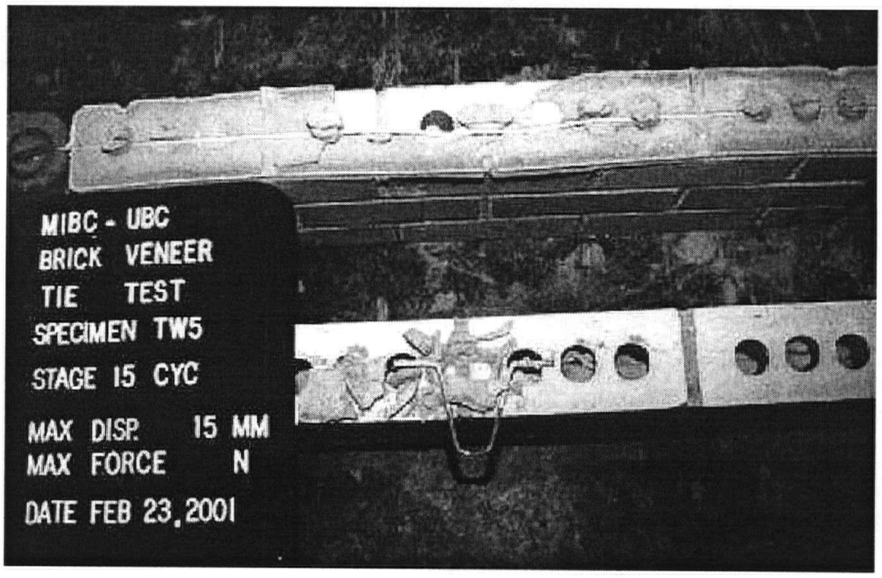


Fig. 4.6 – Photograph of a failed specimen with tie and horizontal joint reinforcement no-clips, showing the deflected wire and tie.

In compression, the wire joint reinforcement was engaged by the tie in a way that the tie was pressing directly to the horizontal wire joint reinforcement. After several cycles of loading, the wire joint reinforcement was deflected and the mortar joint was crushed and opened up, this allows the tie to over-rode the wire at some point. With a low surcharge, this occurrence happened in less cycles of loading than in the case of high surcharge load, this was apparent by the area of crushed mortar, which was push-out from the bed joint. This crushed mortar was found to be less in the low surcharge case compare to the high surcharge one. In some cases, mostly in the pilot test specimen, the horizontal joint reinforcement seems to split the mortar bed joint longitudinally. Figure 4.6 shows the failure mode for this type of specimens.

4.4.3 Tie with Clipped Horizontal Joint Reinforcement

Six specimens were built and successfully tested for this type of specimens. Observations from the cyclic loading tests were, in tension, the clips managed to maintain the mechanical connection of the tie and wire and significantly influenced the pull-out resistance on the embedment. The clipped tie could actively engage the horizontal joint reinforcement after several cycles thus crushing more mortar than the previous one. The engagement effect was even more effective in the case of high surcharge load since the load was providing a clamping effect on the system.

In compression, the clips provided a connection between the tie and the horizontal joint reinforcement that can maintain the embedment resistance. However, since there was a gap inside the clips that was originally designed to make the installation of the connection between the tie and the horizontal wire joint reinforcement easier, the gap provided a slack or a free play between the tie and the wire joint reinforcement in compression such that after several cycles of loading the tie again experienced and over-rode effect on top of the wire reinforcement. This over-rode effect was actually the case in low surcharge as there was almost no significant gain in the compression resistance. It was also observed that in larger displacement, the clips started to move along the tie and the wire reinforcement and since there was an over-rode effect of the tie to the joint reinforcement, eventually the clip started to detach from the connection and while this clip was placed near the cores of the brick unit, after some point in the loading cycles the clip just fell into the core of the brick unit. This was almost always the case for this type of specimens, which was verified by examining the bed joint carefully in the visual observations.

The visual observations with the opening up the bed joint of the tie wire reinforcement system showed that at least one of the clips was detached from the tie and the horizontal joint reinforcement and fell into the core. The discussion of this occurrence was explained in the previous paragraph. After the clip failed to provide any connection between the tie and the horizontal wire joint reinforcement, the system then behave similarly as the tie only specimens with a disadvantage that the area of mortar that already crushed was definitely larger than in the tie only case. The figure below shows the typical failure mode of this type of specimens. The unsymmetrical deformed wire was due to the over-rode effect and the detached clip on one side, while the other side still maintain the connection between the tie and the wire reinforcement.

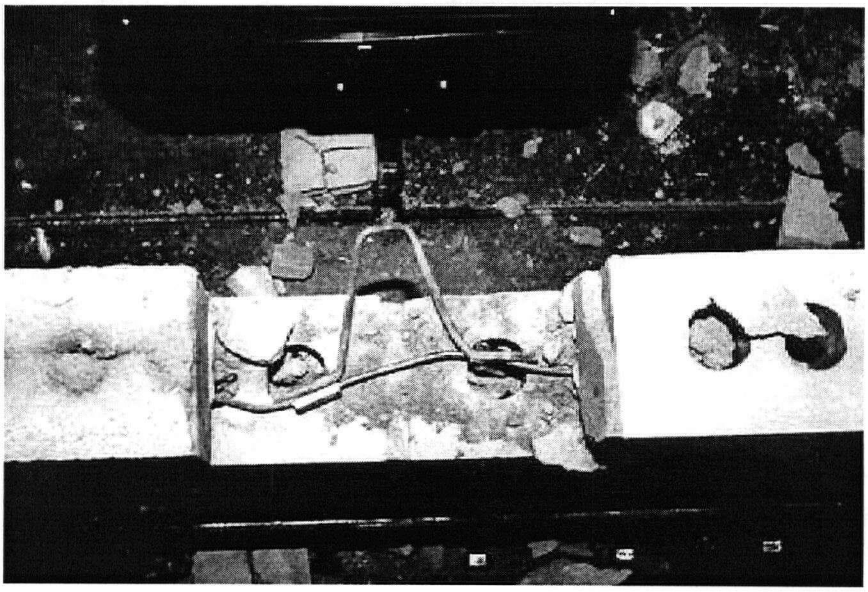


Fig. 4.7 – Photograph of a failed specimen with tie and horizontal joint reinforcement clipped, showing the unsymmetrical deformed wire.

4.5 Results from Different Type of Specimens

The category of tests that is covered in this section is the specific seismic tie type from US manufacturers, triangular tie, conventional corrugated strip tie and the construction tolerance or improper placement of the tie on the mortar bed joint. The discussion will be based on a similar idea as for the main type specimens.

For these specimens, table 4.2 lists the maximum loads and the displacement during the first cycle of loadings and also loads at 5 mm and 10 mm displacements at third cycle of loadings. The corrugated strip tie data was not included in the table, because of the different behaviour of this specimen under reversed cyclic loadings, it will be described more thoroughly later. Also specimen TT2 for triangular tie type was not included on the table because it was a monotonic test in order to examine the backbone envelope curves.

Table 4.2 – Summary of the US seismic tie system & additional specimens.

Specimen Name	Horizontal Bed Joint Reinf.	Surcharge (kPa)	Age (days)	Summary of Results: Tension, 1st Cycle: Peak Load (kN)	Tension, 1st Cycle: Displ. at Peak Load (mm)	Tension, 3rd Cycle: Load at 5 mm (kN)	Tension, 3rd Cycle: Load at 10 mm (kN)	Compression, 1st Cycle: Peak Load (kN)	Compression, 1st Cycle: Displ. at Peak Load (mm)	Compression, 3rd Cycle: Load at 5 mm (kN)	Compression, 3rd Cycle: Load at 10 mm (kN)
TT1	none	4.2	112	2.55	0.75	0.84	0.69	-3.03	-0.39	-0.83	-1.00
S1	wire attached	4.2	118	2.44	0.85	0.59	0.66	-2.06	-0.95	-0.54	-0.79
S2	wire attached	4.2	118	2.65	0.94	0.93	1.00	-2.50	-0.80	-0.87	-0.48
F1	wire attached	4.2	119	3.15	2.35	1.55	1.54	-2.06	-0.95	-0.49	-0.41
F2*	wire attached	4.2	119	1.71	1.52	0.98	0.65	-1.22	-1.32	-0.33	-0.19
OT	none	4.2	111	1.40	0.91	0.21	0.20	-3.73	-2.82	-2.17	-2.45
OTWC	wire – clipped	4.2	112	1.28	0.86	0.40	0.30	-2.70	-2.12	-2.01	-1.10

All specimens were built using type S cement lime mortar.
* Specimen pre-cracked, data included in table but not in figures.

4.5.1 US Specific Seismic Tie Type & Conventional Tie System

The approach that was taken in order to assess the performance of this specific seismic tie type was similar as in the previous study for the main specimens. The envelope curves will be compared to the main three types of specimens and this envelope will be the third cycle loading envelope to take account of the damage caused by previous cycle of loading. As before the comparison was divided between tension envelope and compression envelope. Because the tests for this type of specimens only accounted for one surcharge load (low surcharge – the critical one), there were no parameters based on the surcharge load level.

Figure 4.8 shows the comparison of the load displacement envelope for tension or pull-out embedment strength. With the Triangular type of tie, there is almost no difference in tension envelope with tie only specimens, comparing the long dashes with the triangular symbol with the short dashes on the figure. This Triangular tie can be considered to have a similar pull-out strength as the tie only specimens.

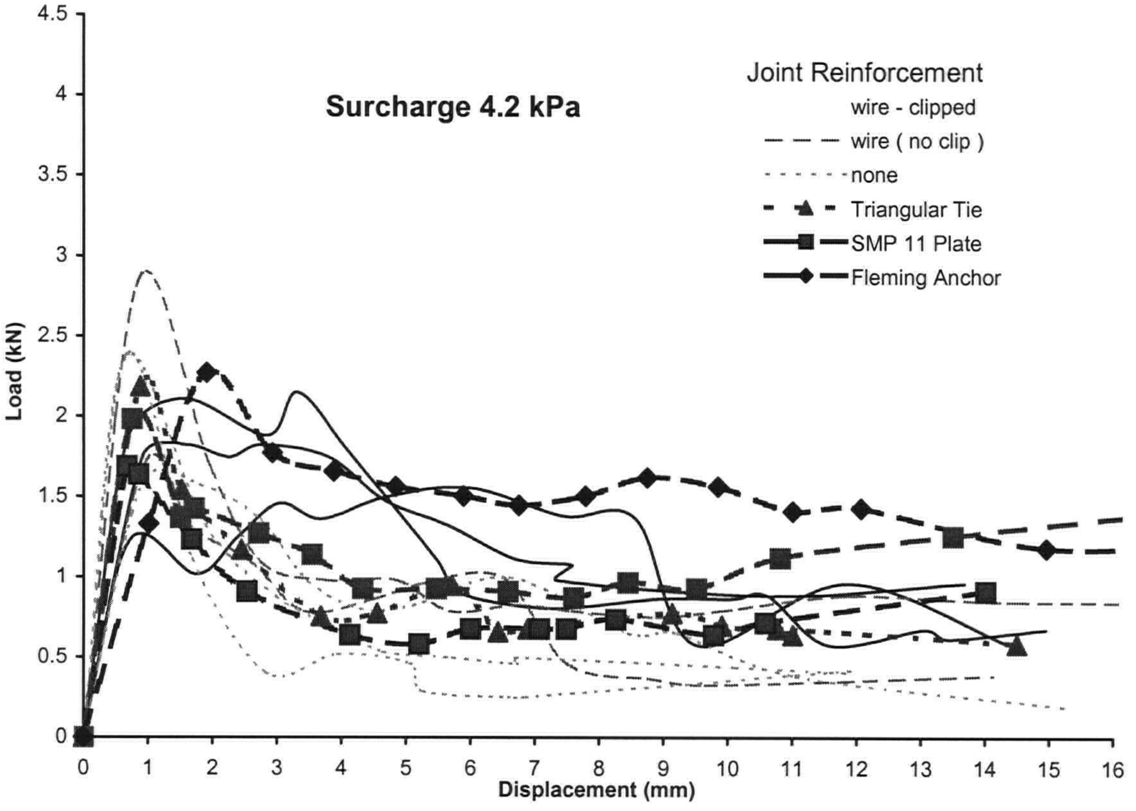


Fig. 4.8 – Comparison of the third cycle tension envelopes for US tie system and triangular tie with the three main type of specimens.

While with the US specific seismic tie type, the modified SMP 11 plate shows a poor performance in their tension envelope compared to the tie with clipped horizontal joint reinforcement, which was designed for seismic purposes also. This could be examined from the Fig. 4.8, which shows the long dashes with the box symbol on it (the SMP 11 plate) could only match the tie only specimens or the tie with horizontal joint reinforcement without the clips. However it shows that at the later part of the tension envelope there is some small increase in resistance. This performance was based on the SMP 11 plate tie that were tested, which were modified to accommodate the testing devices. In the future, there should be a further study of this type of tie in its original shape and using a quite number of specimens to examine the variations in the tests.

The Fleming anchor specimen was definitely shows a good performance on the tension envelope. This was observed from Figure 4.8, which shows the long dashes with the diamond symbol (the Fleming Anchor) was almost always above the other lines, with a very slight downward slope towards the end of the envelope. The peak load of the Fleming anchor envelope also almost matched the tie only specimens, which is considered to be the highest in peak loads. The Fleming anchor has an approximately an increase about a factor of two in load for all measured displacements compared to the tie only specimens, this was also outmatched the performance of the tie with clipped horizontal joint reinforcement. Unfortunately the other specimen for this Fleming anchor was pre-cracked before the test, thus it was not capable to confirm this behaviour with another specimen of the same type. If this behaviour is confirmed then this type of tie is capable to provides greater pull-out embedment strength in higher displacement compared to the other types of tie.

In the compression envelope curves, as shown in Figure 4.9, both the Triangular tie type and the US specific seismic type that is including the SMP 11 plate and the Fleming anchor were almost similar with the three main types of specimens, except for their peak loads. Notice that the Fleming anchor was considerably lower than the rest of the tie system in compression, this probably due to the design of the Fleming anchor with its attachment of the horizontal wire reinforcement that will only engage on tension or only contributes to the pull-out resistance of the embedded tie. Also with the Triangular type of tie there was a sudden extreme drop on the resistance as it reached a much higher displacement level. Overall it was hard to see any trend in the compression envelope for this type of tie, more specimens need to be tested to obtain more results.

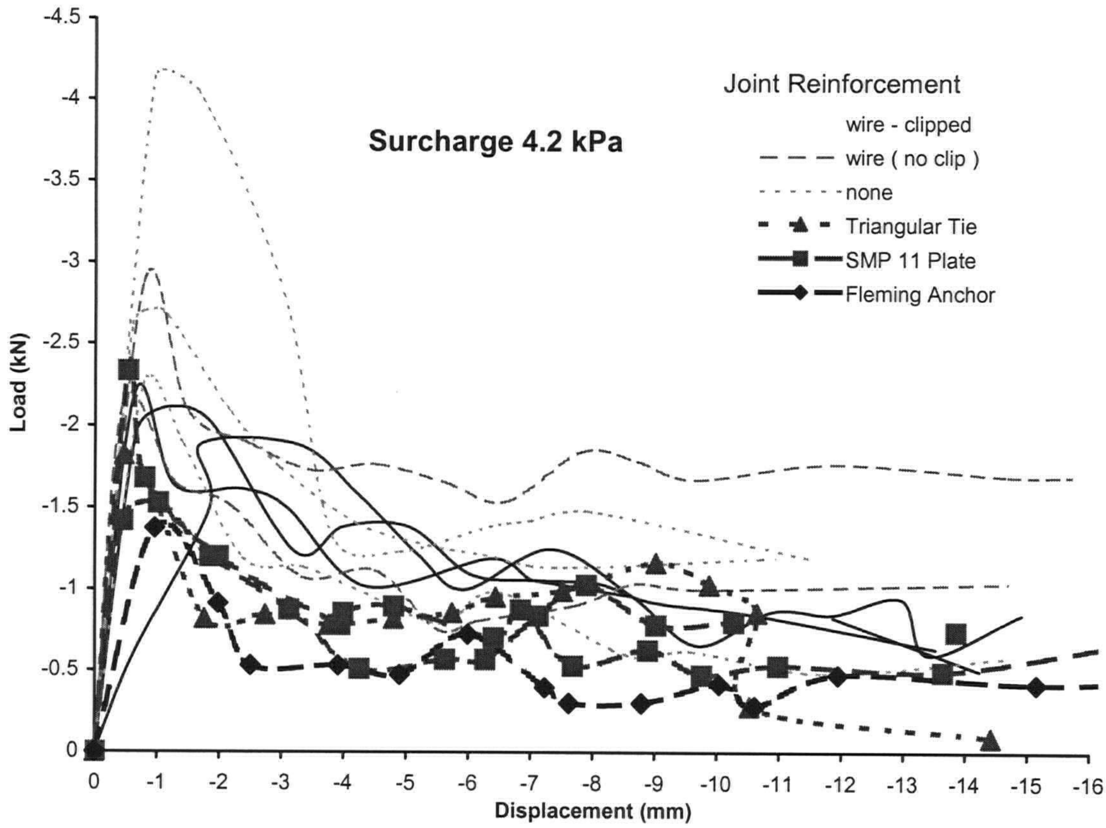


Fig. 4.9 – Comparison of the third cycle compression envelopes for US tie system and triangular tie with the three main type of specimens.

The SMP 11 plate failure mode was a pull-out of tie from mortar bed joint in tension and pull-through mortar bed joint in compression as it is shown in Figure 4.10. The horizontal wire bed joint reinforcement was deformed, which is a proof that it was engaged by the plate, although the attachment of the wire to the tie was very loose because a large amount of space in the lips of the plate. The lips was probably designed to attach the wire with mortar filled the spaces, but as soon as the tie reach its embedment strength that is the mortar embedment resistance (the plate is very stiff), the resistance becomes much lower because the nature of the plate that disturbed the mortar joint around it completely. And with the slack between the horizontal wire joint reinforcement and the lips of the plate, the plate needed more time before it will engage the reinforcement.

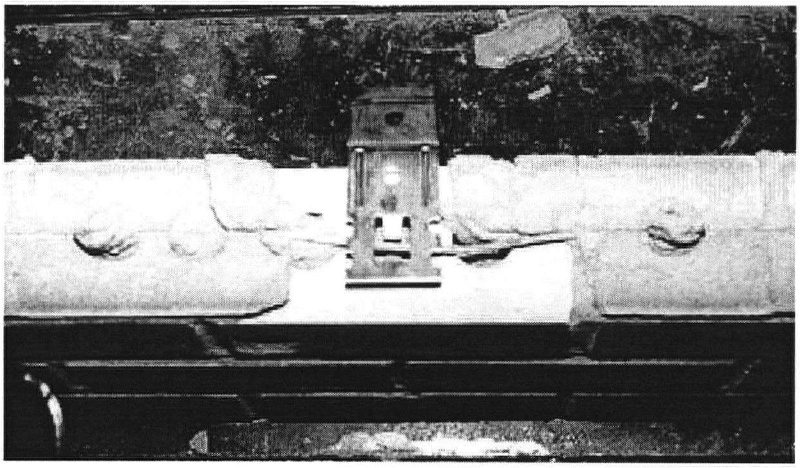


Fig. 4.10 – Photograph of a failed specimen with an SMP 11 plate with horizontal wire bed joint reinforcement.

Failure mode for the Fleming anchor was pull-out of tie from mortar bed joint in tension and push-through mortar bed joint in compression. Figure 4.11 shows a photograph of the Fleming anchor failure mode, it shows that the wire was deformed only in one direction, which is in tension direction only. This is due to the original design of the Fleming tie, which only engages the horizontal wire bed joint reinforcement in tension. The attachment of the wire joint reinforcement to the Fleming anchor itself was considerably tighter with a design of lips that will enclose the wire reinforcement when the wire becomes engaged. The smaller head of the Fleming anchor made only little disturbance to the integrity of the mortar bed joint as it can be seen from the visual observation, this could be an advantage of this type of tie also.

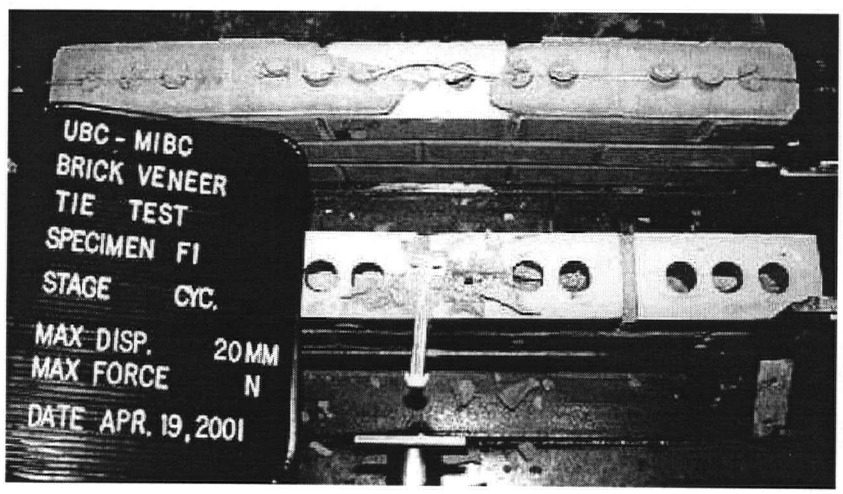
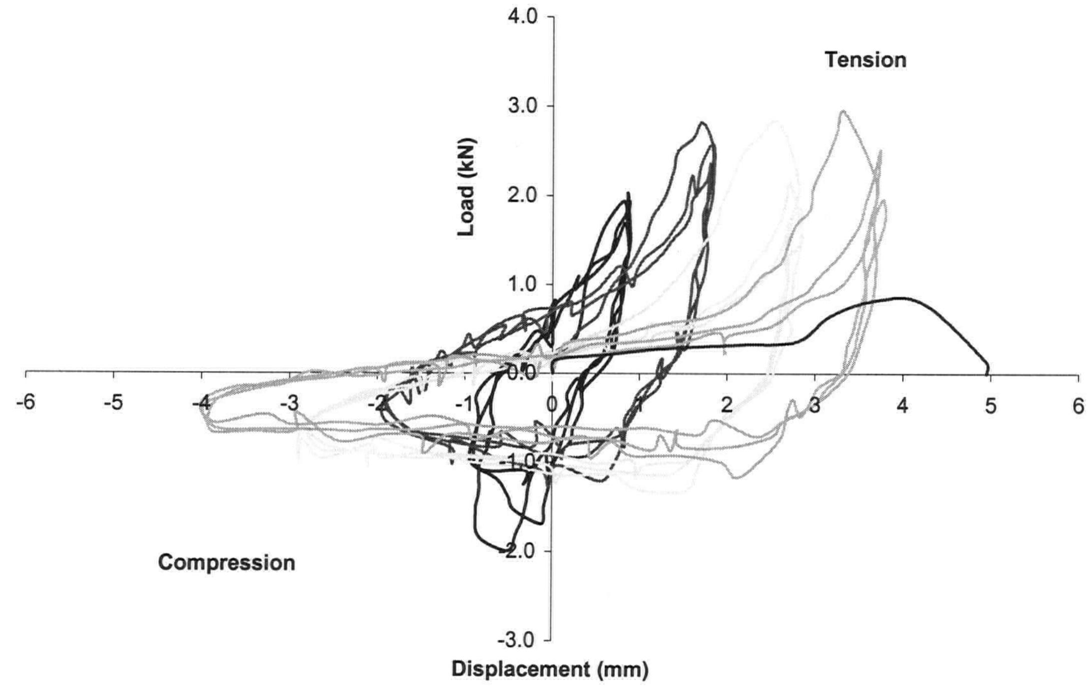
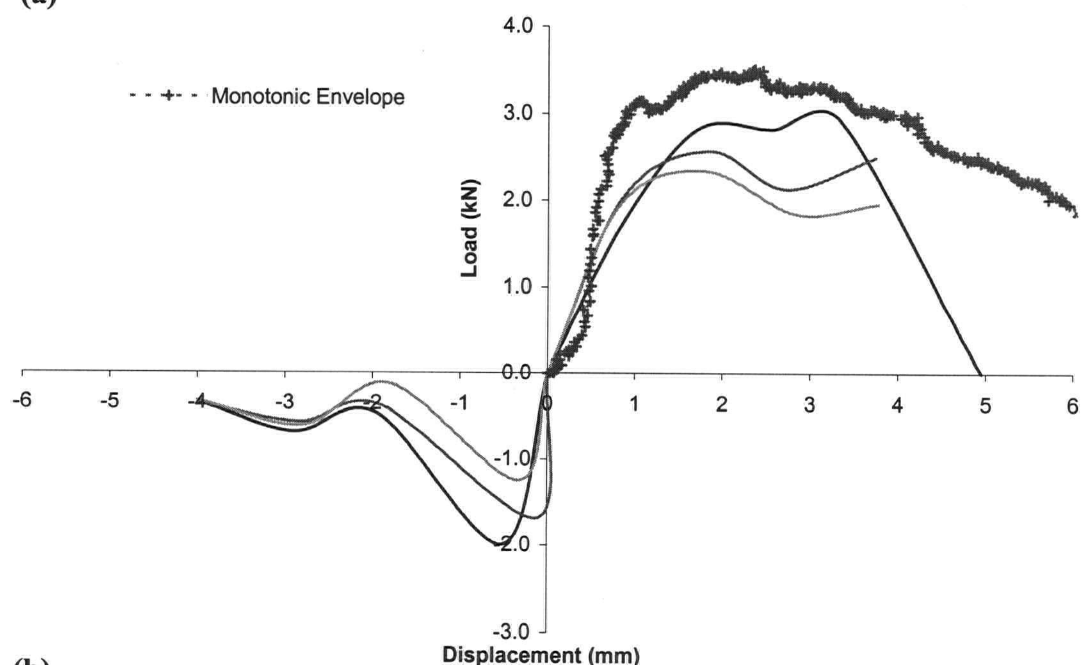


Fig. 4.11 – Photograph of a failed specimen with a Fleming anchor type of tie with horizontal wire bed joint reinforcement.

The corrugated strip tie is probably the most common type of tie. The cyclic loading test of this specimen (C1) was done to assess its performance under simulated earthquake.



(a)

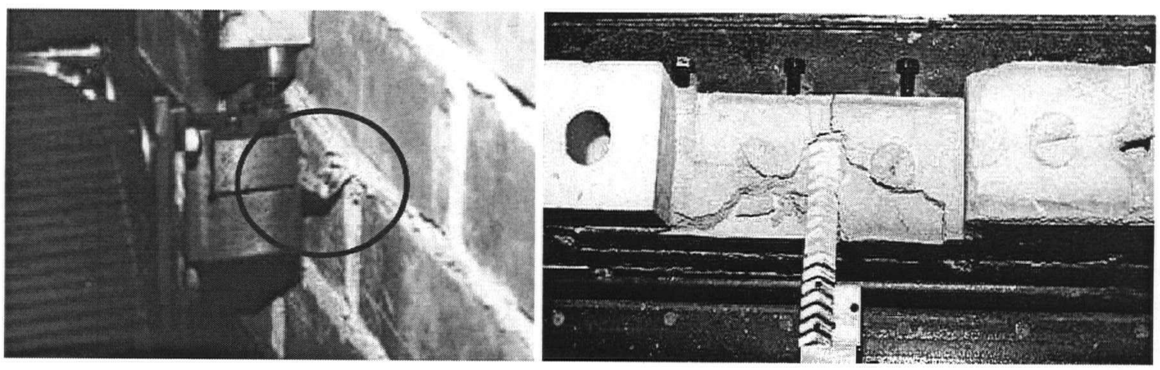


(b)

Fig. 4.12 – Corrugated strip tie specimen (a) hysteresis curves, (b) envelope curves with monotonic envelope.

While it is widely known that the most common failure mode for this type of tie is buckling in compression, it is still interesting to experimentally examine this mode. In this experimental study, the distance between the embedded parts of tie to the clamped portion of tie was 25 mm (1") to consider the air space in a real construction practice.

Figure 4.12 (a) and (b) shows the hysteresis curves and the envelope for the specimen, the envelope also includes the monotonic curve from the specimen C2. From the compression side on the hysteresis curves from Figure 4.12 (a), it clearly shows the buckling failure started as it reaches much greater displacement without any increase in load. The failure mode was then a fatigue failure of the material in tension (detached into two pieces) after 12 cycles of loading. The envelope curve shows after approximately 2.6 mm (0.1"), there was a little increase on the resistance in tension. This was not the embedment strength anymore, but it was the material strength itself, which is near to its fatigue load under the repeated loadings. The C1 specimen under cyclic loadings reached 2.8 kN maximum load at approximately 2.6 mm displacement in tension, while in compression it reached 2 kN at 0.5 mm. The monotonic test from specimen C2 reached a maximum of 3.5 kN at 2.4 mm in tension. Figure 4.13 will shows the failure under cyclic loadings and monotonic loading.



(a) **(b)**

Fig. 4.13 – Photograph of failed specimens with corrugated strip tie (a) under reversed cyclic loading (buckling failure), (b) under monotonic loading.

4.5.2 Effect of Misplaced Tie within Acceptable Tolerances

The two specimens built for this type of tests are the OT and OTWC specimens. They both have the same tolerances or error in placing the tie as much as 19 mm (3/4") from the centreline of the brick unit to the tension side as explained in the previous chapter. As before the load displacement envelope curves of the third cycle loadings for these specimens will be compared to the one for three main types of specimens. Only the tension envelope will be compared as the nature of this off-centre specimens that is they are weaker in tension and will always fail in tension failure or pull-out from their embedment in the mortar bed joint.

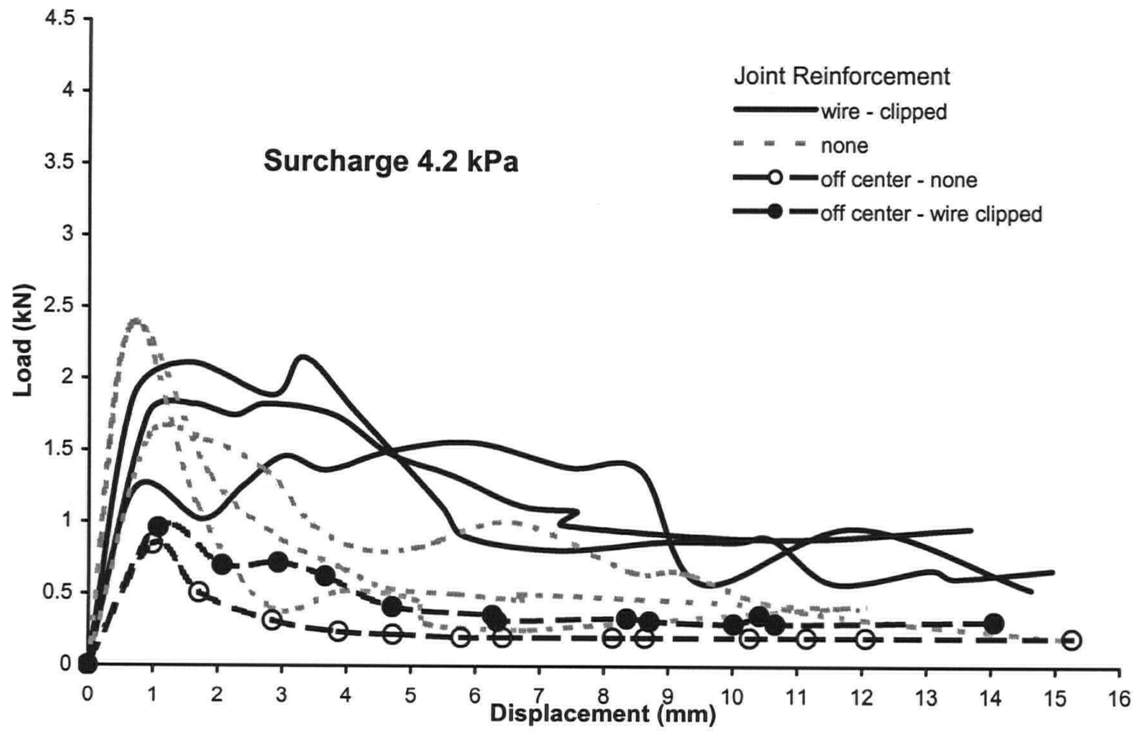


Fig. 4.14 – Comparison of the third cycle tension envelopes for off-centre specimens with the three main types of specimens.

From Figure 4.14 it clearly shows that the improper placement of the tie reduces the tension envelope curves severely. The off-centre tie only (the long dashes with hollow circles symbol) indicates a same trend in the tension envelope as the tie only specimens; the envelope curve reached to a maximum load (peak load) at approximately 1 mm displacement and then it degraded quickly in load resistance with larger displacement. The off-centre specimen with tie and clipped horizontal wire joint reinforcement (the long dashes with solid circles symbol) performs a little bit better than the off-centre tie only. The curve reached a little bit higher in peak load and then degraded almost with the same slope as the off-centre tie only, however at approximately 2 mm displacement the curves had a small increase and formed a plateau between 2 mm to approximately 4 mm. The plateau had a load resistance higher than the off-centre tie only specimen with almost a factor of two.

Comparing the off-centre specimens with each other, the one with the clipped horizontal wire joint reinforcement indicated more reserve pull-out embedment strength especially at higher displacement levels. The severe reduction in peak load and the tension envelope demonstrated the negative effect of misplacing tie outside the embedment tolerance.

4.6 Further Investigation from Phase III Test Results

As explained in the previous chapter these further tests are the Phase III tests, which were conducted after evaluating and analyzing the results from the two previous tests and modification were made to the tie clips in order to improve its behaviour. This is based on the two most important aspects introduced in this experimental study regarding the cyclic behaviour of ties. These two aspects are, first the peak load or maximum load of the wall tie and second, the embedment resistance at larger displacement. Table 4.3 lists the results from the Phase III tests which includes the maximum loads and displacements at first cycle of loadings and the loads at 5 mm and 10 mm displacements at third cycle of loadings. All of the Phase III tests used the 4.2 kPa surcharge load, to consider the most critical condition.

Specimens T7, T8 and T9 are tie only specimens for the monotonic test, this is the reason in which they are not listed in Table 4.3. This is also for specimen TWC7 and TWC8 for the tie with horizontal wire joint reinforcement using original/default clips; and also specimen TWS5 for the tie with horizontal wire joint reinforcement using modified clips, as they were all monotonically tested. All of these monotonic tests are described in the next section.

Table 4.3 – Cyclic test results summary for Phase III specimens.

				Summary of Results							
				Tension				Compression			
				1st Cycle		3rd Cycle		1st Cycle		3rd Cycle	
Specimen Name	Horizontal Bed Joint Reinf.	Surcharge (kPa)	Age (days)	Peak Load (kN)	Displ. at Peak Load (mm)	Load at 5 mm (kN)	Load at 10 mm (kN)	Peak Load (kN)	Displ. at Peak Load (mm)	Load at 5 mm (kN)	Load at 10 mm (kN)
T10	none	4.2	48	3.08	1.22	1.26	0.79	-2.77	-1.30	-0.93	-1.18
T11	none	4.2	48	3.39	1.17	0.69	0.53	-3.97	-1.41	-1.80	-1.21
T12	none	4.2	58	1.73	0.63	0.81	0.63	-3.13	-2.47	-1.82	-1.45
T13	none	4.2	68	1.88	0.69	0.65	0.43	-2.61	-2.87	-1.44	-1.20
TWC9	wire - clipped	4.2	39	2.43	1.38	0.86	0.45	-3.46	-1.63	-1.99	-0.80
TWC10	wire - clipped	4.2	32	2.42	0.80	1.78	0.84	-2.57	-0.95	-2.61	-2.03
TWC11	wire - clipped	4.2	58	1.90	1.03	0.83	0.72	-2.56	-2.28	-1.52	-1.55
TWC12	wire - clipped	4.2	56	3.26	2.86	1.63	0.91	-3.73	-7.88	-2.58	-2.09
TWS1*	wire - clipped	4.2	32	3.80	0.87	1.36	0.83	-4.75	-3.96	-2.02	-1.40
TWS2*	wire - clipped	4.2	55	2.21	2.06	0.86	0.58	-3.20	-2.59	-1.88	-1.45
TWS3*	wire - clipped	4.2	35	3.37	1.43	1.18	0.79	-4.22	-1.33	-2.56	-1.41
TWS4*	wire - clipped	4.2	60	2.93	2.00	1.00	0.74	-4.08	-2.62	-1.91	-1.20
TWS6*	wire - clipped	4.2	59	2.03	2.17	1.22	1.01	-2.55	-7.84	-1.60	-1.62

All specimens were built using type S cement lime mortar.
All brick ties were 80 mm V-Tie.
* The clips were modified as explained in chapter 3.

4.6.1 Monotonic Tension Envelopes

These tests were conducted to examine the maximum load in tension and load displacement envelope under monotonic rather than cyclic loading. The monotonic loading test is the common test use to measure the embedment strength of a veneer wall tie, as can be found in all the manufacturers catalogues.

Figure 4.15, which is a comparison of all monotonic tension envelopes shows that even for the tie only specimens (dashes), there was hardly any trend at all. The envelopes were very variable with peak loads ranging from 2.8 kN to 4.2 kN with displacements between 0.8 mm to 1.5 mm. The range of the peak loads was around the values obtained from the cyclic loading tests. Note that all monotonic envelopes have a maximum displacement of 15 mm.

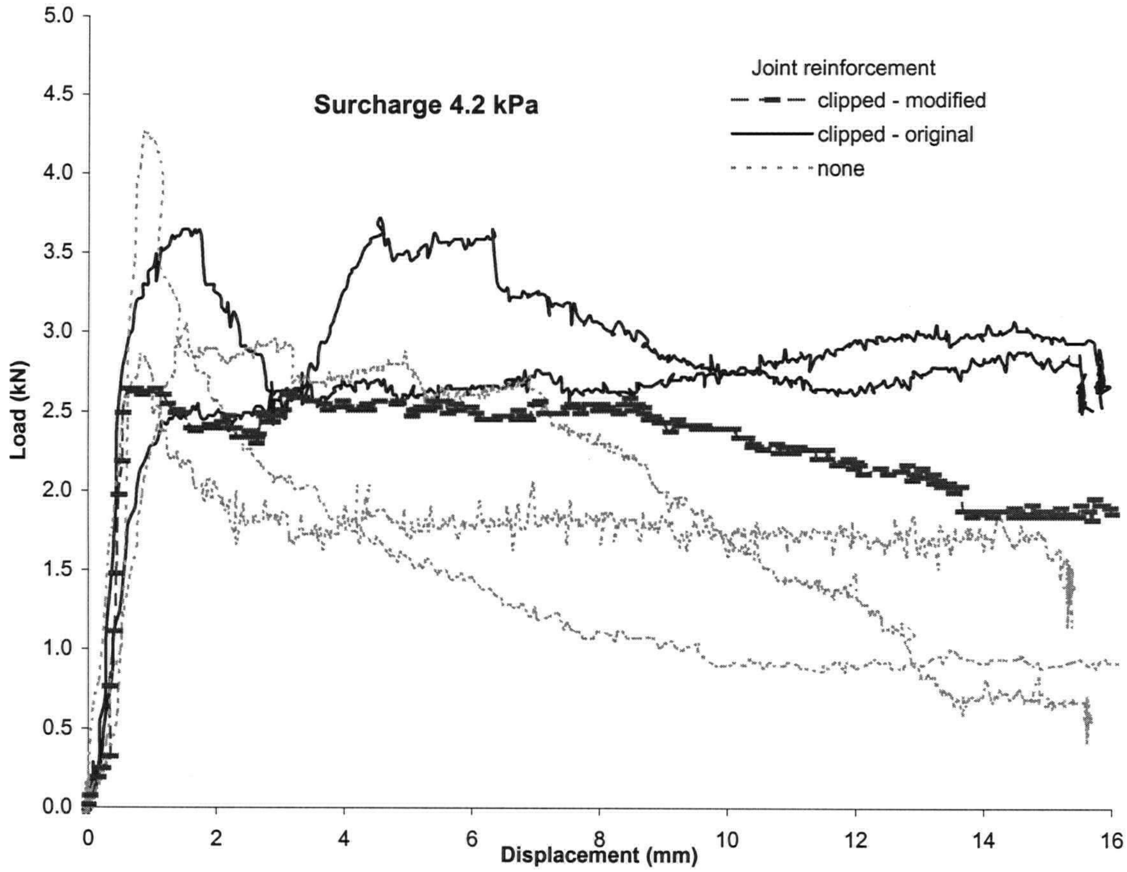


Fig. 4.15 – Comparison of the monotonic tension envelopes for specimens T7, T8, T9 for tie only; TWC7 and TWC8 for tie with original clips; TWS5 for modified clips.

Observations from the envelopes for the tie only specimens showed that, the failure modes were a combination of mortar crushing and deformation of the tie itself. If the mortar were strong enough (possibly with some sort of clamping effect) to maintain the embedment strength, the tie would then deform, as this was probably the case for the large plateau on one of the envelope line. If the embedment strength was quite weak, the tie was just crushing the mortar area around it, as was the case for the envelope line that degraded after it has reached the maximum load.

For the specimens with the original clips attaching the tie to the horizontal wire joint reinforcement (solid line), one envelope shows a twin peaks in load. These two peak loads can be described as the effect of the gap inside the clips. When the clip was installed in the bed joint with the tie and horizontal wire joint reinforcement attached and then covered with mortar, this gap was also filled with mortar. The first peak indicated that the tie reached the maximum load by crushing the mortar inside the gap first. When mortar inside the gap crushed, the resistance dropped. The tie then engaged the clip and reached a second peak in load. The envelope then just degraded as the mortar around the reinforcement was crushed. The small increase in resistance at the end of the envelope was probably the effect of the horizontal joint wire reinforcement that was deforming.

There was only one specimen (TWS5) tested monotonically for the modified clips one. This specimen suffered a damage and before testing and were repaired. The tie was bent back to its original position, and although the process seemed to be successful, the integrity of the mortar joint was already disturbed with some small cracks. This explains the low peak load or maximum load in the tension envelope that was achieved by this specimen during the test. However, the tension envelope was still showing a valid trend, with the clipped horizontal wire joint reinforcement improving the resistance at large displacement.

4.6.2 Comparison of Maximum Loads

One of the most important aspects of the brick veneer ties being tested is the maximum load. In this experimental research, the maximum loads were recorded and compared. This was done as a way to assess the performance of the different specimens with the three different conditions. The maximum loads are the first cycle peak loads given in Table 4.1 and Table 4.3. It should be realized that these peak loads that were measured are a function of the loading protocol that was used. In a sense that if a different step size had been used for the target displacements, or if the test had been a monotonic test in tension or compression, different maximum loads may have been obtained. This subject is evaluated in the effect of loading history section.

Fig. 4.16 shows the comparison of maximum loads for each type of specimens from Phase II tests based on the surcharge load level. The figure shows that without horizontal joint reinforcement the embedment strengths were quite similar regardless of the surcharge level. All the left side data points were positioned close to each other.

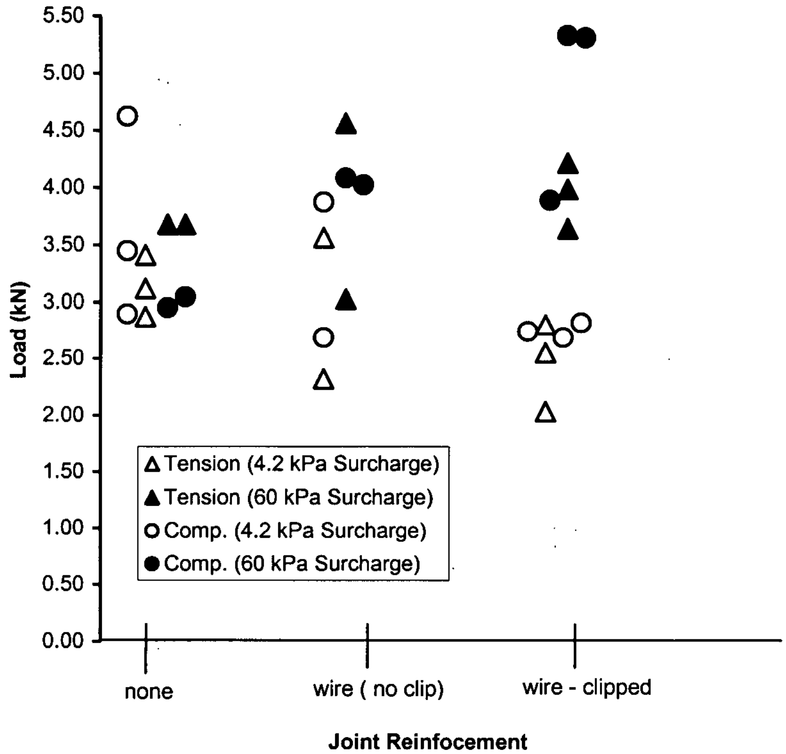


Fig. 4.16 – Maximum loads for Phase II specimens for the main study based on surcharge load.

While for the right side data points, which represent the tie with clipped horizontal joint reinforcement, there is a separation between the low surcharge and high surcharge condition. The surcharge load had a significant influence on the embedment strength for the specimens with clipped horizontal joint reinforcement. It was also evident that for the low surcharge condition (hollow markers), there was a slight reduction in embedment strength as joint reinforcement was added, as compare to the high surcharge (solid markers) there was an increase, particularly for the compression case as can be seen in the figure that the solid circles had an upward trend. Overall the strengths in tension and compression were similar in magnitude.

Figure 4.17 shows the comparison of maximum loads from Phase III and Phase II specimens. All of these results were from specimens with 4.2 kPa of surcharge load. This is because Phase III specimens were tested with 4.2 kPa surcharge load to represent the critical condition on the top row of ties. Note that the unclipped wire joint reinforcement was not on the figure, due to their observed behaviour, which is similar from the tie only specimens.

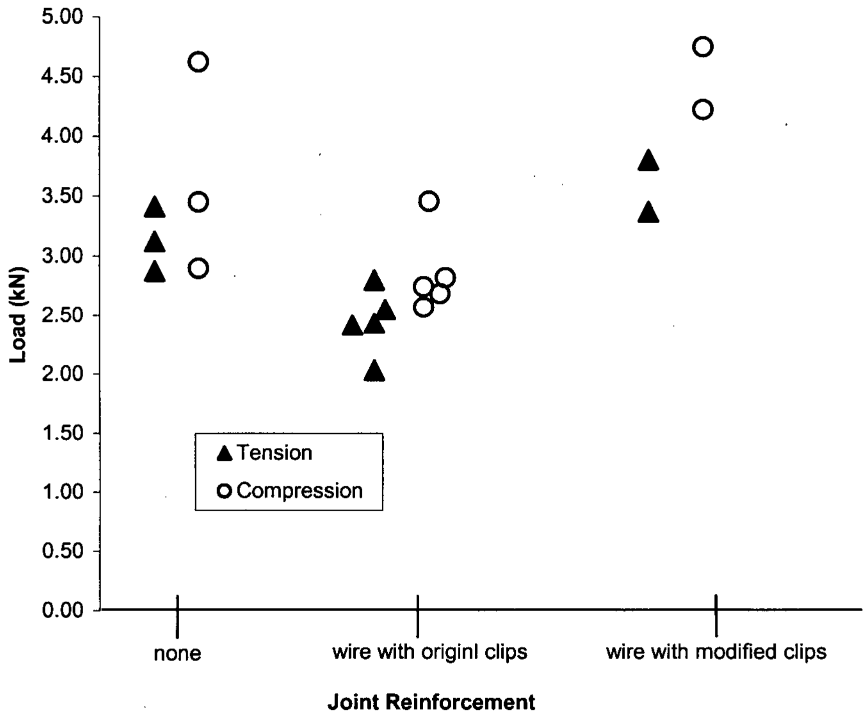


Fig. 4.17 – Maximum loads for the main study from Phase II and Phase III specimens (surcharge load 4.2 kPa).

The middle data points show that with the original clipped horizontal joint reinforcement there was a reduction on the embedment strength on both tension and compression. This was also described on Fig. 4.16 for the low surcharge case. While for the Phase III specimens, the modification made to the clip increases the peak embedment strength, as it is shown in Fig. 4.17 by the right side data points. The peak embedment strength is increased in magnitude to the same level as the tie only specimen, for both compression and tension. A tighter connection between tie and horizontal joint reinforcement significantly influenced the peak embedment strength. The strength in compression was also higher than in tension for the tie with modified clipped wire joint reinforcement.

4.6.3 Influence of Mechanical Connection of Clips

The mechanical connection between tie and horizontal wire joint reinforcement has proven to be an important factor in the performance of the brick veneer tie under reversed cyclic loadings. This connection is an important aspect that is being evaluated in this experimental study, which is the embedment strength at large displacement. The connection, in this case was the lateral clips, allowed the tie to engage the horizontal wire to take advantage of the resistance from the joint reinforcement, thus greater resistance in embedment strength was achieved at large displacement. All of this was observed from the Phase II tests.

A weakness of the design of the original clips for this type of tie (V tie) and horizontal wire joint reinforcement was also discovered. There is a tendency of the clips to detach or disengage after several cycles of loadings; this was also observed during and after test, when visual observations were conducted. The tie wire and the horizontal wire joint reinforcement were allowed to override each other and eventually became unclipped. The gap or slack between the tie wire and the horizontal wire joint reinforcement when they were installed was the potential cause of this problem.

To remedy this situation, the clips were modified to provide secure tight connection by closing the gap in the clips, thus the two materials can only be inserted from the side (see Fig 3.19 in Chapter 3). With this modification it was expected that the clips would provide a better connection, engaging the wire joint reinforcement more effectively without becoming detached. The specimens with this kind of clips were tested, and load displacement envelopes in tension and compression were examined.

Figure 4.18 compares the envelopes from the third cycle loading for specimens with modified clips and original clips, with part (a) for tension envelope and part (b) for compression. The loading protocol applied was the same as the previous phase of test for the main study of this experimental program. For the tension envelope, the modified clips shows an improvement on the maximum load or peak load resistance, the envelopes shows an increase about a factor of 1.5 on the peak load, compare the long dashes line to the solid line. This was also described in previous Chapter, which comparing the peak loads on all main type specimens. There was just a small difference between the envelopes of the modified clips specimens and the original ones apart from the higher peak load. The embedment resistance in larger displacement for the modified clips specimens was at least the same with specimens with original clips.

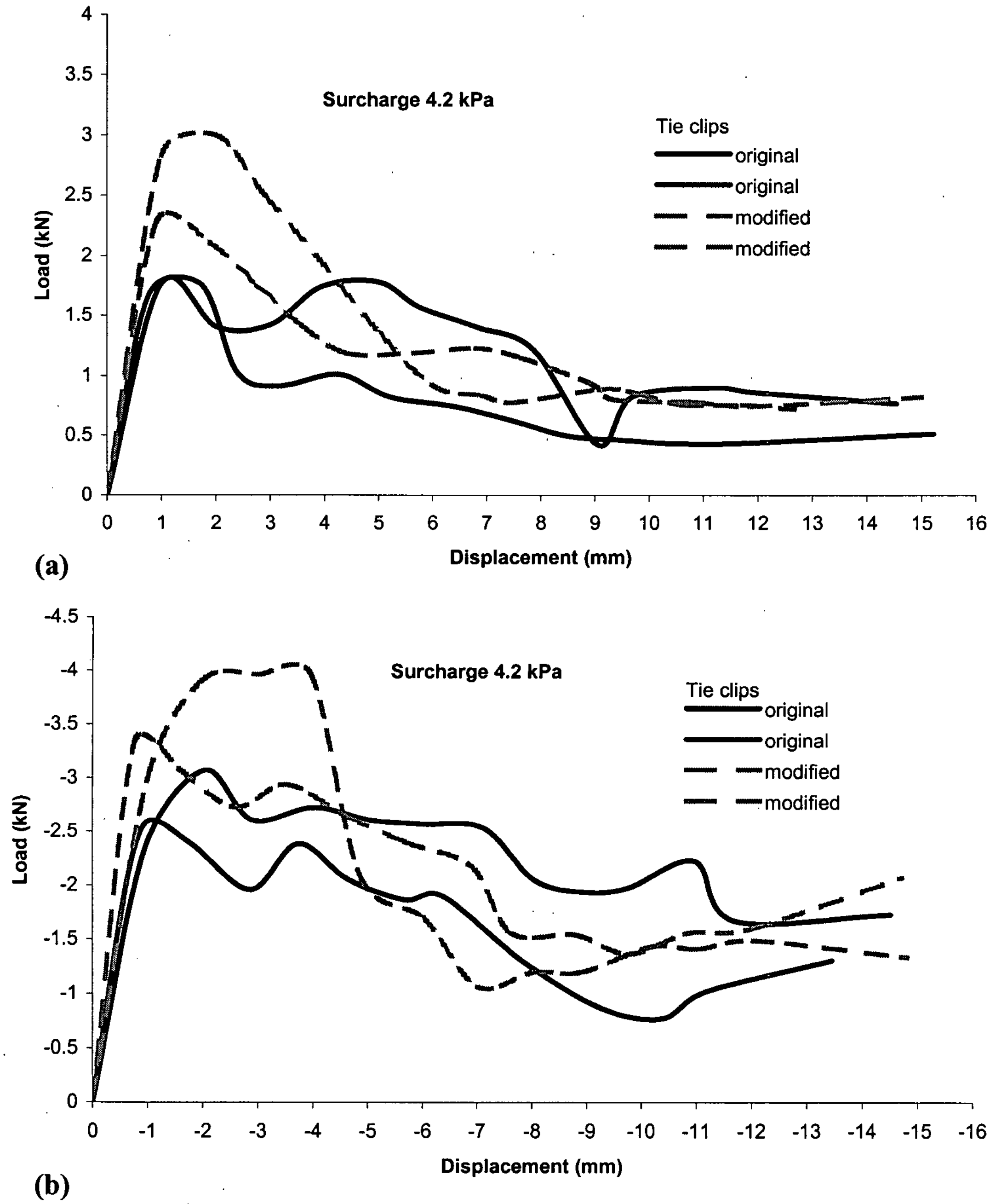


Fig. 4.18 – Effect of modified clips on the third cycle envelopes: (a) tension, (b) compression.

For compression envelopes, there was no significant difference between all the specimens, except for one specimen with modified clips, which achieved higher peak embedment resistance. The modification made to the clips helped to increase the maximum load on the embedment strength of the tie with horizontal wire joint reinforcement, and there were not any case of clips become detached or disengaged from the tie and horizontal wire joint reinforcement. This is an indication that even with tighter connection of the clips and no possibility of the tie and the wire being disengaged, the embedment resistance at larger displacement is not influenced significantly. Thus no improvement in embedment resistance at large displacement would be expected from lack of disengagement of tie with horizontal wire joint reinforcement.

4.6.4 Effect of Alternate Cyclic Loading History

As stated before, this section describes the effects of changing the loading history or loading protocol. The revised loading method intended to cycle below the peak load first, reach maximum load and finally do some number of cycles of loading with larger step size displacements, which means a reduce number of loading cycles.

To address the variability of the maximum loads obtained by using different loading history, the first cycle peak loads given in Table 4.2 and Table 4.3 will be compared in Figure 4.19 below. The parameter is the different loading protocol.

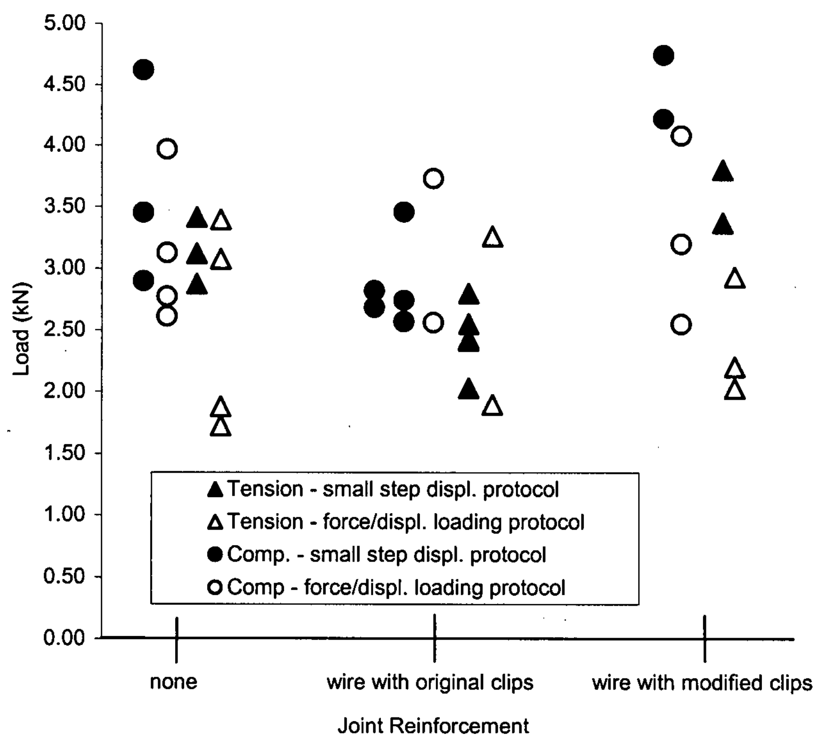


Fig. 4.19 – Comparison of maximum loads for the main type specimens, based on loading protocol.

Figure 4.19 indicated that the results for the tie without joint reinforcement were a little bit scattered with two data points for tension with the force/displacement loading protocol located below the rest of the data points. These were the specimens that did not perform very well after being repaired. But overall the embedment strengths were similar regardless the method of loading. With the right most data points (the tie with modified clips), the effect of loading history gave a slight reduction in the embedment strength for both tension and compression, this was due to the fact that before reaching the peak load, there were already several cycle of loading applied to the specimen.

For the specimens tested with the small displacement loading history (solid markers), there was a significant increase between the tie with joint reinforcement using original clips and the tie using modified clips, as was shown before in Figure 4.17. This is valid for both tension and compression of the embedment strengths. There is a possibility that this is due to the fact that with the small displacement loading history, the specimens do not undergo cyclic loadings below their peak load.

To further evaluate the effect of loading history to the overall behaviour of tie with wire reinforcement, the load displacement envelopes of the Phase III specimens were compared. Figure 4.20 shows the comparison for tension and compression envelope for specimens with tie and horizontal joint reinforcement with original clips. It shows that the force/displacement loading protocol follows the same trend as the small step displacement loading protocol, for both tension and compression. The peak loads were in the same range of value between the two protocols and the degradation of the envelopes was very similar, except that for the revised protocol (the force/displacement loading protocol) it was much more smoother because of the effect of reduced cycles of loadings by using larger step size displacement.

For specimens with tie and horizontal wire joint reinforcement with modified clips, Figure 4.21 shows the comparison for the tension and compression envelope. Again there was a similar trend between the two protocols both in tension and compression, with similar peak loads in tension and not a big difference in compression.

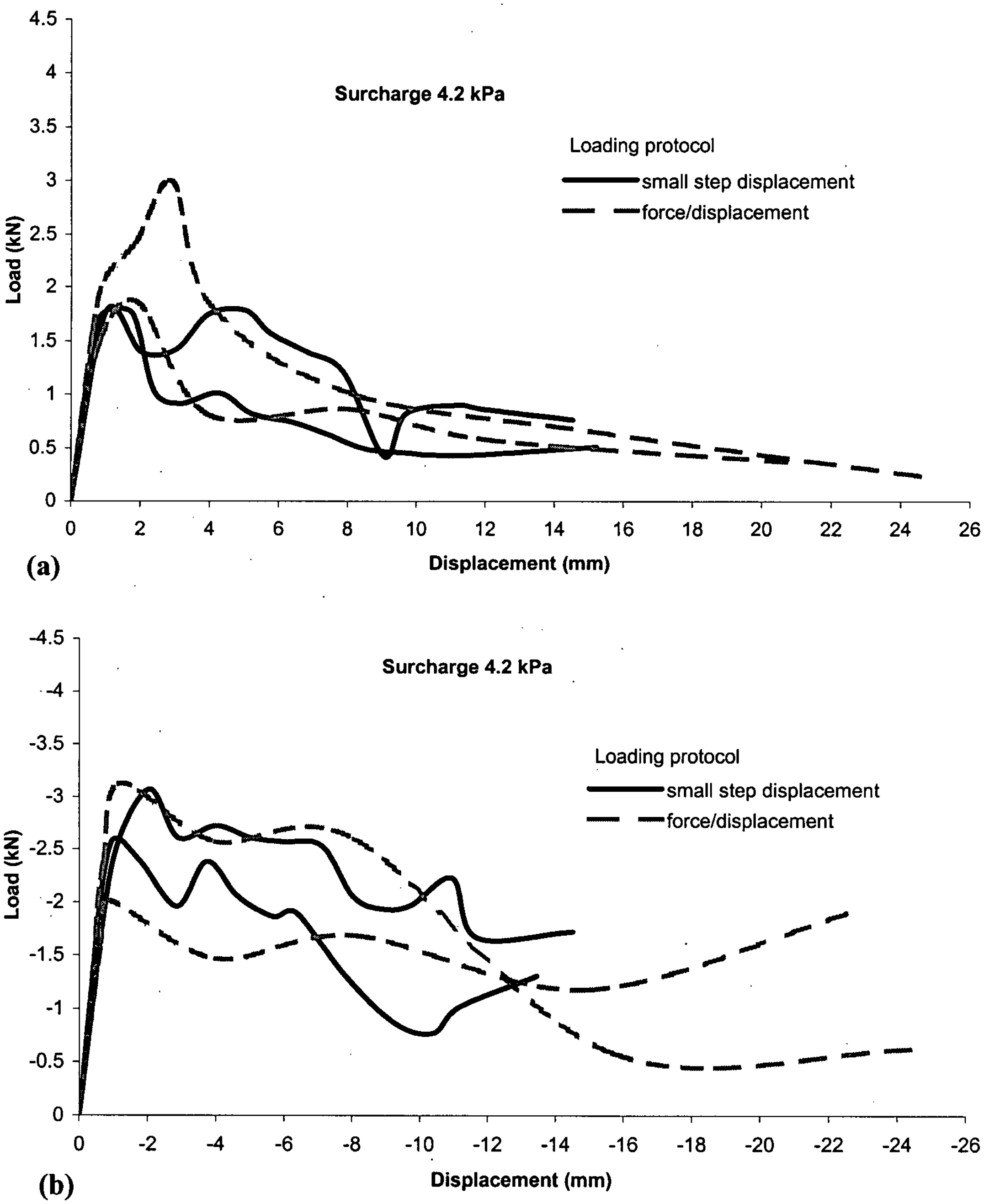


Fig. 4.20 – Influence of loading protocol on the third cycle envelopes for specimens with original clips: (a) tension, (b) compression.

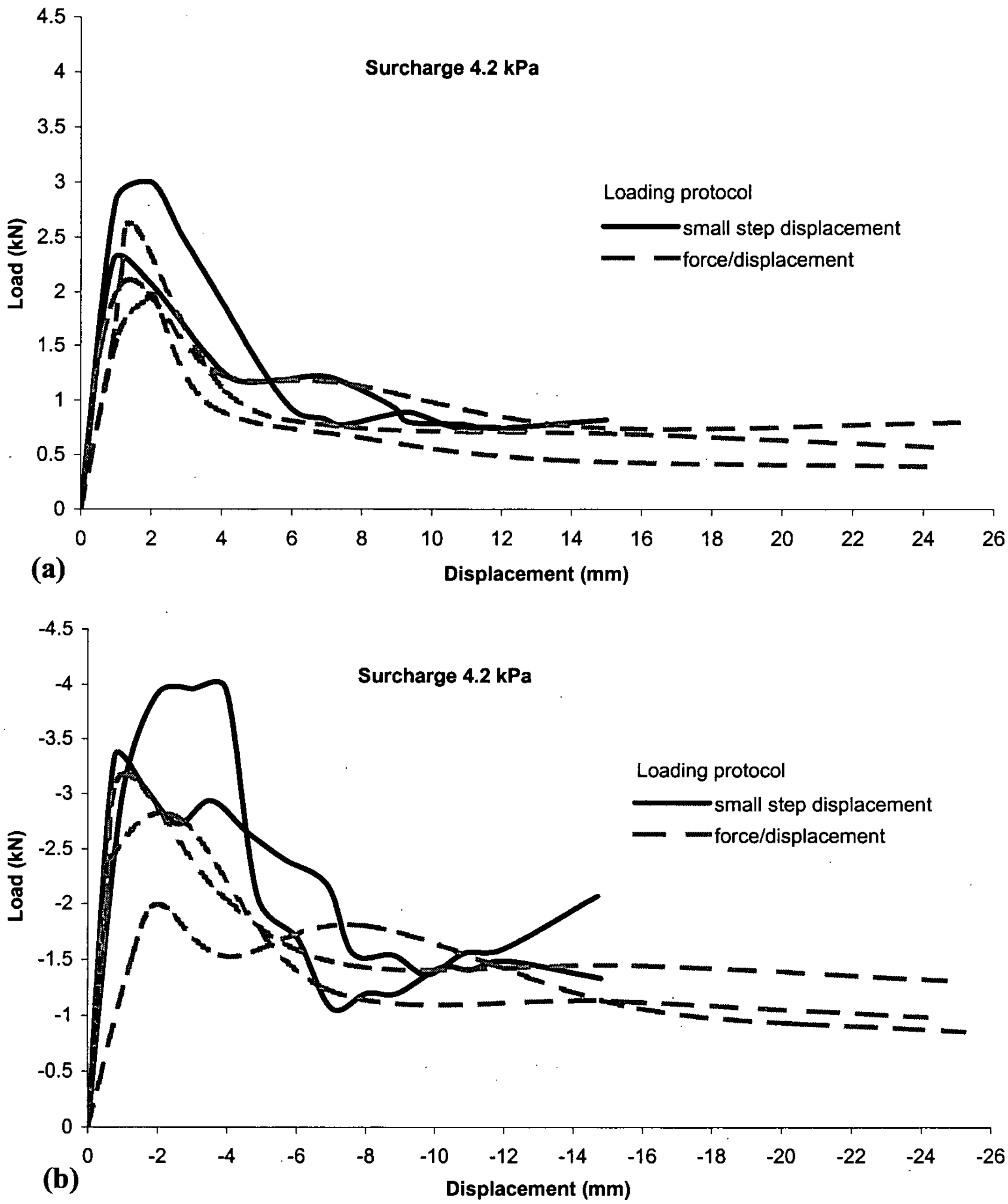


Fig. 4.21 – Influence of loading protocol on the third cycle envelopes for specimens with modified clips: (a) tension, (b) compression.

The two figures (Fig. 4.20 and Fig. 4.21) indicate that the effect of changing the loading history or loading protocol only has a small influence on the performance of the embedment strength of the brick tie with horizontal wire joint reinforcement. Overall the response of the system was quite similar for the load displacement envelope for both cases of specimens (original clips and modified clips).

Chapter 5 Conclusions & Design Recommendations

5.1 Conclusions from the Experimental Study

This experimental study examined the effect of joint reinforcement on the embedment strength of brick ties subjected to simulated seismic loading. The embedment failure is one link in a series of possible failure mechanisms of a tie system, which include metal failure, buckling of tie and fastener failure. The experimental study focused on the results of 18 specimens and an additional 19 specimens on the embedded wire portion of a two-piece adjustable tie commonly used in Western Canada. While several additional tests on other types of tie was also conducted to provide a general description on the behaviour of this type of tie. In addition, the effect of misplacing the tie within the acceptable tolerance was also studied.

The two important aspects considered in the behaviour of veneer wall ties in this experimental study are the peak or maximum load and the embedment strength at large displacement. The following conclusions can be made on the embedment strength of these ties based on these two aspects.

1. Adding horizontal wire joint reinforcement will only significantly affect the embedment strength of the tie, if it is mechanically connected to the tie. There is no gain in the embedment strength if the horizontal wire joint reinforcement is not attached to the brick tie.
2. Peak embedment strength is affected by the tightness of the connection between the tie and the wire joint reinforcement. The lower peak embedment strength of the tie with clipped wire joint reinforcement can be described as the effect of the clips, which had a loose connection (due to the gap) to the horizontal wire joint reinforcement. When this situation was corrected by modifying the clips to have a secure and tight connection, the peak embedment strength showed a significant increase in tension and compression but it was only comparable to the tie without wire reinforcement at all. This is an indication that the detail of the attachment of the tie to the wire joint reinforcement is critical to the peak embedment strength.

3. The maximum loads observed in the current tests are of similar magnitude of other possible mechanisms of failure. Therefore the embedment strength is clearly an important factor in the capacity of brick tie subjected to reversed cyclic loadings.
4. The clips ensure that the tie engages the horizontal wire joint reinforcement in the mortar bed joint. By engaging the wire joint reinforcement, the area of mortar that gives embedment resistance will increase to some extent. This will in turn provide higher embedment resistance or capacity at larger displacement. However, the tightness of the connection does not actually influence the large displacement embedment resistance.
5. Brick ties with low surcharge (condition near top of a wall) will be the most critical considering that these ties will have the highest demand load and lowest brick tie embedment resistance. The experimental study showed lower embedment strength for the low surcharge condition. With clipped (original design) horizontal wire joint reinforcement added, the peak embedment strength reduces, thus lowering the embedment resistance. However, the clipped wire joint reinforcement does increase the embedment resistance at large displacements in tension, but not significantly in compression.
6. For the high surcharge condition that occurs near the base of a one-story wall, the embedment strength is generally higher. With the addition of clipped wire joint reinforcement, increased embedment strength was achieved as expected. The clamping effect of the surcharge load made the clipped wire joint reinforcement to work more effectively.
7. The modes of failure for the tie consisted of two components. First, the failure of the bond between mortar and the brick where the tie is located because of the movement of the tie. Second, the failure of the tie material itself, however because the tie is much stiffer, this case only occurs for a high bond of mortar joint due to good workmanship or high clamping effect from surcharge load.
8. Based on small number of tests on US tie systems and conventional ties (triangular and corrugated strip tie), detail of connection between the brick tie and the joint reinforcement can be considered as the main component that will influence the embedment strength. The plate tie system showed poor performance due to the

looseness of the connection while the Fleming anchor achieved good embedment strength in tension despite its weakness in compression. With stiffer ties, the embedment strength depends more on the bond of the bricks and mortar joint. The area of mortar joint occupied by the ties also affected the bond of the mortar, as more material in the mortar joint may weaken the bond strength of the bed joint. This was the case for the plate type ties compared to the Fleming and conventional ones.

For flexible ties such as the corrugated strip ties, compression failure always governed by buckling of ties. While in cyclic loadings the flexible ties will fail in fatigue because of the repeated loadings.

9. It was proven from the limited number of tests that the location of the embedment of the tie really affects its performance. Loss of embedment strength might occur if the tie is not embedded properly as indicated by the code, which is ± 13 mm from the centreline of the brick. A slight increase in embedment strength is achieved by adding clipped horizontal wire joint reinforcement.

5.2 Design Recommendations

This section will address the design of a wall tie based on the embedment capacity as evaluated in the experimental study. It does not take into account the type of backup wall, sheathing, or the type of fasteners. The recommendations will only act as a guide to achieve better anchorage performance of veneer wall ties using the horizontal wire joint reinforcement.

Mechanical anchorage of brick ties to embedded items in the bed joint is needed to provide a positive anchorage and to maintain their strength after the masonry has cracked. The function of horizontal wire joint reinforcement from this approach is to maintain the anchorage after cracks occurred. Therefore the connection between the brick tie and the horizontal wire joint reinforcement becomes crucial.

Several key points that have to be taken into account when considering adequate anchorage of the tie with horizontal wire joint reinforcement are:

1. The embedment location of the brick ties should be within the middle third of the brick unit. The more offset the location of embedment of the tie from the centreline of the brick, the less the embedment strength will be.

2. The size of ties should consider the area of mortar in the bed joint that will be disturbed, as this affect the bond strength between the bricks. Adequate stiffness of the brick ties will permit the tie to deform and redistribute the load to the adjacent tie before it overloads and fail.
3. The connection system for the tie and the horizontal wire joint reinforcement should allow the tie to engage the horizontal wire reinforcement in compression and tension.
4. In this experimental study, clips as a device to maintain the connection of ties to the wire joint reinforcement allowed the tie to override the wire joint reinforcement and finally become disengaged. Therefore, if clips are used, they should have a tight connection to enclose the brick tie and the horizontal wire joint reinforcement.
5. The bond of the mortar joint and the brick unit is a very important factor in the embedment strength or embedment capacity of the ties, therefore the more reliable the bond strength is the more embedment capacity will be achieved. Workmanship plays an important role in this case. Also the core of the brick unit could provide a mechanical bond between the bricks in flexure and shear. This is due to the core filling with mortar, thus locking the bricks together.

5.3 Recommendations for Future Research

The research program that was conducted on the performance of brick veneer ties with horizontal joint reinforcement covered several key issues regarding embedment strength. Future research should also consider the following items:

1. More specimens representing the US specific seismic tie systems, to confirmed the behaviour of these types of ties. This includes modifying the test apparatus so it could accommodate the Dur-o-wal tie system without modifying it as described in Chapter 3, in order to obtain the real behaviour of the tie.
2. Effect of the backup wall on the overall system, by including components of a backup wall in the specimens if applicable. The backup wall should be varied between rigid backup (concrete masonry unit) and flexible backup (steel stud and wood backing).
3. An experimental study on full-scale veneer wall specimens attached to a backup wall with several ties supporting the veneer under a simulated earthquake loading, which can be accomplished by using a shake table. There should be a variation of the

specimens to include tie only specimens and tie with horizontal wire joint reinforcement, which was the approach on the experimental research that had been conducted. Also it will also be useful to vary the spacing of ties, in order to see the effect of horizontal joint reinforcement in providing positive anchorage for the tie. This experimental study will provide more in-depth observations on the real performance of the brick veneer tie with horizontal wire joint reinforcement, not only testing the embedment strength of the tie wire but also the second opinion, that the use of horizontal wire joint reinforcement maintains the integrity of the veneer wall after cracks occur. Also the distribution of tie forces along the veneer wall can be examined, for un-cracked and post-cracked condition. Investigation on the effect of horizontal wire joint reinforcement to the distribution of tie forces on a wall can also be analyzed.

4. Analytical study of the behaviour of the tie with horizontal wire joint reinforcement by modelling the system can provide important result in order to predict the behaviour of the veneer wall with the system installed. The result from this analytical study can then be compared to the results from the experimental full-scale veneer wall test. This will lead to a real design guide of the system, which then can be incorporated into the code.

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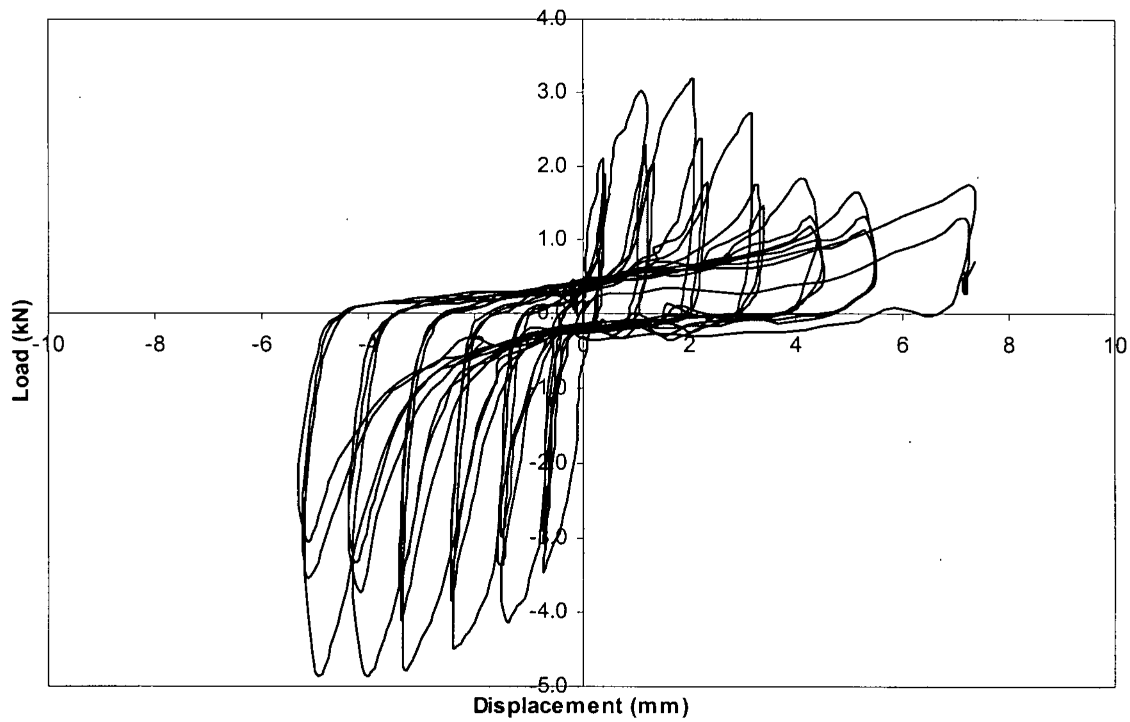
APPENDIX A

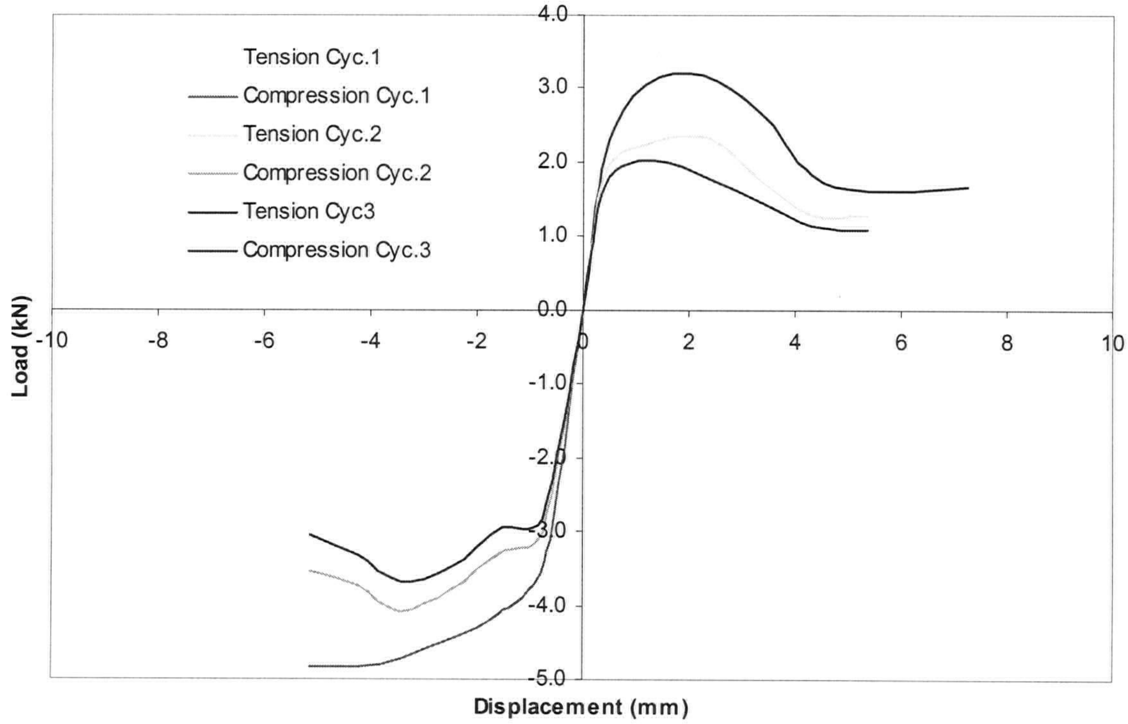
Experimental Data

Specimen	PL2
Characteristics	V-Tie only
	V-Tie length 60 mm
	Type N mortar
Test Date (age)	November 17th, 2000 (140 days)
Surcharge Load	60 kPa
Maximum Force	
Tension	3.17 kN
Compression	4.84 kN
Displacement at Maximum Force	
Tension	2.09 mm
Compression	4.98 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship

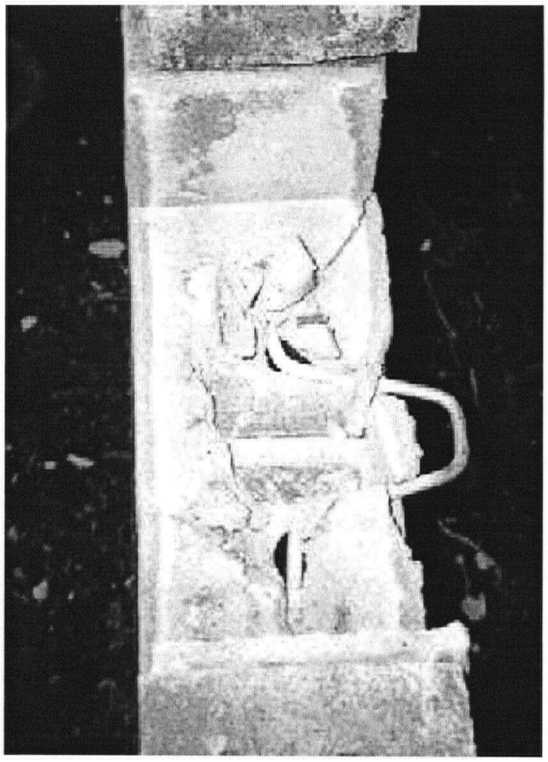


Load-Displacement Envelope Curve



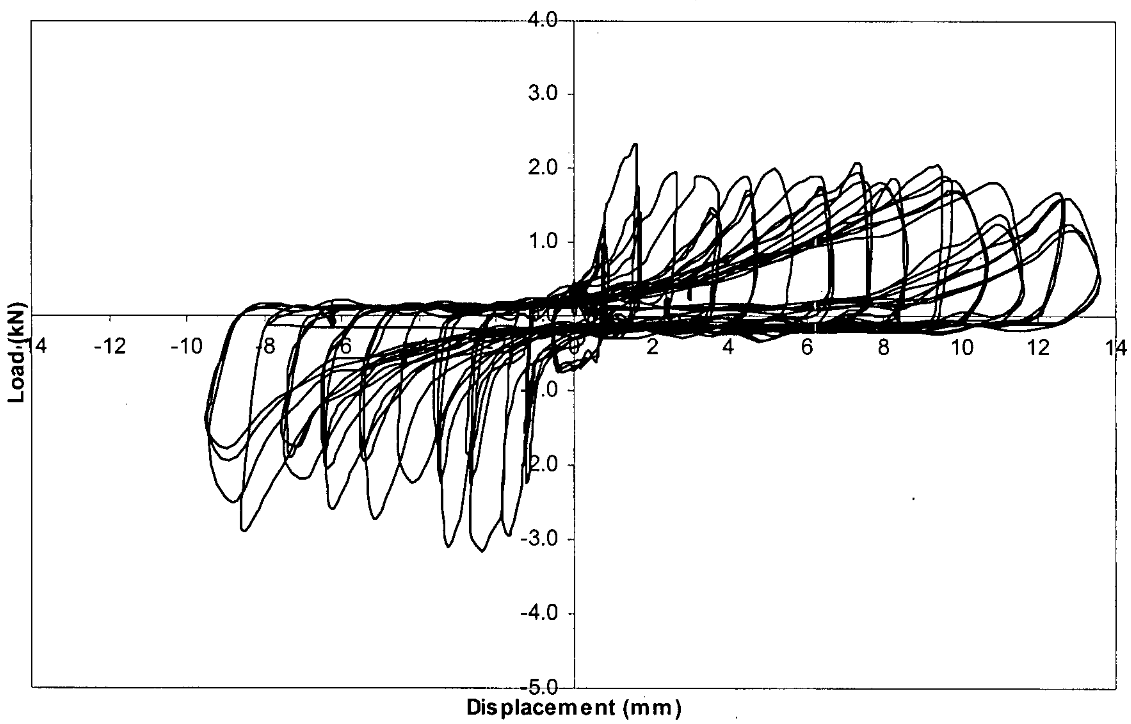
Description of Test Observations:

- Test was done in a continuous way with every stage consisted of 3 cycle of loadings.
- The observations were done after the test was stopped.
- It was observed that the loading guide was not capable to allow the movement of the rod to larger displacement, due to the limited length of the case. Another problem was the length of the tie, which only has a little space to the brick panel. This caused the clamping device to collides with the brick after the tie deformed.
- There were cracks formed and pull-out of crushed mortars from the bed joint at the tie face (tension side). While at the tooled joint face (compression side), push-through of mortars was the failure mode.

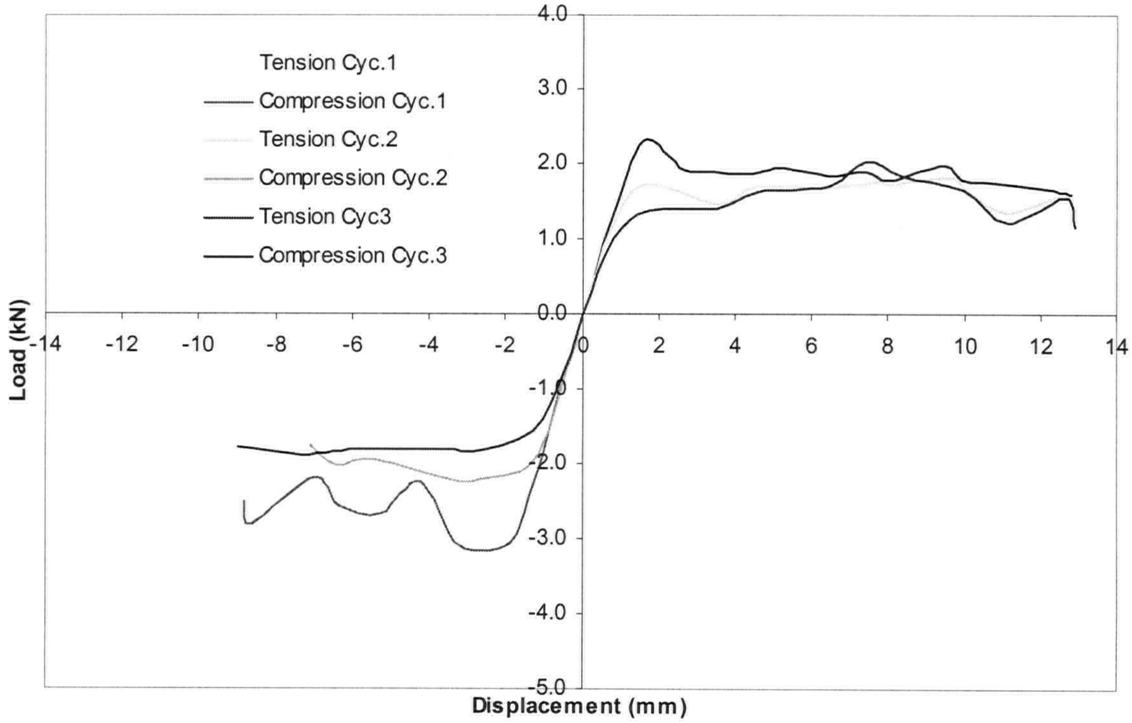


Specimen	PL3
Characteristics	V-Tie only
	V-Tie length 60 mm
	Type N mortar
Test Date (age)	November 22nd, 2000 (145 days)
Surcharge Load	13 kPa
Maximum Force	
Tension	2.29 kN
Compression	3.17 kN
Displacement at Maximum Force	
Tension	1.59 mm
Compression	2.39 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship

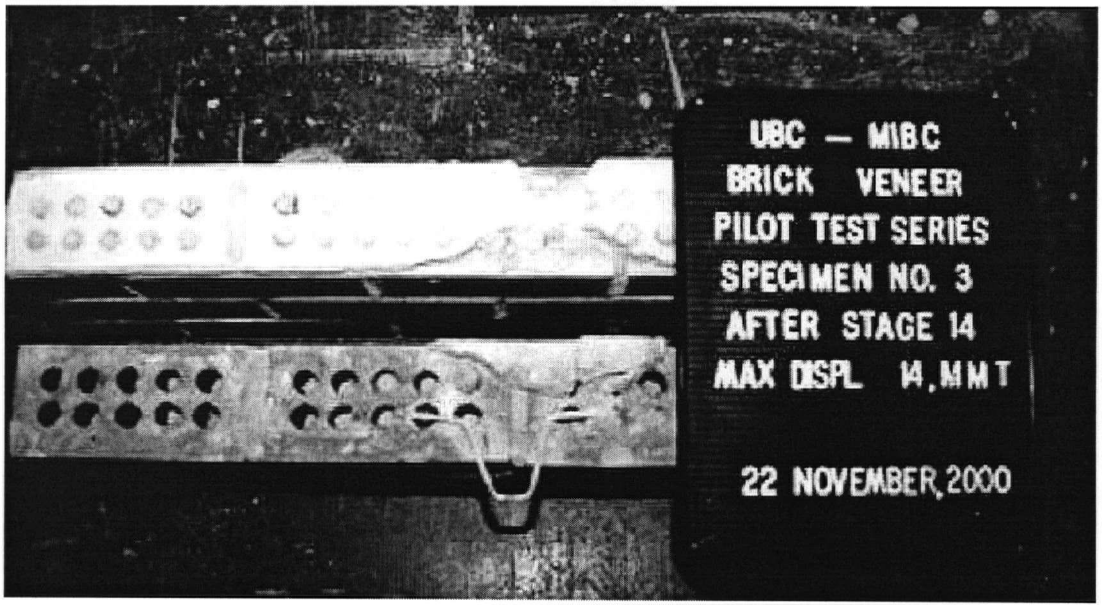


Load-Displacement Envelope Curve



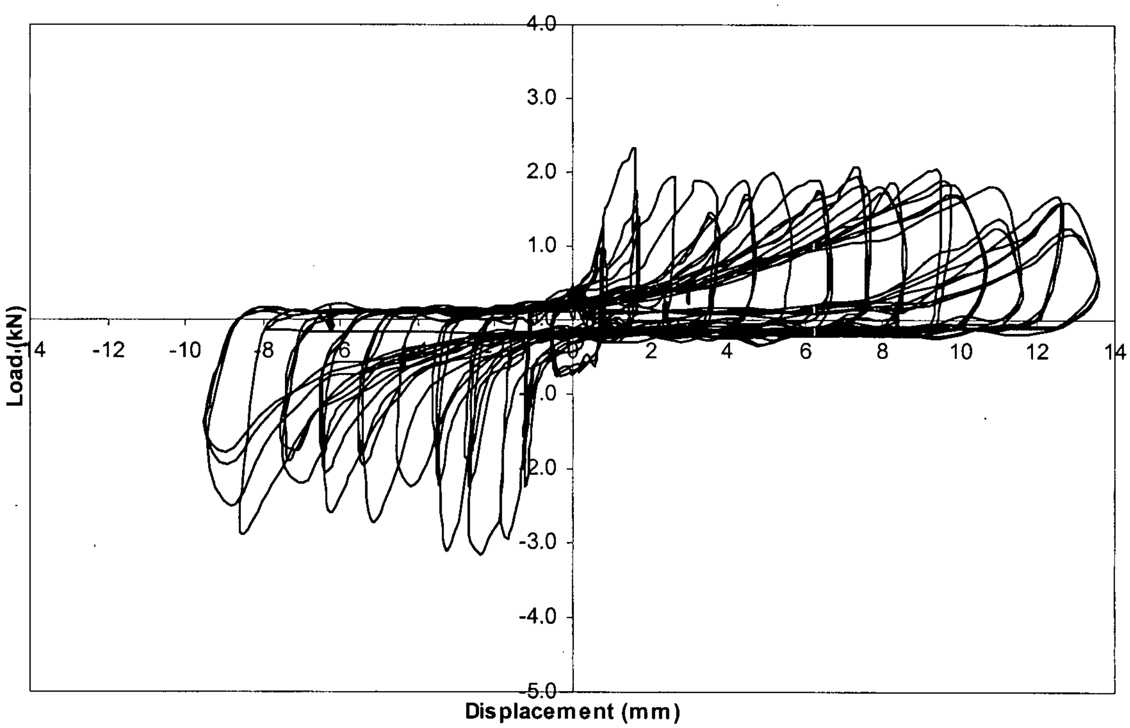
Description of Test Observations:

- Test was done in a continuous way with every stage consisted of 3 cycle of loadings.
- The observations were done after the test was stopped.
- It was observed that the loading guide was not capable to allow the movement of the rod to larger displacement level, due to the limited length of the case.
- Observations after the test stopped revealed that cracks occurred at the tie face (tension side) with some spalling of the crushed mortars from the bed joint. At tooled joint face (compression side), a push-through failure of the mortar bed joint was evident.

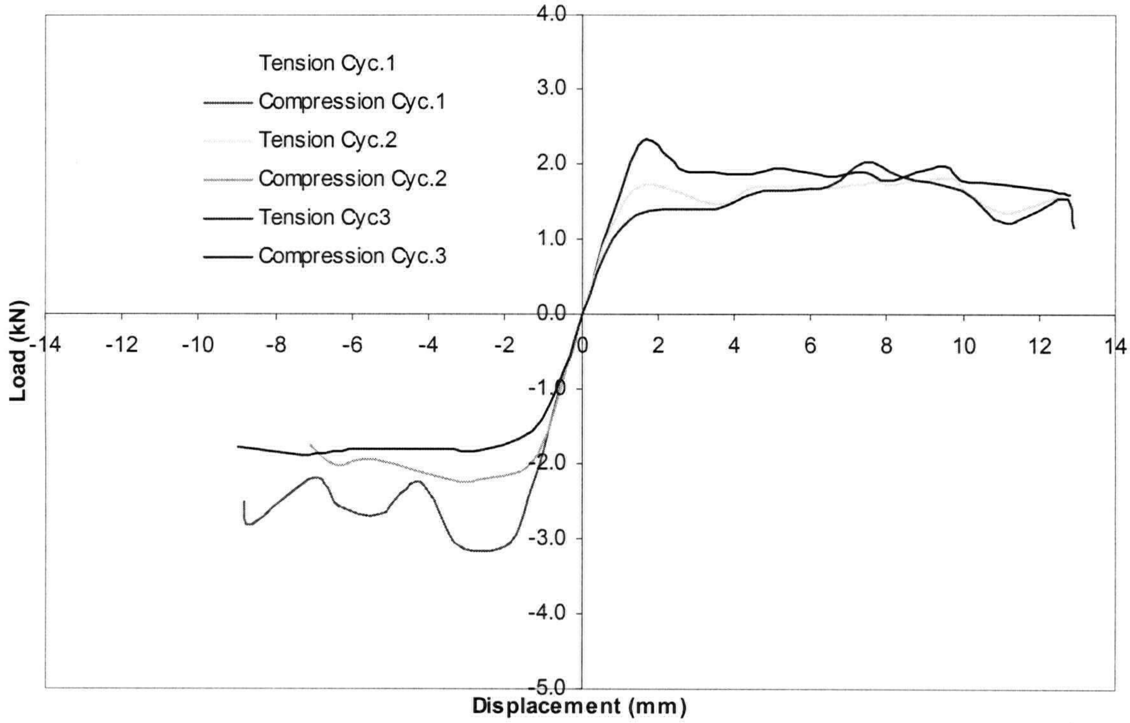

UBC — MIBC
BRICK VENEER
PILOT TEST SERIES
SPECIMEN NO. 3
AFTER STAGE 14
MAX DISPL 14.MMT
22 NOVEMBER,2000

Specimen	PL4
Characteristics	V-Tie with horizontal wire reinforcement unclipped V-Tie length 60 mm Type N mortar
Test Date (age)	November 21st, 2000 (144 days)
Surcharge Load	60 kPa
Maximum Force	
Tension	2.29 kN
Compression	3.17 kN
Displacement at Maximum Force	
Tension	1.59 mm
Compression	2.39 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-out through mortar bed joint

Load-Displacement Relationship

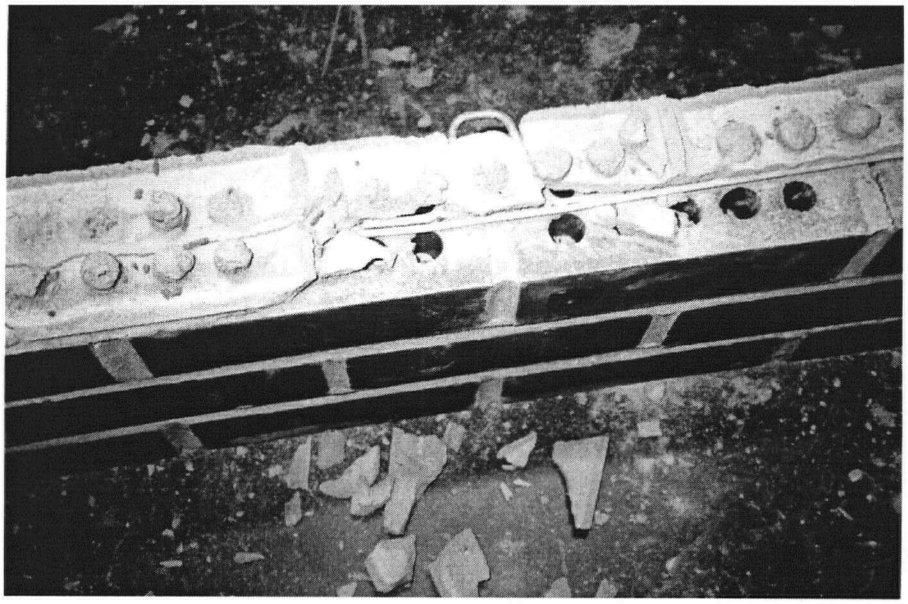


Load-Displacement Envelope Curve



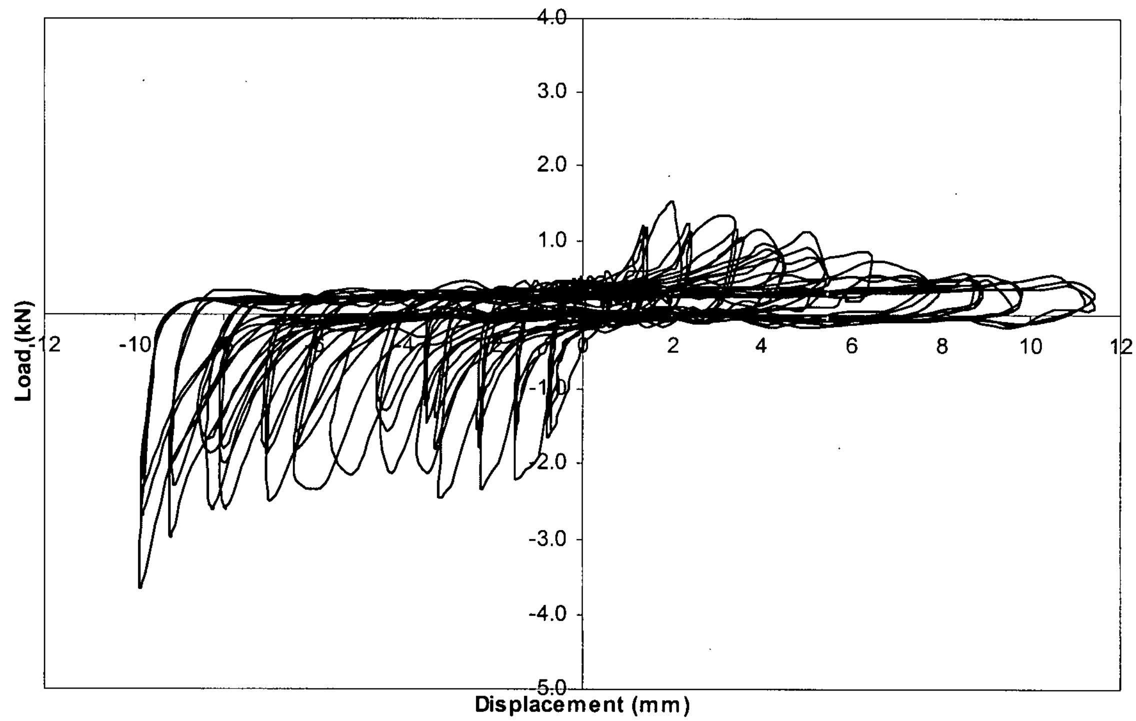
Description of Test Observations:

- Test was done in a continuous way with every stage consisted of 3 cycle of loadings.
- The observations were done after the test was stopped.
- Due to the length of the loading guide, test was stopped at certain displacement in which the limit of the displacement was reached.
- Observations indicated that mortars were pulled out and spalled from the bed joint at the tie face (tension side). At the tooled joint face (compression side), extended area of crushed mortars was being pushed through from the bed joint. When the bed joint with the embedded tie was open, it showed that the horizontal wire joint reinforcement somehow had split the mortar bed into two parts, one in tension side and the other in compression.

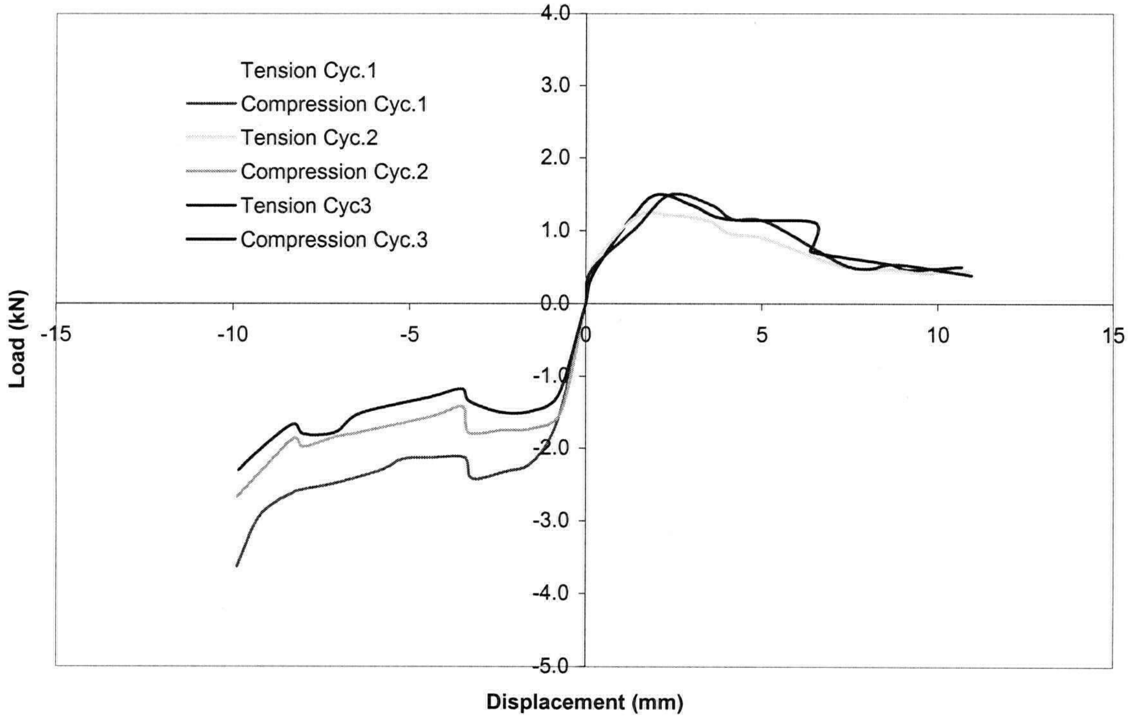


Specimen	PL5
Characteristics	V-Tie with horizontal wire reinforcement unclipped
	V-Tie length 60 mm
	Type N mortar
Test Date (age)	November 23rd, 2000 (146 days)
Surcharge Load	13 kPa
Maximum Force	
Tension	1.50 kN
Compression	2.42 kN
Displacement at Maximum Force	
Tension	2.03 mm
Compression	3.23 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-out through mortar bed joint

Load-Displacement Relationship

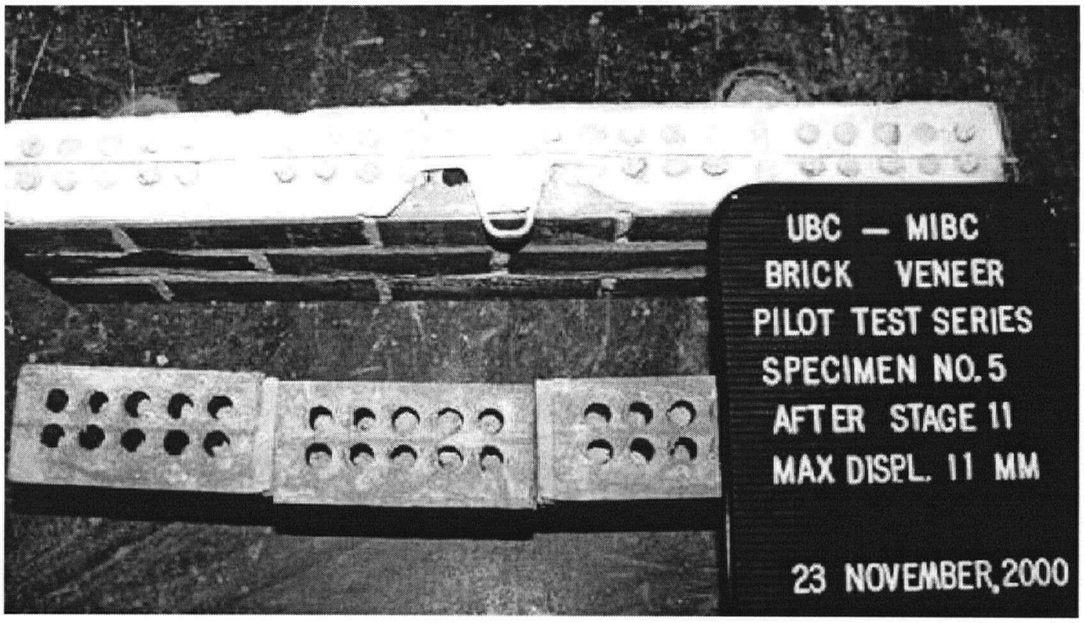


Load-Displacement Envelope Curve



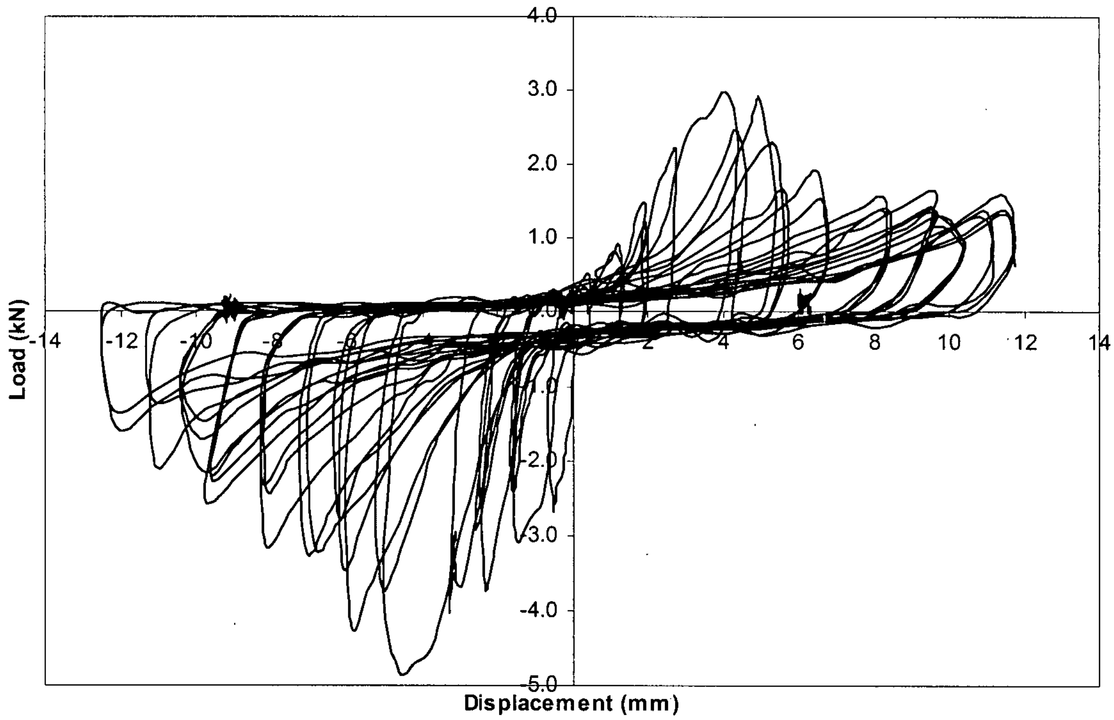
Description of Test Observations:

- Test was done in a continuous way with every stage consisted of 3 cycle of loadings.
- The observations were done after the test was stopped.
- The specimen already suffered some cracks before the test was conducted due to the failure in handling of the specimen (it was very brittle).
- Limited length of the loading guide forced the test to be stopped earlier.
- From the observations after the test was stopped, an indication of pull-out mortars that were crushed by the movement of the tie was occurred at the tie face or tension side. While in compression side at the tooled joint face, push-through of crushed mortars from the bed joint was also apparent.

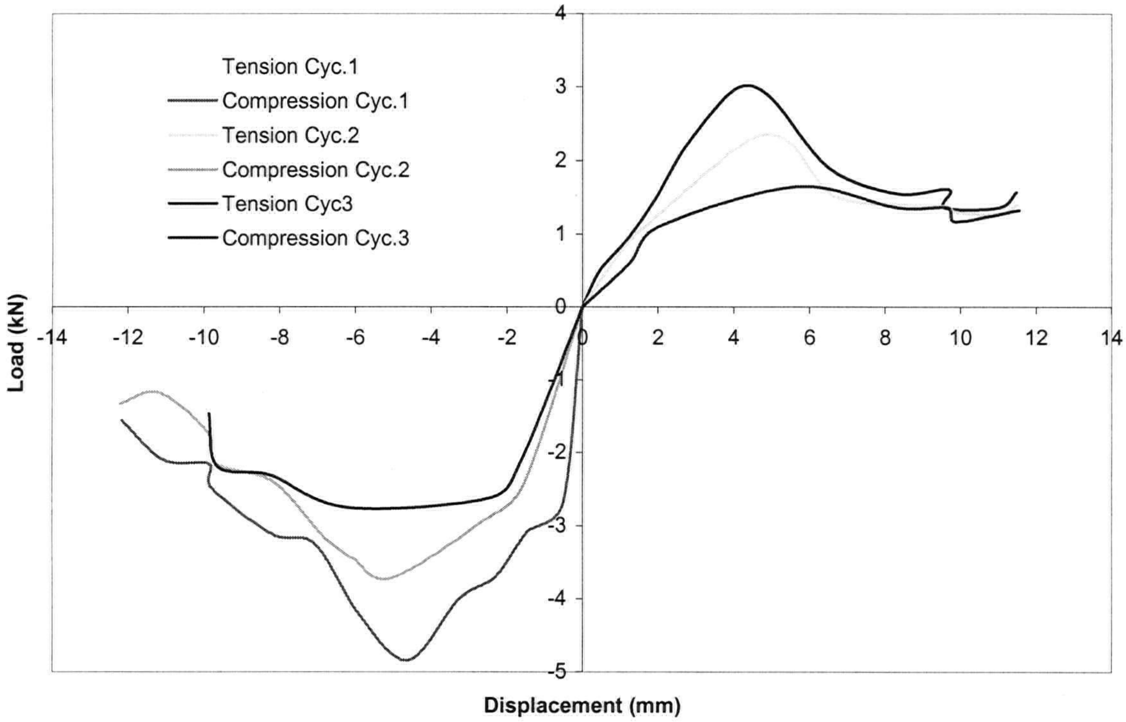

UBC — MIBC
BRICK VENEER
PILOT TEST SERIES
SPECIMEN NO. 5
AFTER STAGE 11
MAX DISPL. 11 MM
23 NOVEMBER, 2000

Specimen	PL6
Characteristics	V-Tie with horizontal wire reinforcement clipped V-Tie length 60 mm Type N mortar
Test Date (age)	November 24th, 2000 (147 days)
Surcharge Load	60 kPa
Maximum Force	
Tension	2.97 kN
Compression	4.85 kN
Displacement at Maximum Force	
Tension	4.06 mm
Compression	4.61 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-out through mortar bed joint

Load-Displacement Relationship



Load-Displacement Envelope Curve

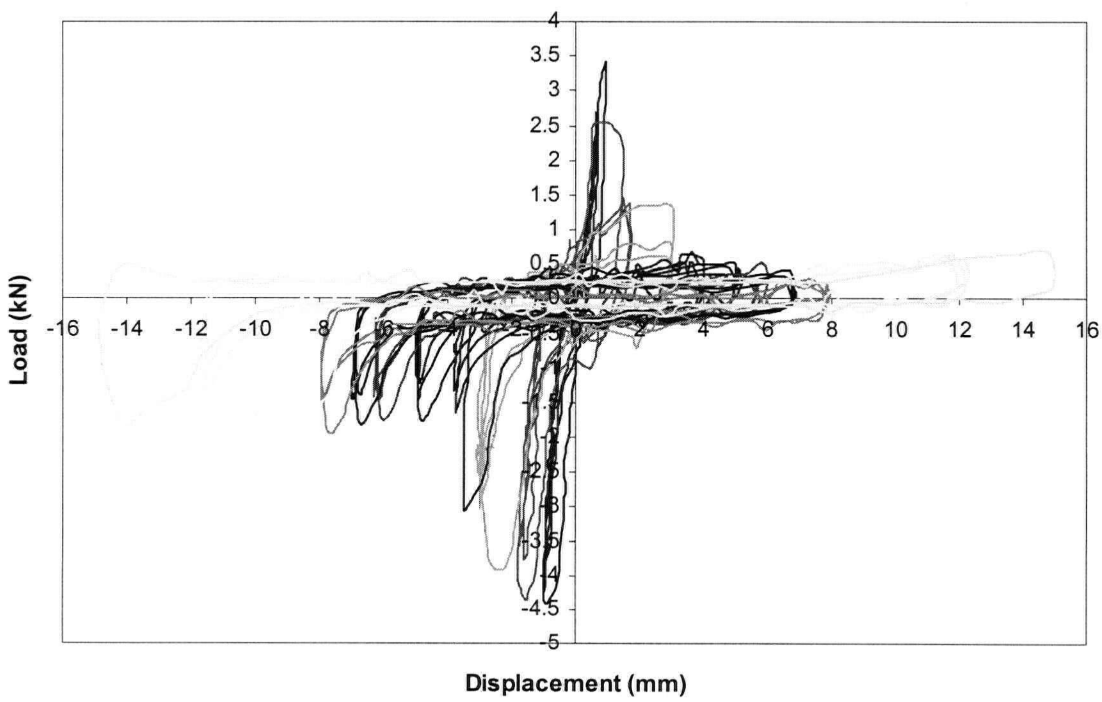


Description of Test Observations:

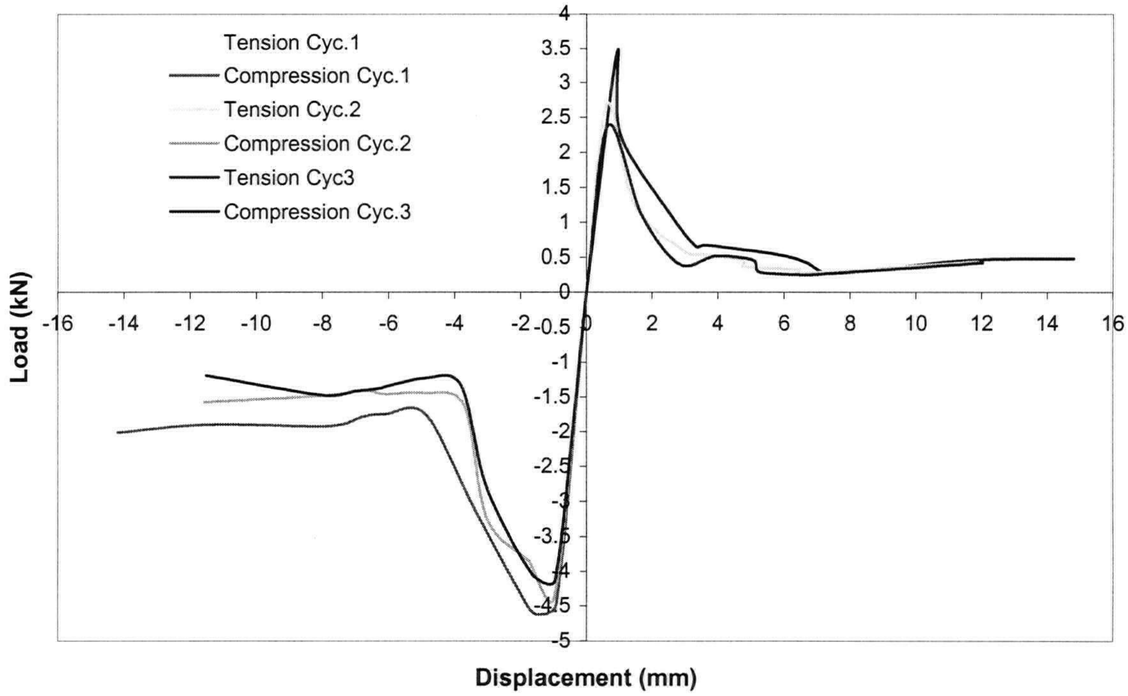
- Test was done in a continuous way with every stage consisted of 3 cycle of loadings.
- The observations were done after the test was stopped.
- Loading guide limits the movement of the rod, which was connected to the tie and the actuator. This resulted in a limited target displacement level.
- Observations indicated at the tie face in tension side, more area of mortars were crushed and pulled out (spalled) from the bed joint where the tie with the clipped wire reinforcement located. Similar condition occurred in compression side, where more crushed mortars were pushed through from the bed joint.
- Opening up the bed joint where the embedment of the tie with clipped horizontal wire reinforcement located indicated that clips were still attached and wire was also slightly deformed. This was an indication that the tie had engaged the wire at the loading cycle.

Specimen	T1
Characteristics	V-Tie only embedded at the centre, length 80 mm Type S mortar
Test Date (age)	February 5th, 2001 (47 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	3.41 kN
Compression	4.62 kN
Displacement at Maximum Force	
Tension	0.92 mm
Compression	1.56 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship



Load-Displacement Envelope Curve

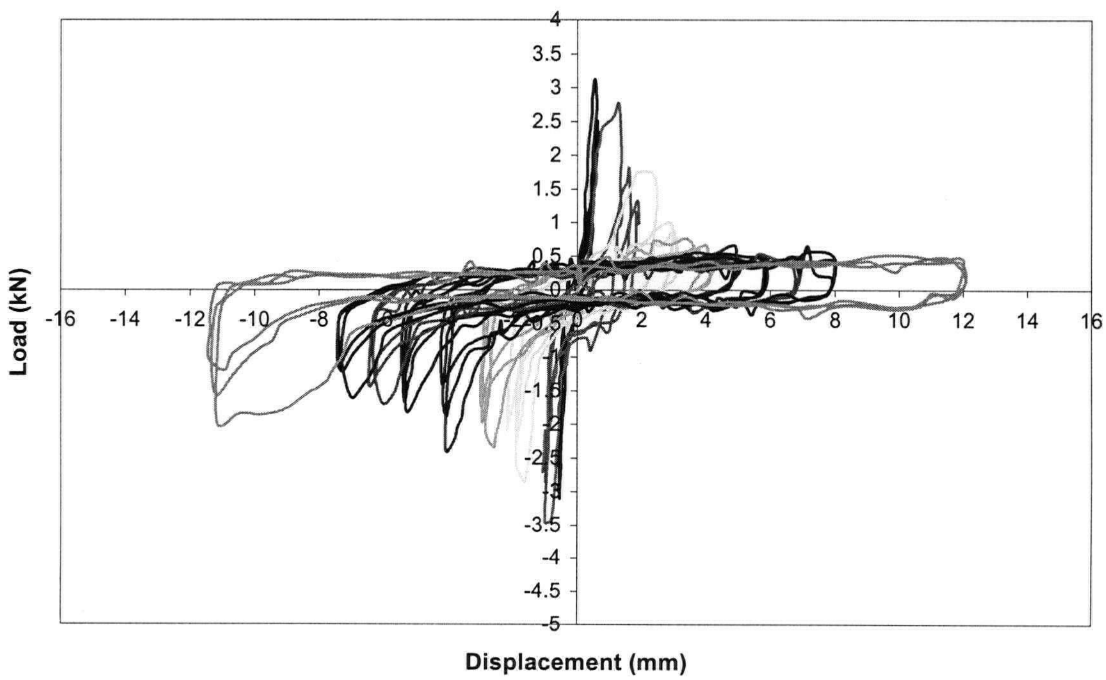


Description of test observations:

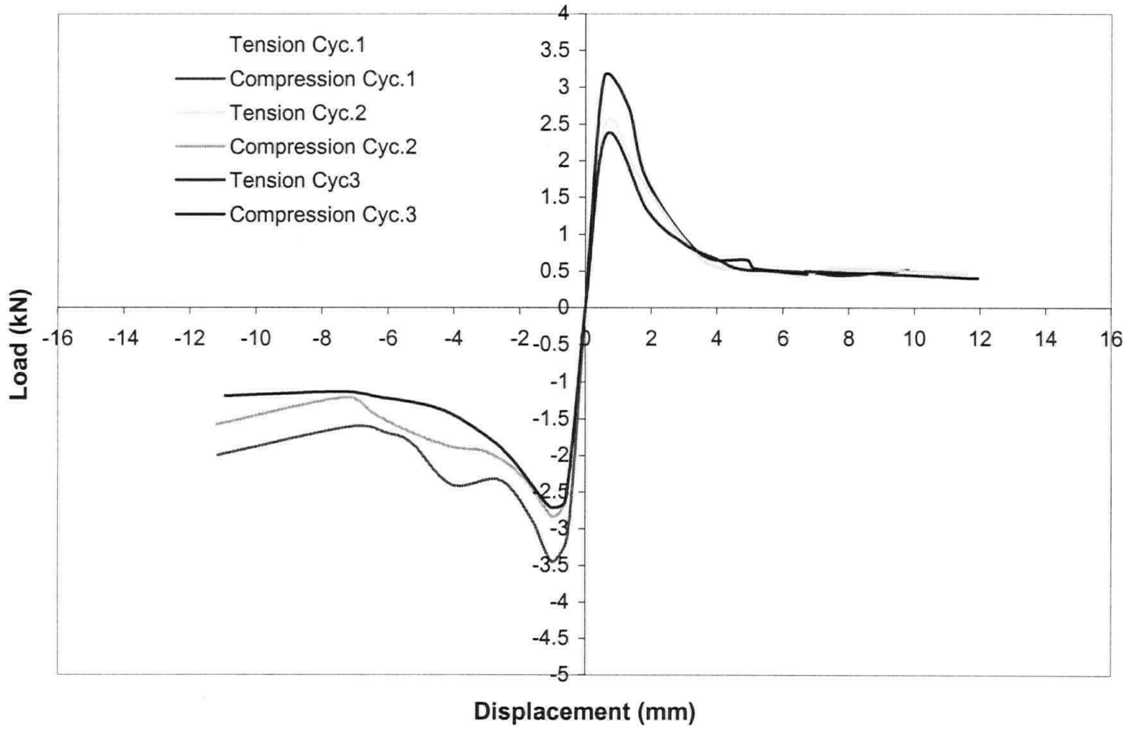
- At 2nd stage (i.e. 2 mm displacement) second cycle, the cracks were formed on the mortar bed joint on location of the tie (at tie face). There were still no mortars spalling occurred.
- At stage 4 with 4 mm displacement, the cracks became bigger, and in 2nd cycle tension side there was a portion of crushed mortars that spalled from the tie side, while in the tooled joint face, several cracks had been formed.
- At stage 6 (6mm displacement), part of the crushed mortars in the bed joint were pull-out at the tension side (at the tie face) and spalled, while in the tooled joint face, there were 4 separate pieces of crushed mortars formed with some pieces already push-through.
- At 9 mm displacement (stage 9), with each cycle, both on the tension side and compression side, every piece of crushed mortars spalled, and can be taken out easily from the bed joint. But there was still some resistance due to mortars that interlocked with the core of bricks.
- Test was stopped when displacement reached 15 mm, as it was considered an excessive displacement for the tie with a very low resistance from the tie.
- Opening up the bed joint did a further study of the area of mortars being crushed and the deformation of the tie.
- The observations showed that the mortars in the area of the leg of the V-Tie being crushed and both legs of the V-tie were bent in the weak joint of the body of the tie (the V part) and the leg in the tension direction.

Specimen	T2
Characteristics	V-Tie only embedded at the centre, length 80 mm Type S mortar
Test Date (age)	February 14th, 2001 (54 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	3.12 kN
Compression	3.45 kN
Displacement at Maximum Force	
Tension	0.55 mm
Compression	1.00 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship



Load-Displacement Envelope Curve

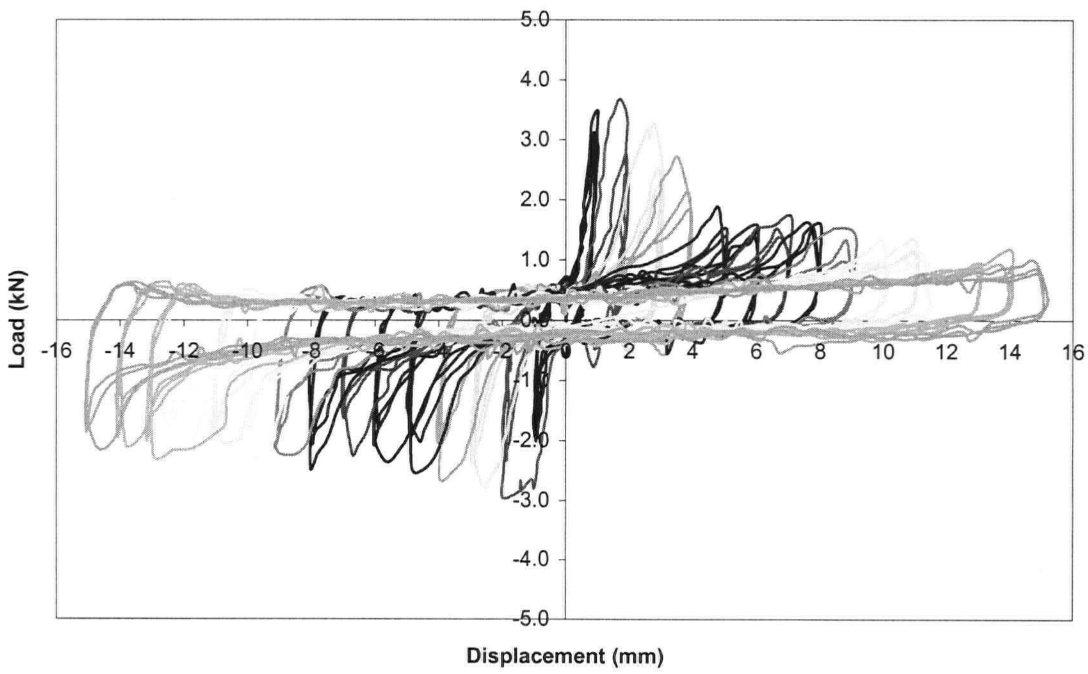


Description of test observations:

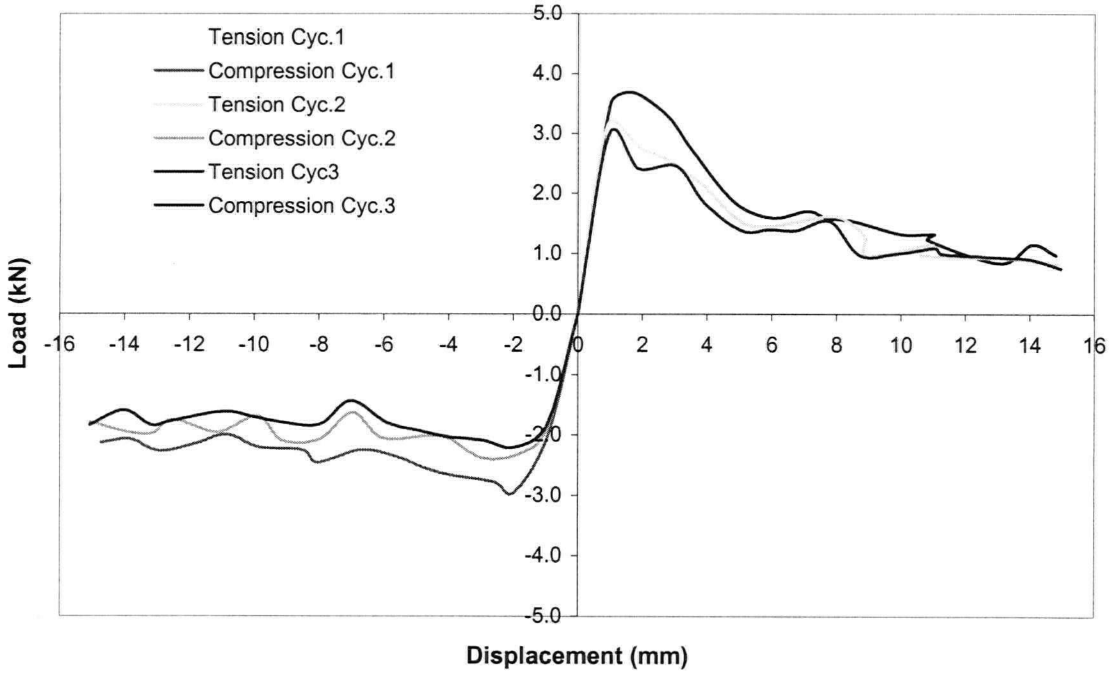
- At 2 mm displacement stage 2, on 3rd cycle in tension pull-out, cracks were formed near area of the tie, while in compression some small cracks were formed.
- Crushed mortars from the bed joint spalled in tension side at stage 5, cycle 1 with 5 mm displacement. On the tooled joint face while in compression, push-through of crushed mortars on the centre of bed joint were apparent.
- After stage 8 with 8 mm displacement, the test was continued directly to 12 mm displacement, this was done due to the observed mortars that already spalling out from the tie face and the indication that there was very little resistance in tension after 7 mm, and this was evident from the hysteresis curve. While in compression there was also some spalling occur and cracks were formed that divided the bed joint into several pieces of crushed mortars.
- At 12 mm displacement almost all the crushed mortar pieces on the tie face was spalled and it can be seen that only some resistance from the interlocking mortars with the cores of the brick that held the tie and also gave some axial compression on the tie. At the tooled joint face in compression, push-through of mortar joint occurred and pieces of crushed mortar started to spall.
- The test was stopped at 12 mm displacement due to excessive displacement and little resistance showed on the tension side of the tie.

Specimen	T4
Characteristics	V-Tie only embedded at the centre, length 80 mm Type S mortar
Test Date (age)	February 16th, 2001 (58 days)
Surcharge Load	60 kPa
Maximum Force	
Tension	3.68 kN
Compression	2.95 kN
Displacement at Maximum Force	
Tension	1.71 mm
Compression	1.98 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship



Load-Displacement Envelope Curve

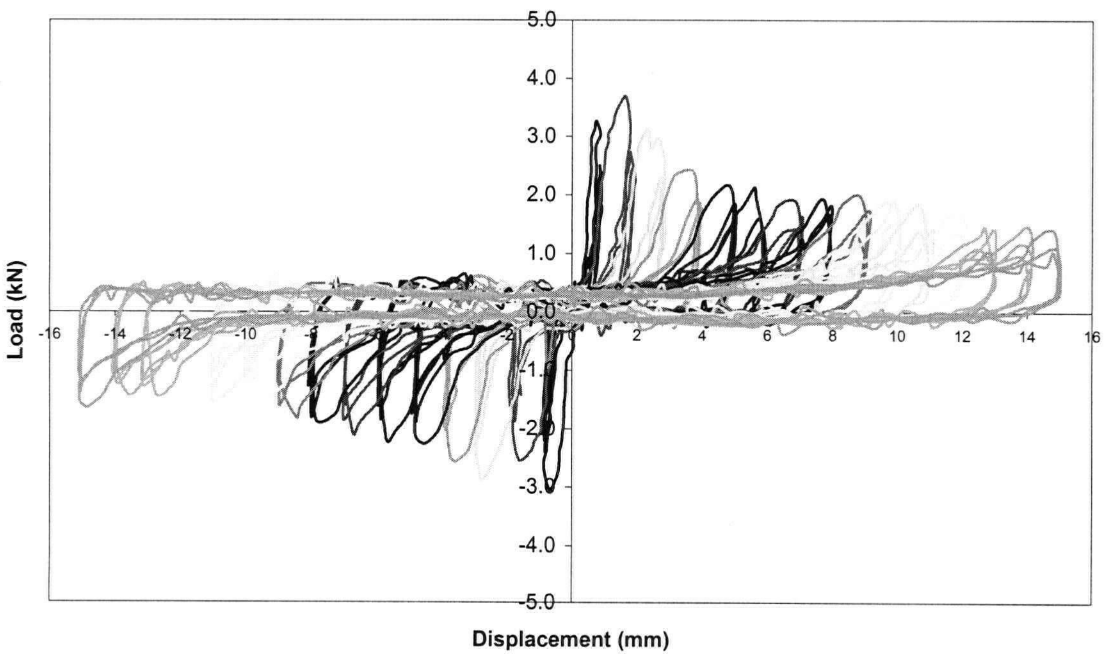


Description of test observations:

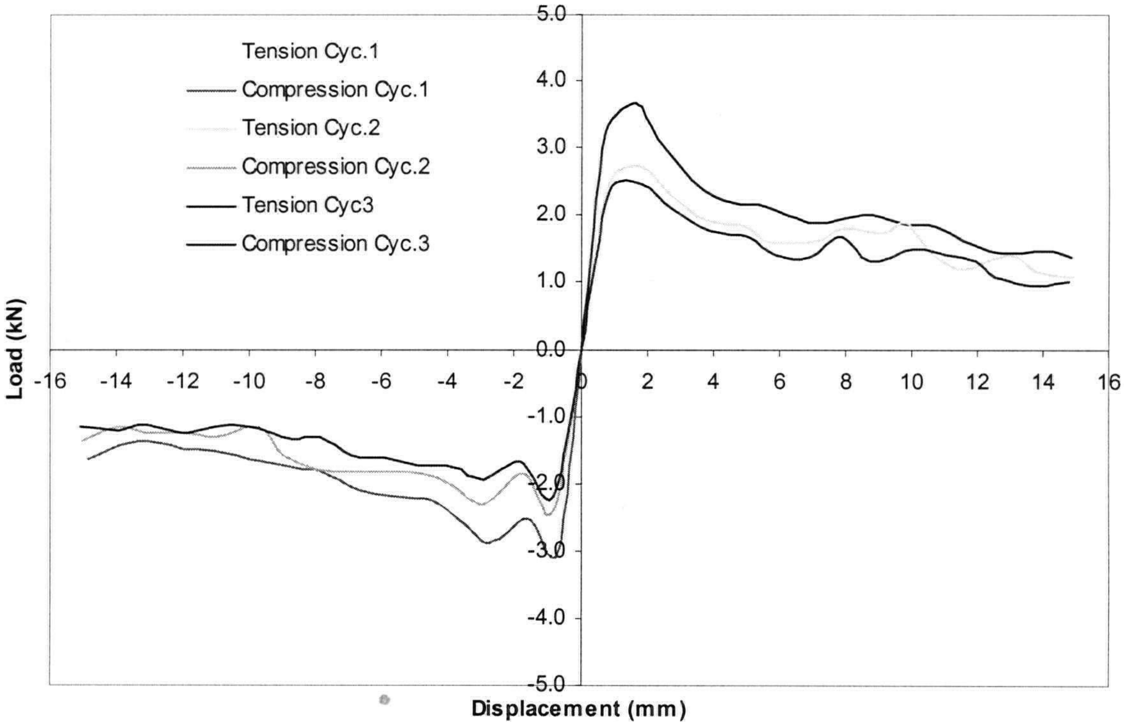
- At 2nd stage (2 mm displacement), 2nd cycle, on tie face or in tension cracks started to form in the area where the legs of the tie are, while in compression, on the tooled joint face, cracks were formed in the centre bed joint with the distance between each crack approximately one brick unit.
- At 6 mm displacement, the mortars already crushed on both faces but the surcharge load maintained to limit the opening of the cracks in the bed joint and clamped the tie.
- At Stage 9 (9 mm displacement) cycle 1, the mortar pieces spalled on the tie face in tension, and in compression the crushed mortar pieces started to become loose, and just pushing through from the bed joint.
- At stage 12 (12 mm displacement), the tie face lost most of the crushed mortar due to spalling, and on the compression face, the crushed mortar can be seen pushed through from the joint.

Specimen	T5
Characteristics	V-Tie only embedded at the centre, length 80 mm Type S mortar
Test Date (age)	February 22nd, 2001 (64 days)
Surcharge Load	60 kPa
Maximum Force	
Tension	3.68 kN
Compression	3.05 kN
Displacement at Maximum Force	
Tension	1.66 mm
Compression	0.68 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship

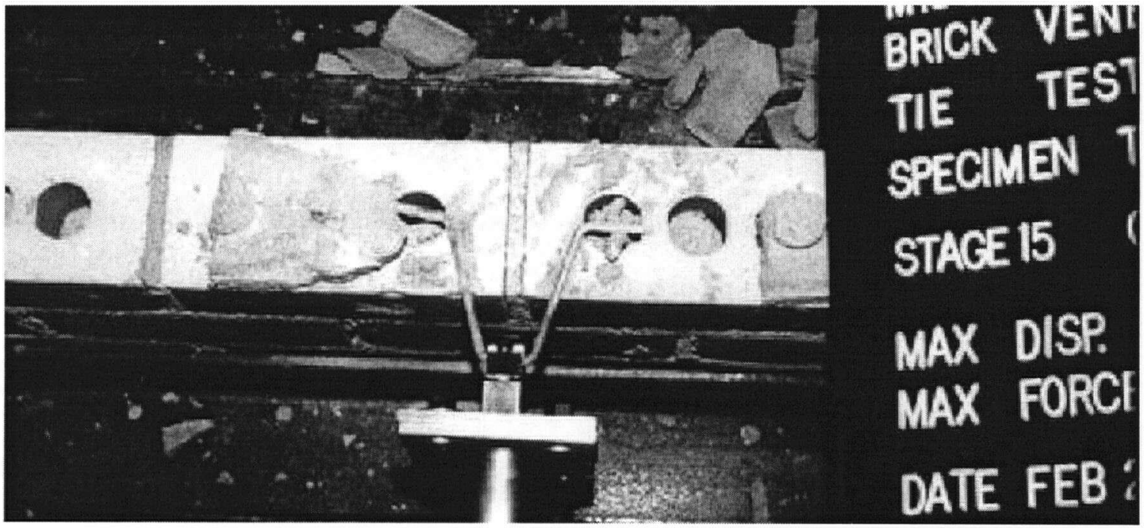


Load-Displacement Envelope Curve



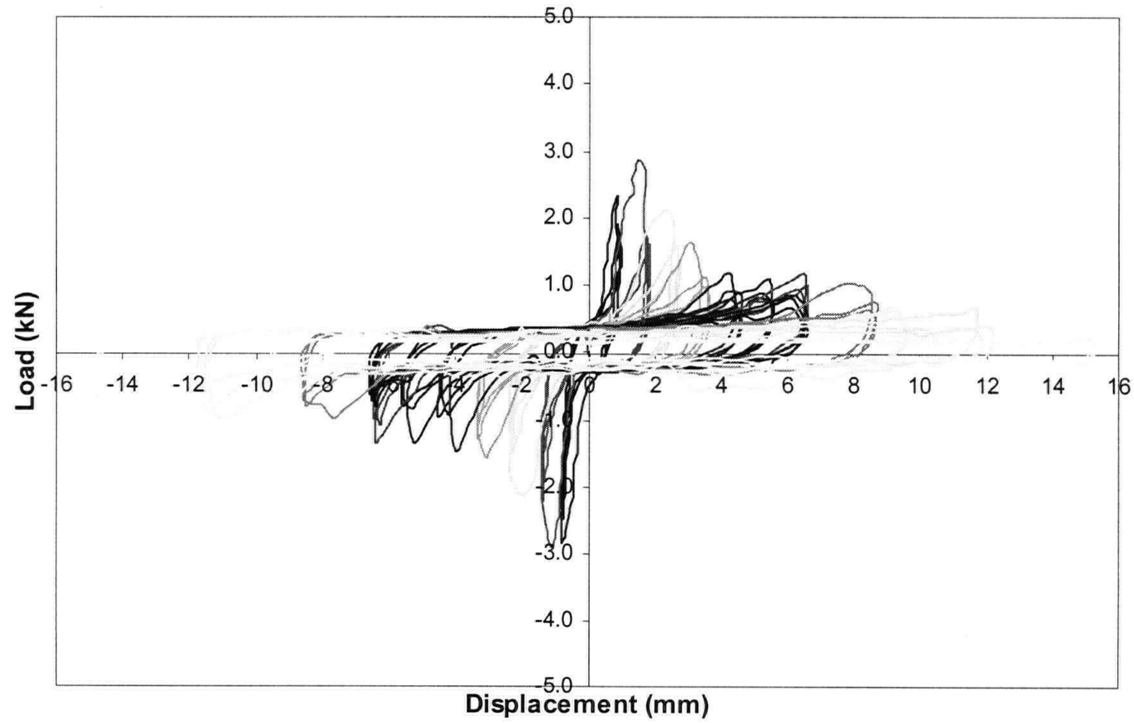
Description of test observations:

- Cracks started to form at the bed joint near the area of the tie on the tie face at 2 mm displacement i.e. at stage 2 2nd cycle, on the tooled joint side, two vertical cracks were evident on the centre of the mortar joint.
- At 5 mm displacement, stage 5 3rd cycle, pieces of crushed mortars already pulled out and on the tooled joint face, pushed-through of mortars from the bed joint was apparent.
- At 9 mm displacement, stage 9, some mortars on the tie face were spalled in tension; while on the other side pushed through crushed mortar pieces was evident.
- At 15 mm the test was stopped and continued by observing the mortar joint of the tie, this was done by opening up the bed joint. Although the opening up of the bed joint was done carefully, it was hard to maintain the crushed mortars pieces not to be disturbed due to their brittle behaviour. The observations showed that the leg of the tie bent onto the tension side. The bent was in the joint between the leg and the body of the V-Tie.

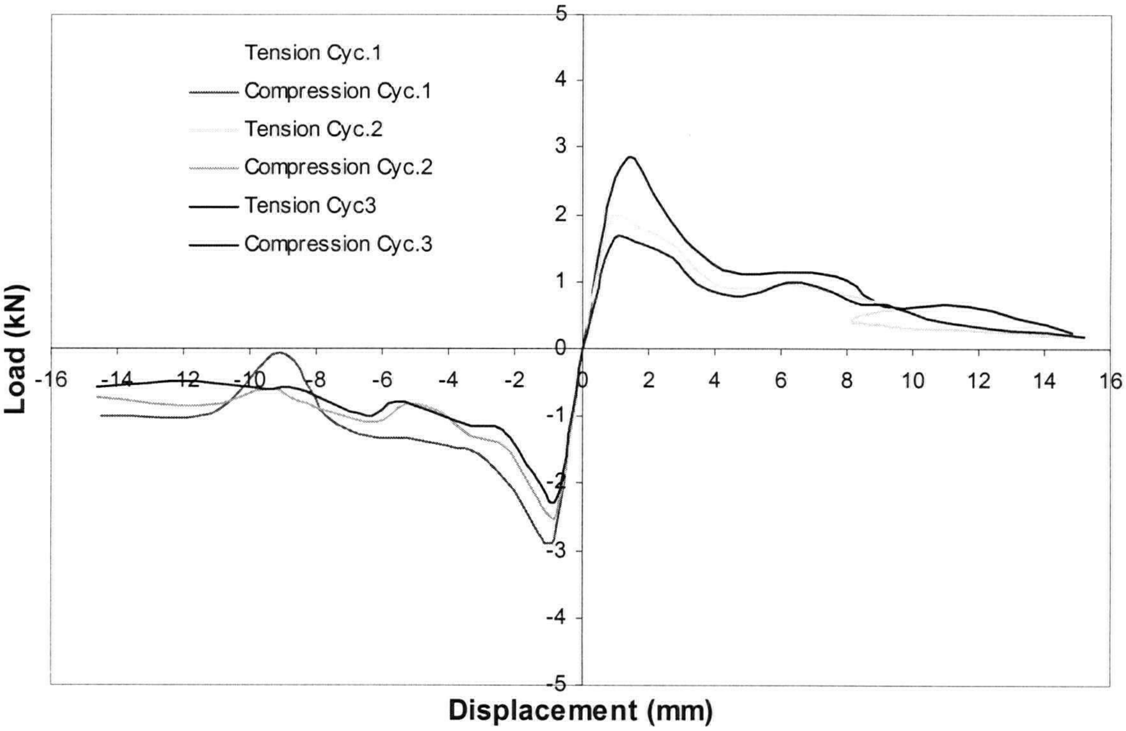

BRICK VEN
TIE TEST
SPECIMEN
STAGE 15
MAX DISP.
MAX FORC
DATE FEB

Specimen	T6
Characteristics	V-Tie only embedded at the centre, length 80 mm Type S mortar
Test Date (age)	March 5th, 2001 (75 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	2.87 kN
Compression	2.90 kN
Displacement at Maximum Force	
Tension	1.5 mm
Compression	1.12 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship

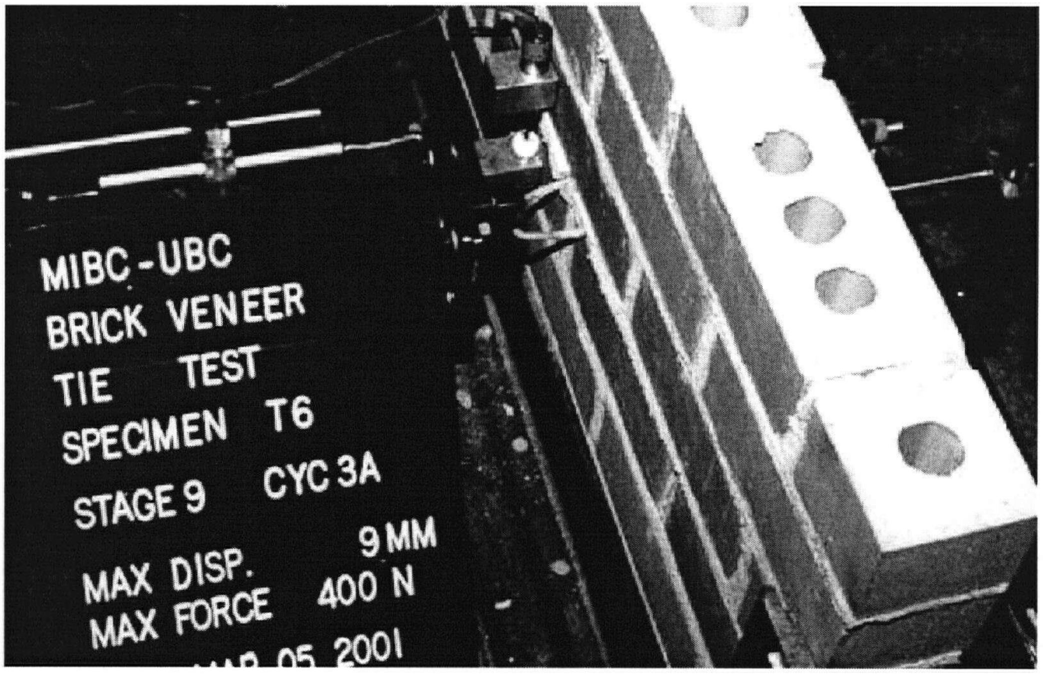


Load-Displacement Envelope Curve



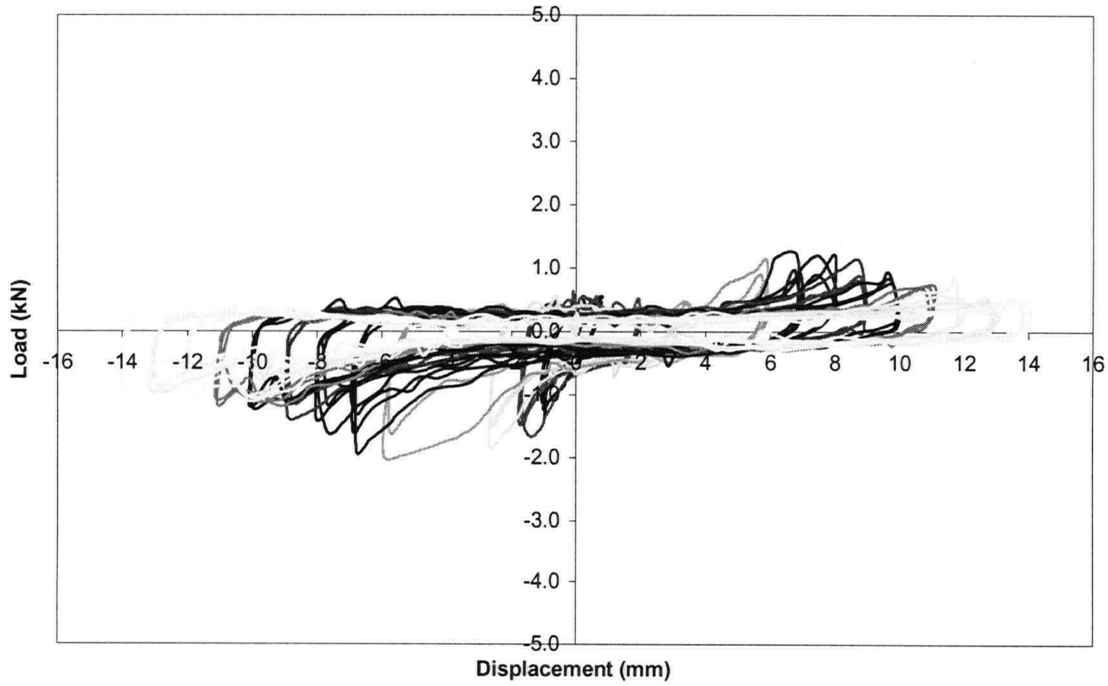
Description of test observations:

- At 1 mm displacement cycle 3, in compression, there were cracks formed on the tooled joint face of the panel.
- At stage 2, 2nd cycle with 2 mm displacement, cracks formed in tension at the tie face of the panel near the area of the legs of the tie. While in compression, previous cracks started to open up and several more cracks were also formed at the tooled joint face.
- At 4 mm stage 4, 1st cycle, more cracks were formed, in compression at the tooled joint side, one crack propagated to the top course of the brick panel. This finally led into a diagonal shear cracks of the panel specimen. In tension, at the tie face cracks became bigger and some pieces already spalled.
- Stage 8 was intentionally set to 8 mm displacement, but accidentally it was only displaced to 7 mm because of failure on the setting of the controller. So both stage 7 and 8, were similar that is 7 mm displacements.
- At 9 mm displacement, pull-out of crushed mortars occurred in tension. The tooled joint face mortar bed were broken apart into several pieces of loose crushed mortars, with pushed-through of crushed mortars occurred.
- After stage 10 the test was continued to 12 mm directly, skipping the 11 mm displacement, this was done basically because there was little resistance anymore in tension side and compression side.
- At stage 12 and 15, large number of crushed mortar pieces spalled, leaving a big hole on the mortar bed joint, both at the tie face in tension and at the tooled joint face in compression.

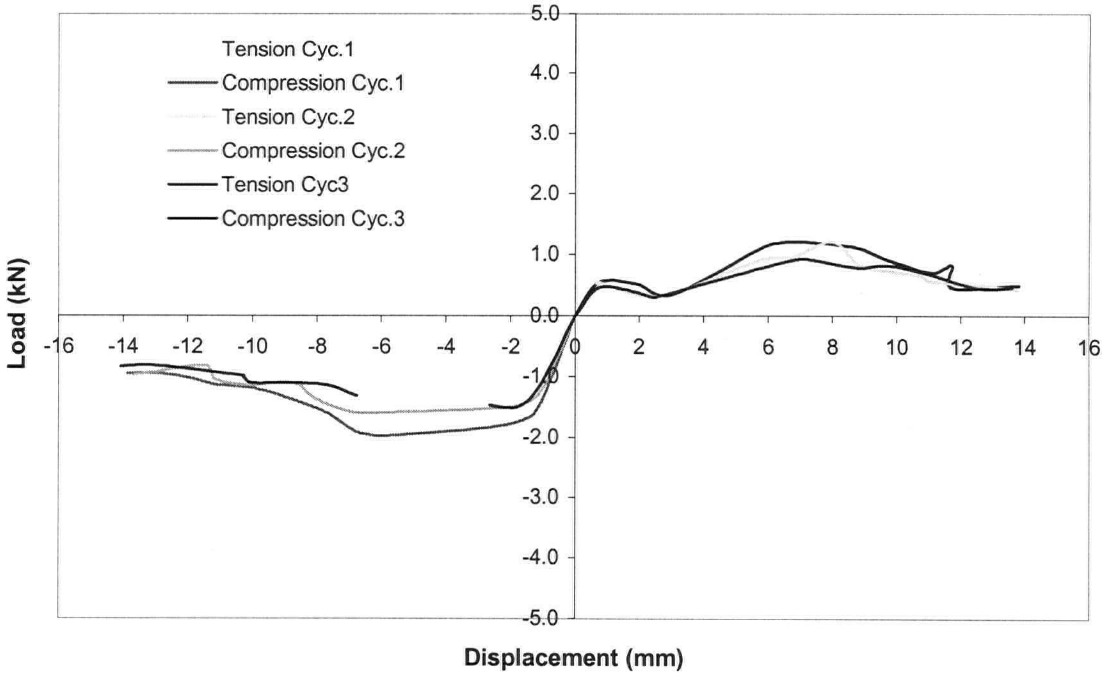

MIBC-UBC
BRICK VENEER
TIE TEST
SPECIMEN T6
STAGE 9 CYC 3A
MAX DISP. 9 MM
MAX FORCE 400 N

Specimen	TW1
Characteristics	V-Tie with horizontal wire reinforcement unclipped
	V-Tie length 80 mm
	Type S mortar
Test Date (age)	February 9th, 2001 (51 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	1.21 kN
Compression	1.98 kN
Displacement at Maximum Force	
Tension	6.81 mm
Compression	5.93 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship



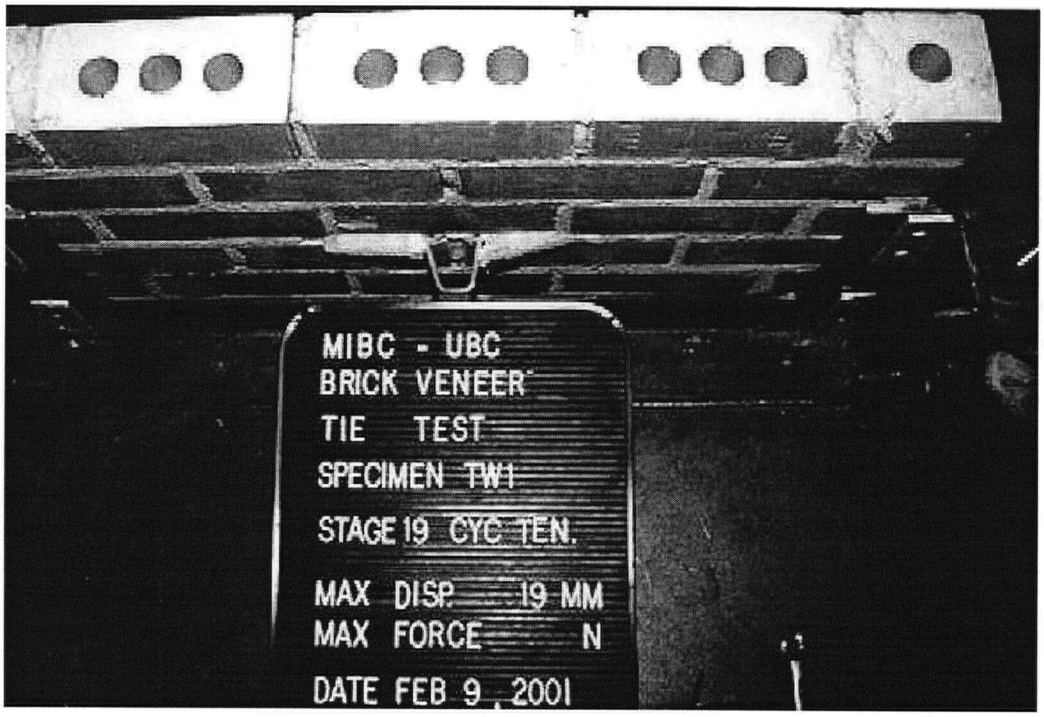
Load-Displacement Envelope Curve



Description of test observations:

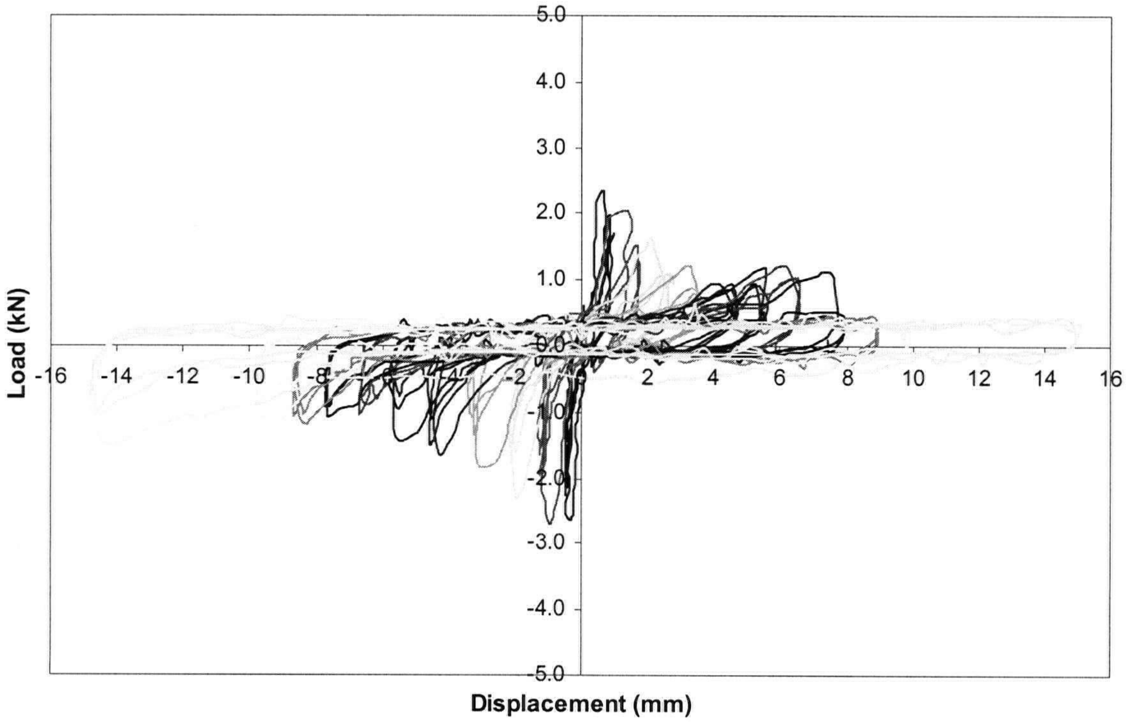
- The Specimen was pre-cracked / damage due to installation failure, there was a mishandling of the specimen when it was installed on the U-frame bracket, the specimen was dropped and the tie was accidentally pushed out due to the impact with the apparatus.
- The test was conducted at the usual stages, until it reached 3mm (Stage3), where it was observed that there was very little resistance on the tension side (presumably it was due to the internal cracks on the mortars in the bed joint located in the area of the tie). In compression (the tooled joint face), cracks were started to open up at 3 mm.
- Due to the reason of little resistance, the stages of test were changed, and the tie was displaced directly to 6 mm displacement after 3 mm displacement. It was observed that the tie resistance was increased in tension. While in compression there was no more cracks formed at the tooled joint face.
- At 8 mm displacement, there were new pieces of crushed mortars at the tie face that started to spall. While in compression pieces of crushed mortars started to push-through the bed joint.
- At 10 mm displacement, in compression the crushed mortars at the tooled joint face started to push out more, without any further cracks being formed. Similar occurrences happened at the tie face in tension.
- With the displacement being increased, the pieces of crushed mortars were just pushed through and pulled out very loosely without any further cracks being formed.

- The test was actually done with 19 mm in tension, but it was observed there was no more resistance and no more cracks were formed. So the data presented here is only until 14 mm displacement.

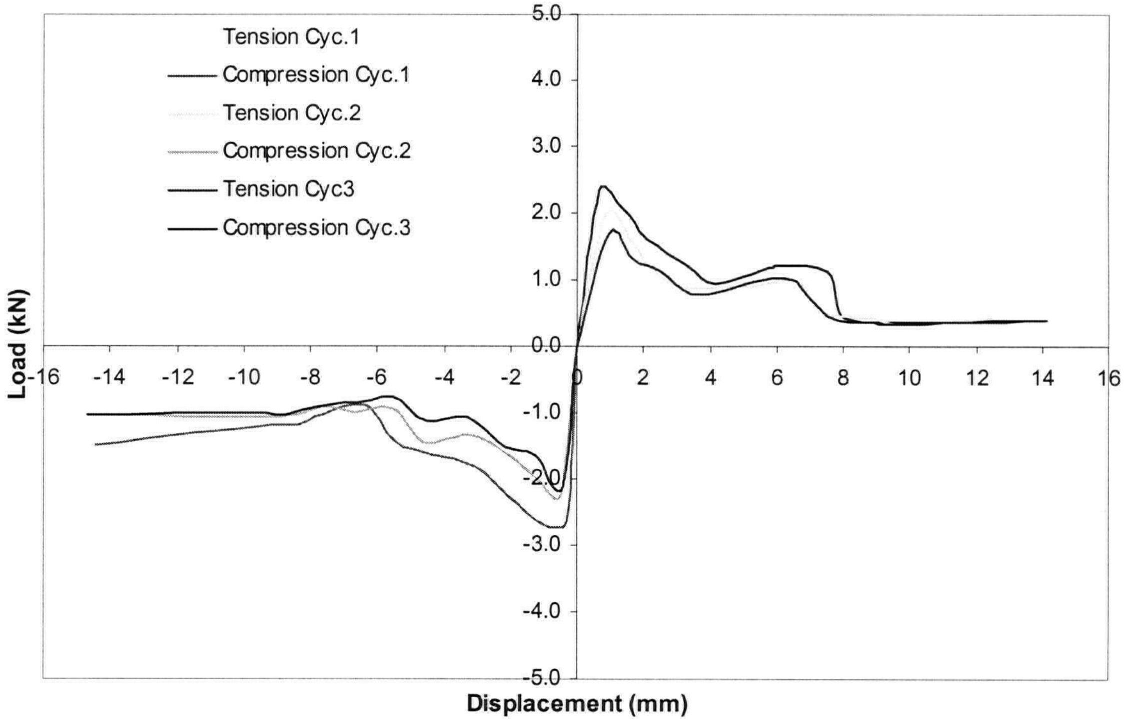


Specimen	TW2
Characteristics	V-Tie with horizontal wire reinforcement unclipped
	V-Tie length 80 mm
	Type S mortar
Test Date (age)	February 14th, 2001 (56 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	2.32 kN
Compression	2.68 kN
Displacement at Maximum Force	
Tension	0.65 mm
Compression	0.95 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship

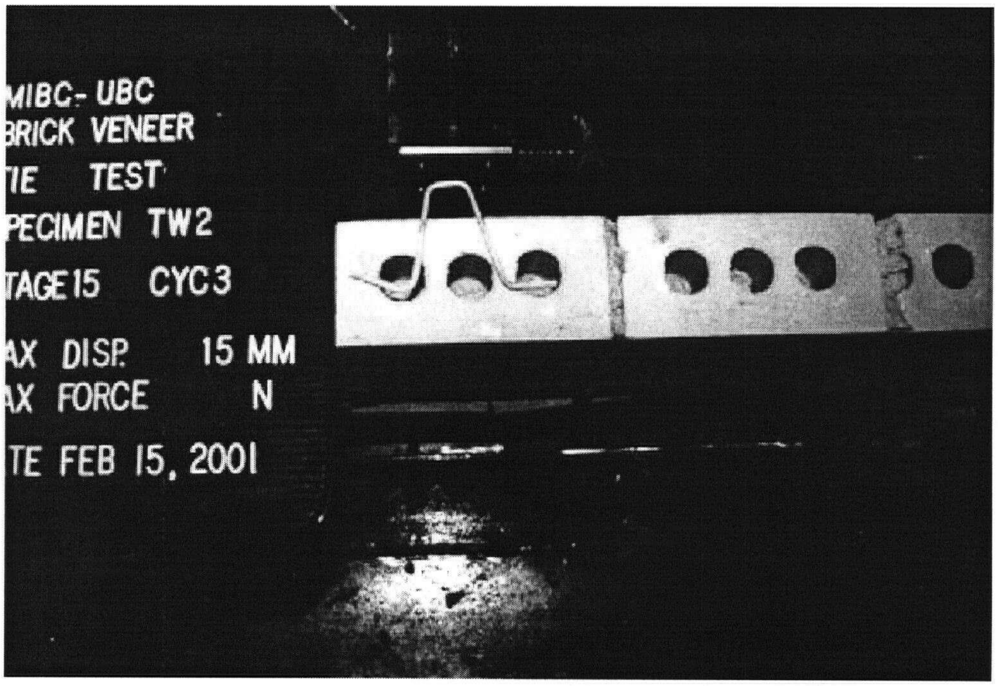


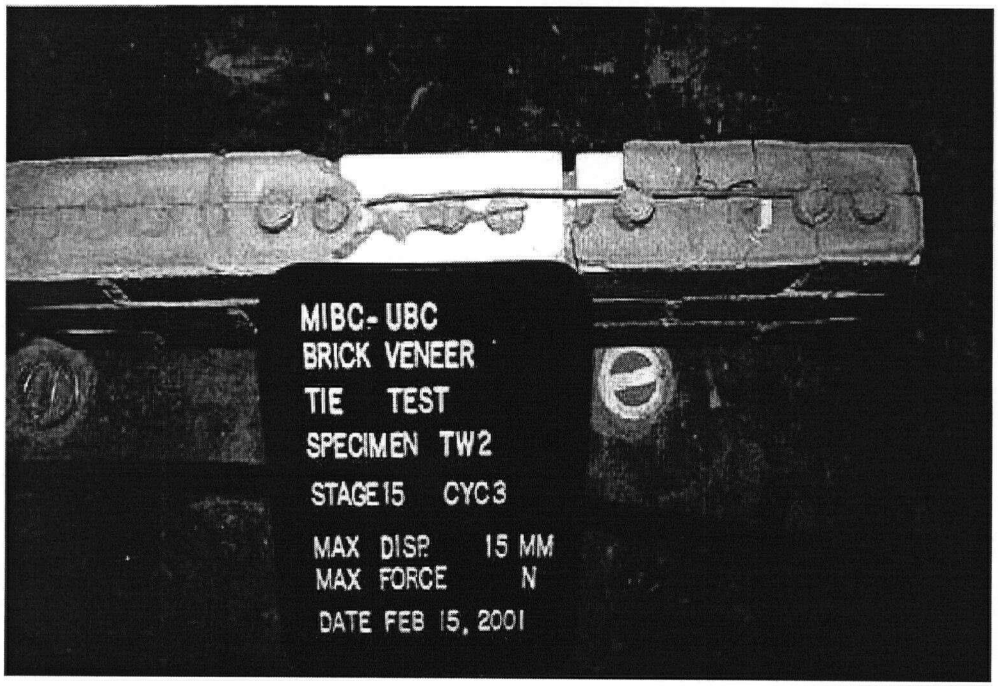
Load Displacement Envelope Curve



Description of test observations:

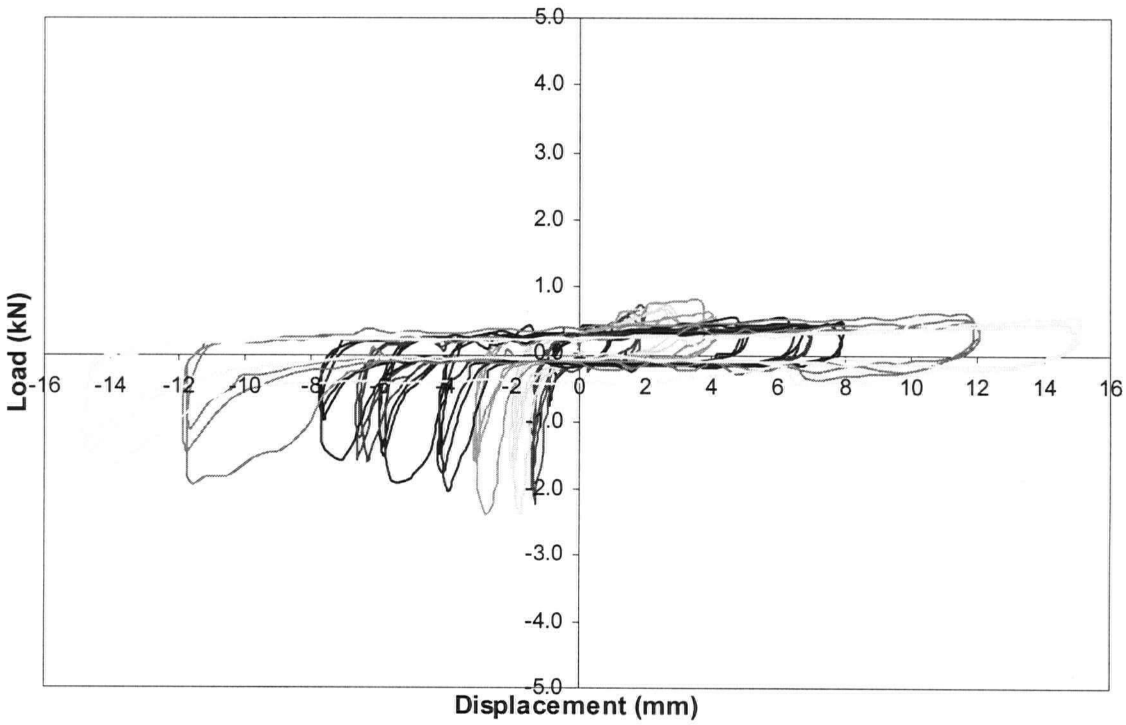
- At 2 mm stage 2, 2nd cycle, cracks were formed on both faces, a crack pattern was formed and some of the crushed mortars on the tie face were spalling.
- At 4 mm displacement stage 4, crushed mortar joints at the tie face started to pull out. While in compression, pieces of crushed mortars started to push-through from the bed joint at the tooled joint face.
- At 5 mm displacement stage 5 in tension, the crushed mortars at the tie face finally spalled. While in compression, more pieces of crushed mortars started to push through the bed joint.
- At 8 mm displacement, stage 8 cycle 1, there was a loud noise came from the specimen, then the curve started to show a tension resistance lost. At 3rd cycle of loadings, loose pieces of crushed mortars at the tie face were removed. This method permitted the observation of the tie while it was under compression or tension.
- At stage 9 with 9 mm displacement, it was observed that the tie itself with the mortars being crushed in the tie area, was actually just slip on top of the wire reinforcement in compression, so basically there was no such an engagement on the wire at larger displacement. And in tension, the tie was deforming at the joint where the leg of the tie and the body met, while crushing the mortar, thus the wire was not actually engaged in tension.
- With the 10 mm displacement, an indication of same occurrence happened at both faces of the panel (pull-out and push-through more of the crushed mortars from the bed joint) with less resistance, the test was then incremented directly to 15 mm displacement, and stopped.


IBC- UBC
RICK VENEER
IE TEST
PECIMEN TW2
TAGE 15 CYC3
AX DISP 15 MM
AX FORCE N
TE FEB 15, 2001

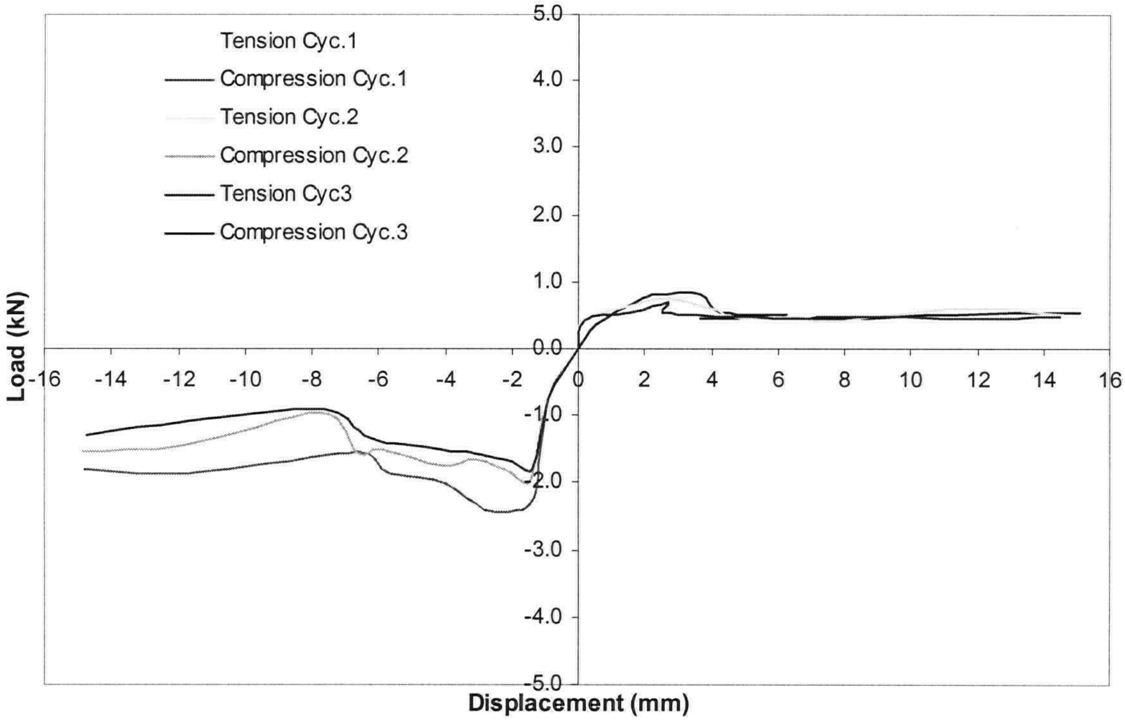

MIBC- UBC
BRICK VENEER
TIE TEST
SPECIMEN TW2
STAGE 15 CYC3
MAX DISP 15 MM
MAX FORCE N
DATE FEB 15, 2001

Specimen	TW3
Characteristics	V-Tie with horizontal wire reinforcement unclipped V-Tie length 80 mm Type S mortar
Test Date (age)	March 2nd, 2001 (72 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	0.81 kN
Compression	2.40 kN
Displacement at Maximum Force	
Tension	3.68 mm
Compression	2.81 mm
Failure modes	
Tension	Pull-out through mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship



Load Displacement Envelope Curve

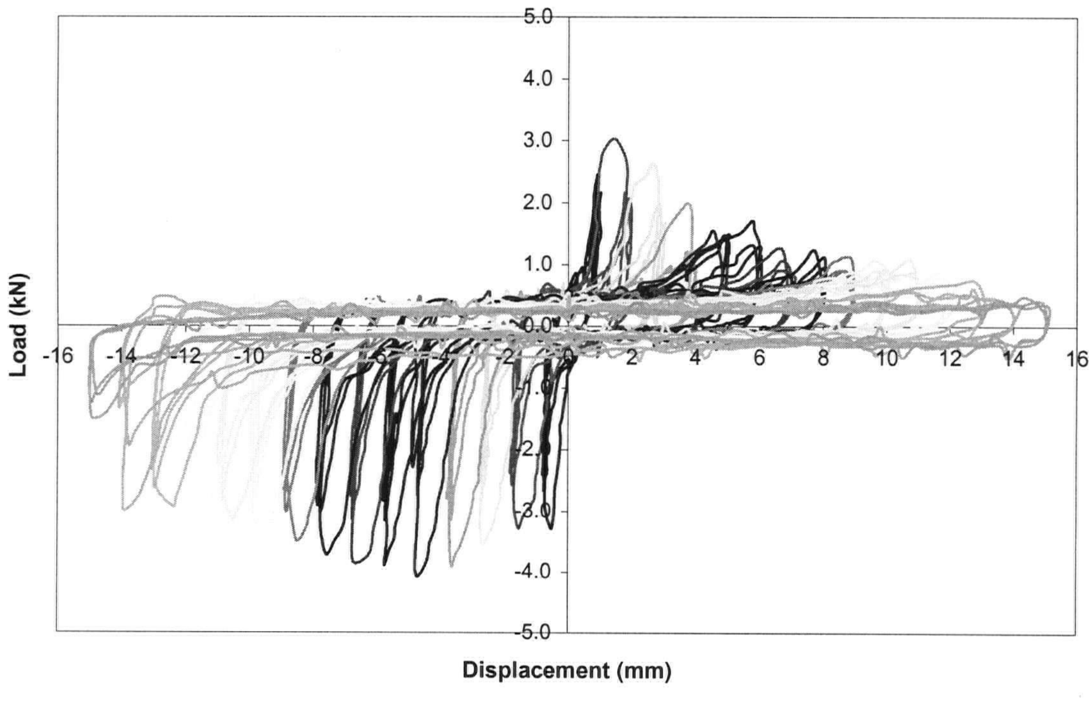


Description of test observation:

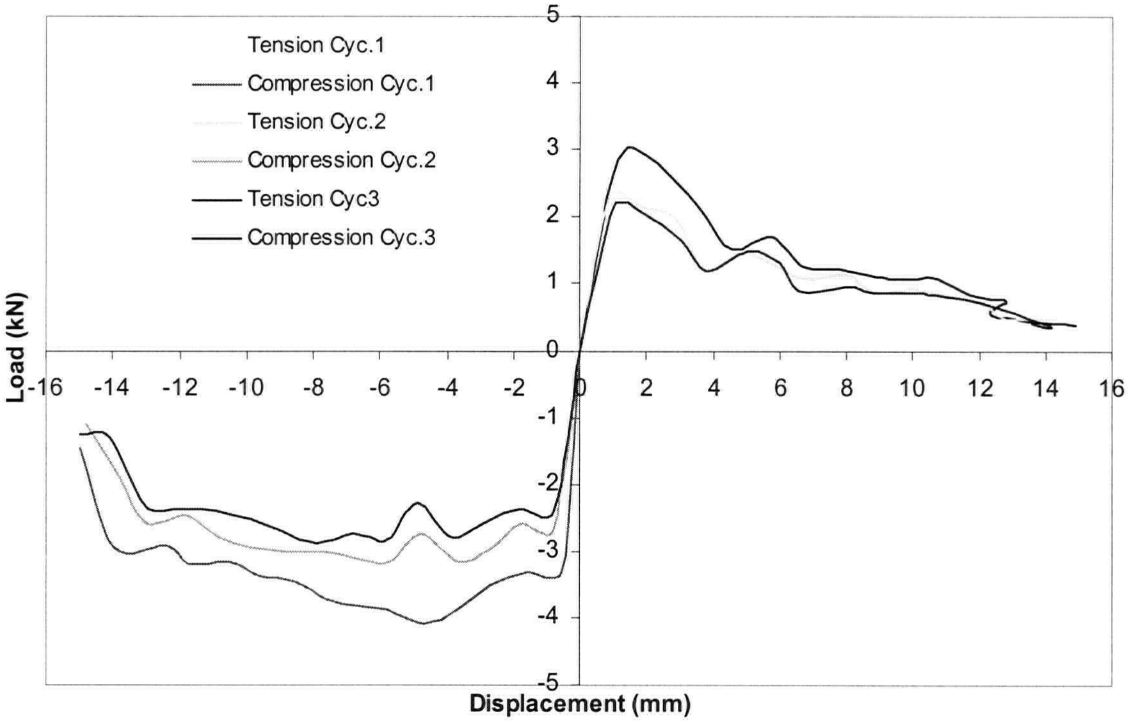
- The specimen was damaged due to a loading problem on the testing apparatus. When the specimen was installed inside the frame and was connected to the hydraulic actuator, the actuator pushed the tie suddenly and damaged the tie embedment. The test was still conducted to see the how the specimen would perform.
- At stage 1 to stage 8 (1 mm to 8 mm displacement), the cracks from the loading problem just open up and some of the crushed pieces of mortar spalled in tension at the tie face at 5 mm displacement, while in compression at the tooled joint side, the cracks that were already formed getting wider and push-through of crushed mortar occurred.
- The test was continued directly to 12 mm, which was observed no difference from the last stages, except in compression more crushed mortar pieces being pushed-through from the bed joint.
- The test was stopped at 15 mm with almost all the crushed mortars at the tie face spalled from the bed joints. While in compression, the crushed mortars at the tooled joint face were pushed-through from the bed joint.

Specimen	TW4
Characteristics	V-Tie with horizontal wire reinforcement unclipped V-Tie length 80 mm Type S mortar
Test Date (age)	February 20th, 2001 (62 days)
Surcharge Load	60 kPa
Maximum Force	
Tension	3.03 kN
Compression	4.08 kN
Displacement at Maximum Force	
Tension	1.46 mm
Compression	4.72 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship

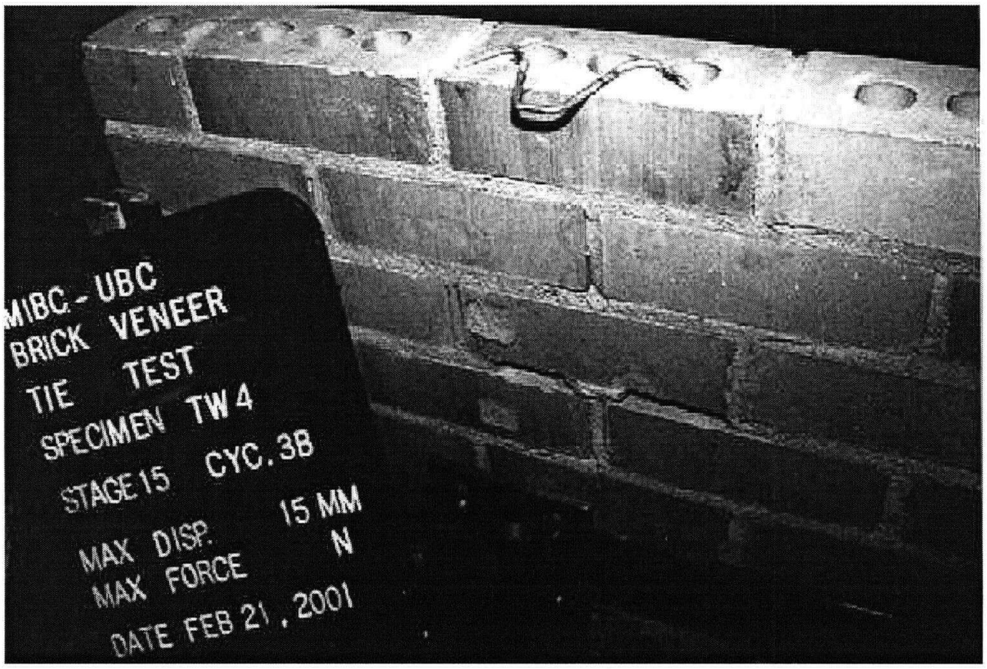


Load Displacement Envelope Curve



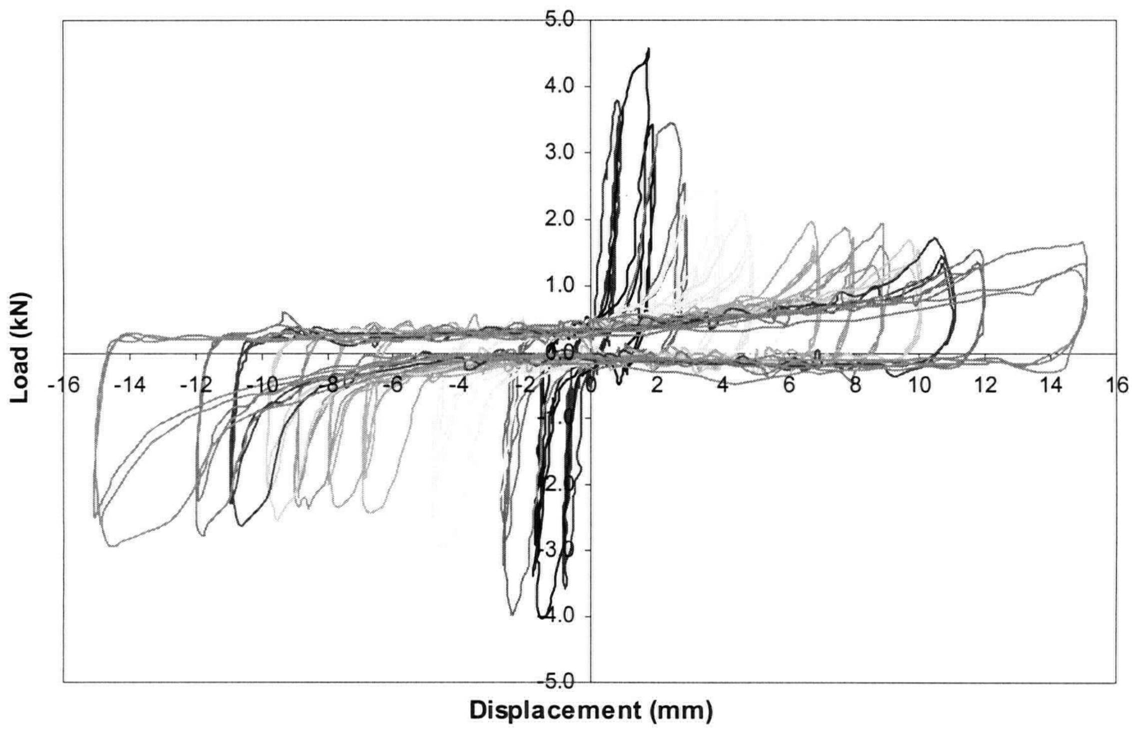
Description of test observation:

- At 2mm displacement stage 2, 2nd cycle in tension, at the tie face there were cracks formed in area where the leg of the tie was embedded. While in compression there were no cracks formed yet.
- At 4 mm displacement stage 4, in tension crushed mortars at the tie face started to spall, while in compression there were some cracks formed at the tooled joint face of the brick panel.
- At stage 5 with 5 mm displacement, pull-out of mortar joint in tension and spalling of pieces of crushed mortars occurred. In compression, more cracks of the mortar joint were formed but the joint was still completely intact (the width of the cracks were very small), this was due to the clamping force from the surcharge load.
- At stage 8, 8 mm displacement, all the crushed mortars at the tie face were spalled and removed, again the opening of the joint permitted the observations of the tie while it was loaded in tension or compression. It was evident that the tie was pushing up against the horizontal wire reinforcement in compression with mortar still intact around the perimeter of the reinforcement. This gave stronger resistance in compression and allows the tie to deform its legs into the compression side (the stronger direction). While in tension, pull-out of crushed mortars in the bed joint occurred.
- The test was continued until 15 mm displacement where it was observed that all the mortars were spalled at the tie face while in compression push-through of the crushed mortars from the bed joint was occurred.

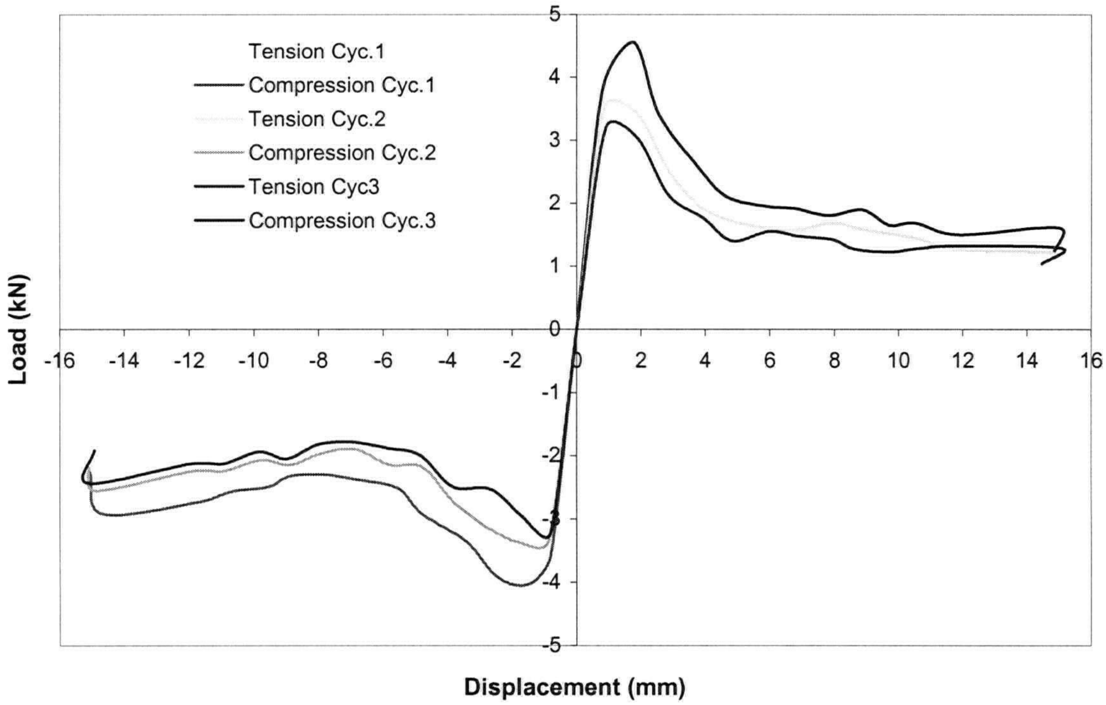

MIBC - UBC
BRICK VENEER
TIE TEST
SPECIMEN TW 4
STAGE 15 CYC. 38
MAX DISP. 15 MM
MAX FORCE N
DATE FEB 21, 2001

Specimen	TW5
Characteristics	V-Tie with horizontal wire reinforcement unclipped
	V-Tie length 80 mm
	Type S mortar
Test Date (age)	February 23rd , 2001 (65 days)
Surcharge Load	60 kPa
Maximum Force	
Tension	4.56 kN
Compression	4.03 kN
Displacement at Maximum Force	
Tension	1.77 mm
Compression	1.47 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship

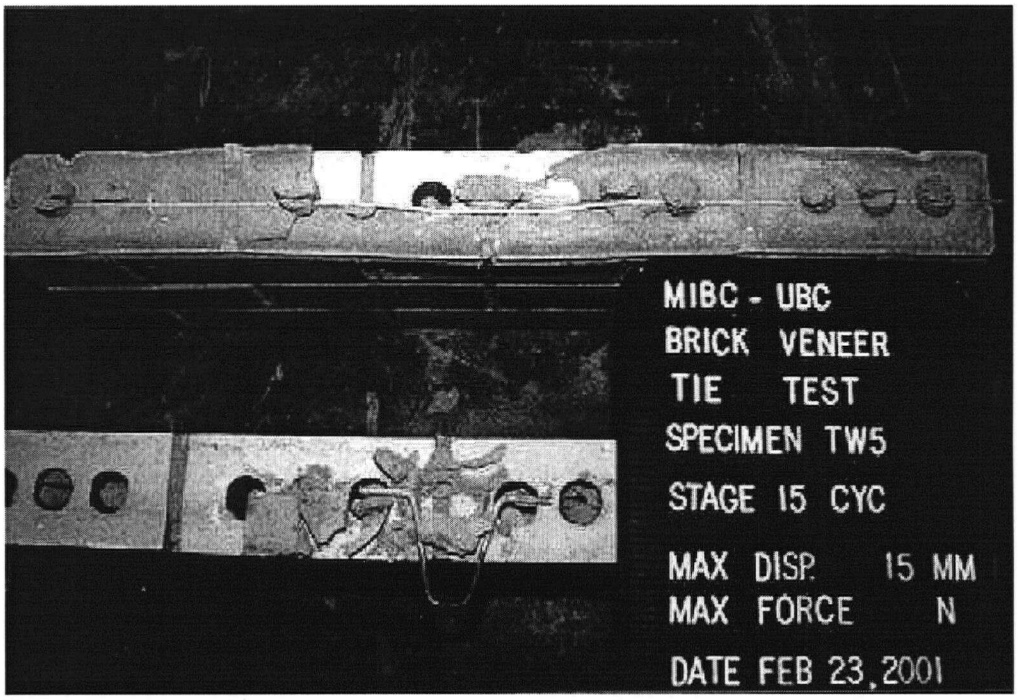


Load Displacement Envelope Curve



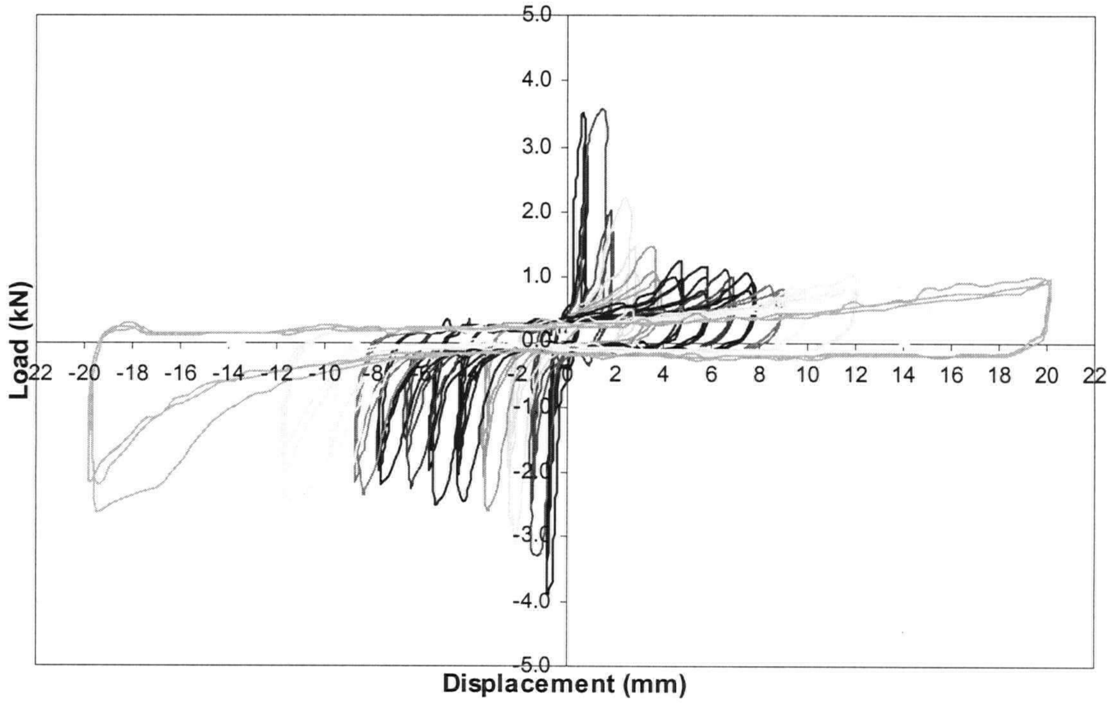
Description of test observation:

- At 2mm displacement stage 2, 2nd cycle in tension, at the tie face there were cracks formed in the embedment location of the tie.
- At 5 mm displacement, in stage 5, in tension, more cracks were formed especially near the tie leg embedment but no spalling of crushed mortars occur yet. While in compression there were some little cracks formed at the tooled joint face of the brick panel. Again it was evident that the clamping force from the surcharge load helps the embedment capacity of the tie
- At stage 7, 7 mm displacement, pull-out of crushed mortars from the bed joint and spalling of pieces of crushed mortar occurred in tension. In compression, more cracks of the mortar joint were formed but the joint was still intact.
- At stage 10, 10 mm displacement, more pieces of crushed mortars at the tie face started to pull-out and spalled in tension. Compression side suffered from cracks and a small amount of crushed mortars were push-through from the bed joint.
- Test continued until 12 mm and then directly went to 15 mm, with spalling occurred on the bed joint at the tie face in tension, while in compression only push-through of mortars from the bed joint was apparent, without any spalling due to clamping force from surcharge load.

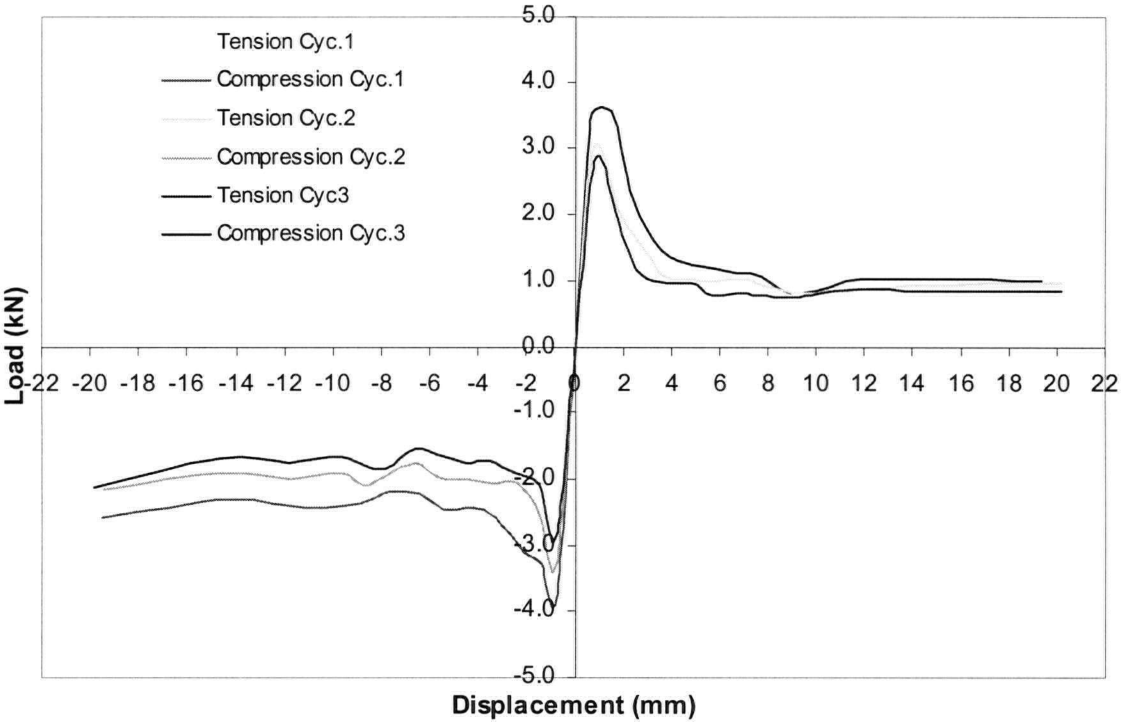

MIBC - UBC
BRICK VENEER
TIE TEST
SPECIMEN TW5
STAGE 15 CYC
MAX DISP. 15 MM
MAX FORCE N
DATE FEB 23, 2001

Specimen	TW6
Characteristics	V-Tie with horizontal wire reinforcement unclipped
	V-Tie length 80 mm
	Type S mortar
Test Date (age)	March 5th, 2001 (75 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	3.57 kN
Compression	3.87 kN
Displacement at Maximum Force	
Tension	1.57 mm
Compression	0.76 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship

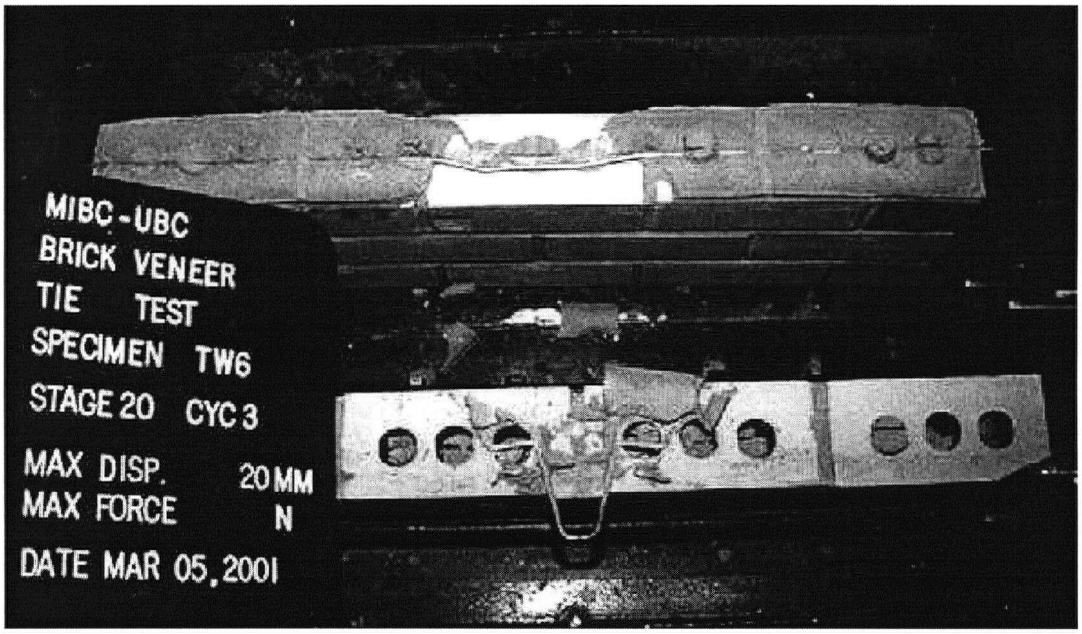


Load Displacement Envelope Curve



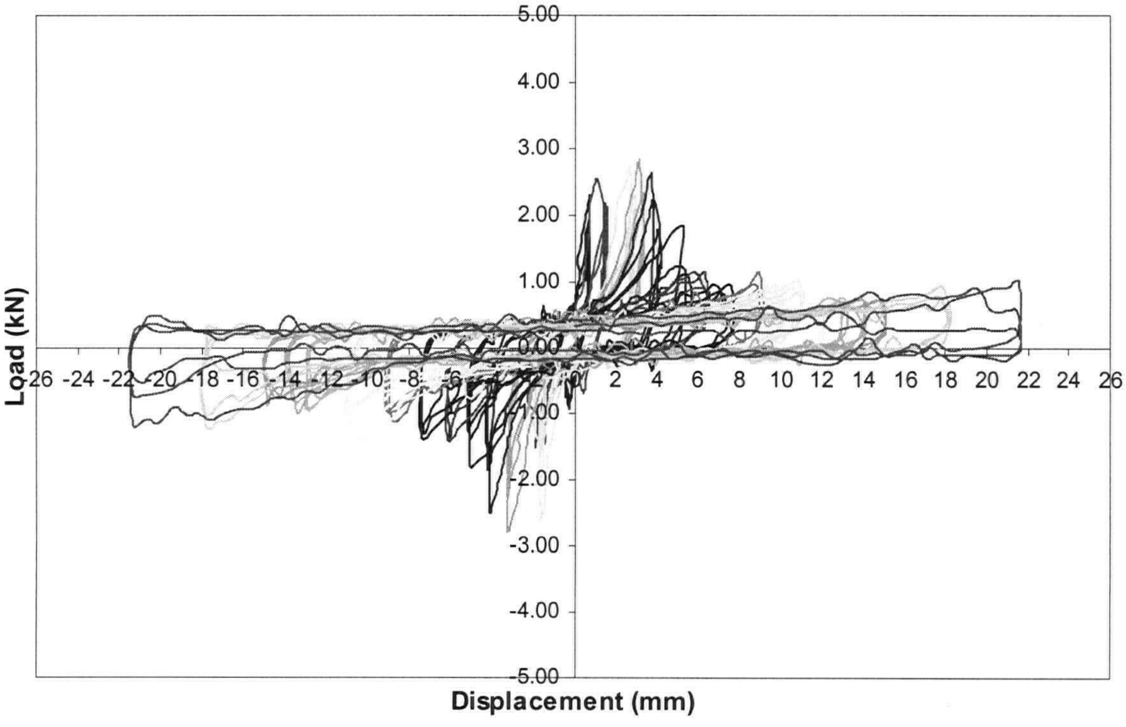
Description of test observations:

- At 2mm displacement stage 2, 2nd cycle in tension, cracks were formed in the area where the leg of the tie was embedded. While in compression several cracks were propagated in the bed joint at the tooled joint face and formed several pieces of loose crushed mortars.
- At 4 mm displacement stage 4, in tension several pieces of crushed mortar were formed, while in compression there were some more cracks formed.
- At stage 6, 6 mm displacement, pull-out of mortars from bed joint in tension and spalling of some pieces of crushed mortar occurred. In compression, push-through of mortars was evident.
- At stage 7 with 7 mm displacement in compression, some pieces of crushed mortars at the tooled joint face were spalled.
- The test was continued until 12 mm displacement then increased directly to 15 mm and further up to 20 mm displacement.
- Observations were made by opening up the bed joint where the tie was located. From the observations, it showed that all the mortar was spalled at the tie face (i.e tension side) by pull-out, and in compression push-through of the crushed mortar joint was occurred. The tie showed that the leg was deformed to the compression side (stronger direction).

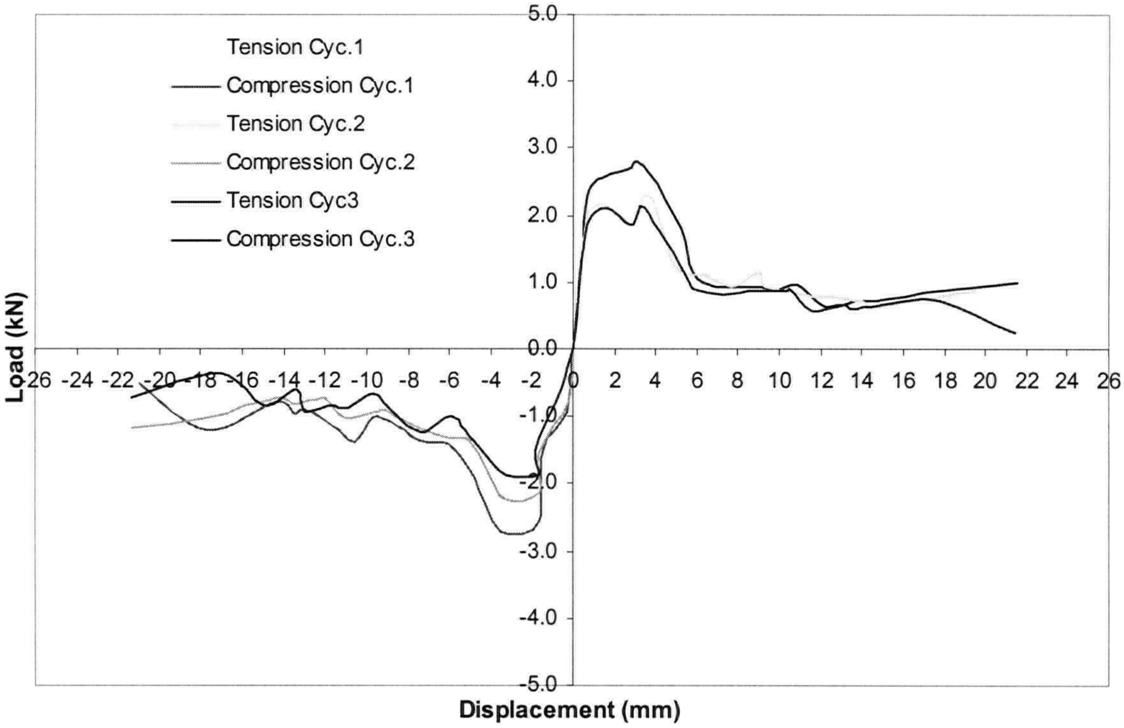

MIBC-UBC
BRICK VENEER
TIE TEST
SPECIMEN TW6
STAGE 20 CYC 3
MAX DISP. 20 MM
MAX FORCE N
DATE MAR 05, 2001

Specimen	TWC1
Characteristics	V-Tie with horizontal wire reinforcement clipped
	V-Tie length 80 mm
	Type S mortar
Test Date (age)	February 9th, 2001 (51 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	3.57 kN
Compression	3.87 kN
Displacement at Maximum Force	
Tension	1.57 mm
Compression	0.76 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship



Load Displacement Envelope Curve

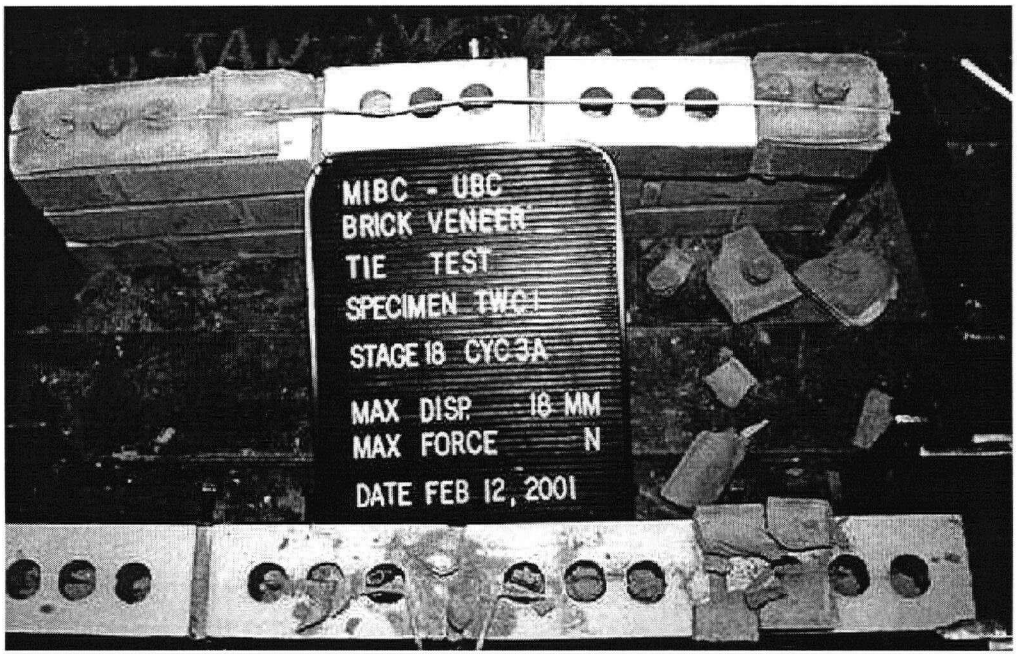


Description of test observations:

- At stage 2, 2nd cycle in tension, the mortar joint at the tie face where the legs were embedded, showed some small cracks.
- At stage 3, in tension, cracks located at tie face close to the legs of tie were more apparent, in compression there were small cracks formed in the mortar bed joint.
- At 4 mm displacement stage 4, more cracks were formed at the tie face, and in compression at the tooled joint face, several cracks were also formed.
- At 7 mm displacement stage 7, pieces of crushed mortar at the tie face started to spall and pull-out of crushed mortars were evident. While in compression side, cracks, which were formed in earlier stages, became bigger and started to push-through from the bed joint.
- At 12 mm displacement, stage 12, pieces of crushed mortar at the tie face were spalled so there were holes that made it possible to examine the tie movement with the wire attached to it by the clip. It was observed that one of the clip had disengaged the wire and fell off to the core hole on the brick unit, and the mortar around that side of the leg and the clip were all crushed and gone. While on the other side (the tooled joint face), the mortars were push-through severely and almost spall.
- At 13 mm crushed mortars spalling occurred at the tooled joint face in tension, the gap made possible to examine the movement from the backside of the tie. From the observations, it was apparent that the tie slipped in a way inside the bed joint where all the mortar already been crushed.
- The test was continued to 15 mm and then directly to 18 mm. Observations at 18 mm displacement through the hole showed that the clip that still attached started

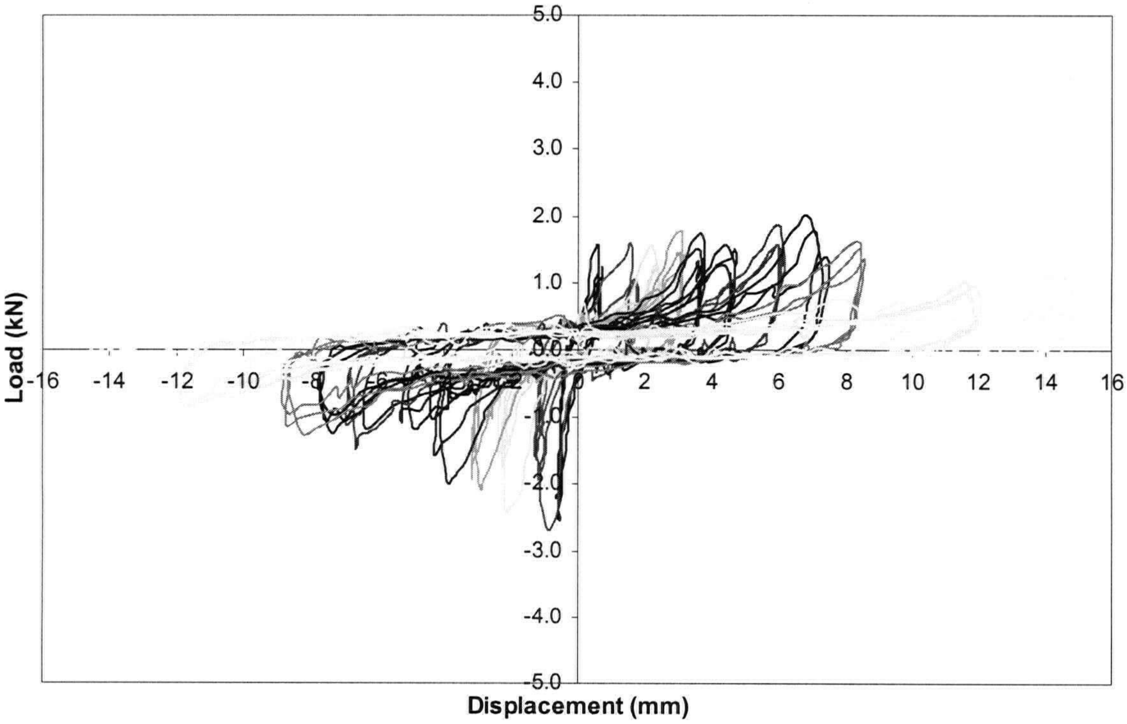
to disengage and the leg was riding up against the wire in compression. The override effect in larger displacement was made possible because the device (in this case it was the clip) that attaches or encloses the wire was finally detach or missing.

- Final displacement was 25 mm, but without any significant resistance of the tie. The observations continued by opening up the brick panel on the bed joint where the tie was embedded, and it showed that the wire engaged a wider area of mortar, thus the area of crushed mortars becomes larger. The horizontal wire joint reinforcement was also deformed showing that it had been engaged by the tie.

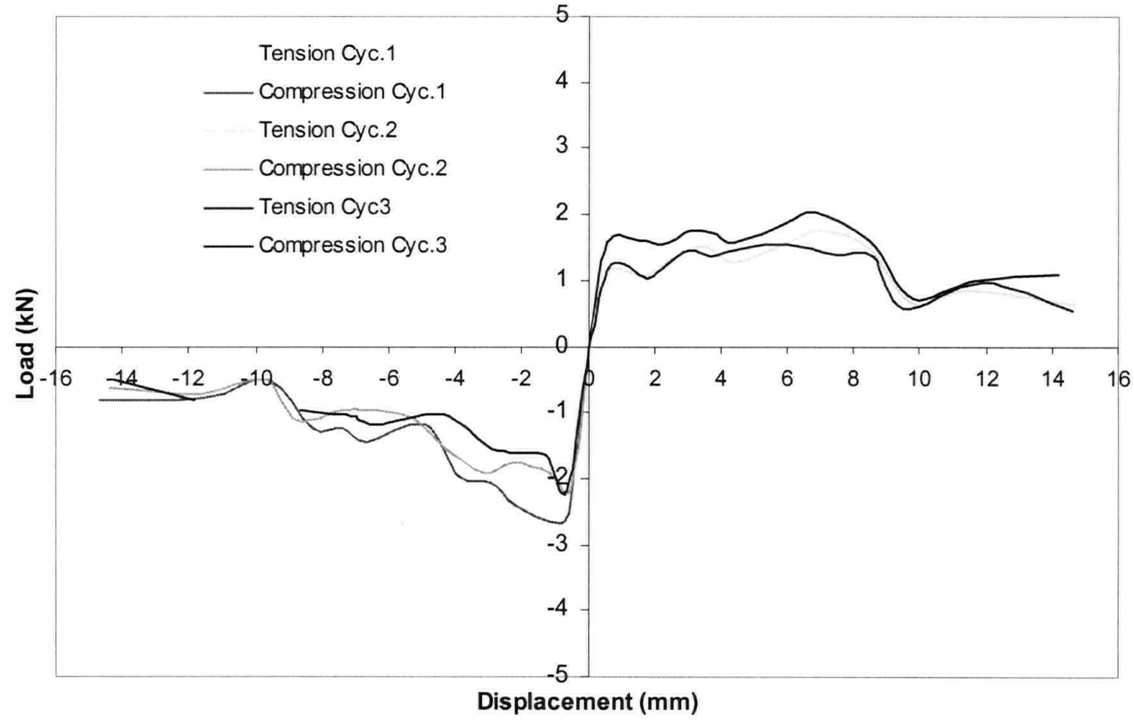


Specimen	TWC2
Characteristics	V-Tie with horizontal wire reinforcement clipped
	V-Tie length 80 mm
	Type S mortar
Test Date (age)	February 15th, 2001 (57 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	2.03 kN
Compression	2.68 kN
Displacement at Maximum Force	
Tension	6.85 mm
Compression	0.89 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship



Load-Displacement Envelope Curve



Description of Test Observations:

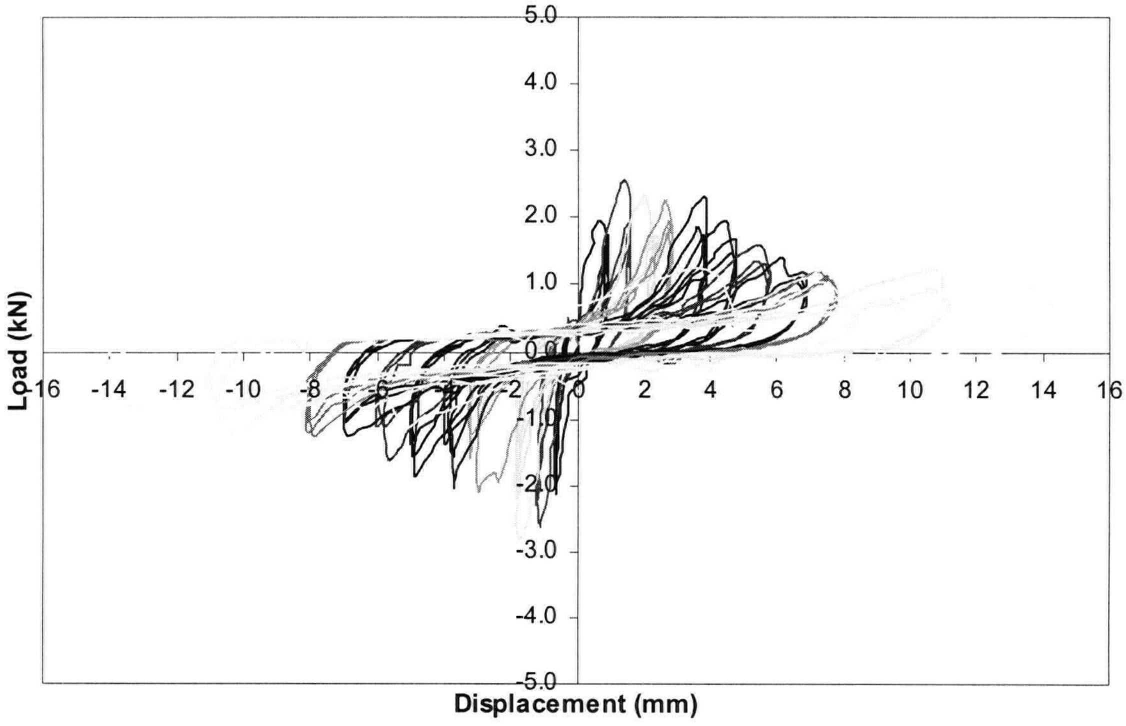
- It was observed at stage 2 with 2 mm displacement, cracks started to form in the area of the embedded leg of the tie in tension, while in compression there were several cracks formed on the tooled joint face.
- At 4 mm displacement stage 4, in tension, pieces of crushed mortar started to form, while in compression more cracks propagated in the bed joint of the tooled joint face.
- When the test reached 6 mm displacement at stage 6, pieces of crushed mortar at the tie face started to spall. In compression, push-through of mortar joint was starting to occur.
- At stage 7 with 7 mm displacement in compression, the push-through mortar on the tooled joint face became more extreme while in tension more crushed mortar joint were spalling from the tie face.
- At the end of stage 9, all the pieces of crushed mortar joint on tension side were removed, the holes enabled to examine the movement of the tie with the clipped joint reinforcement.
- At the start of stage 10, with 10 mm displacement, it was observed that one of the clips had already disengaged and fell into the core hole of the brick unit. All mortars around the area were crushed and in compression the crushed mortars on the tooled joint face were push-through further to the outside.
- Test was continued with 12 mm displacement at stage 12, while it was observed that in tension only one leg of the tie was engaging the wire joint reinforcement thus pulling out more mortar from the bed joint, while the other leg without the clip just moved to the target displacement. In compression, the 12 mm

displacement made it possible for the tie to over-ride the wire joint reinforcement without the clip attached to it.

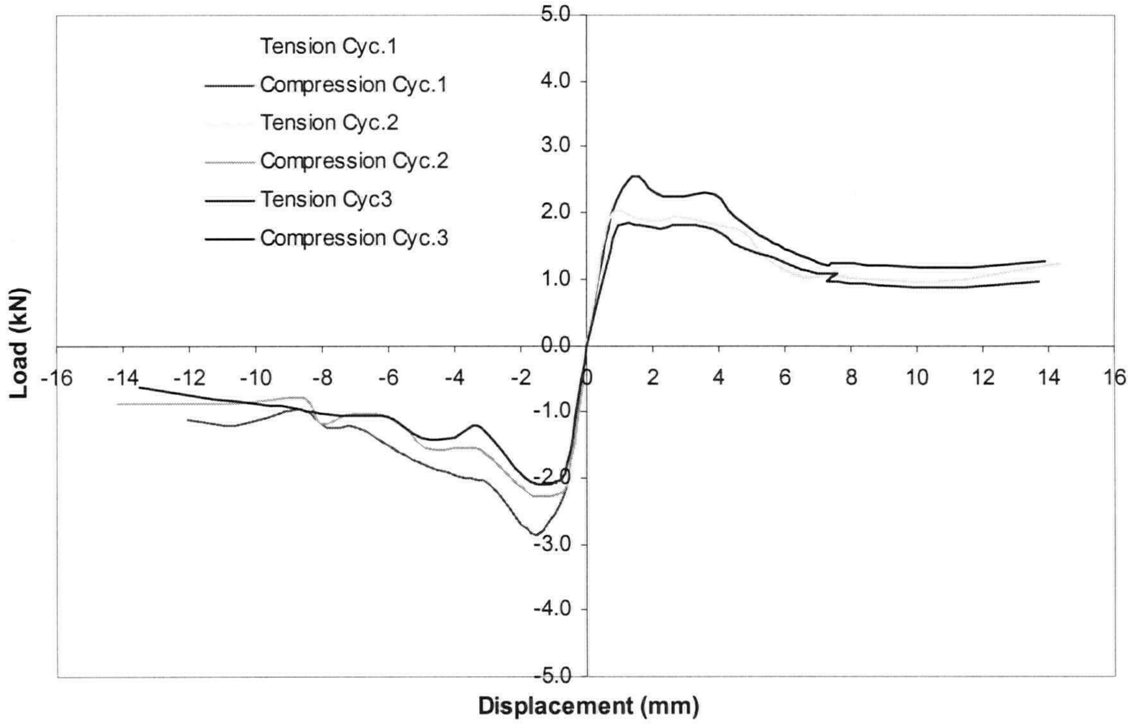
- The final displacement of the test was 15 mm, without any significance difference from the last stage of loading, all the mortars were already spalled on the tie face, while in compression, push-through of mortar joint was extreme on the tooled joint side with some mortars spalled.

Specimen	TWC3
Characteristics	V-Tie with horizontal wire reinforcement clipped V-Tie length 80 mm Type S mortar
Test Date (age)	March 2nd, 2001 (72 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	2.55 kN
Compression	2.82 kN
Displacement at Maximum Force	
Tension	1.40 mm
Compression	1.72 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship



Load-Displacement Envelope Curve

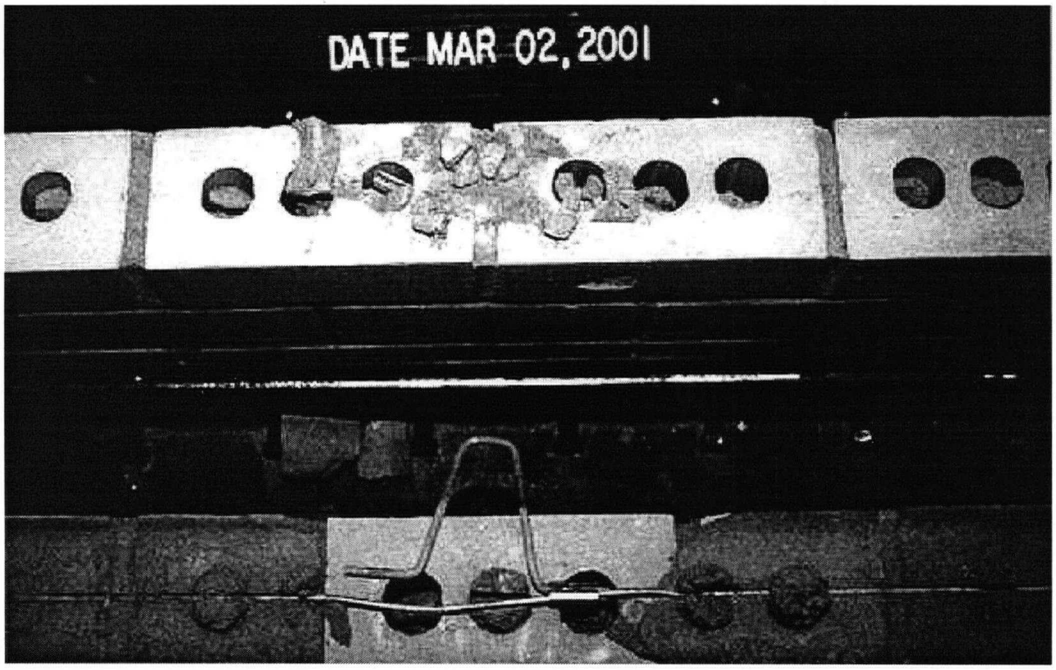


Description of Test Observations:

- At stage 2 with 2 mm displacement, 1st cycle, in tension several small cracks were formed on the tie face, which mostly concentrated in the area where the leg of the tie was embedded. In compression, two vertical cracks in the mortar bed where the tie was embedded started to open.
- In stage 4, 4 mm displacement, cracks started to open up and the movement of tie started to pull-out the crushed mortars from the bed joint in tension. While in compression cracks, which were formed earlier started to get bigger and propagated in the mortar joint on the tooled joint face.
- Before stage 5 was started, pieces of crushed mortar on both sides of the embedded leg of the tie wire were removed, the gap made possible to examine the movement of the tie with wire joint reinforcement enclosed by the clip. It was evident that at this stage both clips were still intact and provided the attachment between the tie and the wire joint reinforcement.
- At stage 7 with 7 mm displacement, the push-through mortar joint on the tooled joint side became more apparent in compression.
- When it reached 10 mm displacement at stage 10, crushed mortar joint on the tooled joint side push-through extremely with some almost spall, while in tension the tie still engaged the wire with the clip attached.
- The test was continued to 12 mm displacement and then directly to 15 mm displacement. In 15 mm displacement, in compression the tie was over-riding the wire joint reinforcement with the clip being pressed on the bottom, while the test

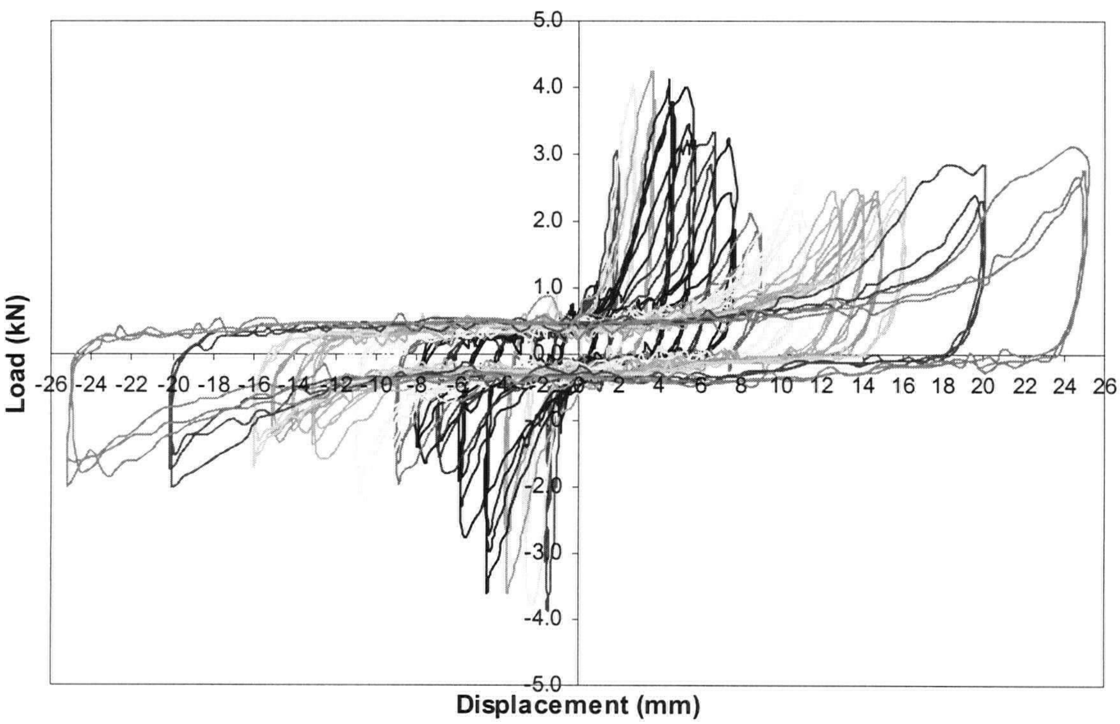
continued to the next cycle in tension, this clip moved and fell down into the core hole in the brick unit.

- Observations by opening up the mortar joint where the tie was located revealed that one clip detached and fell into the hole, while the other one still attached. The wire reinforcement showed the deformation caused by the effect of engagement from the tie.

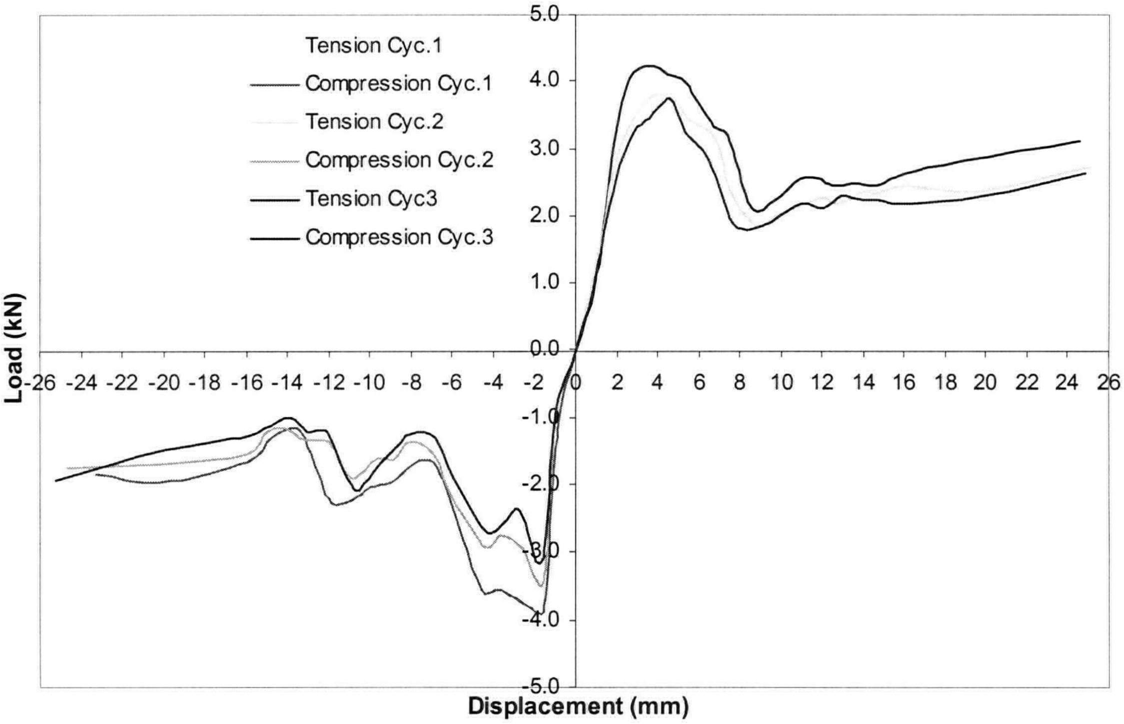


Specimen	TWC4
Characteristics	V-Tie with horizontal wire reinforcement clipped V-Tie length 80 mm Type S mortar
Test Date (age)	February 21st, 2001 (63 days)
Surcharge Load	60 kPa
Maximum Force	
Tension	4.22 kN
Compression	3.89 kN
Displacement at Maximum Force	
Tension	3.64 mm
Compression	1.53 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship



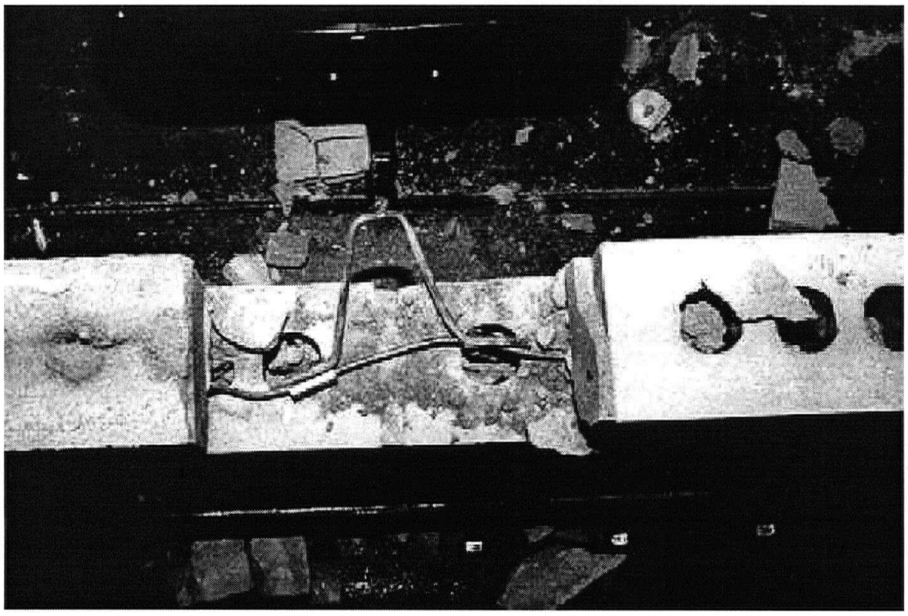
Load-Displacement Envelope Curve



Description of Test Observations:

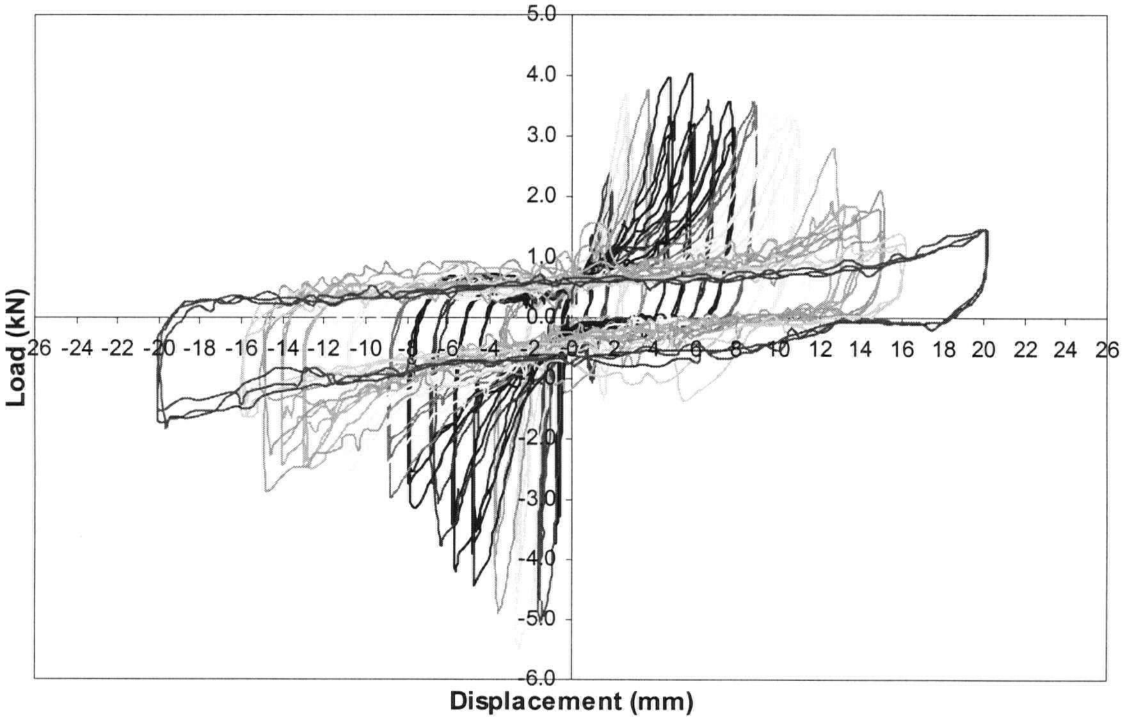
- The test was conducted with a surcharge load on top of the specimen. Small cracks were formed at stage 2 with 2 mm displacement in both tension and compression.
- When the displacement was 4 mm i.e. stage 4, spalling occurred on the tie face in tension, the spalled mortar was located on the area of the embedded leg. In compression more cracks were formed and previous cracks were getting bigger.
- At stage 8 with 8 mm displacement, in tension pull-out of mortar joint was evident on the tie face, while in compression push-through of the crushed mortar joint started to appear.
- The effect of surcharge load was evident in providing a clamping effect on the tie. As the test reached 12 mm displacement, all the mortar joint in each side were completely loose from the effect of pulling out and pushing through the tie from the brick panel. With the removal of the crushed mortar, the gap provided a closer observation on the mechanical connection (the clips) of the tie and the wire joint reinforcement. It was evident that both clips were still providing the connection between the tie and the horizontal wire reinforcement.
- Test was continued with 13, 14, 15, then to 16 mm and directly to 20 mm and to the final displacement of 25 mm. It was observed that in 16 mm displacement, one of the clips was disengaged at tension then fell off to one of the core hole of the brick unit, thus only one side was providing the resistance at tension. On higher displacement that is 20 and 25 mm, the leg of the tie without the clip was overriding the wire joint reinforcement in compression. While the other side with clip still giving resistance.

- Further observations included opening up the bed joint where the tie and the wire was located. It was discovered that the wire deformed in unsymmetrical manner, where the side without the clip deformed to tension position, while the one with clip deformed to compression position or almost back to its initial position (this was due to the cycle of loading which at the end of the cycle the actuator always move back to its initial position). The observation picture below will show this mode of deformation on the horizontal wire reinforcement.
- The unsymmetrical deformation was due to the mechanical connection that was lost during the larger displacement, so the tie only overrode the wire in compression on one side, which the clip had disengaged.

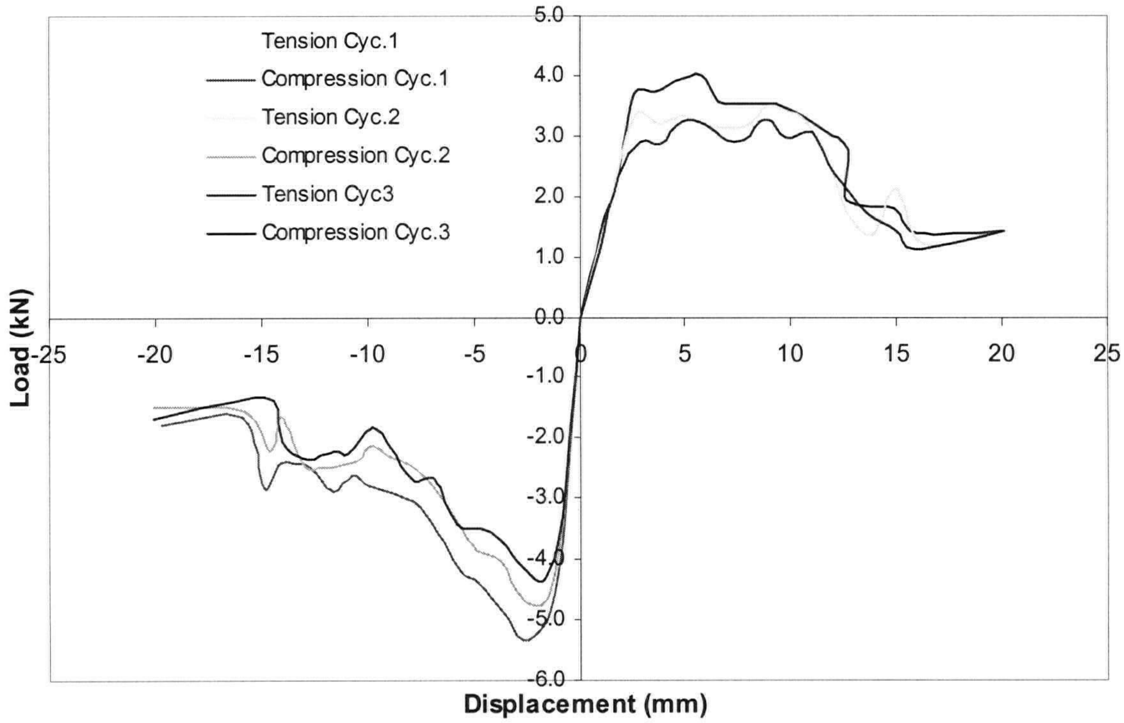


Specimen	TWC5
Characteristics	V-Tie with horizontal wire reinforcement clipped
	V-Tie length 80 mm
	Type S mortar
Test Date (age)	February 28th, 2001 (70 days)
Surcharge Load	60 kPa
Maximum Force	
Tension	3.99 kN
Compression	5.33 kN
Displacement at Maximum Force	
Tension	5.78 mm
Compression	2.74 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship



Load-Displacement Envelope Curve

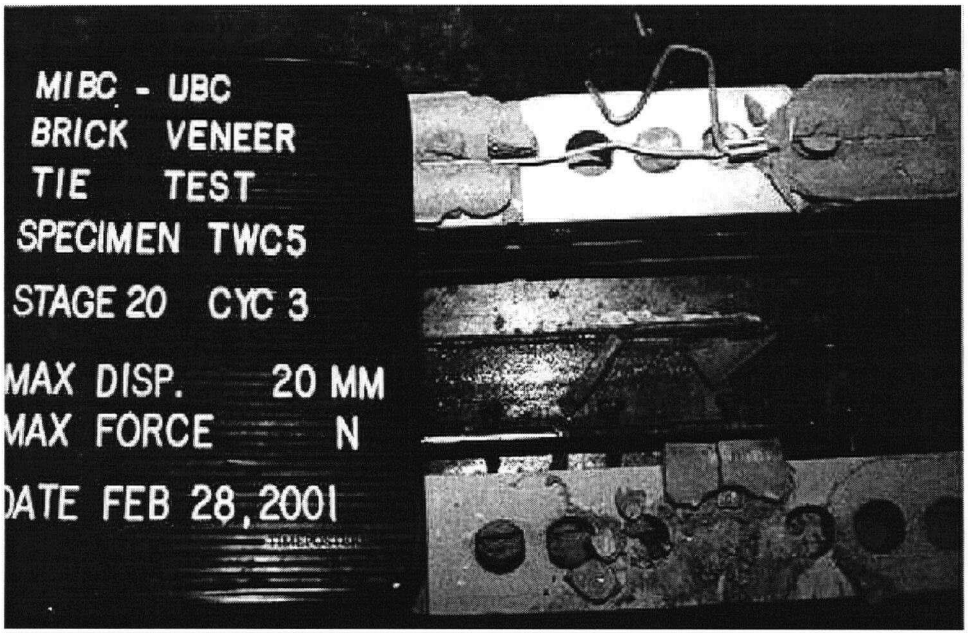


Description of Test Observations:

- At 2 mm displacement at stage 2, small cracks started to form both in tension and compression. The cracks were concentrated in the location of the tie.
- At 3 mm displacement, a crack propagated into the brick on the tooled joint face in compression, more cracks started to open and there was an evidence of ladder type of cracks in compression from the location of tie to the top of the brick panel.
- With 4 mm displacement, pull-out of mortars from the bed joint became more apparent on tension at the tie face, while in compression push-through of mortars from the bed joint was also more apparent. More cracks were formed at this stage of loading.
- At 9mm displacement, crushed mortars at the tie face in tension spalled and left a gap, which through this gap the clip could be observed. It was evident that the clip was still intact at this displacement. The effect of surcharge load in providing a clamping effect on the tie was evident at this stage.
- When the test reached 12 mm displacement from close observation of the clip (it was possible from the gap), the tie was overriding on top of the wire in compression with the clip was still holding both the wire and the tie, this was a proof that the space/gap in the clip between the tie and the horizontal wire reinforcement was also contribute to the integrity of the attachment between the tie and the wire joint reinforcement.
- The test then continued with 13, 14, 15 and 16 mm displacement, then it went directly to 20 mm for the final displacement. While it reached the 16 mm displacement, it was observed that on one leg of the tie, the clip started to loose, hit the brick in compression. This was happening because the tie after it overrides

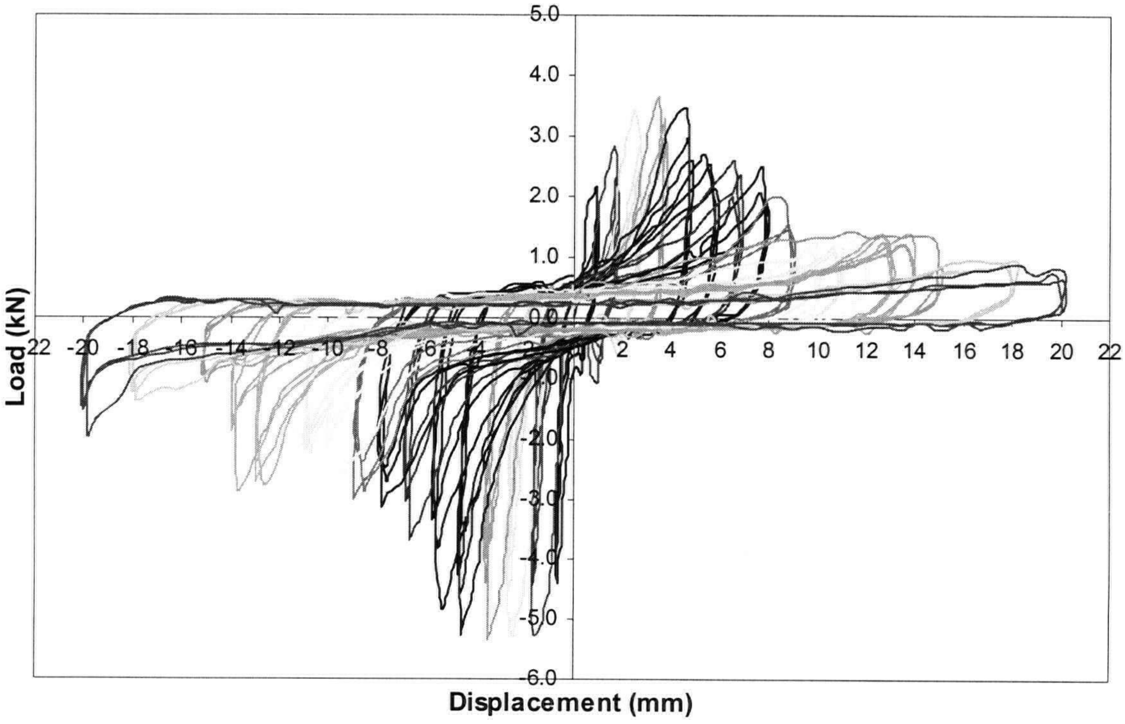
the wire was being pressed between the clip and the brick. This occurrence happened in one cycle and then followed by the clip fell into the core.

- At 20 mm displacement it was observed that one of the clip was disengaged from the tie and the horizontal wire reinforcement. This made it possible for the tie to overrides the wire reinforcement and actually hit the brick in compression.
- After the final displacement reached (i.e. 20 mm), close observation was performed by opening up the bed joint where the tie was located. It showed that one leg was deformed in compression side; this was the side, which the clip was detached and failed to maintain the connection. The deformation of one leg of the tie clearly shows that in compression, this leg was hitting the brick and thus gives more resistance in compression while it was deforming itself.

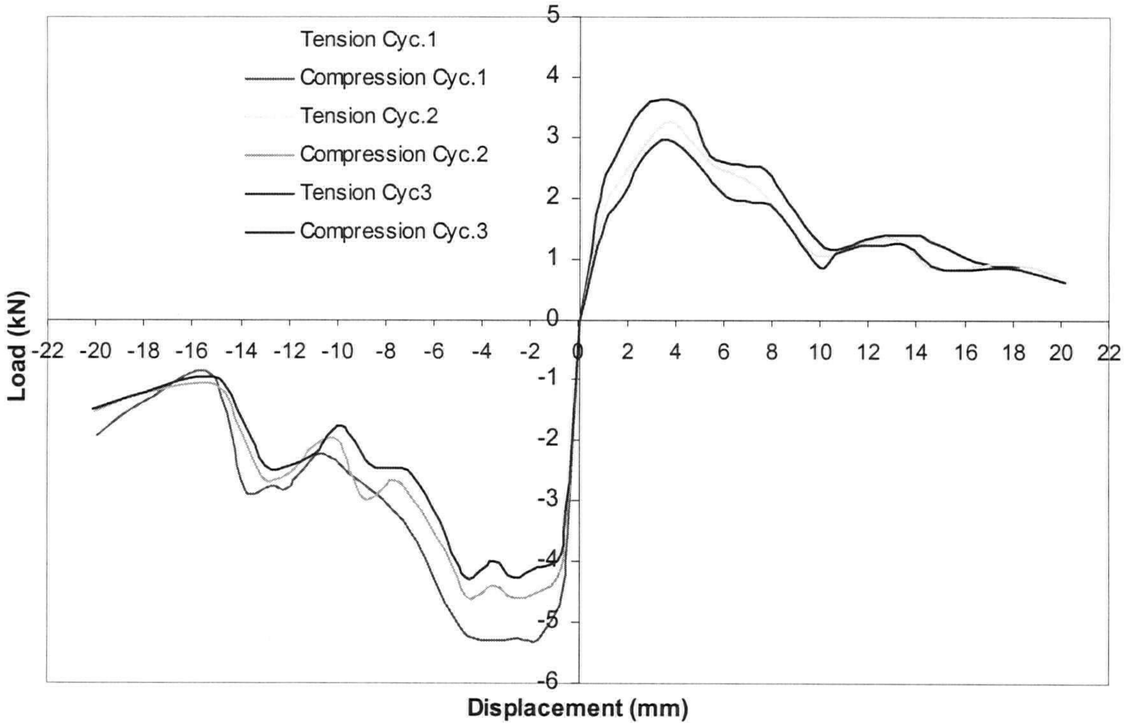


Specimen	TWC6
Characteristics	V-Tie with horizontal wire reinforcement clipped
	V-Tie length 80 mm
	Type S mortar
Test Date (age)	March 8th, 2001 (78 days)
Surcharge Load	60 kPa
Maximum Force	
Tension	3.64 kN
Compression	5.31 kN
Displacement at Maximum Force	
Tension	3.31 mm
Compression	3.32 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship



Load-Displacement Envelope Curve



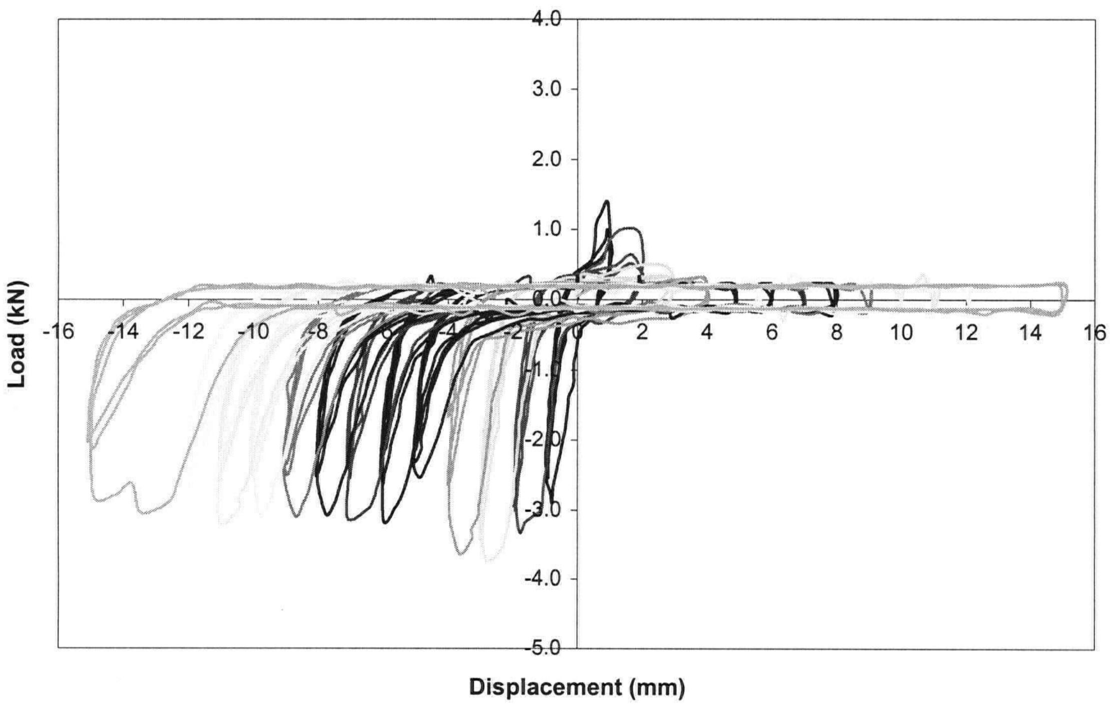
Description of Test Observations:

- At 2 mm displacement cracks started to form both in tension and compression at tie face and the tooled joint face. The cracks were concentrated on the location where the tie was embedded.
- At 4 mm displacement more cracks were formed.
- Spalling occurred at 6 mm displacement in tension on the tie face, it was observed that pull-out of crushed mortars from bed joint was started and also on compression push-through of mortars from bed joint in the vicinity of the tie was evident.
- Surcharge load provided a clamping effect on the tie; this was evident by observing the spalling of the crushed mortar joint, which occurred in larger displacement. At 11 mm, the mortar bed joint in the location of the tie was very loose because the movement of the tie already crushed them. Removing the crushed mortar leaves a gap where the tie was embedded in, that enabled a closer look at the clip at that side. It was observed that both clips are still intact providing the connection between the tie and the horizontal wire joint reinforcement.
- At 12 mm displacement, in tension, one of the clips was disengaged and fell into the core. Thus in compression, the overrides effect of the tie against the wire reinforcement was occurring.
- The test was continued with 13, 14 and 15 mm displacement then it went directly to 18 mm and to the final displacement which was 20 mm. At 15 mm displacement it was evident that the tie was overriding the horizontal wire

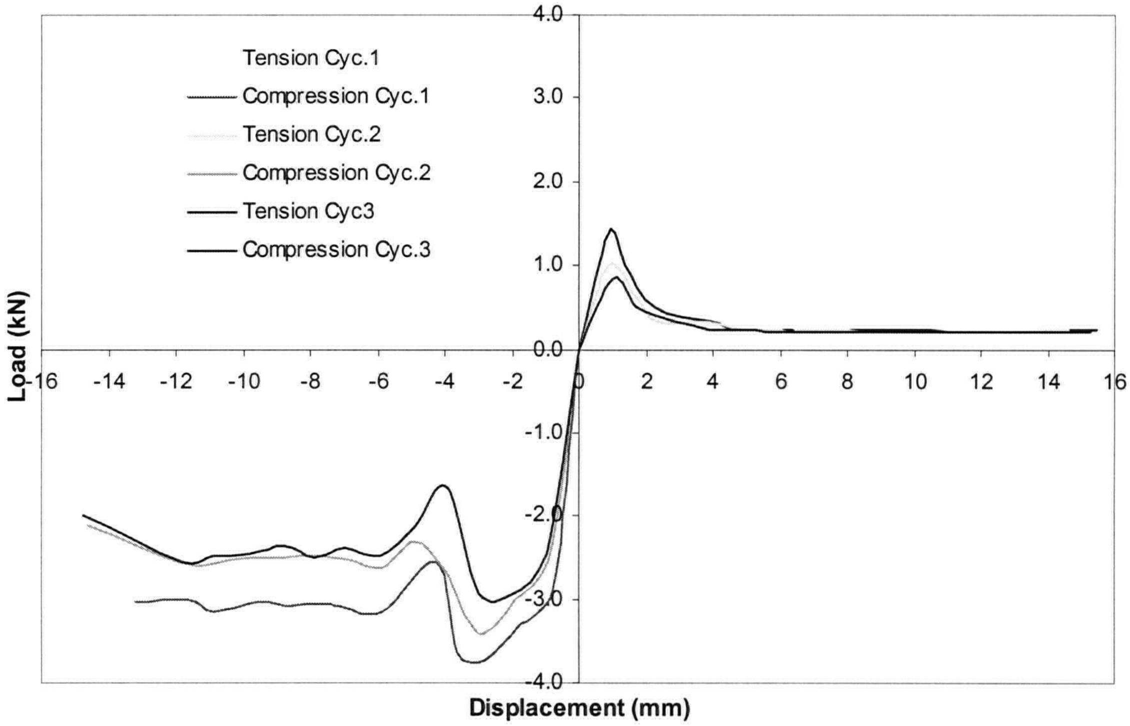
reinforcement and got pressed between the brick unit and the wire joint reinforcement, and finally the clip fell into the core.

Specimen	OT
Characteristics	V-Tie only embedded at 19 mm (3/4") from centre V-Tie length 80 mm Type S mortar
Test Date (age)	April 11th, 2001 (111 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	1.40 kN
Compression	3.73 kN
Displacement at Maximum Force	
Tension	0.91 mm
Compression	2.82 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Not Applicable

Load-Displacement Relationship

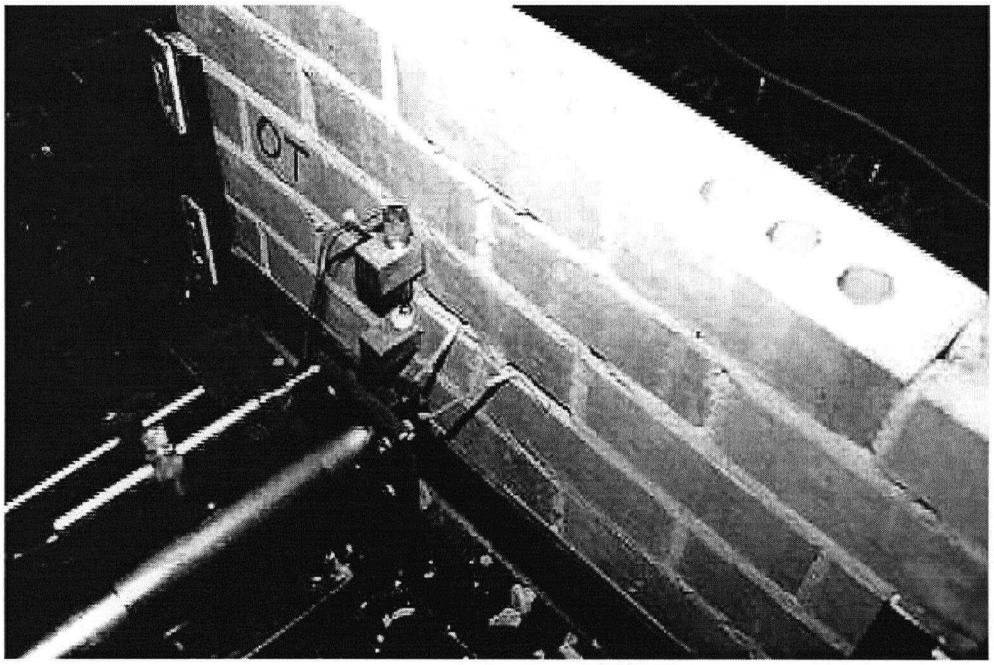


Load Displacement Envelope Curve



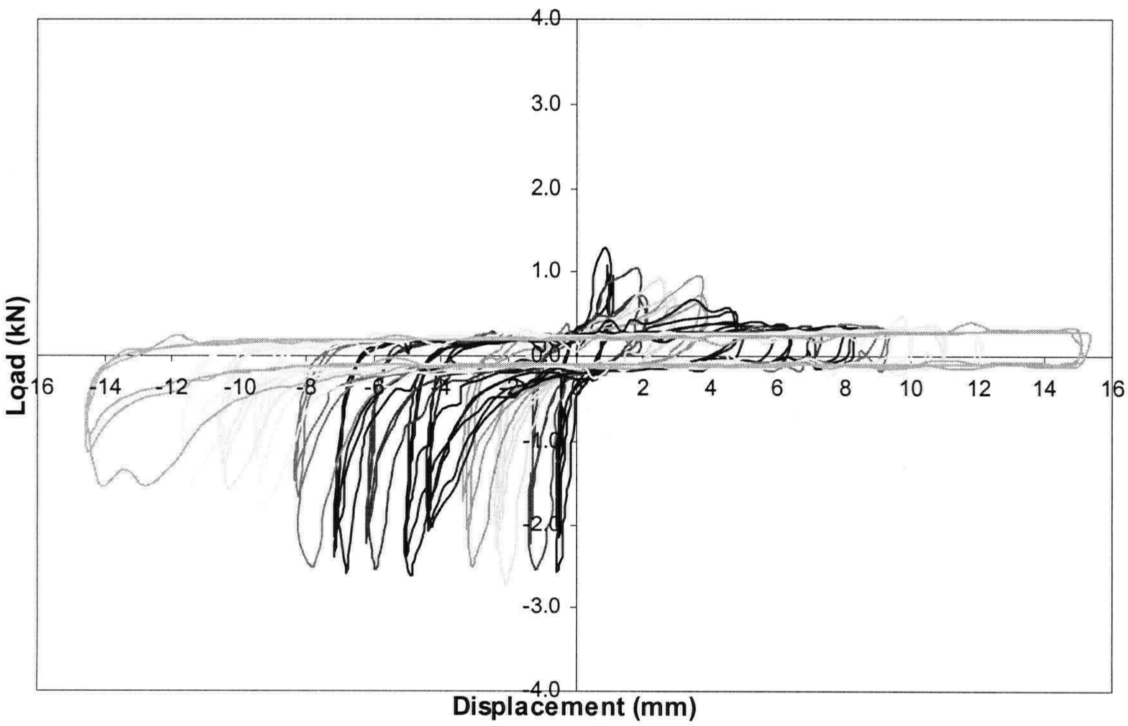
Description of test observation:

- At 2 mm displacement, cracks started to form in tension at the tie face, while in compression no cracks were formed.
- At 3 mm displacement in tension, pieces of loosely crushed mortar started to spall, this was due to the location of embedment of the tie. While in compression at the tooled joint face a ladder crack formed from the corner of top course of brick to the centre where the tie was located.
- At 6 mm displacement, in tension at the tie face, all pieces of crushed mortar were pulled out from the bed joint and spalled. While in compression, the bricks started to move along the ladder crack that was formed earlier.
- At 9 mm, in tension almost no resistance at all, the embedment strength had fail, all the mortar around the area of tie were crushed and spalled. In compression the tie could not crushed any more mortar, the tie just pushed against the mortar bed joint that still intact and relatively pushed the whole brick that got separated by the ladder crack. This was no means of a failure in compression.
- Test was stopped at 15 mm, and visual observation was conducted by opening up the mortar bed joint where the tie was embedded.
- From the observation, it was recognized that area of mortar being crushed was only in tension or at the tie face side. However, in compression, the tie deformed the legs towards the compression side.

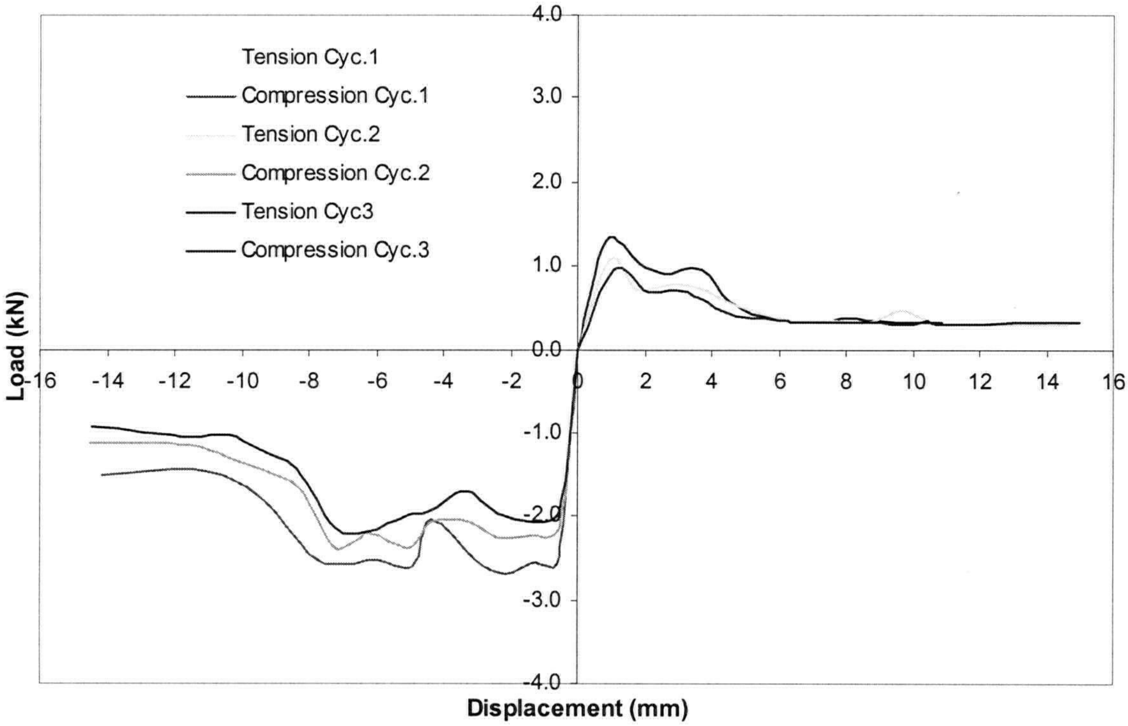

OT

Specimen	OTWC
Characteristics	V-Tie with horizontal wire reinforcement clipped embedded at 19 mm (3/4") from centre V-Tie length 80 mm Type S mortar
Test Date (age)	April 12th, 2001 (112 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	1.28 kN
Compression	2.70 kN
Displacement at Maximum Force	
Tension	0.86 mm
Compression	2.12 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship



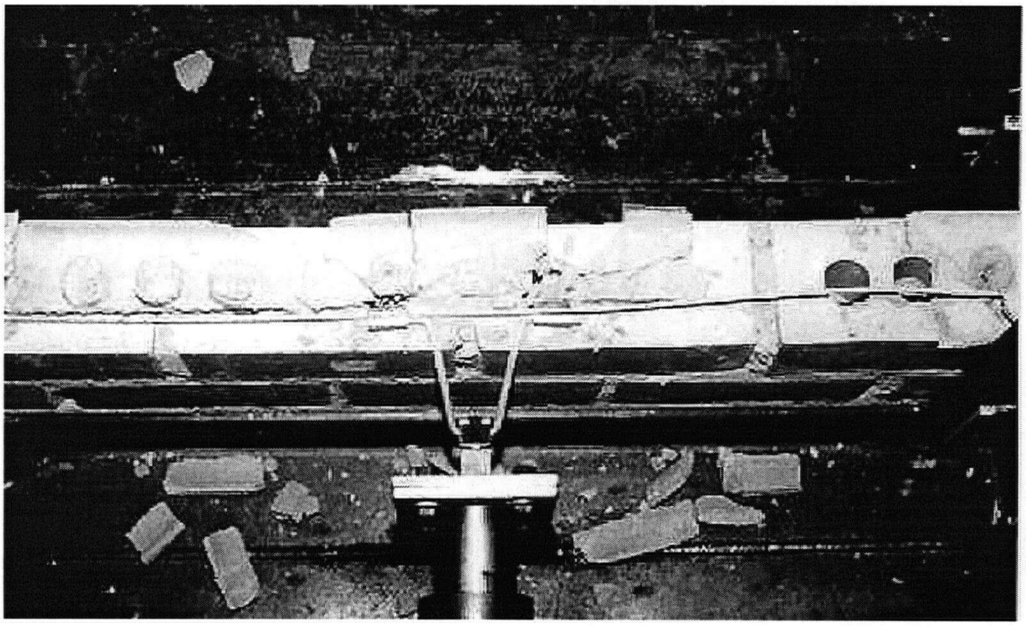
Load Displacement Envelope Curve



Description of test observation:

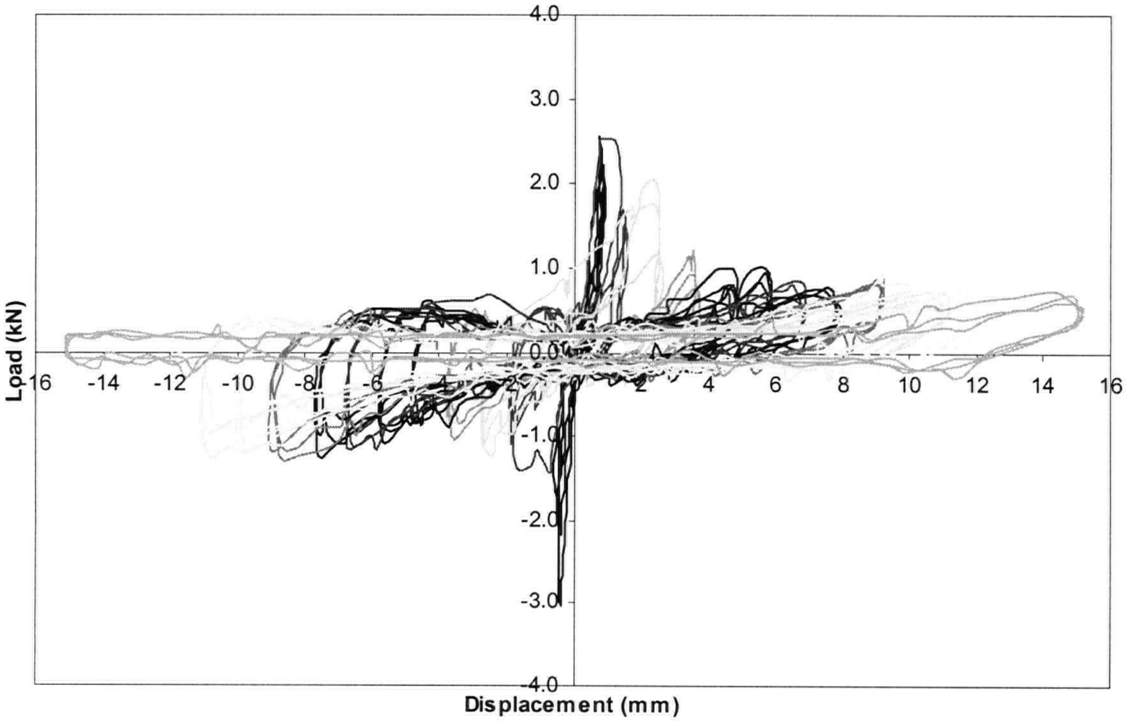
- Cracks started at 2 mm displacement in tension at the tie face, evidence of the tie engaged the horizontal wire reinforcement by observing the area of mortars cracked and starting to loosely pulled out from the bed joint. In compression small cracks formed in the location of the embedment.
- At 3 mm displacement, in tension several cracks occurred forming pieces of loosely crushed mortar at the tie face. While in compression ladder crack appeared from top course of bricks to the centre where the embedment located. This ladder cracks was caused by punching action of the tie on compression.
- At 5 mm displacement, in tension all the loose crushed mortars were spalled leaving a hole in the bed joint. Observed from this hole, the clips were effectively engaged the horizontal wire reinforcement in tension. In compression several horizontal cracks along the bed joint were formed. There was still no indication of push through mortar pieces from the bed joint.
- At 9 mm displacement, more crushed mortar were engaged in tension, resulting in a pull-out mortar from bed joint. In compression another ladder crack formed on from the other top corner of the bricks to the centre. This clearly indicated that the tie load in compression gave a point load and a punching shear occurred. A small portion of loose crushed mortars were push through from the bed joint.
- Test was stopped at 15 mm displacement and observation was conducted by opening up the bed joint where the embedded tie located.
- It was discovered that the horizontal wire joint reinforcement helped by extending the area of the mortar that being crushed at the tie face in tension. The horizontal

wire joint reinforcement was also deformed. In the compression side, pieces of crushed mortar were pushed through the bed joint by the clipped tie horizontal wire joint reinforcement.

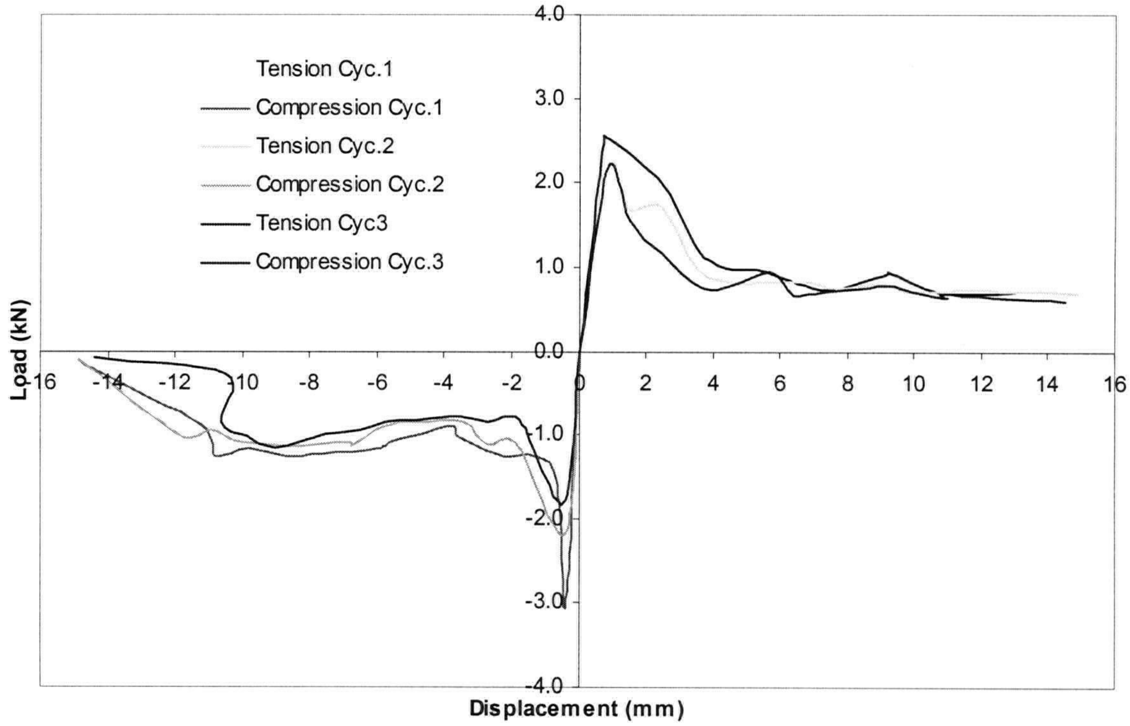


Specimen	TT1
Characteristics	Triangular Tie – 100 mm length Type S mortar
Test Date (age)	April 12th, 2001 (112 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	2.55 kN
Compression	3.03 kN
Displacement at Maximum Force	
Tension	0.75 mm
Compression	0.39 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship

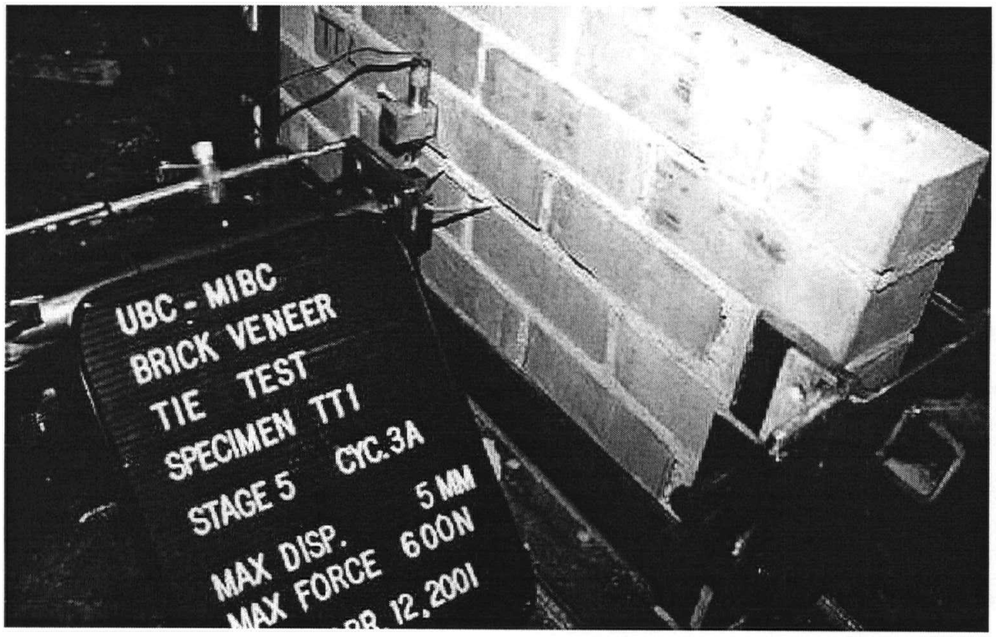


Load Displacement Envelope Curve



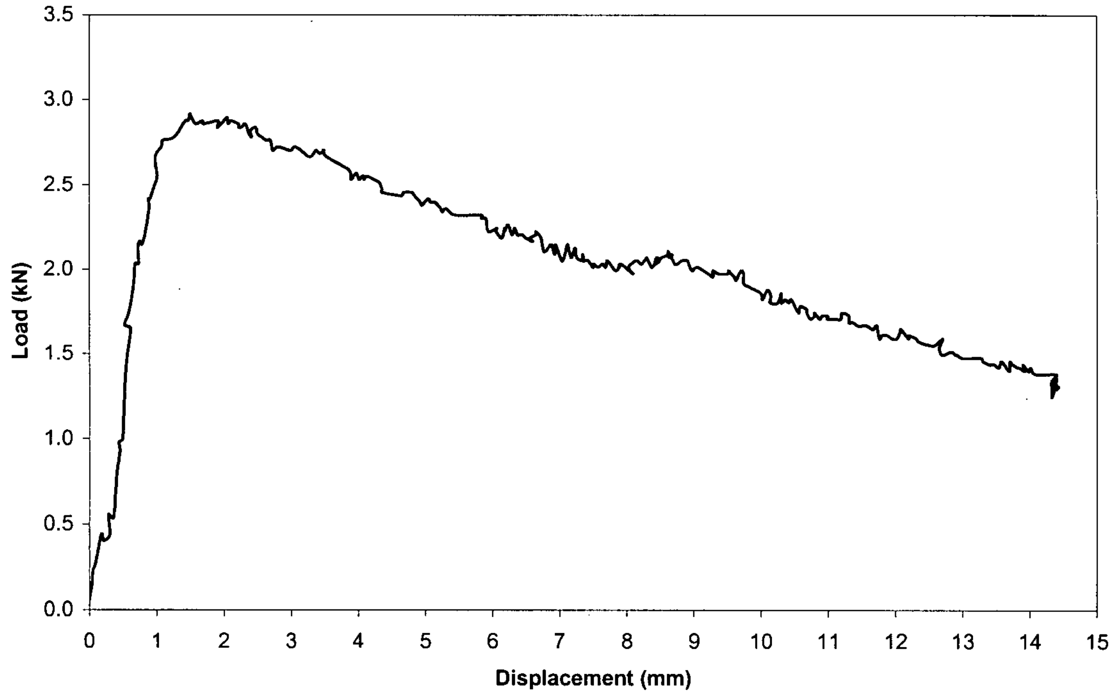
Description of test observation:

- At 2mm displacement in tension and compression, cracks started to occur in the embedded tie location.
- At 3 mm displacement, in tension, several pieces of loosely crushed mortar were formed and started to pull-out. In compression also a large piece of crushed mortar joint was formed and started to push out from the bed joint.
- With 4 mm displacement, the loose pieces of crushed mortar were spalled in tension and also the same thing happened in compression side.
- In 5 mm displacement, several vertical cracks were formed in tension and compression.
- Test was continued until 12 mm displacement and directly went to 15 mm displacement as the final target displacement.
- There was no observation by opening the bed joint for this specimen.


UBC - MIBC
BRICK VENEER
TIE TEST
SPECIMEN TT1
STAGE 5 CYC.3A
MAX DISP. 5 MM
MAX FORCE 600N

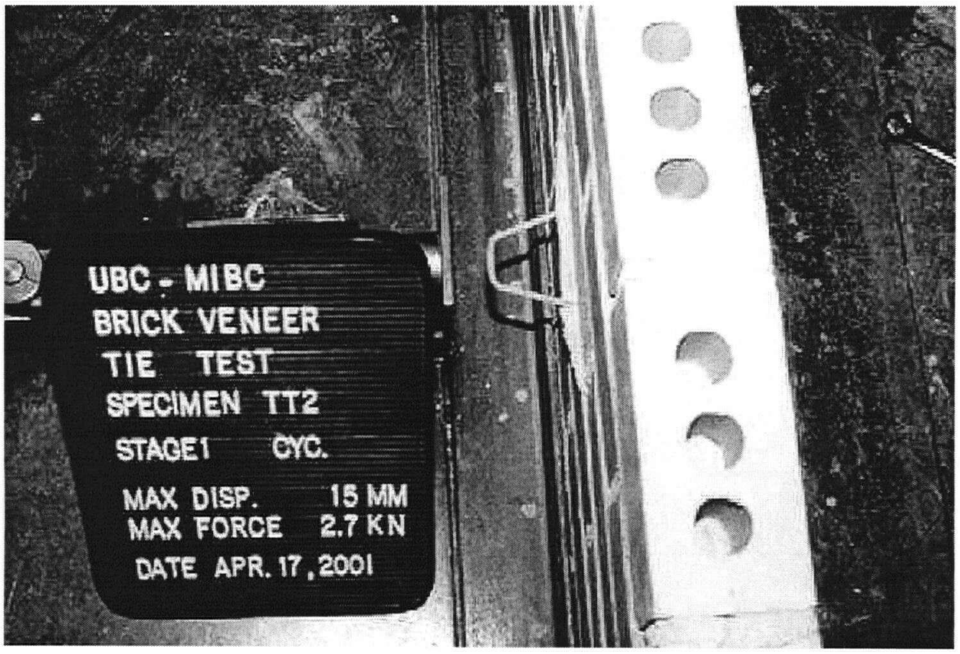
Specimen	TT2 - Monotonic
Characteristics	Triangular Tie – 100 mm length Type S mortar
Test Date (age)	April 17th, 2001 (117 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	2.90 kN
Displacement at Maximum Force	
Tension	2.04 mm
Failure modes	
Tension	Pull-out from mortar bed joint

Load-Displacement Relationship



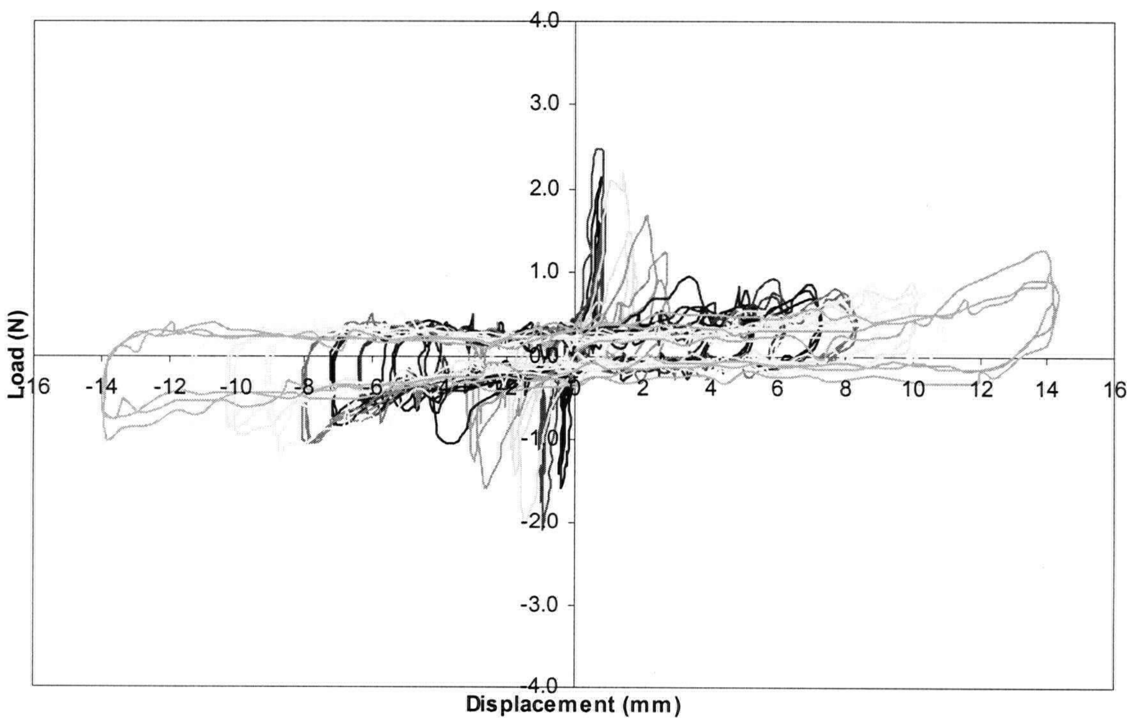
Description of test observation:

- Test was done by monotonic pull-out in tension with 15 mm as the target displacement level.
- Mode of failure was pull-out of tie from mortar bed joint with failure of the bond between the mortar and the bricks.

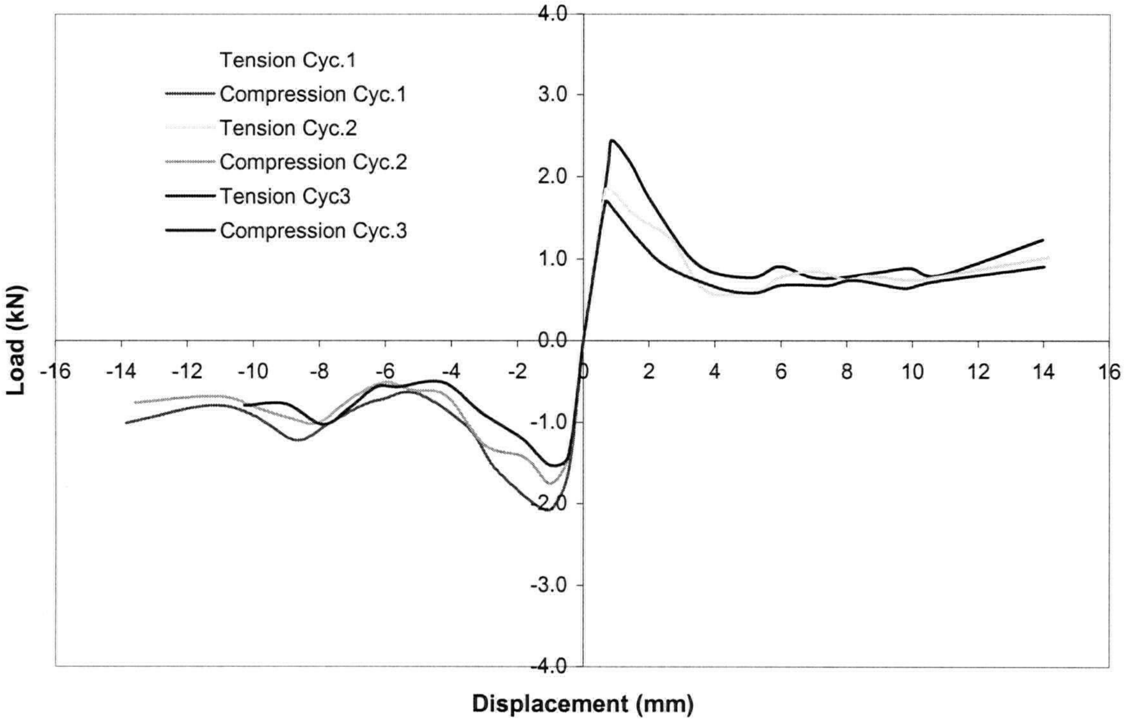

UBC - MIBC
BRICK VENEER
TIE TEST
SPECIMEN TT2
STAGE1 CYC.
MAX DISP. 15 MM
MAX FORCE 2.7 KN
DATE APR. 17, 2001

Specimen	S1
Characteristics	Dur-o-Wall SMP 11 plate with horizontal wire reinforcement Type S mortar
Test Date (age)	April 18th, 2001 (118 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	2.44 kN
Compression	2.06 kN
Displacement at Maximum Force	
Tension	0.85 mm
Compression	0.95 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship

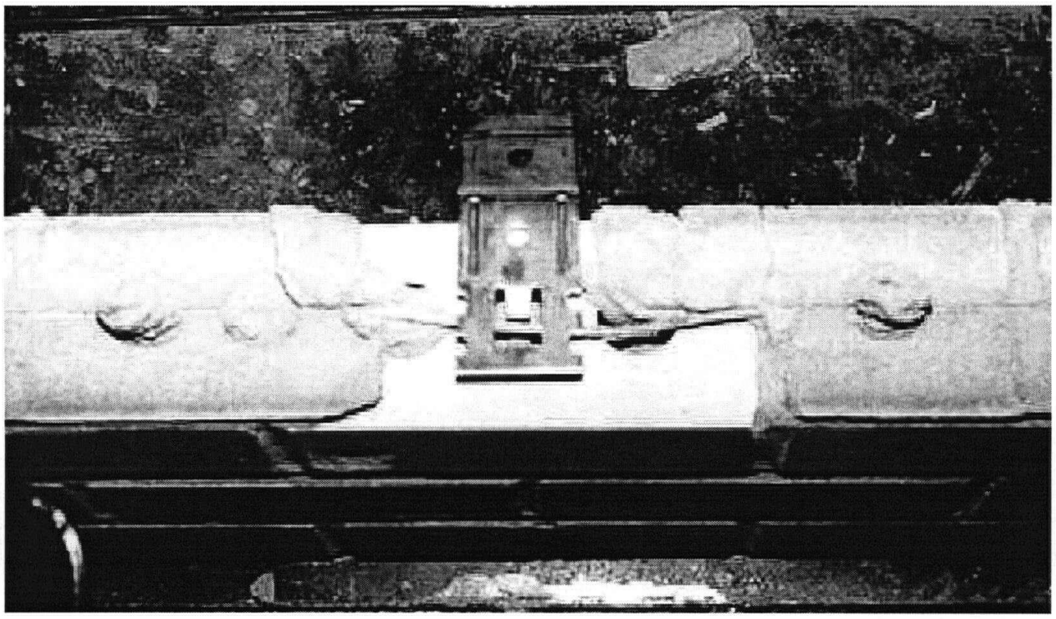


Load Displacement Envelope Curve



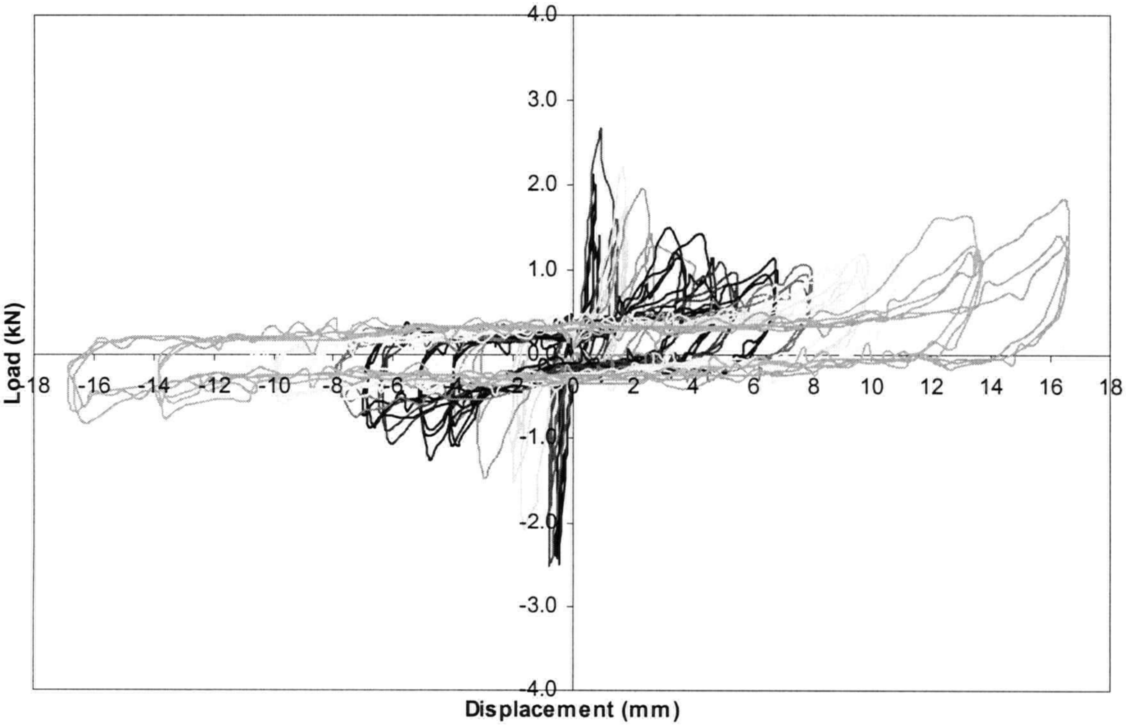
Description of test observation:

- At 2 mm displacement, in tension cracks started to occur in the area of the embedded plate. The very rigid plate crushed the mortar bed joint area on top of the plate and formed a rectangular piece of crushed mortar with a width equal to the width of the plate. In compression, same situation occurred. The plate crushed the mortar joint and formed a loose rectangular piece of mortar with an equal width as of the plate.
- At 4 mm displacement, in tension and compression, the crushed mortar was loosely pulled-out and push through the bed joint along with the movement of the plate. Several vertical cracks were formed in compression but not in tension at this level of displacement.
- At 7 mm displacement, both pieces of the loose crushed mortar were spalled.
- Test was continued with a final displacement of 18 mm. Visual observation was conducted by opening up the bed joint where the plate was embedded.
- The observation revealed that the area of crushed mortar was entirely located around the plate and the wire did not actually helped to extend the area of crushing. However, the wire was deformed, showing that the plate did engage the wire joint reinforcement. The lips or hooks of the plate, which intended to engage the wire reinforcement, had a big gap or space before it can actually engage the wire joint reinforcement.

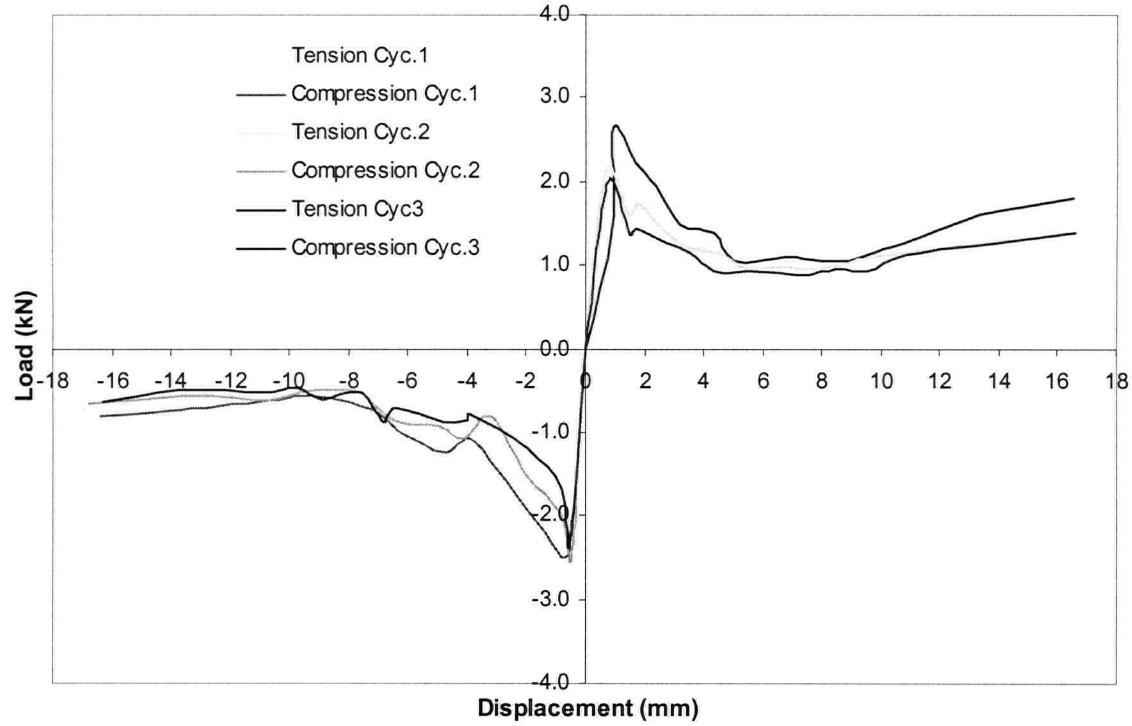


Specimen	S2
Characteristics	Dur-o-Wall SMP 11 plate with horizontal wire reinforcement Type S mortar
Test Date (age)	April 18th, 2001 (118 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	2.65 kN
Compression	2.50 kN
Displacement at Maximum Force	
Tension	0.94 mm
Compression	0.80 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship

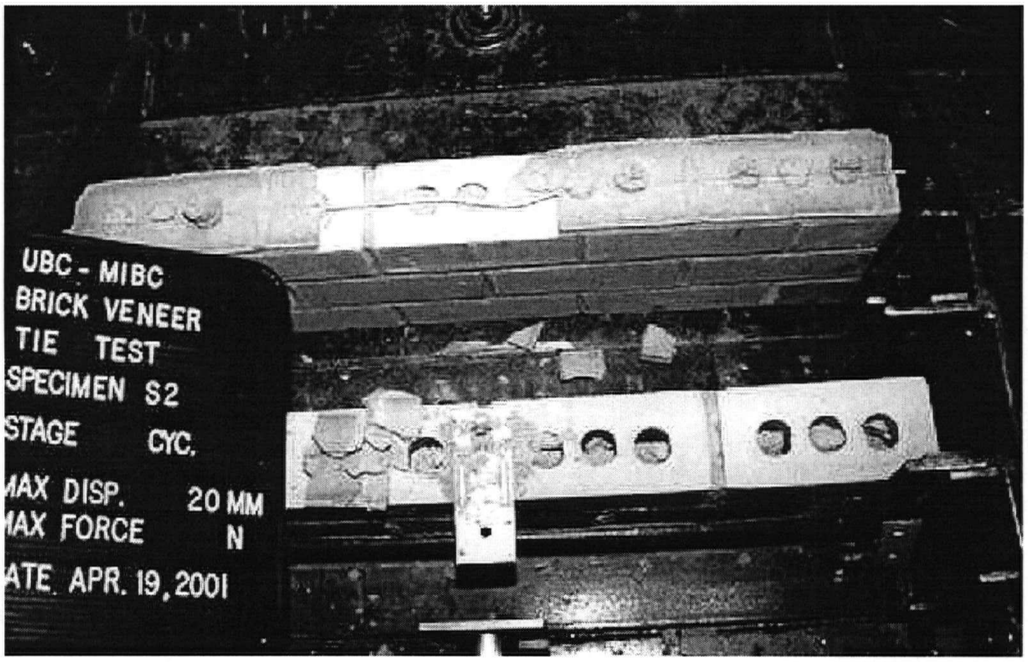


Load Displacement Envelope Curve



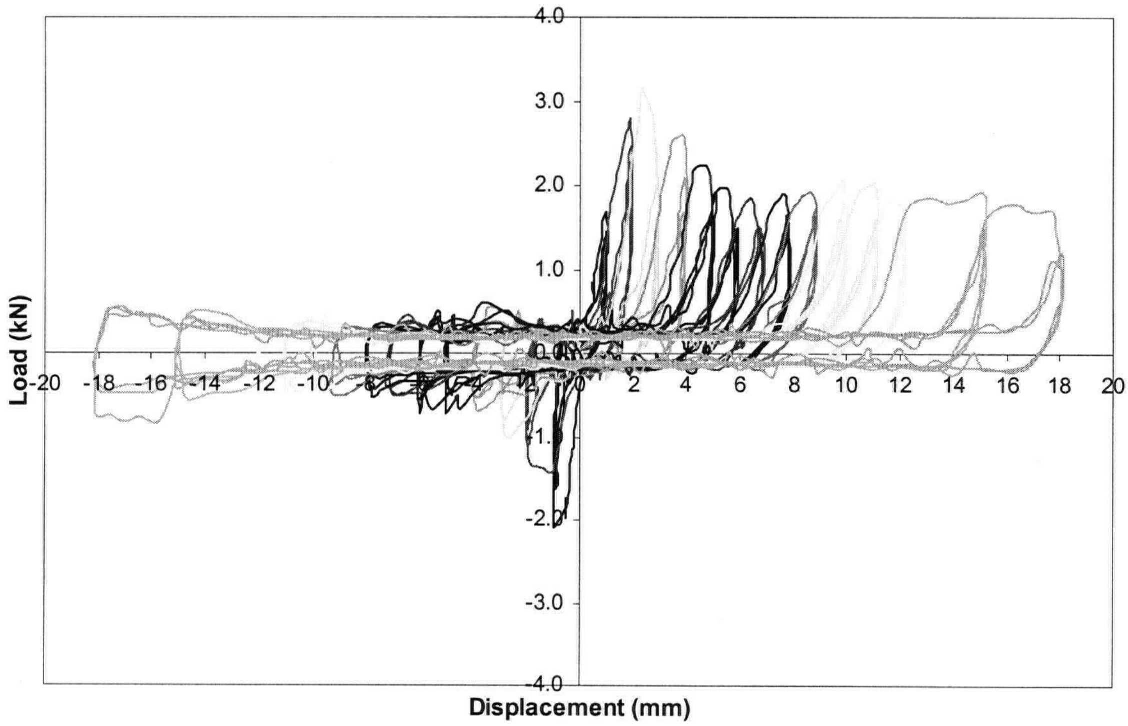
Description of test observation:

- At 2 mm displacement, in tension, cracks were occurred and formed a rectangular piece of loosely crushed mortar with an equal width as the plate. In compression the plate also crushed the mortar joint and formed a similar piece of loosely crushed mortars.
- At 4 mm displacement, the piece of crushed mortar was pulled out from the mortar bed joint in tension and pushed through in compression. More horizontal cracks were formed in compression.
- At 7 mm, both pieces of crushed mortar were spalled in tension and compression.
- At 9 mm displacement, another piece of crushed mortar spalled in compression at the tooled joint face.
- Test was continued and the final displacement reached was 20 mm.
- Visual observation was conducted by opening up the bed joint of the embedded plate. It was revealed that most of the mortar crushed by the stiff plate were only located at the area of the embedded plate. Although the wire joint reinforcement deformed, but the engagement of the wire did not seem to extend the area of the crushed mortar in the bed joint.

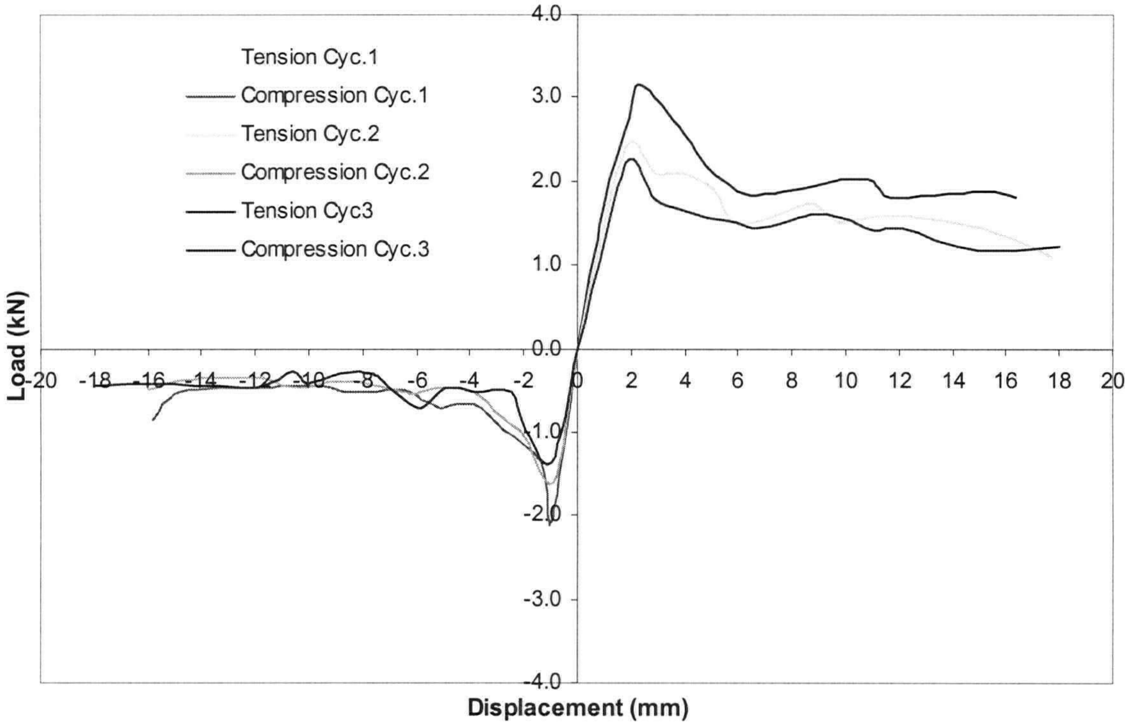

UBC - MIBC
BRICK VENEER
TIE TEST
SPECIMEN S2
STAGE CYC.
MAX DISP. 20 MM
MAX FORCE N
ATE. APR. 19, 2001

Specimen	F1
Characteristics	5" Fleming Anchor with wire reinforcement Fleming Anchor length 130 mm (5 1/8") Type S mortar
Test Date (age)	April 19th, 2001 (119 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	3.15 kN
Compression	2.06 kN
Displacement at Maximum Force	
Tension	2.35 mm
Compression	0.95 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship

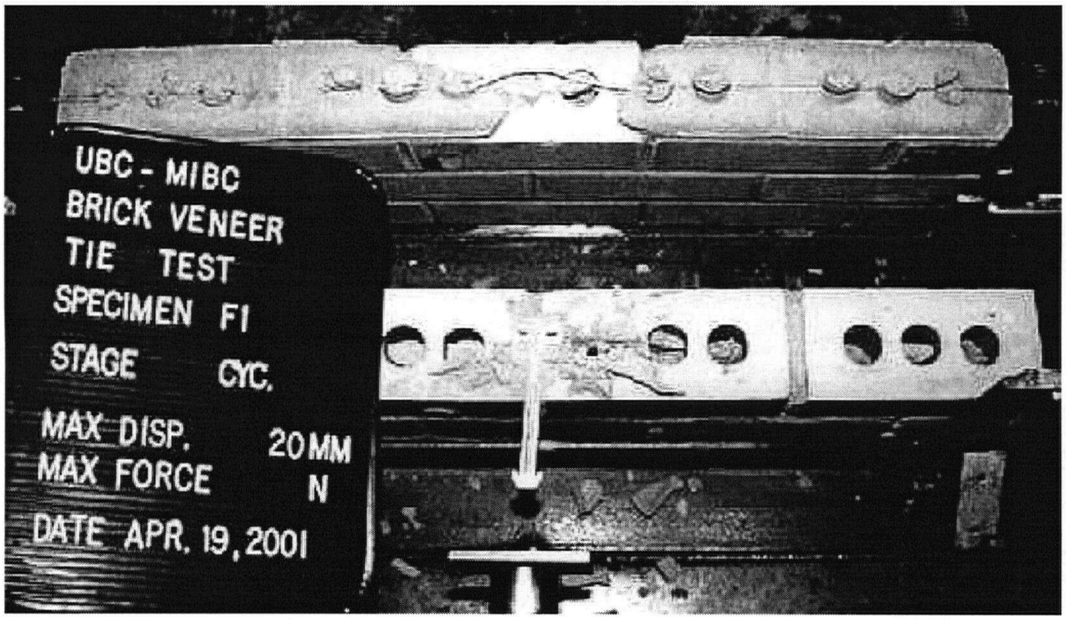


Load Displacement Envelope Curve



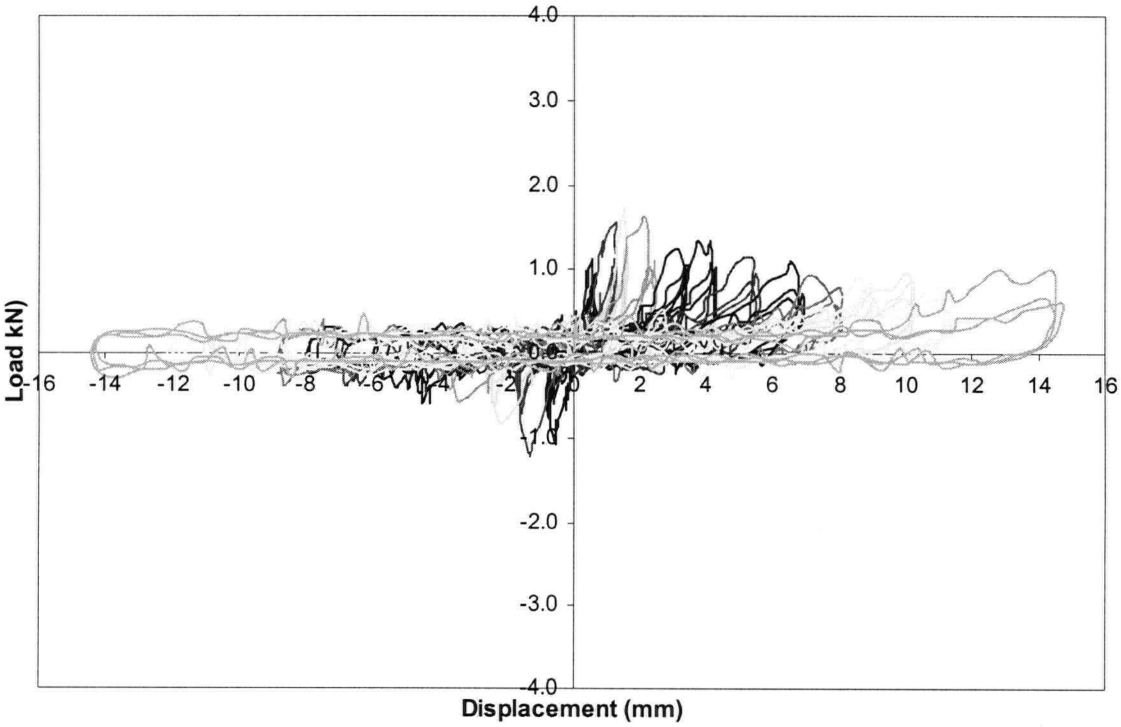
Description of test observation:

- At 2nd stage in compression, at the tooled joint face, cracks occurred and formed two separate pieces of very loose crushed mortar.
- Cracks started to form in 4 mm displacements at stage 4 in tension at the anchor face, there was an evidence of pull-out mortar joint with crushed mortars starting to loosely pulled out from the bed joint.
- At 6 mm displacement in tension, another crack formed at the tie face close to the previous crack located in the vicinity of Fleming Anchor. This crack eventually formed a piece of loose mortar that easily moved with the movement of the tie. This piece was actually moved by the movement of the horizontal wire joint reinforcement that was engaged by the anchor. In compression the crushed mortar became very loose and started to push-through the bed joint along with the movement of the anchor.
- At 9 mm displacement in tension, at the anchor face, the piece of loosely crushed mortar spalled. Test was stopped at 18 mm displacement.
- Observation by opening up the bed joint where the tie was located revealed that the horizontal wire joint reinforcement only deformed to one side, that is the tension side. This is due to the design of the Fleming Anchor, which will only engage the wire reinforcement in tension. While the crushed mortar showed an extended area more than one brick unit.

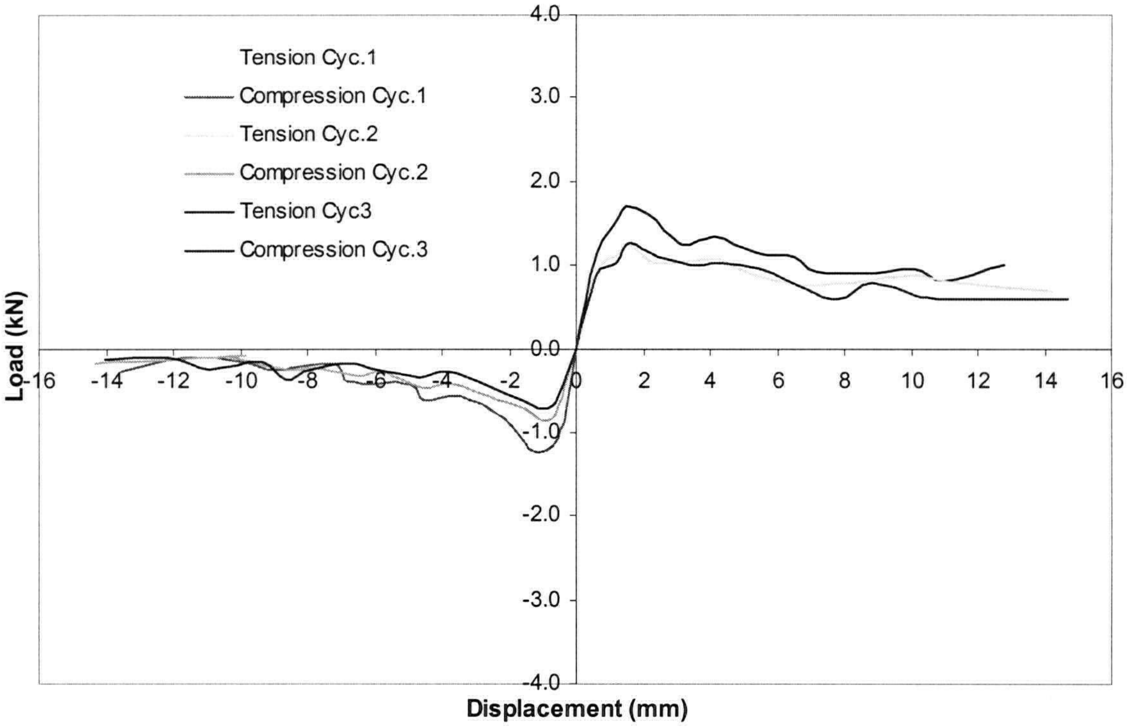

UBC - MIBC
BRICK VENEER
TIE TEST
SPECIMEN F1
STAGE CYC.
MAX DISP. 20MM
MAX FORCE N
DATE APR. 19, 2001

Specimen	F2
Characteristics	5" Fleming Anchor with wire reinforcement Fleming Anchor length 130 mm (5 1/8") Type S mortar
Test Date (age)	April 20th (120 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	1.71 kN
Compression	1.22 kN
Displacement at Maximum Force	
Tension	1.52 mm
Compression	1.32 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship

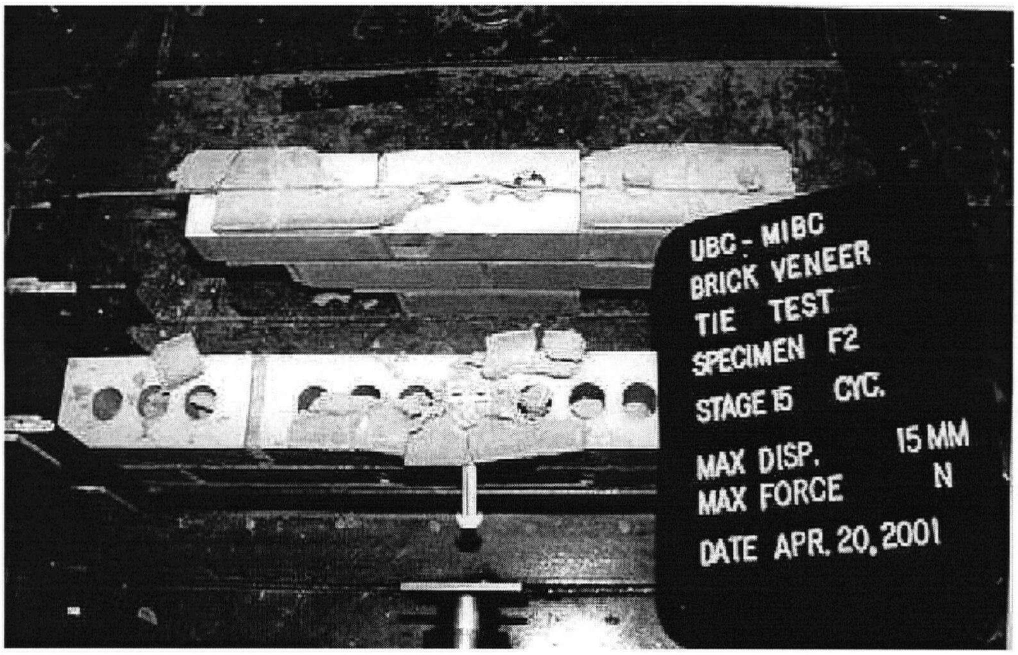


Load Displacement Envelope Curve



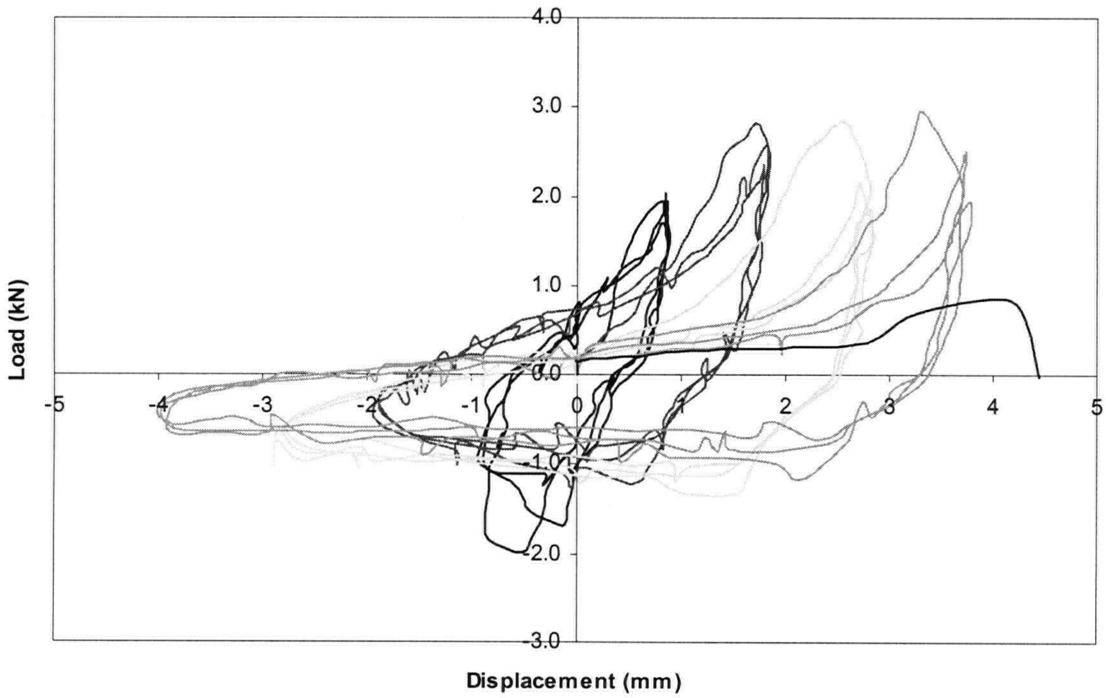
Description of test observation:

- The specimen was pre-cracked before the test was conducted, due to an installation failure.
- At 2 mm displacement, at the tooled joint face in compression, cracks were formed into separate pieces of loosely crushed mortar.
- At 3 mm displacement in tension at the anchor face, cracks occurred and pull-out of mortar joint became evident with the movement of the pieces of crushed mortar along with the anchor.
- At 6 mm displacement, all pieces of crushed mortar started to pull-out in tension or push through in compression of the bed joint. There was an indication of excessive number of loosely crushed mortar due to the pre-crack condition of the bed joint.
- With larger displacement the loosely crushed mortars moved along with the anchor, pull-out in tension and push-through in compression.
- Test was stopped at 15 mm displacement and observation was conducted.
- Observation by opening up the bed joint where the anchor was embedded showed that deformation of the horizontal wire joint reinforcement only to one side that is the tension side, due to the design of this Fleming Anchor. Several loosely crushed mortars in the area of the anchor indicated pre-cracking weaken the embedment strength of the anchor, in tension and compression.

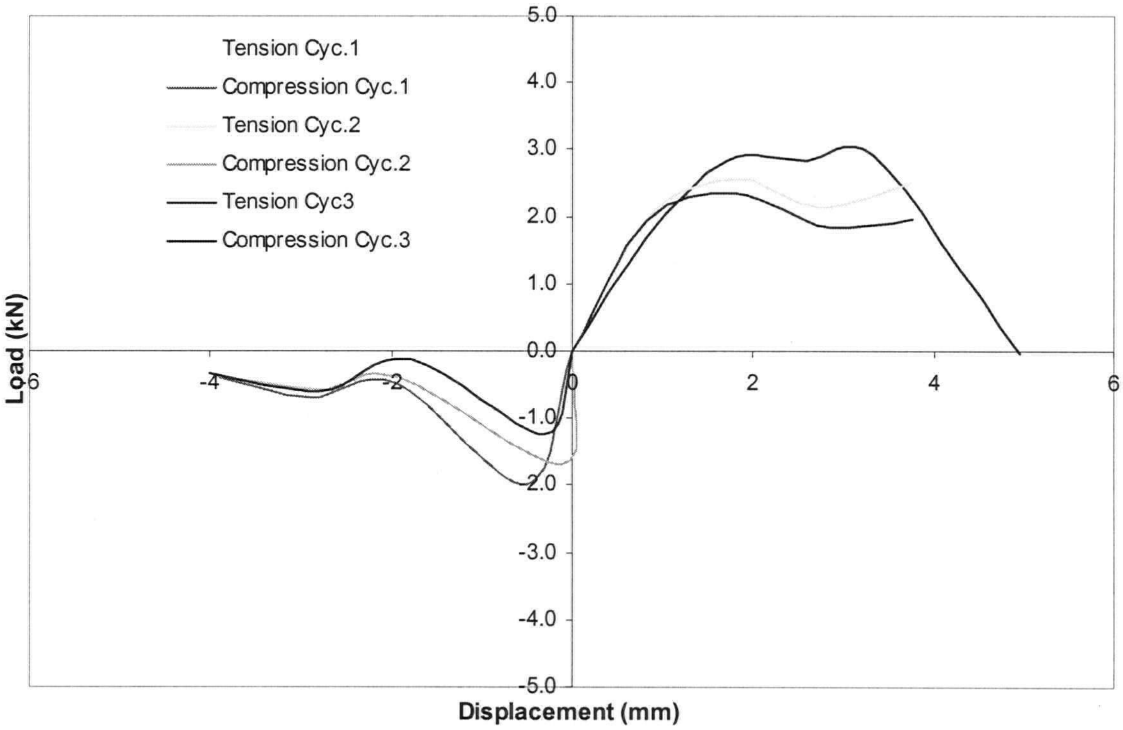

UBC - MIBC
BRICK VENEER
TIE TEST
SPECIMEN F2
STAGE 15 CYC.
MAX DISP. 15 MM
MAX FORCE N
DATE APR. 20, 2001

Specimen	C1
Characteristics	Corrugated Strip Tie 22 gauge Embedded length 64 mm (2.5") Type S mortar
Test Date (age)	April 20th, 2001 (120 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	2.93 kN
Compression	1.99 kN
Displacement at Maximum Force	
Tension	3.33 mm
Compression	0.5 mm
Failure modes	Buckled in compression and reach fatigue failure (material failure) in tension.

Load-Displacement Relationship

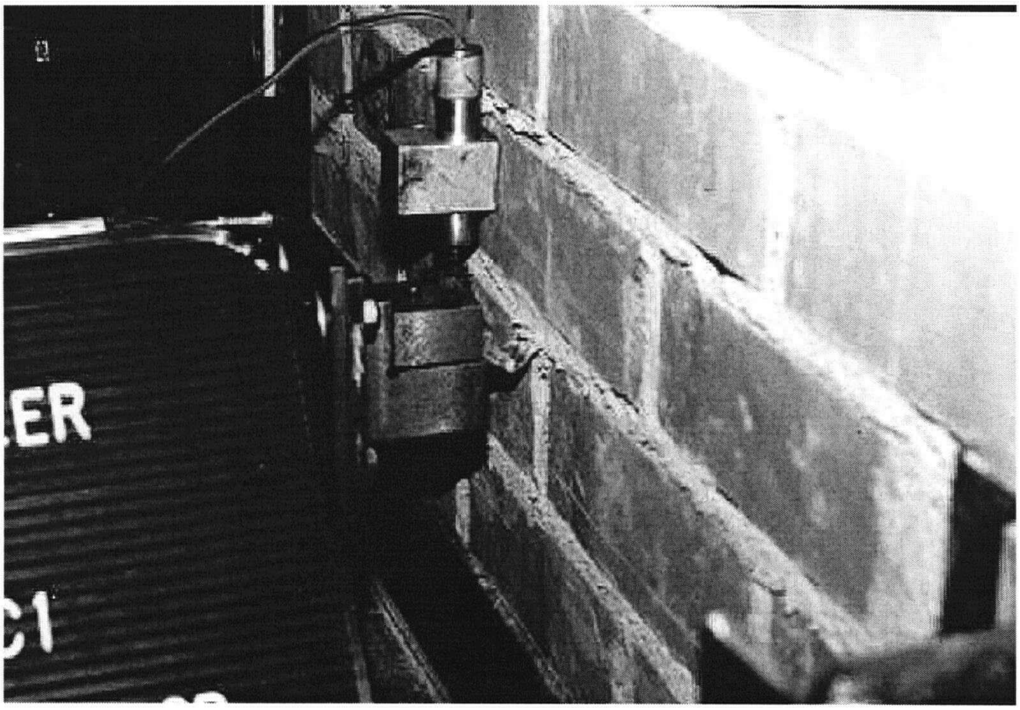


Load-Displacement Envelope Curve



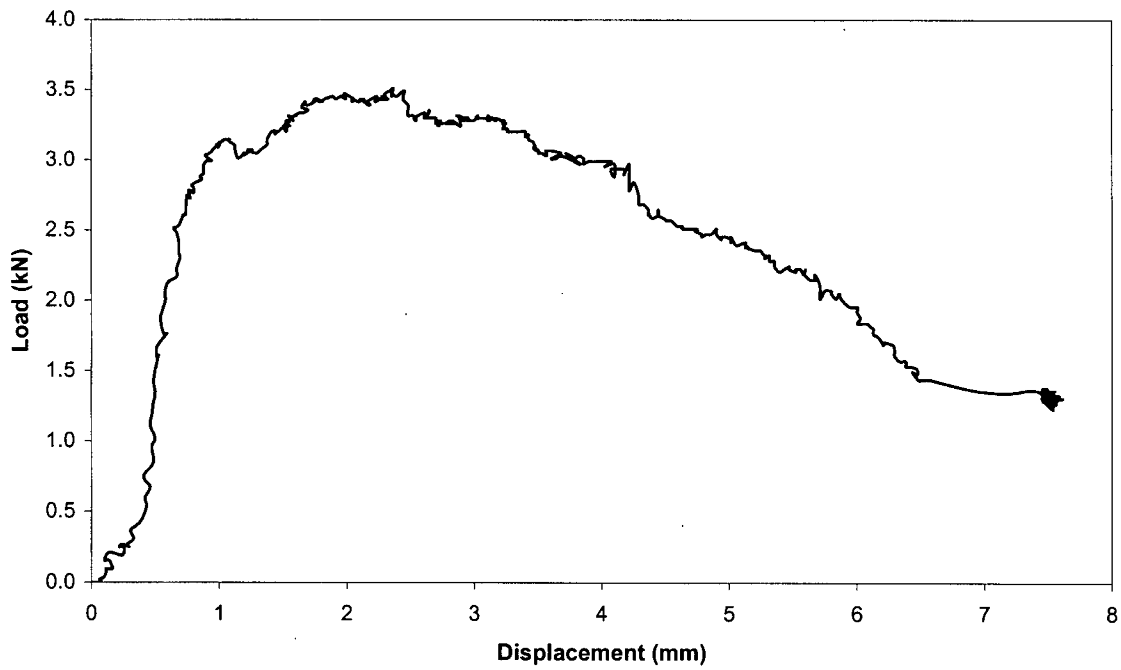
Description of Test Observations:

- At 2nd cycle in compression (2 mm displacement), the tie started to buckle without any cracks formed at the tooled joint side.
- In 3rd stage at 3 mm displacement in tension at the tie face, shows some cracks formed in the area of the tie.
- At 4 mm displacement in compression, the tie buckled and loss its resistance in compression without any cracks formed at the tooled joint side. In tension there was a sign of material fatigue on the tie because of the repeated loadings.
- At 5 mm displacement fatigue failure reached and the tie fractured into two parts.


ER
C1

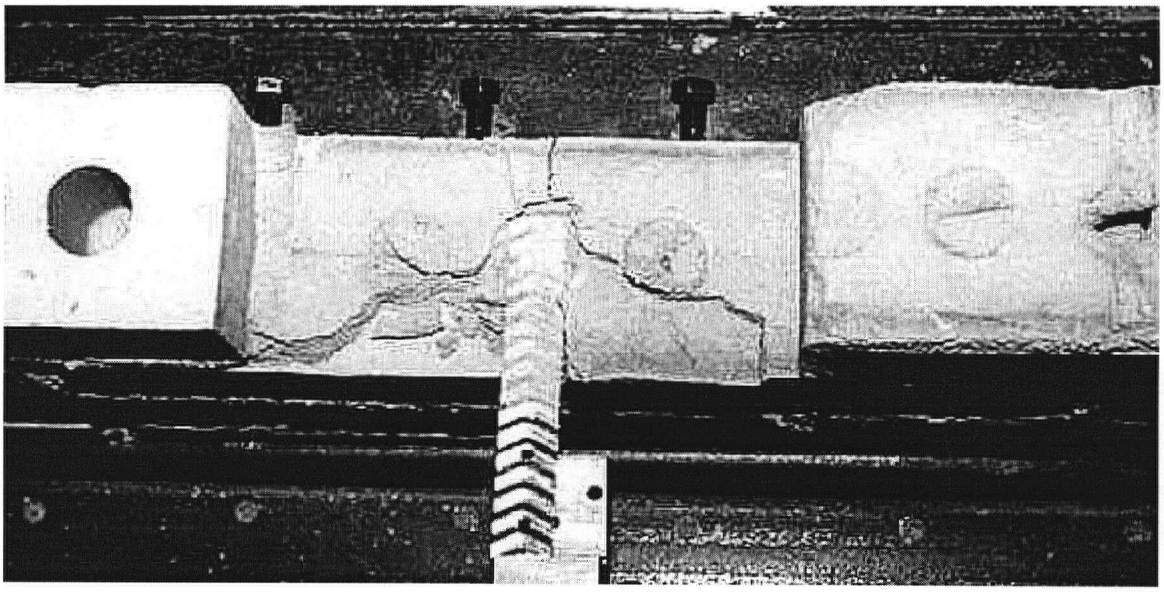
Specimen	C2 – Monotonic Test
Characteristics	Corrugated Strip Tie 22 gauge Embedded length 64 mm (2.5") Type S mortar
Test Date (age)	April 23rd, 2001 (123 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	3.51 kN
Displacement at Maximum Force	
Tension	2.36 mm
Failure modes	
Tension	Pull-out from mortar bed joint

Load-Displacement Relationship



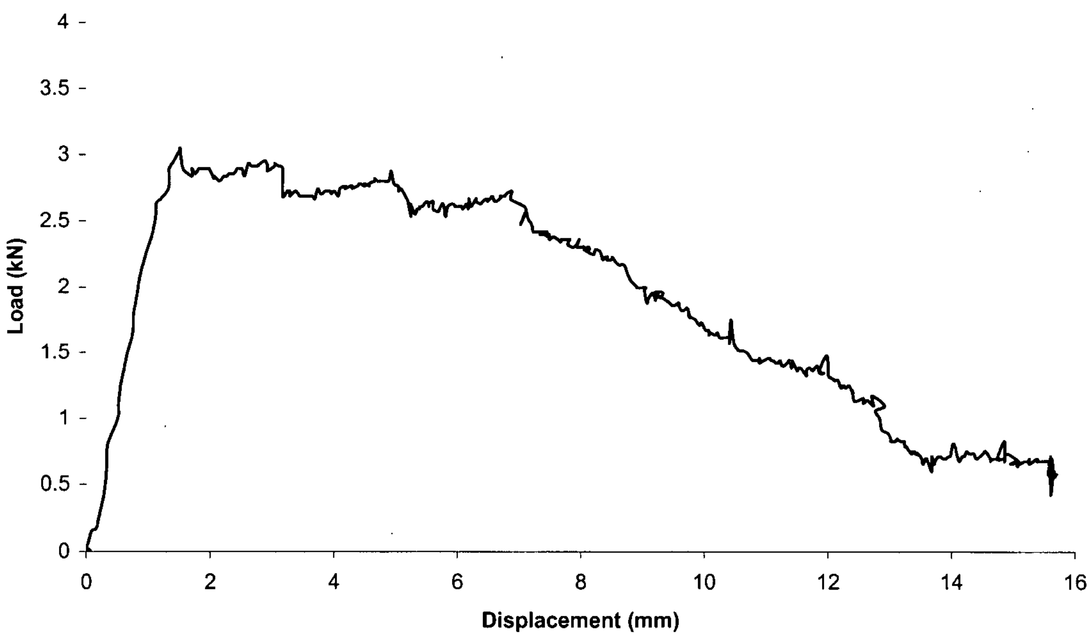
Description of Test Observations:

- This test was a monotonic pull-out test of the corrugated strip tie.
- The test was initially planned to reach 15 mm displacements, however failure of setting the controller resulted only 8 mm displacements.
- The mode of failure was dominantly an embedment failure. This mode was a failure of the bond in the mortar joint that clamped the tie.



Specimen	T7 - Monotonic
Characteristics	V-Tie only embedded at the centre, length 80 mm Type S mortar
Test Date (age)	May 28th, 2001 (31 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	3.05 kN
Displacement at Maximum Force	
Tension	1.52 mm
Failure modes	
Tension	Pull-out from mortar bed joint

Load-Displacement Relationship

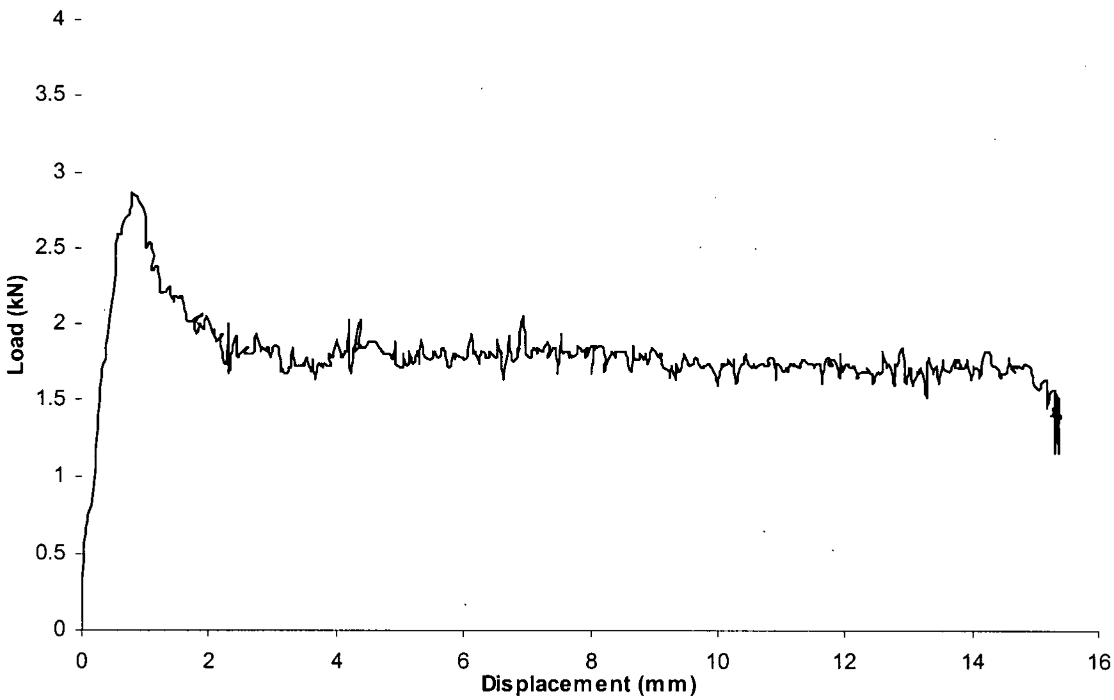


Description of test observation:

- Test was done by monotonic pull-out in tension with 15 mm as the target displacement level.
- Mode of failure was a pull-out of tie from mortar bed joint with a combination of the bond failure between the mortar and the bricks and deformation of the tie material.

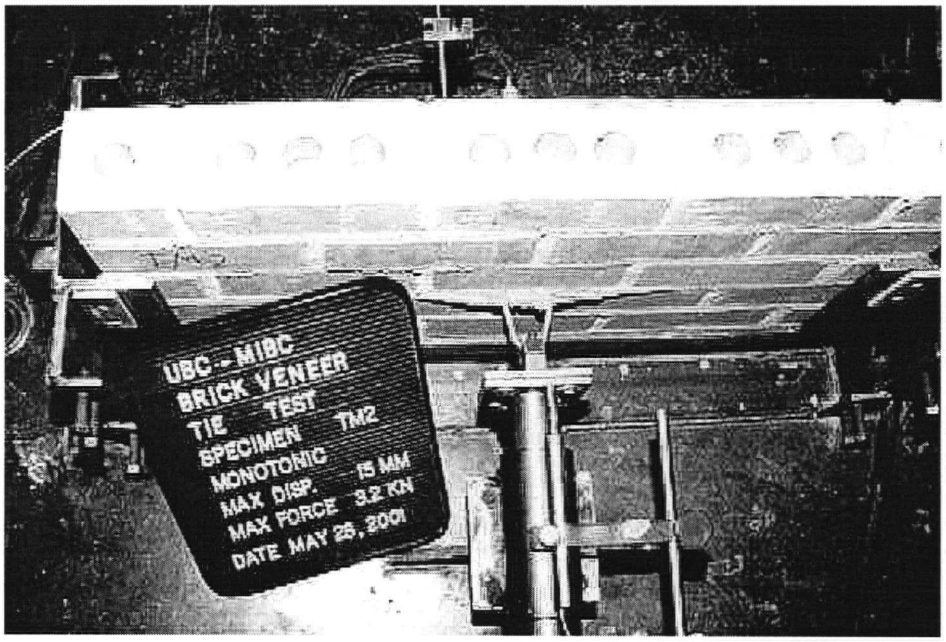
Specimen	T8 - Monotonic
Characteristics	V-Tie only embedded at the centre, length 80 mm Type S mortar
Test Date (age)	May 28th, 2001 (31 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	2.86 kN
Displacement at Maximum Force	
Tension	0.80 mm
Failure modes	
Tension	Pull-out from mortar bed joint

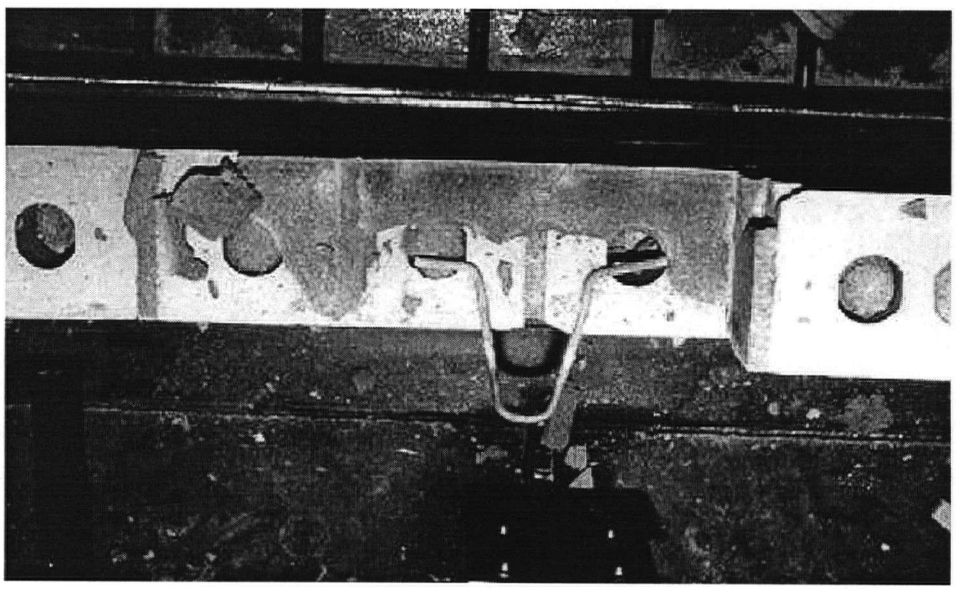
Load-Displacement Relationship



Description of test observation:

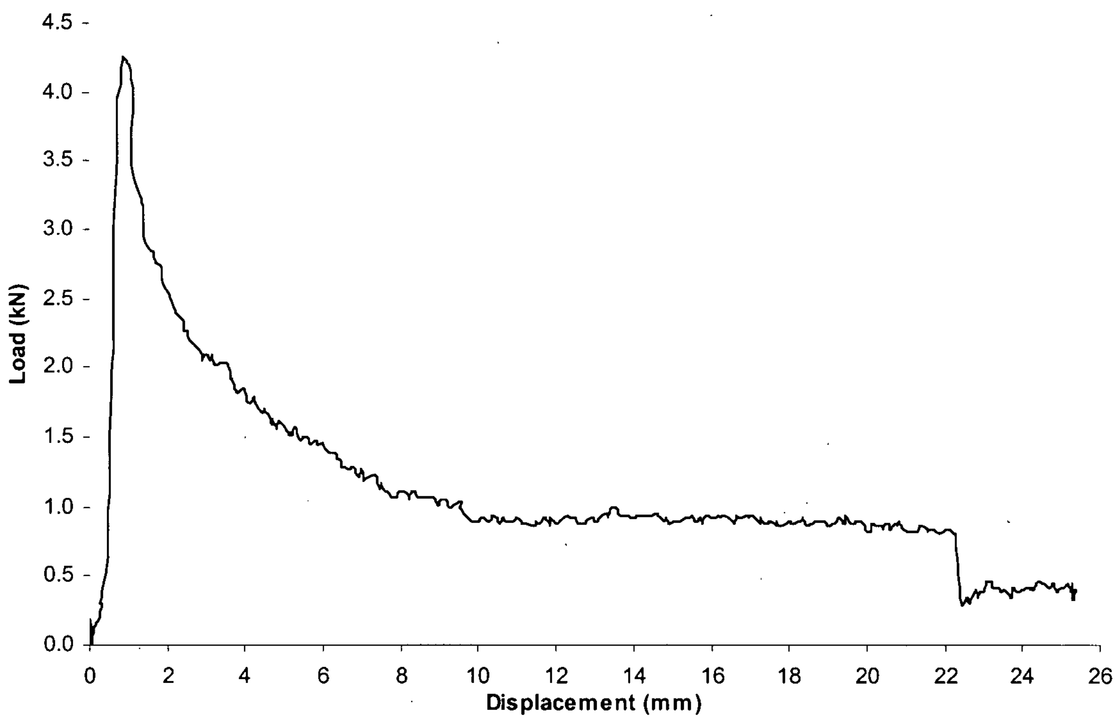
- Test was done by monotonic pull-out in tension with 15 mm as the target displacement level.
- Mode of failure was a pull-out of tie from mortar bed joint with a combination of the bond failure between the mortar and the bricks and deformation of the tie material.


UBC - MIBC
BRICK VENEER
TIE TEST
SPECIMEN TM2
MONOTONIC
MAX DISP. 15 MM
MAX FORCE 3.2 KN
DATE MAY 25, 2001



Specimen	T9 - Monotonic
Characteristics	V-Tie only embedded at the centre, length 80 mm Type S mortar
Test Date (age)	June 21st, 2001 (55 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	4.26 kN
Displacement at Maximum Force	
Tension	0.86 mm
Failure modes	
Tension	Pull-out from mortar bed joint

Load-Displacement Relationship



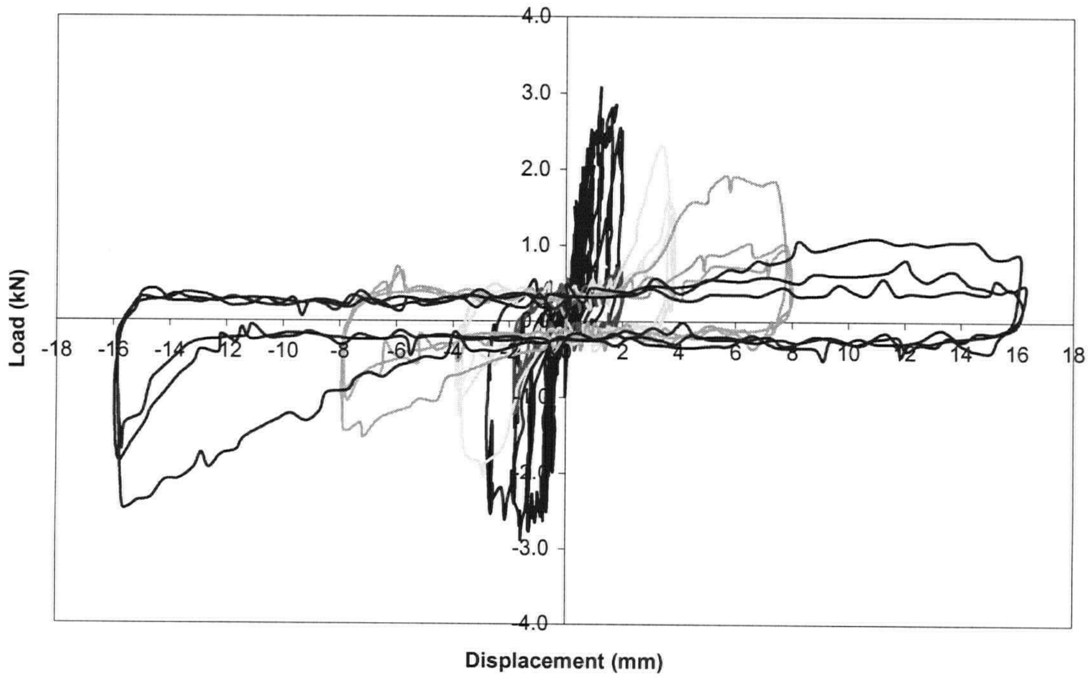
Description of test observation:

- Test was done by monotonic pull-out in tension with 25 mm as the target displacement level.
- Mode of failure was a pull-out of tie from mortar bed joint with a combination of the bond failure between the mortar and the bricks and deformation of the tie material.

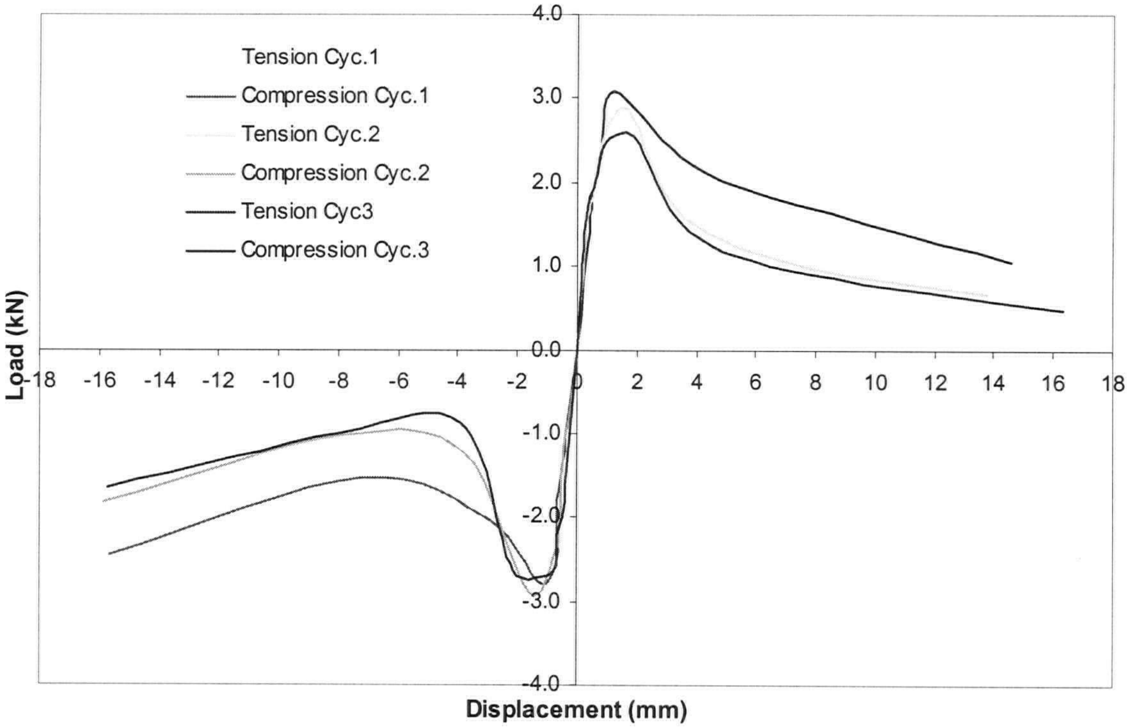

TM3
MIBC - UBC
BRICK VENEER
TIE TEST
SPECIMEN
MONOTONIC
MAX DISPL 25MM
DATE JUNE21, 2001

Specimen	T10
Characteristics	V-Tie only embedded at the centre, length 80 mm Type S mortar
Test Date (age)	June 14th, 2001 (48 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	3.08 kN
Compression	2.77 kN
Displacement at Maximum Force	
Tension	1.22 mm
Compression	1.30 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship



Load-Displacement Envelope Curve



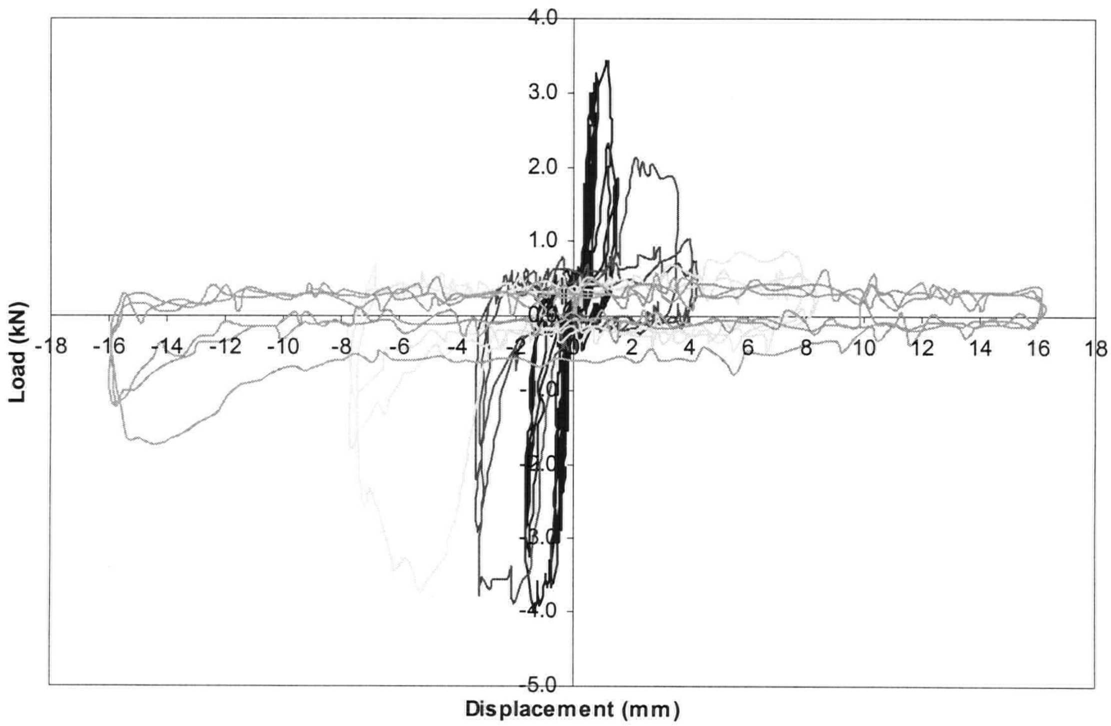
Description of test observations:

- Test was conducted using the subsequent loading protocol.
- In the force-controlled part, there was not any indication of cracks in the bed joint, as the loading cycled in tension and compression to the target force level until it reached 2 mm displacement.
- There was an additional 2 mm displacement target level at the displacement-controlled part due to the setting of the controller. This was then to be corrected to limit the loading not more than 3 cycles for the 2 mm displacement target level. This was done by setting the displacement target level directly to 4 mm displacement.
- At 4 mm displacement, cracks formed in tension and spalling started to occur, followed by a pull-out action of the crushed mortar joint that becomes very loose. In compression several cracks were formed and evidence of ladder cracks started to appear from both top course corner of the bricks to the centre of the bed joint where the embedded tie located.
- At 8 mm displacement, in tension at the tie face, spalled crushed mortars occurred and more pieces of loosely crushed mortar being pulled out of the bed joint. While in compression horizontal cracks formed along the course where the tie was embedded. Some push-through of crushed mortars occurred in the tie embedment location at the tooled joint face.

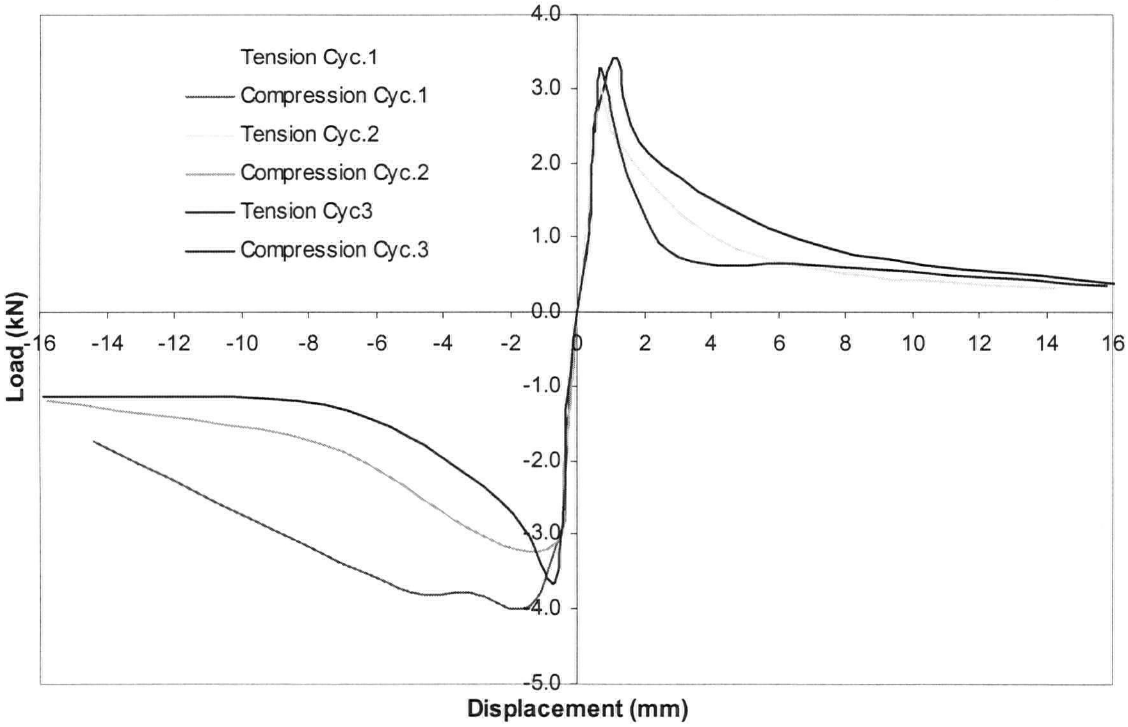
- With the final displacement at 16 mm, in tension, severe pull-out of crushed mortar from the bed joint were evident. And in compression push-through of loose crushed mortars were also apparent.
- Visual observations of the specimen by opening up the bed joint where the tie was embedded, indicated large pieces of crushed mortar were formed by the larger step size of loading with the movement of tie. The tie legs was deformed or bent to the compression side, which is stronger.

Specimen	T11
Characteristics	V-Tie only embedded at the centre, length 80 mm Type S mortar
Test Date (age)	June 14th, 2001 (48 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	3.39 kN
Compression	3.97 kN
Displacement at Maximum Force	
Tension	1.17 mm
Compression	1.41 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship



Load-Displacement Envelope Curve

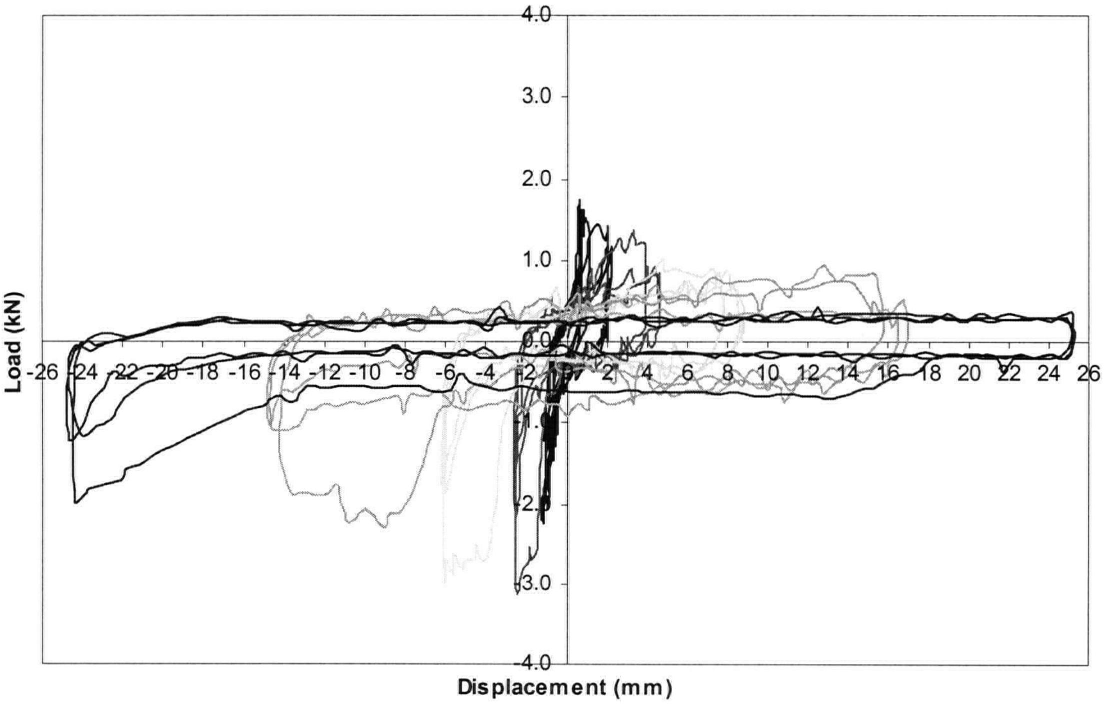


Description of test observations:

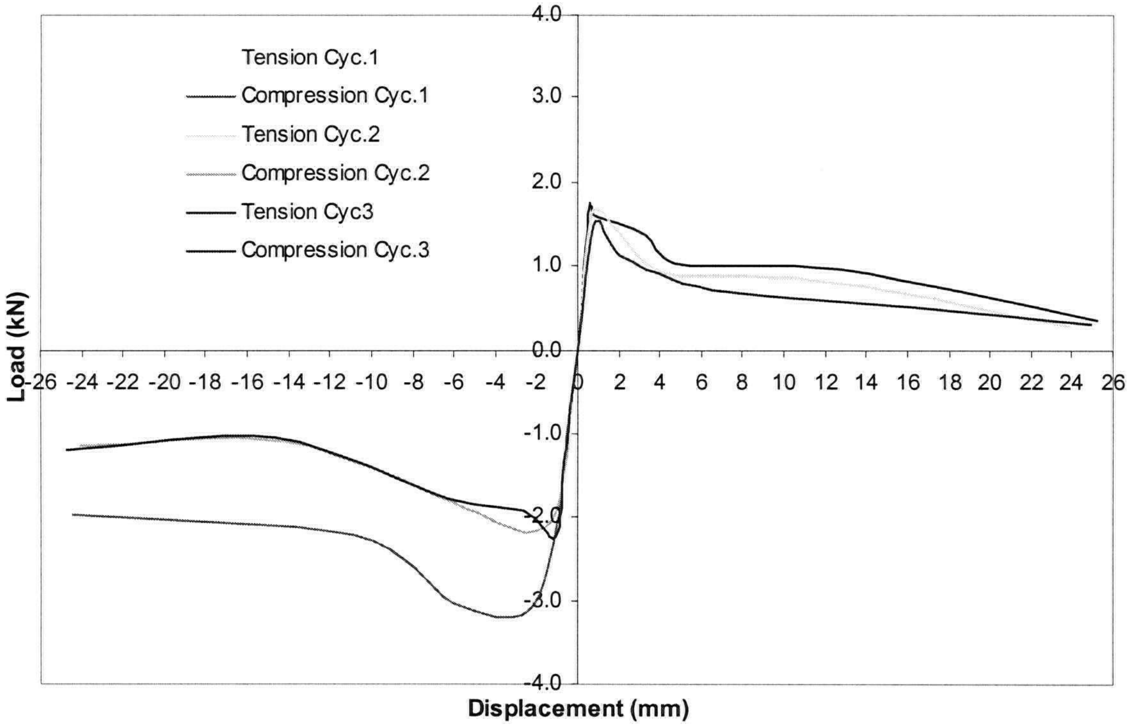
- This was the first specimen to be tested using the new loading protocol that consisted of two parts, force-controlled and displacement controlled.
- In the first part of loading, which is the force-controlled part, the final loading cycles were actually stopped when it reached 3.5 kN, though the displacement at that target force level had not reached 2 mm displacement yet.
- There were almost no cracks formed in the force-controlled part of loading.
- At 4 mm displacement, loose crushed mortar at the bed joint started to pull-out and spall in tension, while in compression more mortar were crushed and pushed through the bed joint.
- With 8 mm displacement, more crushed mortar spalled in tension and several vertical cracks were formed in the head joints, while in compression push-through of crushed mortar from the bed joint were evident.
- Final displacement was at 16 mm, where in tension almost all crushed mortar were spalled, with the compression side showing push-through mortar joint failure.
- No observation by opening up the bed joint was conducted for this specimen.

Specimen	T12
Characteristics	V-Tie only embedded at the centre, length 80 mm Type S mortar
Test Date (age)	June 25th, 2001 (58 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	1.73 kN
Compression	3.13 kN
Displacement at Maximum Force	
Tension	0.63 mm
Compression	2.47 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship

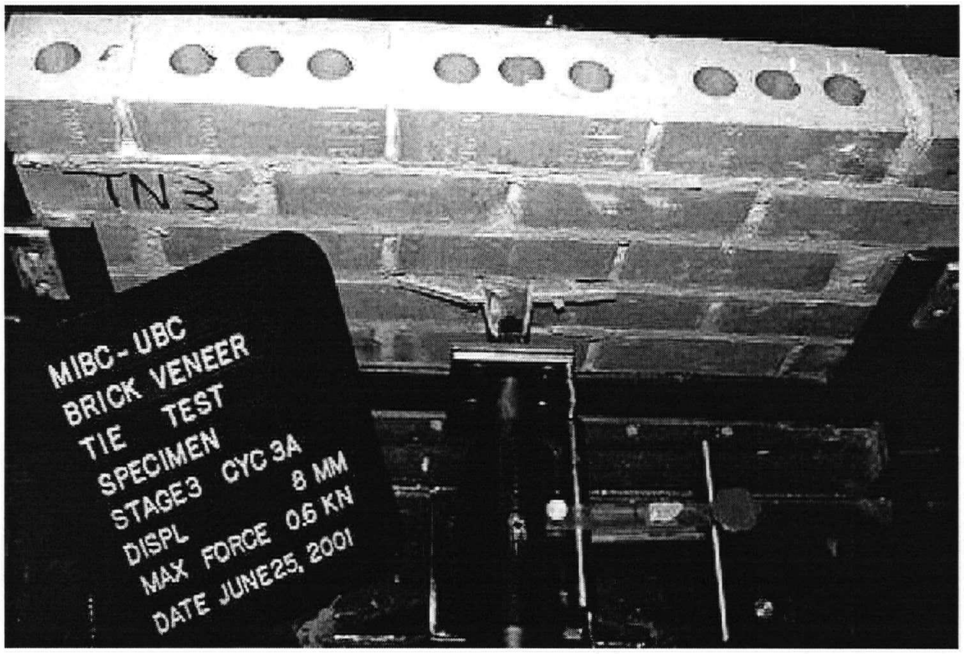


Load-Displacement Envelope Curve



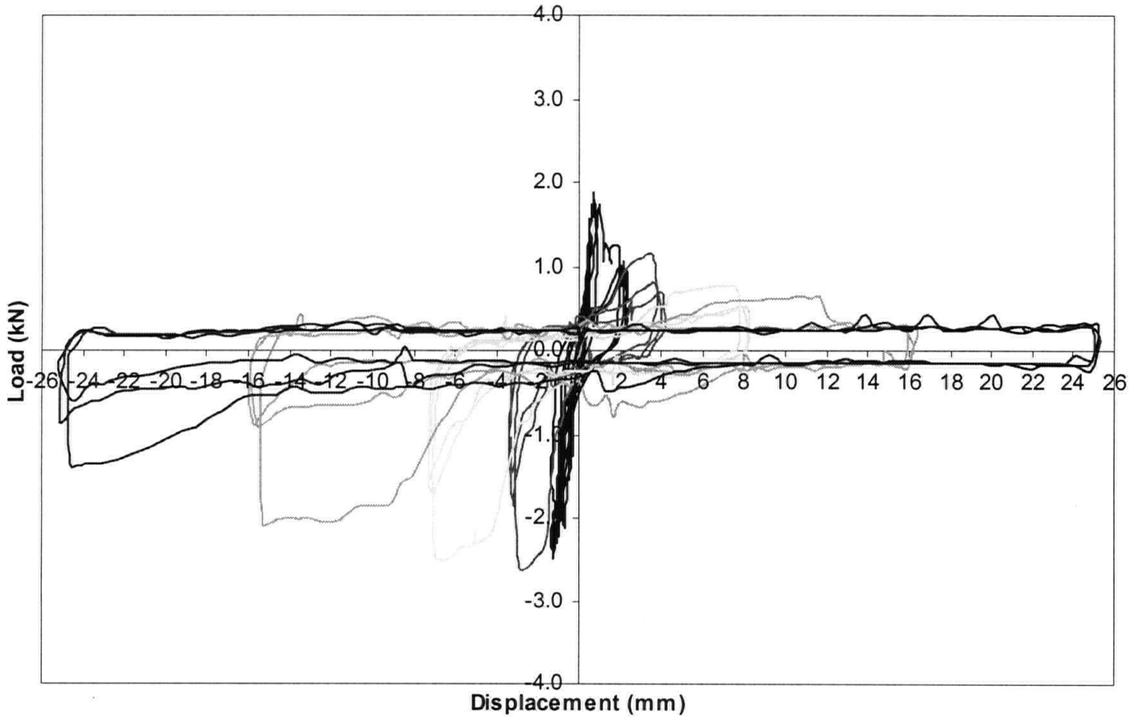
Description of test observations:

- Test was conducted using the new loading protocol.
- Force-controlled loading part indicated a low peak resistance in tension, while the displacement reached 2 mm. This was due to the disturbed area of mortar bed joint in the embedment of tie caused by the suspected damage of the specimen. There were cracks formed in the force-controlled part of loadings.
- At 4mm displacement, several crushed mortars were pulled out from the bed joint in tension and some spalling occurred. In compression, pieces of crushed mortar were pushed-through the bed joint by the movement of the tie. Horizontal cracks along the bed joint were also formed in compression at the tooled joint face.
- At 8 mm displacement, more spalling occurred in tension, while in compression push-through of mortars from the bed joint occurred.
- At 16 mm, no more cracks were formed in tension and almost all the mortars located close to the vicinity of the tie were lost due to spalling. In compression, loose mortar was pushed-out through the bed joint.
- Final displacement was 25 mm, with all mortars spalled in the bed joint at the tie face in tension, and push-through mortars from the bed joint in compression.

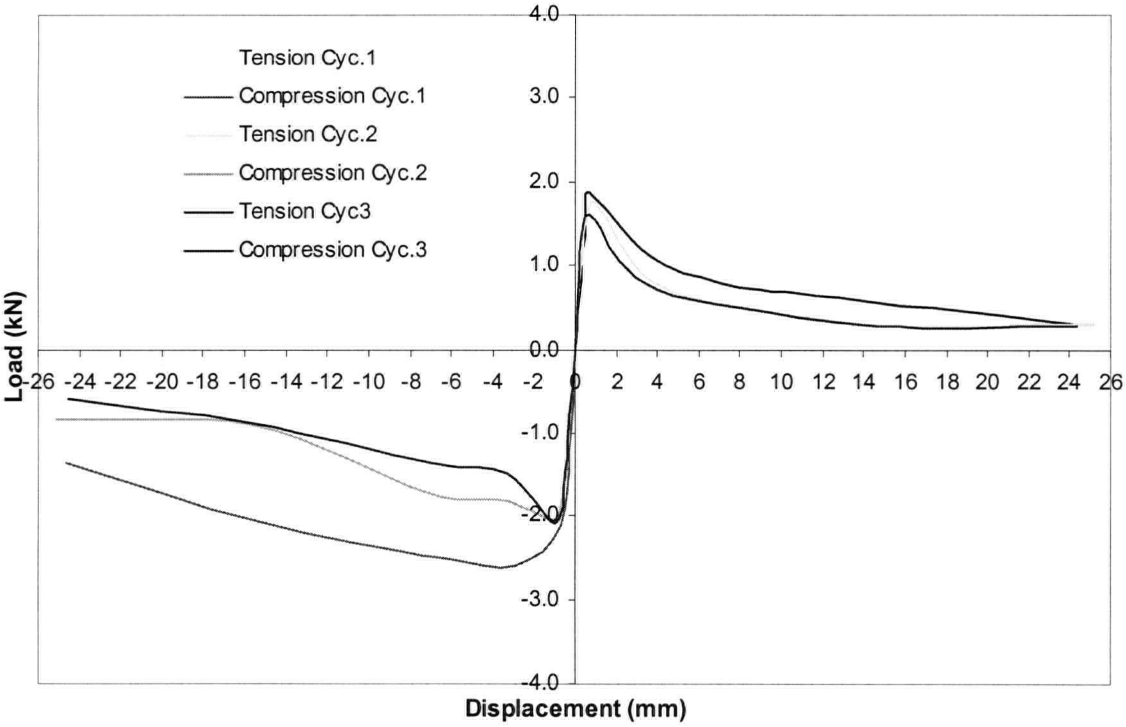

TN3
MIBC - UBC
BRICK VENEER
TIE TEST
SPECIMEN
STAGE3 CYC 3A
DISPL 8 MM
MAX FORCE 0.6 KN
DATE JUNE25, 2001

Specimen	T13
Characteristics	V-Tie only embedded at the centre, length 80 mm Type S mortar
Test Date (age)	July 4th, 2001 (68 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	1.88 kN
Compression	2.61 kN
Displacement at Maximum Force	
Tension	0.69 mm
Compression	2.87 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship



Load-Displacement Envelope Curve



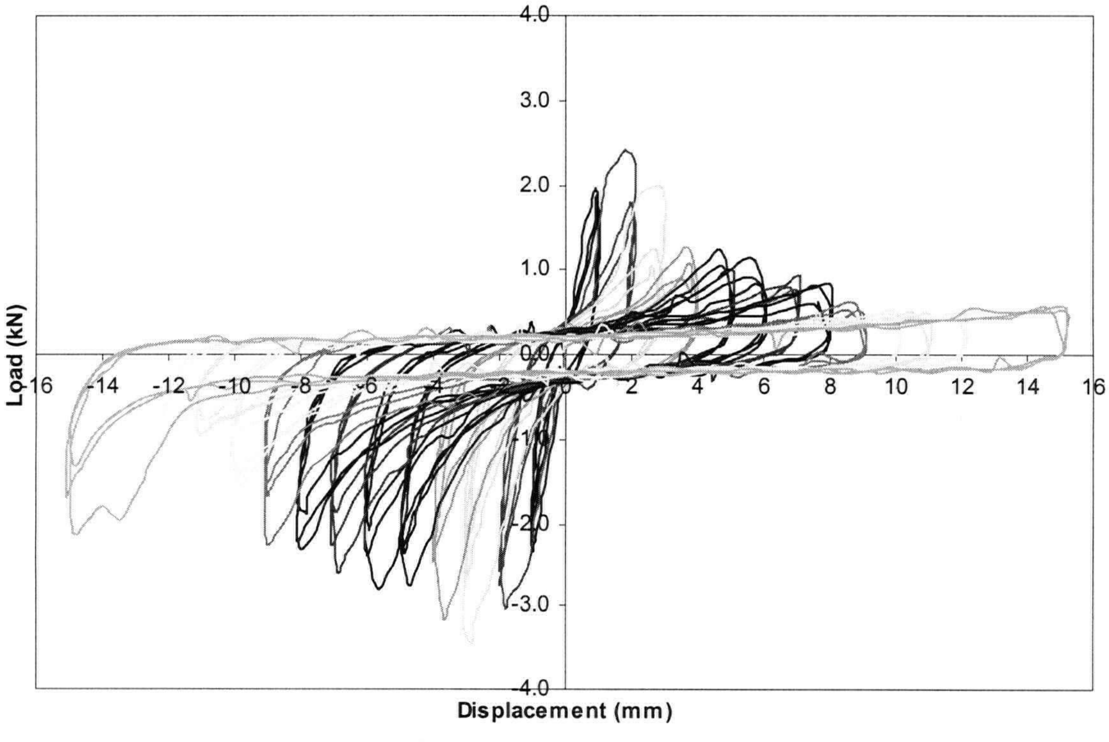
Description of test observations:

- Test was conducted using the new loading protocol.
- The low peak load resistance obtained in the force-controlled loading part was due to the disturbed area of mortars around the embedment of the tie due to the accident. There were some cracks formed in tension at this part of loadings.
- At 4 mm displacement in the displacement-controlled loading part, pieces of crushed mortar were formed in tension along the bed joint where the tie was embedded. The loosely pieces of crushed mortar were starting to pull-out from the bed joint as the tie moved in tension. There was an indication of ladder cracks formed from the top course of brick to the centre at the embedded tie location in compression.
- More pull-out of crushed mortars occurred at 8 mm displacement stage in tension. In compression cracks were formed in the location of embedment of the tie with ladder cracks getting wider. The ladder cracks was an indication of a punching shear action of the tie in compression to the brick panel specimen.
- At 16 mm displacement, the pieces of crushed mortars were spalled in tension, while in compression push-through of crushed mortars was also evident.
- Final displacement of the test was 25 mm. With in tension, all mortars were spalled from the bed joint and in compression push-through of crushed mortars occurred.

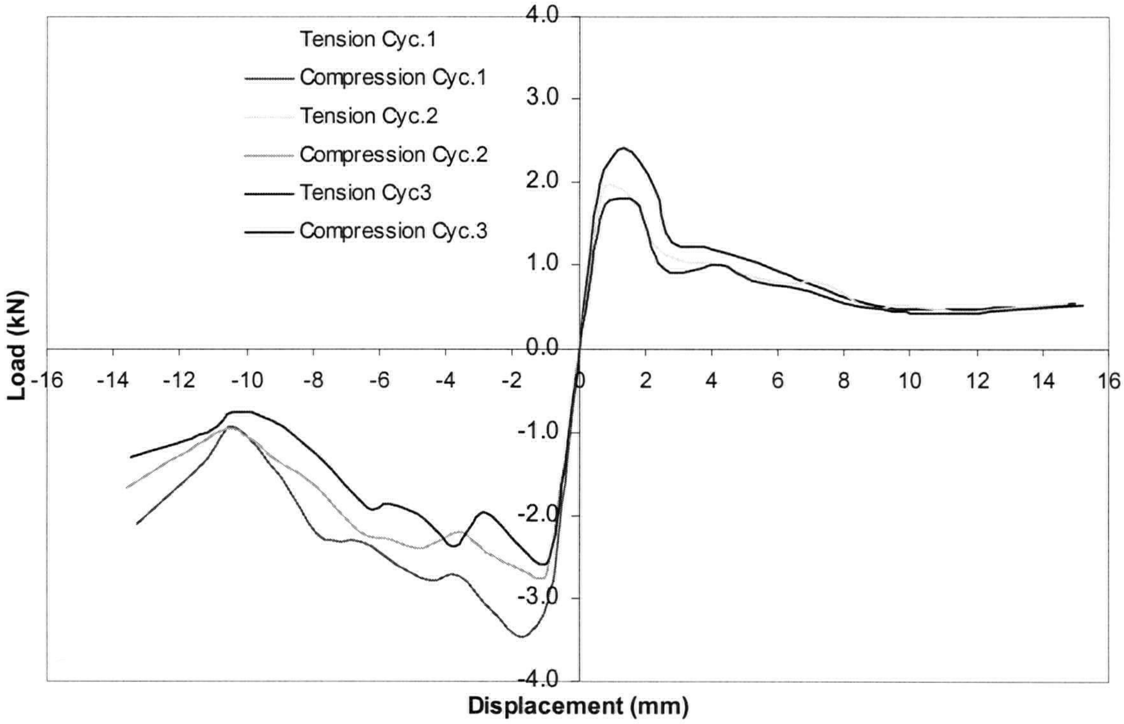

TM4
MIBC - UBC
BRICK VENEER
TIE TEST
SPECIMEN T13
STAGE3 CYC3A
DISPL 8 MM
MAX FORCE 0.5 KN
DATE JULY4 , 2001

Specimen	TWC9
Characteristics	V-Tie with horizontal wire reinforcement clipped
	V-Tie length 80 mm
	Type S mortar
Test Date (age)	June 5th, 2001 (39 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	2.43 kN
Compression	3.46 kN
Displacement at Maximum Force	
Tension	1.38 mm
Compression	1.63 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship



Load-Displacement Envelope Curve

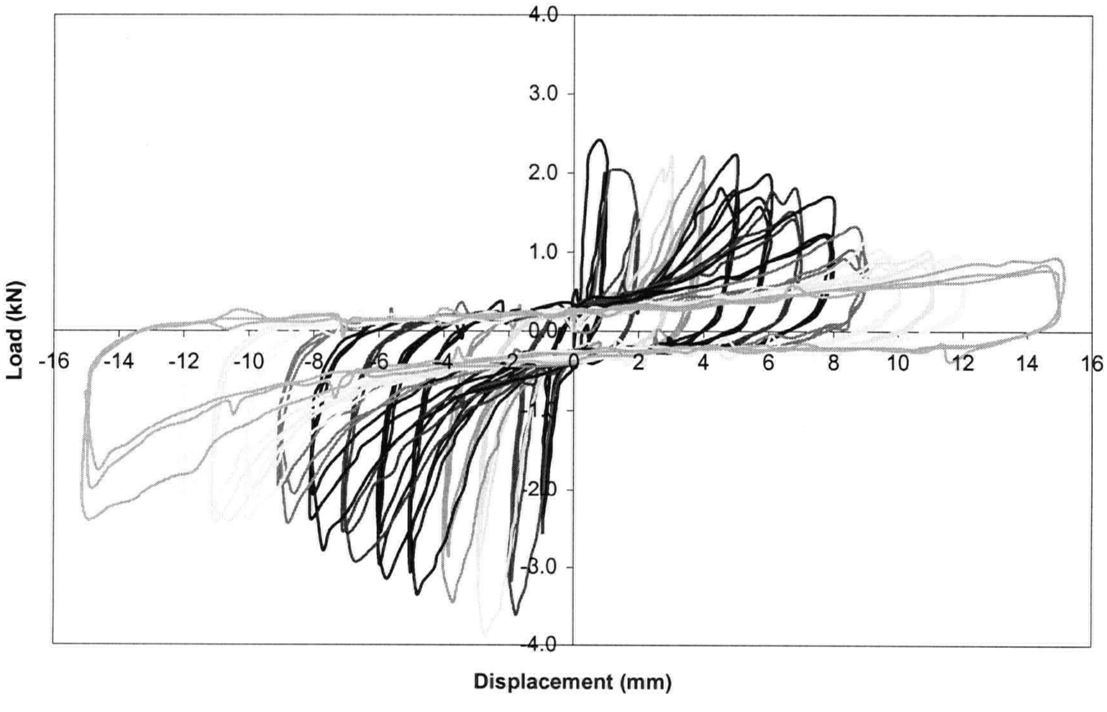


Description of Test Observations:

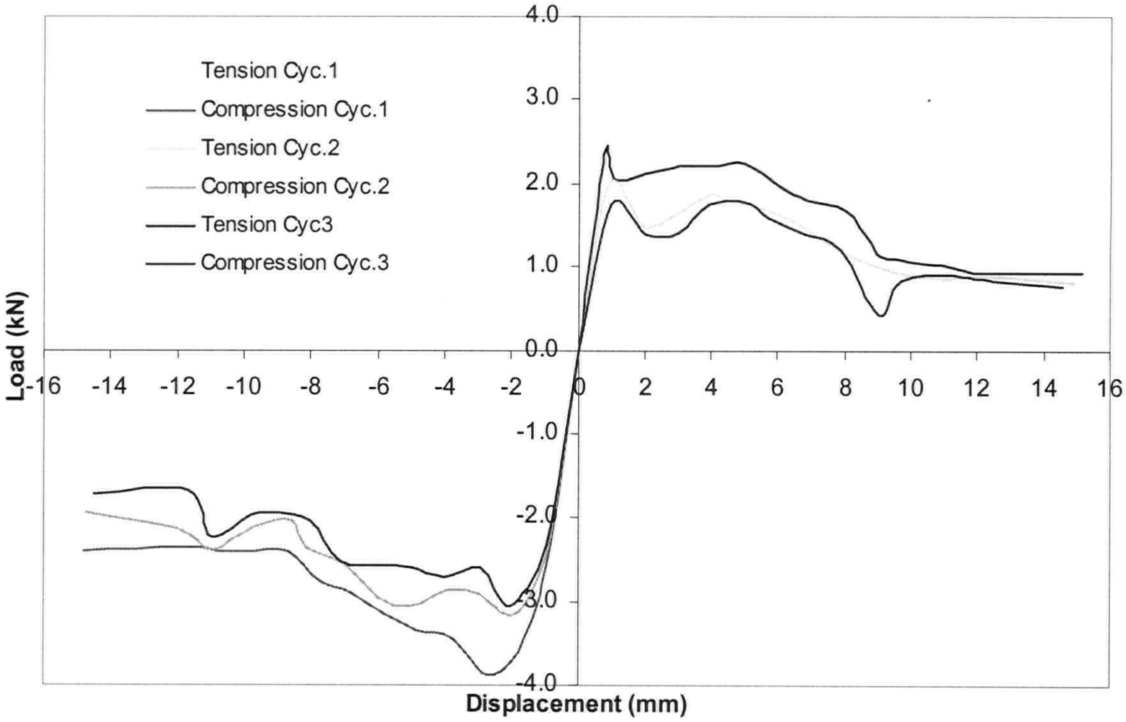
- At stage 2 with 2 mm displacement, cracks started to appear in tension and compression especially near the embedment of the tie.
- At 4 mm displacement stage 4, in tension, pieces of crushed mortar started to pull-out from the bed joint. While in compression more cracks propagated vertically and horizontally from the tie embedment location in the bed joint at the tooled joint face.
- Stage 5 with 5 mm displacement, loose pieces of crushed mortars on the tie face were started to spall. In compression, more cracks appeared.
- At 8 mm displacement in stage 8 of loading, tension indicated more spalled crushed mortars from the bed joint at vicinity of tie embedment. In compression side, push-through crushed mortars was started to occur.
- At 12 mm displacement, more spalling occurred in tension and finally left a gap or a hole in the area of the tie legs, which allowed visual observation of the clipped tie with the joint reinforcement.
- In the final displacement at 15 mm, in compression, there was an override condition occurred between the tie and the horizontal wire joint reinforcement. This was possible due to the gap in the clips as the connection. In compression at large displacement with all the mortars inside the gap already crushed and also in the bed joint, the clips started to become loose and allowing the tie to override the wire at some point. This was also the case of an unclipped condition of the system because of the clips detached from the two wires.

Specimen	TWC10
Characteristics	V-Tie with horizontal wire reinforcement clipped V-Tie length 80 mm Type S mortar
Test Date (age)	May 32nd, 2001 (39 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	2.42 kN
Compression	2.576 kN
Displacement at Maximum Force	
Tension	0.80 mm
Compression	0.95 mm
Failure modes	
Tension	Pull-out from mortar bed joint.
Compression	Slight push-through of mortar bed joint with punching shear failure of the brick panel specimen

Load-Displacement Relationship



Load-Displacement Envelope Curve

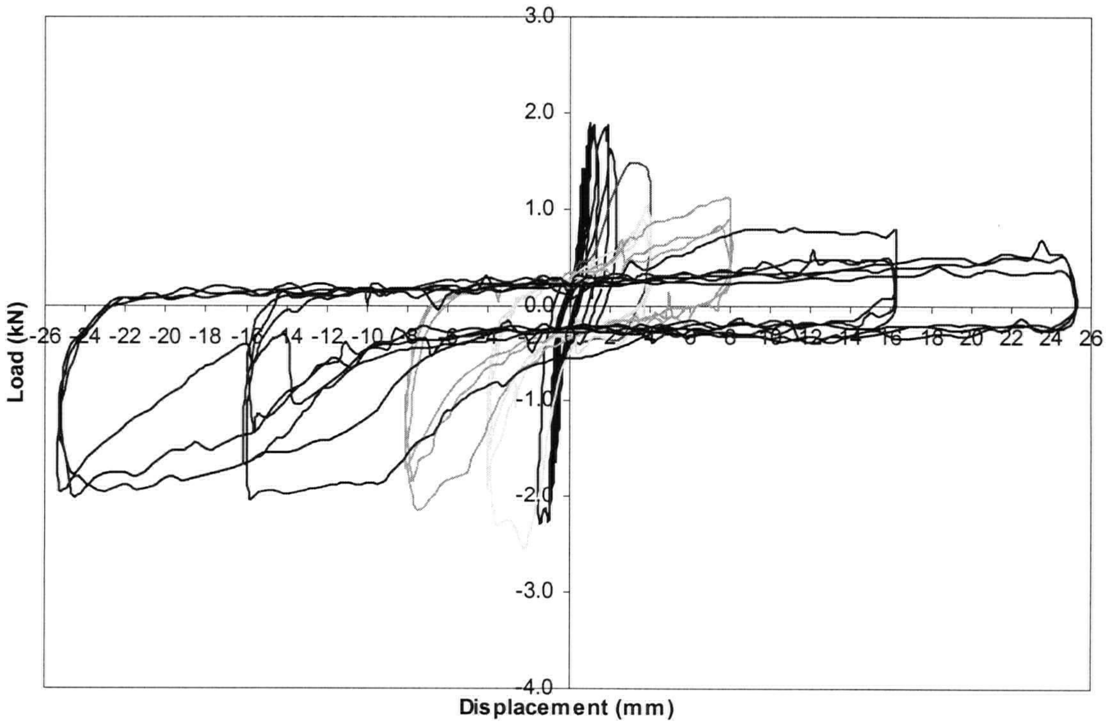


Description of Test Observations:

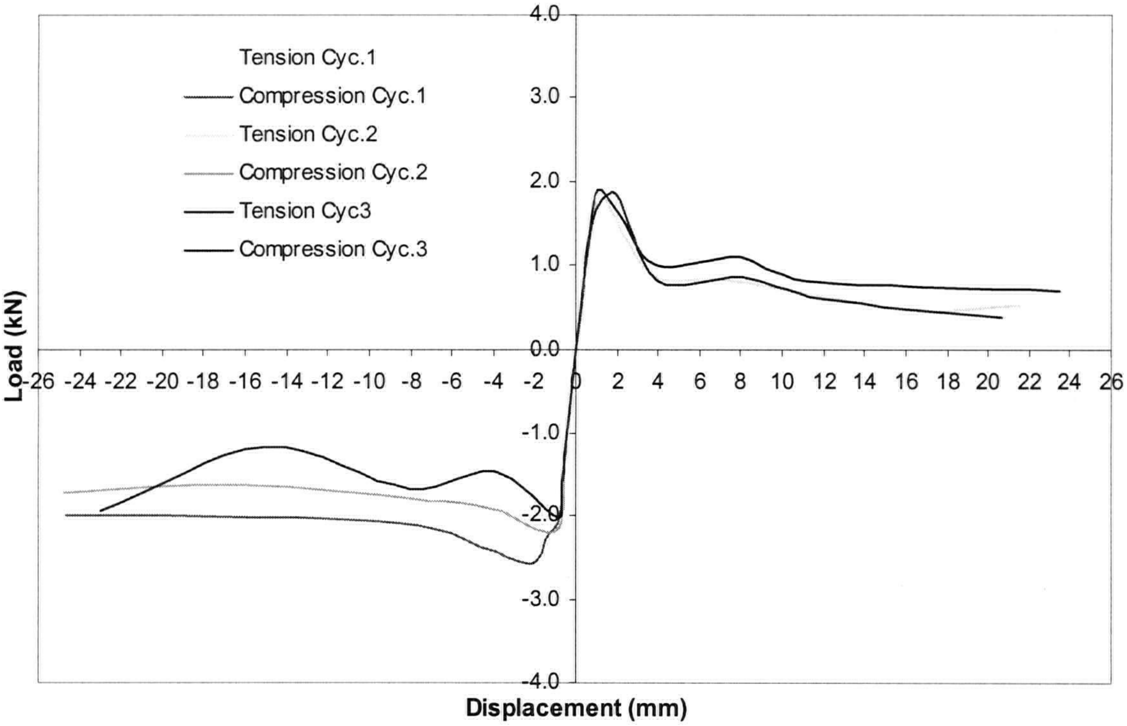
- Cracks started to appear in 2 mm displacement at stage 2 in tension at the tie face. In compression there were no cracks formed yet.
- At 4 mm displacement stage 4, in tension, pieces of crushed mortar started to pull-out from the bed joint. In compression more cracks propagated diagonally from the centre near the tie embedment to the top course of the brick. This was an indication of ladder cracks at the tooled joint face.
- At 7 mm displacement, most of the loose crushed mortar were spalled in tension and left a gap on the tie embedment location at the tie face. In compression, push-through of crushed mortars from the bed joint occurred and the diagonal cracks getting wider, thus allowed the specimen to be divided into two parts (wedge shape). The top part of the wedge was slightly lifted up in compression.
- At 9 mm displacement, in tension, a diagonal crack formed from the tie embedment to the top course of specimen. The cracks were propagated through the bricks, leaving a slightly spalled bricks.
- At 12 mm displacement, almost all of the loosely crushed mortars were spalled in tension due to the pull-out action. While in compression the tie pushed the top part of the wedge out.
- In 15 mm displacement as the final stage of loading, tension indicated no more crushed mortars were formed and in compression a pushed out top part of brick specimen was occurred.

Specimen	TWC11
Characteristics	V-Tie with horizontal wire reinforcement clipped V-Tie length 80 mm Type S mortar
Test Date (age)	June 25th, 2001 (58 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	1.90 kN
Compression	2.56 kN
Displacement at Maximum Force	
Tension	1.03 mm
Compression	2.28 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint with diagonal cracks on one side of specimen (punching shear)

Load-Displacement Relationship

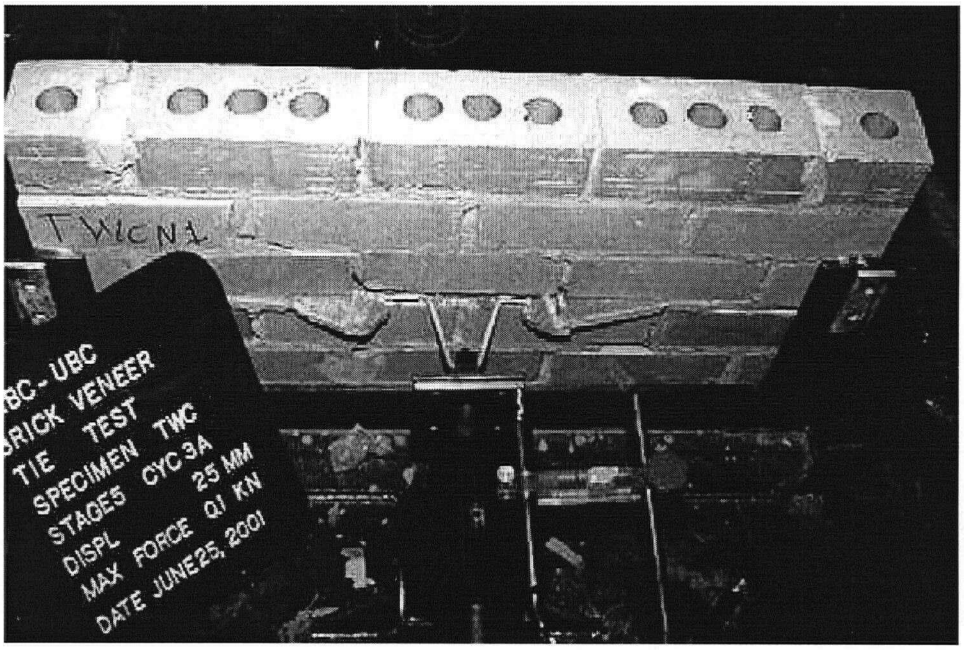


Load-Displacement Envelope Curve



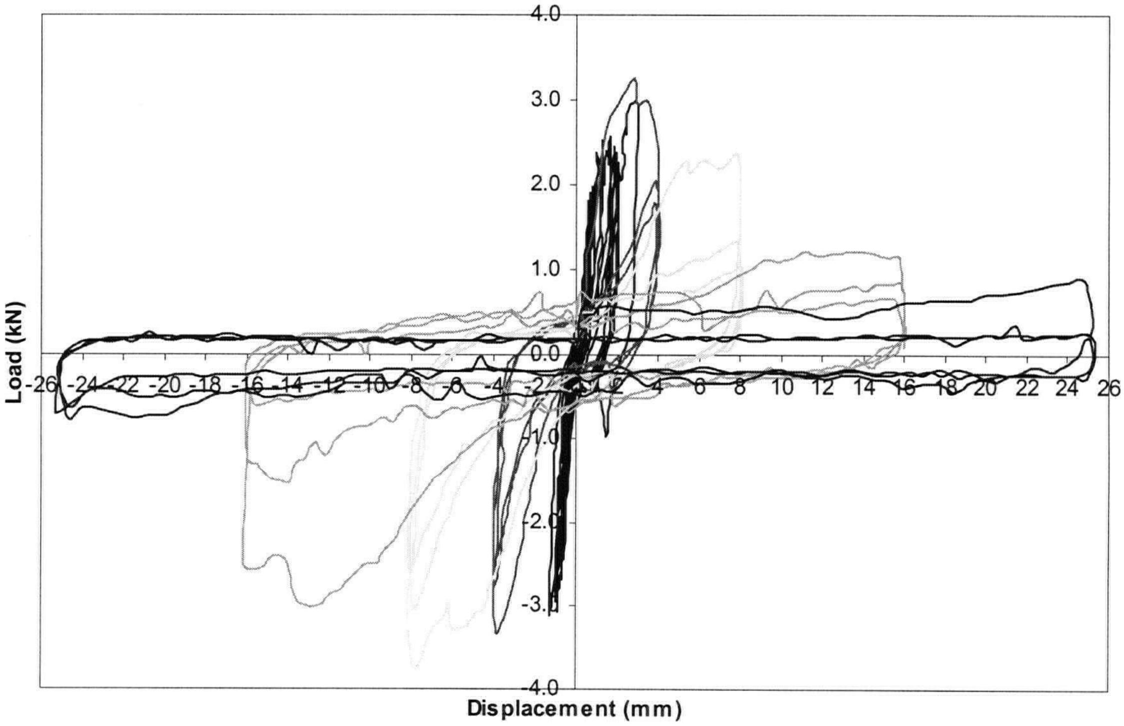
Description of Test Observations:

- Test was conducted using the new loading protocol.
- There was a low peak load obtained from the force-controlled part of the loading protocol. This was suspected from the damage of the specimen before the test. Some disturbances on the area of mortar embedment of tie might be the possibility of the low maximum load.
- At 4 mm displacement, several cracks appeared and formed pieces of loose crushed mortar in the bed joint, with an indication of pull-out from the bed joint. In compression diagonal cracks were formed and vertical cracks in the bed joint of the tie were also forming a loose piece of crushed mortars.
- At 8 mm displacement, pull-out of crushed mortar joint occurred and spalled, while several cracks forming some pieces of crushed mortar joint were also pulled out from the bed joint. In compression the diagonal cracks became a ladder cracks from the top course of brick to the centre of the tie embedment location. The piece of loose crushed mortars was started to push through from the bed joint.
- At 16 mm displacement, excessive spalling occurred in tension, leaving a hole in the bed joint in the embedment of the tie. In compression, push-through of crushed mortars from the bed joint occurred along with movement of the top part of brick specimen that were divided by the ladder cracks on one side.
- Final displacement was 25 mm. Tension indicated all of the crushed mortars were spalled. And compression showed a push-through failure along with the diagonal cracks that allowed parts of the brick specimen to move.

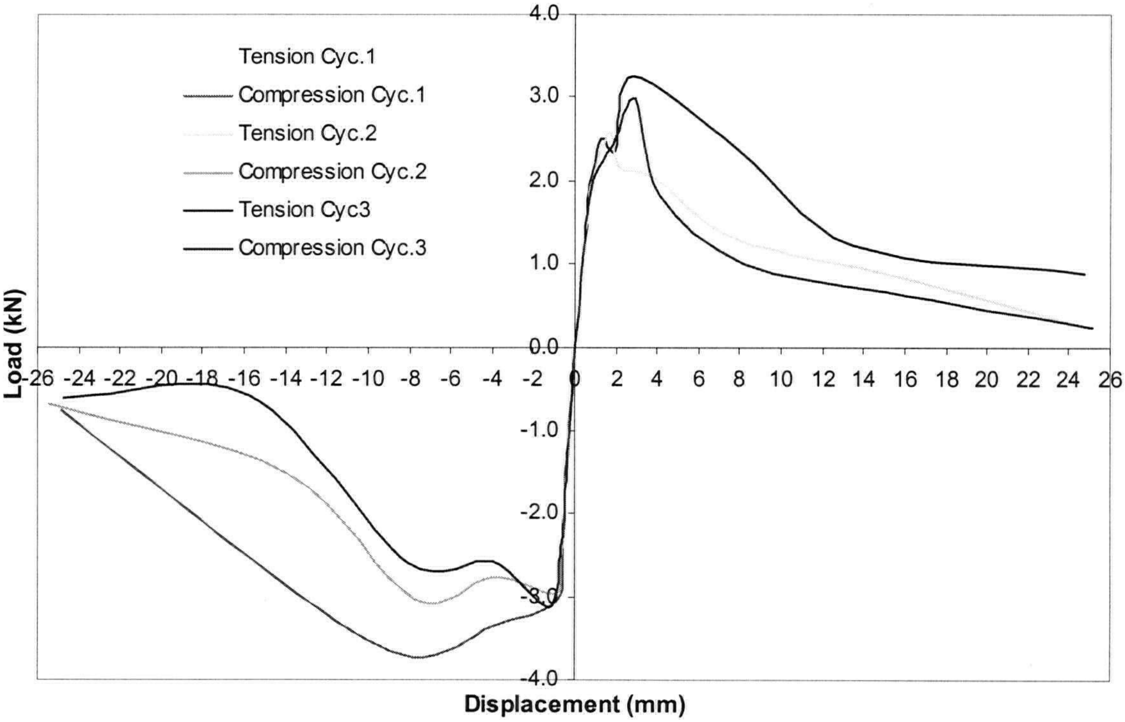

TWCN1
BC - UBC
BRICK VENEER
TIE TEST
SPECIMEN TWC
STAGE5 CYC 3A
DISPL 25 MM
MAX FORCE 0.1 KN
DATE JUNE25, 2001

Specimen	TWC12
Characteristics	V-Tie with horizontal wire reinforcement clipped V-Tie length 80 mm Type S mortar
Test Date (age)	June 22nd, 2001 (56 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	3.26 kN
Compression	3.73 kN
Displacement at Maximum Force	
Tension	2.86 mm
Compression	7.88 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint with diagonal cracks on one side of specimen (punching shear)

Load-Displacement Relationship

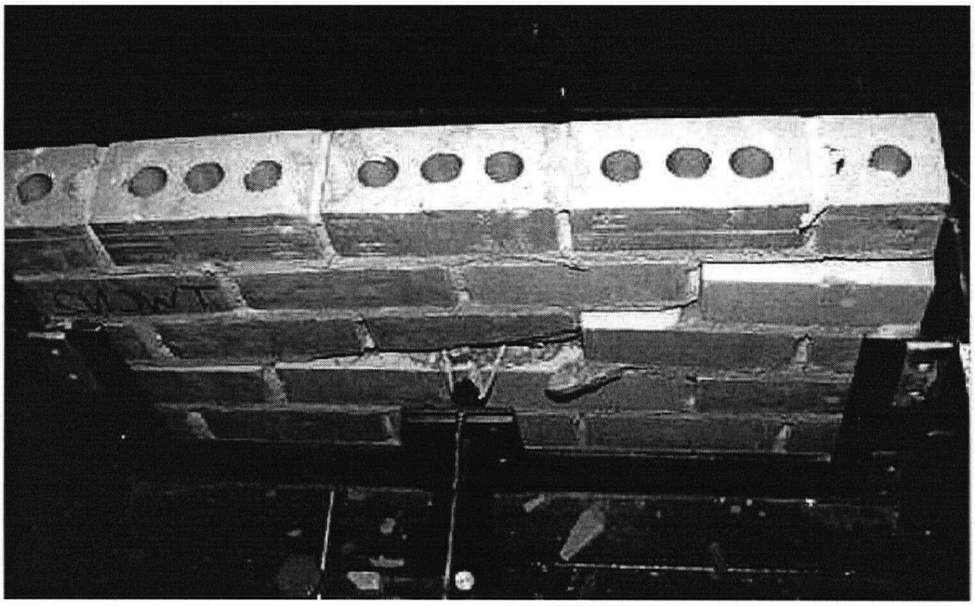


Load-Displacement Envelope Curve



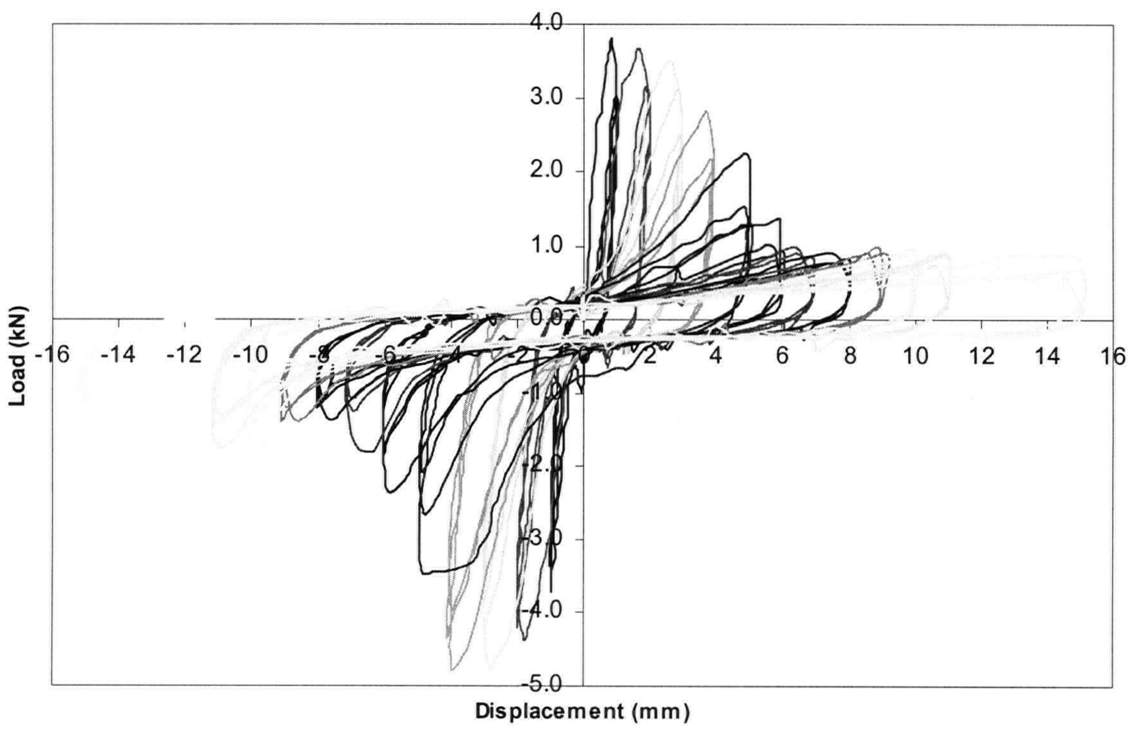
Description of Test Observations:

- Test was conducted using the new loading protocol.
- No cracks were formed in the force-controlled part of the loading protocol.
- At 4 mm displacement with the displacement-controlled loading, several cracks were formed and became pieces of loose crushed mortar in the bed joint. This was also followed with an indication of pull-out from the bed joint. In compression diagonal cracks were formed and several cracks formed loose pieces of crushed mortars.
- At 8 mm displacement, loose pieces of crushed mortars were pulled out from the bed joint in tension. In compression, diagonal cracks started to form a ladder cracks pattern on one side of the specimen, which moved along the movement of the tie.
- At 16 mm displacement, spalling of loose crushed mortar with some brick pieces occurred in tension leaving a hole on the embedment location of tie. In compression, the ladder cracks allowed one side of the specimen to slightly lift up due to the movement of the tie. Some crushed mortars were also pushed-through slightly from the bed joint.
- Final displacement was 25 mm. With all of the crushed mortars spalled in tension and in compression, combination failure of push-through mortar from bed joint along with excessive movement of top part of bricks specimen due to the diagonal cracks.

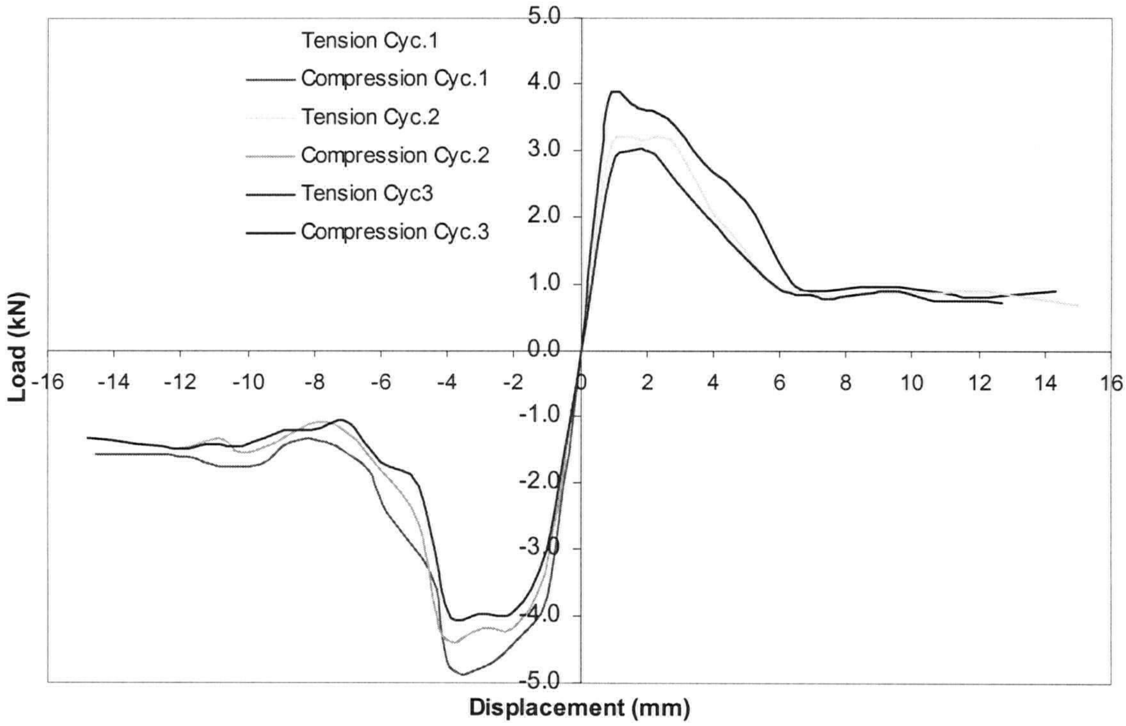


Specimen	TWS1
Characteristics	V-Tie with horizontal wire reinforcement clipped (modified clips) V-Tie length 80 mm Type S mortar
Test Date (age)	May 29th, 2001 (32 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	3.80 kN
Compression	4.75 kN
Displacement at Maximum Force	
Tension	0.87 mm
Compression	3.96 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship

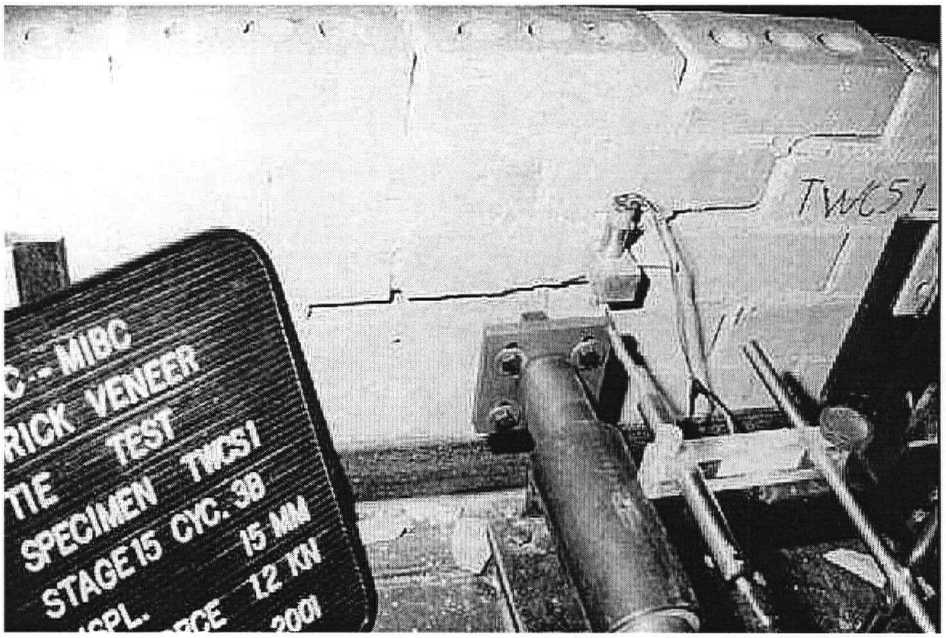


Load-Displacement Envelope Curve



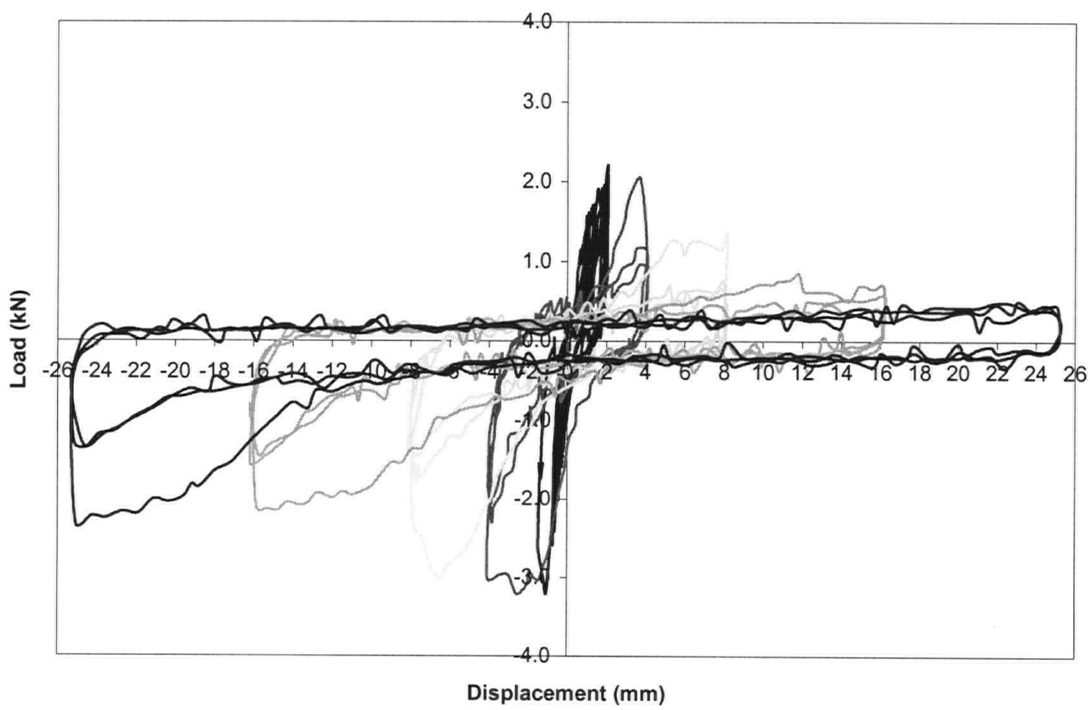
Description of Test Observations:

- At 2 mm displacement, cracks started to form in tension at tie face. No cracks were formed yet in compression.
- Test reached 4 mm displacement with an indication of loose crushed mortars started to pull-out in tension. And in compression horizontal cracks propagated along the bed joint where the embedment of tie located.
- At 5 mm displacement, pieces of loose crushed mortars were spalled in tension, leaving a hole in the location of the embedded tie. While compression showing diagonal cracks forming from top course corner to the centre, which was the embedment location of the tie. This was happening at the tooled joint face.
- At 7 mm displacement, some bricks were cracked and some small pieces were spalled along with vertical cracks. In compression, the diagonal cracks became a ladder cracks and getting wider. This ladder cracks allowed one side of the bricks specimen to move along with the tie.
- At 9 mm displacement, more spalling of crushed mortars occurred in tension.
- At 12 mm displacement, there was an indication of excessive movement of the top part of bricks specimen, which were separated by the ladder cracks. This condition occurred in compression.
- Final displacement was 15 mm. With all pieces of crushed mortar were spalled in tension and compression showed a push-through of crushed mortars from bed joint along with an excessive movement of top part of the specimen due to the diagonal cracks.

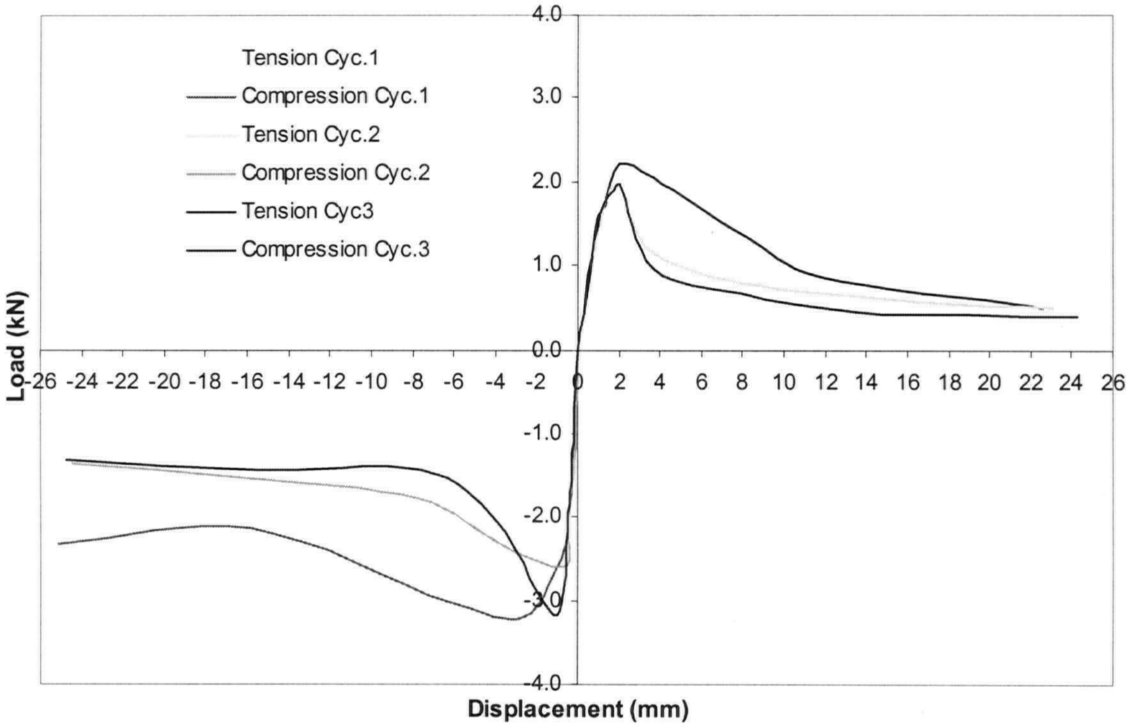

C. - MIBC
RICK VENEER
TIE TEST
SPECIMEN TWCS1
STAGE 15 CYC. 3B
15 MM
12 KN
2001

Specimen	TWS2
Characteristics	V-Tie with horizontal wire reinforcement clipped (modified clips) V-Tie length 80 mm Type S mortar
Test Date (age)	June 21st, 2001 (55 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	2.21 kN
Compression	3.20 kN
Displacement at Maximum Force	
Tension	2.06 mm
Compression	2.59 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship

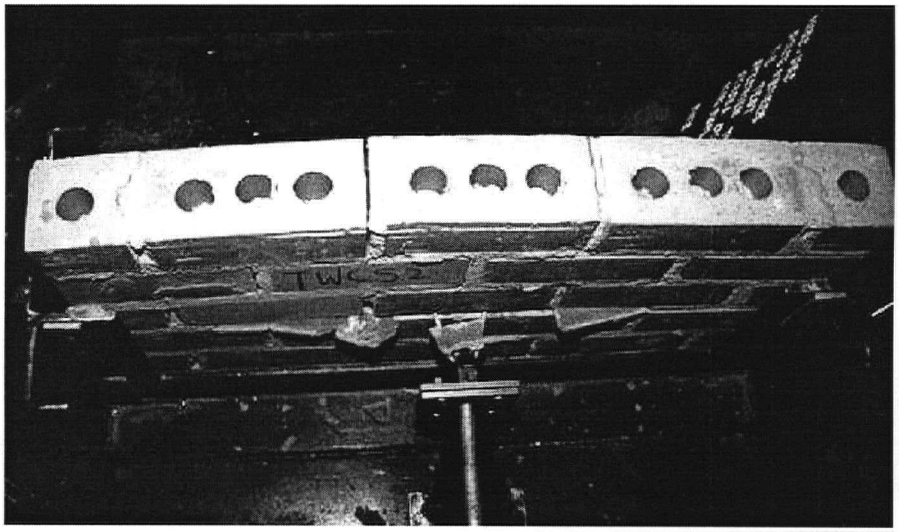


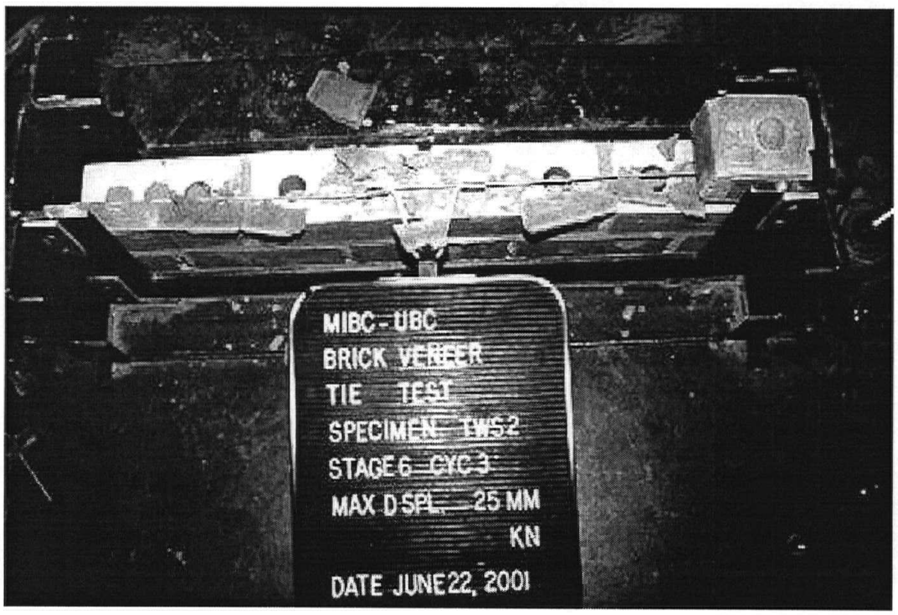
Load-Displacement Envelope Curve



Description of Test Observations:

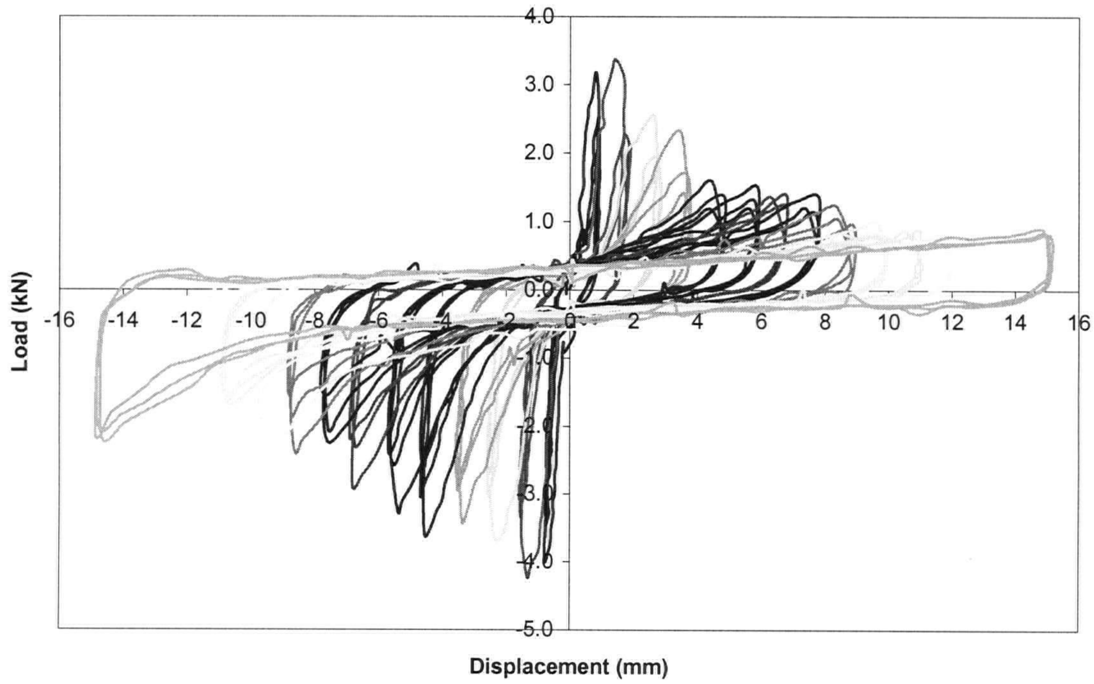
- Test was conducted using the new loading protocol.
- At the end of the force-controlled loading part of the loading protocol, cracks were apparent and a slight pull-out of crushed mortars was occurred.
- At 4 mm displacement, indication of pull-out of crushed mortars from the bed joint was apparent in tension. In compression, cracks were formed along the horizontal bed joint where the embedment of tie located.
- At 8 mm displacement, some spalling of crushed mortars occurred in tension along with more excessive pull-out of pieces of crushed mortars. In compression the horizontal cracks allowed a push-through of the loose pieces of crushed mortars from the bed joint.
- At 16 mm displacement, all pieces of crushed mortars were spalled in tension leaving a hole in the embedded tie location.
- Final displacement was 25 mm displacement. No more crushed mortars were spalled in tension. In compression, push-through of crushed mortars from bed joint were occurred.


TWS2

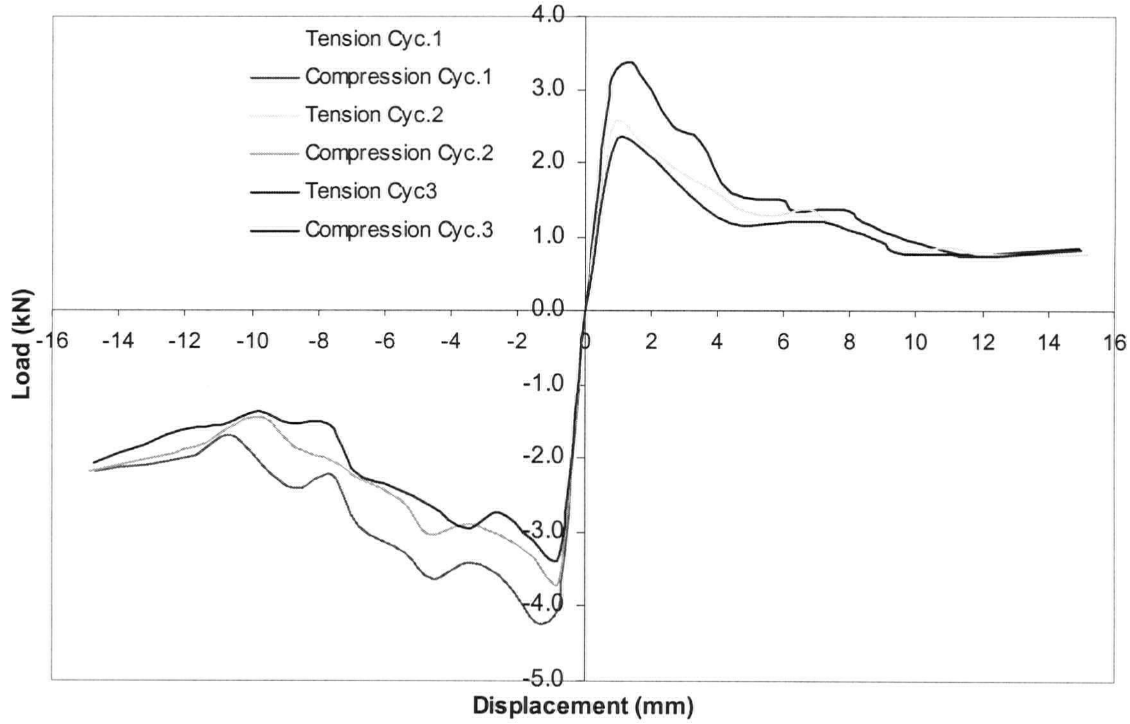

MIBC - UBC
BRICK VENEER
TIE TEST
SPECIMEN: TWS2
STAGE 6 CYC 3
MAX DSPL. 25 MM
KN
DATE JUNE 22, 2001

Specimen	TWS3
Characteristics	V-Tie with horizontal wire reinforcement clipped (modified clips) V-Tie length 80 mm Type S mortar
Test Date (age)	June 1st, 2001 (35 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	3.37 kN
Compression	4.22 kN
Displacement at Maximum Force	
Tension	1.43 mm
Compression	1.33 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship



Load-Displacement Envelope Curve

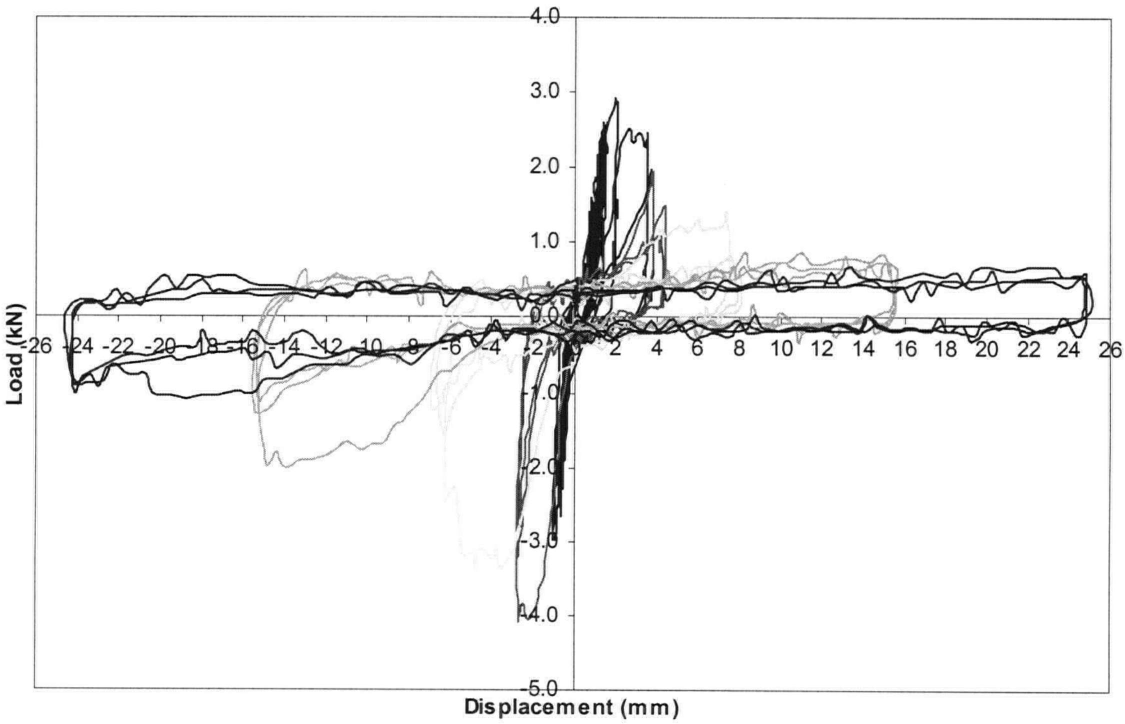


Description of Test Observations:

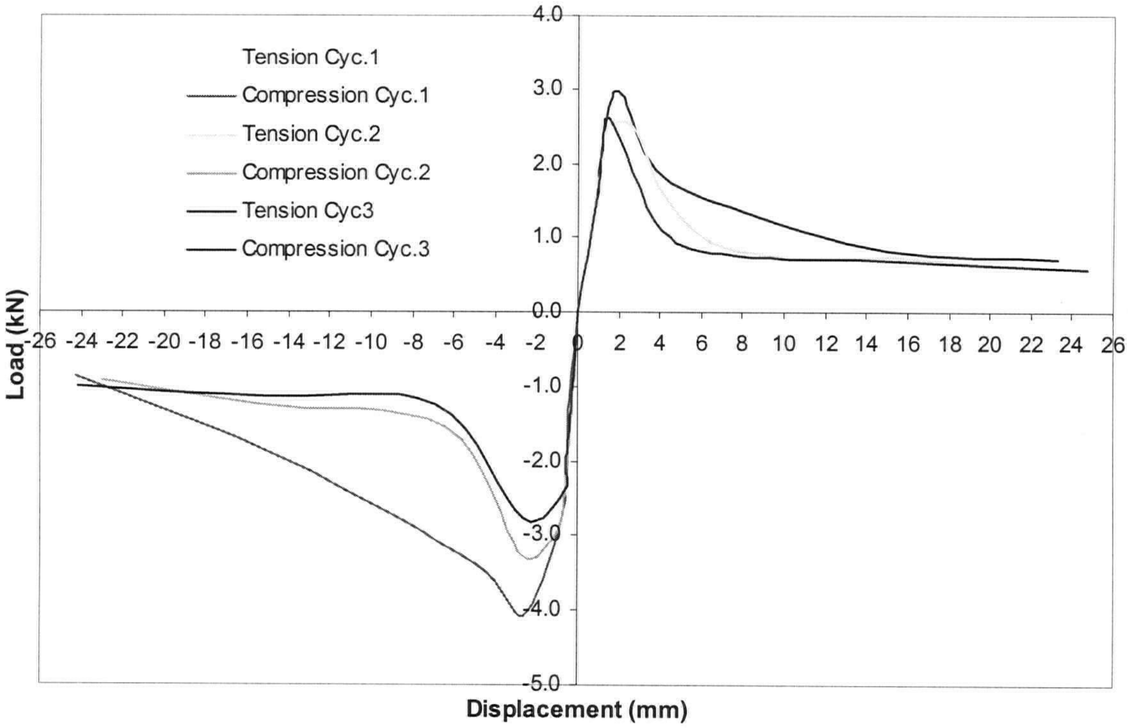
- At 2 mm displacement, cracks started to occur in tension at the location of the tie embedment. The cracks formed pieces of loose crushed mortars at both legs of the tie, which embedded into the bed joint.
- At 3 mm displacement, pieces of loose crushed mortars started to pull-out from the bed joint in tension. While several cracks appeared at the bed joint where the tie embedment located, in compression. This cracks also formed pieces of crushed mortars, which started to push-through the bed joint also.
- At 4 mm displacement, pieces of crushed mortars were spalled in tension.
- At 7 mm displacement, in compression, excessive push-through of crushed mortars from the bed joint force the spalling of this pieces. Also some of the bricks experienced some cracks and some pieces were spalled.
- At 12 mm displacement, almost all pieces of loose crushed mortars were spalled in tension, while compression showed a push-through and spalled condition of the crushed mortars from the bed joint.
- Final displacement was 15 mm with most of the crushed mortars were gone due to spalled in tension. While in compression, push-through of crushed mortars caused some spalling from the bed joint also occurred.

Specimen	TWS4
Characteristics	V-Tie with horizontal wire reinforcement clipped (modified clips) V-Tie length 80 mm Type S mortar
Test Date (age)	June 27th, 2001 (60 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	2.93 kN
Compression	4.08 kN
Displacement at Maximum Force	
Tension	2.00 mm
Compression	2.62 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship

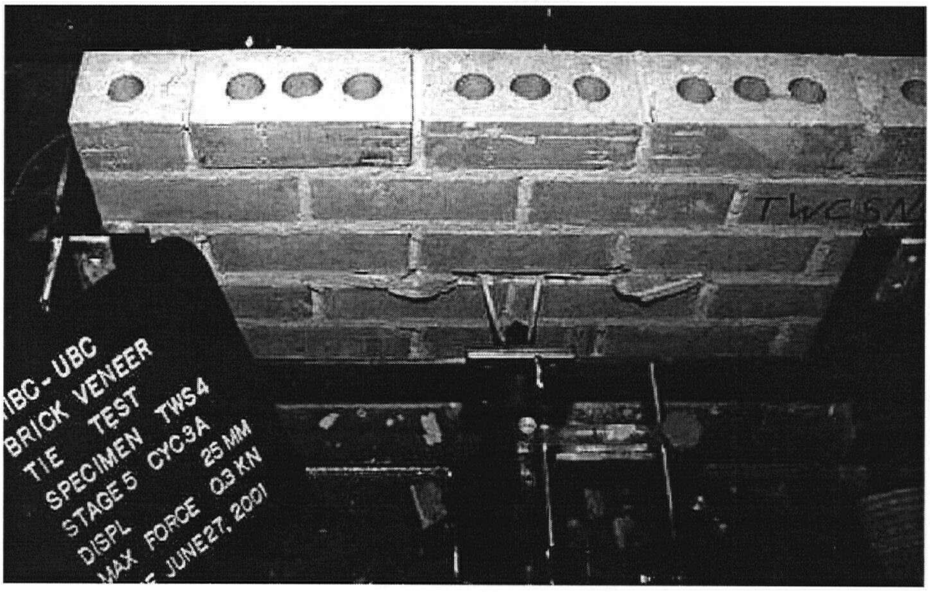


Load-Displacement Envelope Curve



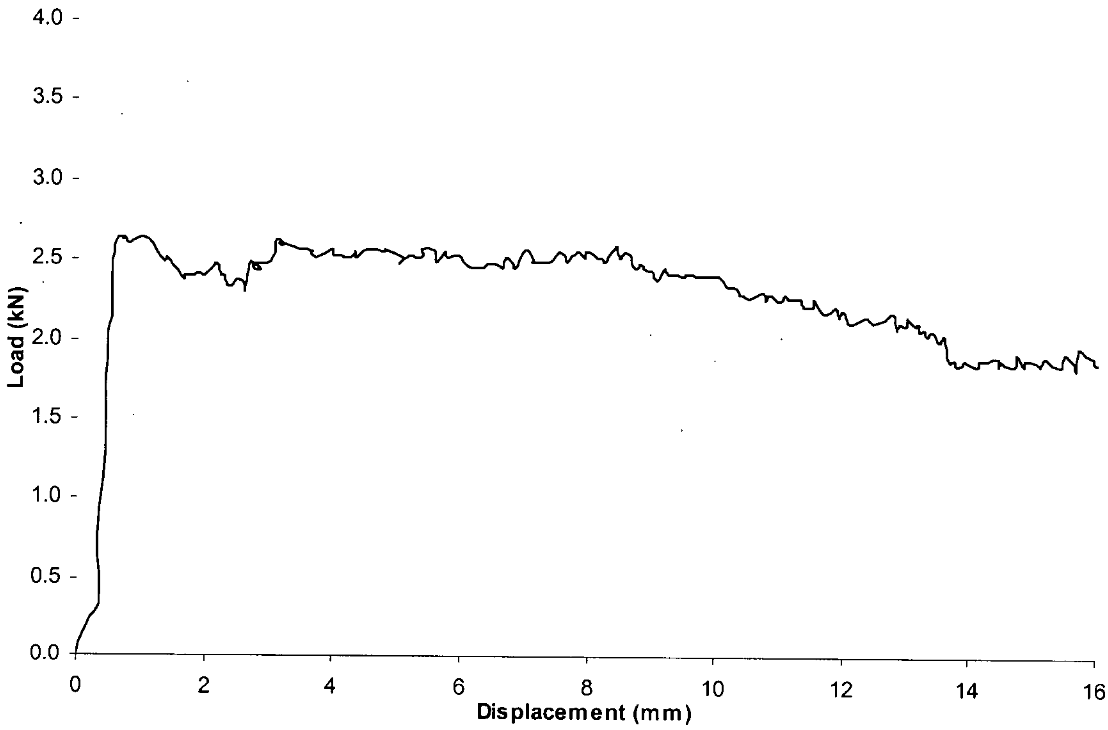
Description of Test Observations:

- Test was conducted using the new loading protocol.
- At the end of the force-controlled part of the loading protocol, vertical cracks appeared at the tie face near the embedment of the tie legs. At the tooled joint face, horizontal cracks were formed along the bed joint of the tie embedment.
- At 4 mm displacement, indication of pull-out of crushed mortars from the bed joint was apparent in tension.
- At 8 mm displacement in tension, more excessive pull-out of crushed mortars along with some spalling occurred. While in compression, horizontal cracks formed one piece of long crushed mortar (almost 2 bricks in length), which were pushed-through from the bed joint.
- At 16 mm displacement, pieces of crushed mortars were spalled in tension, leaving a hole in the area of the tie embedment. In compression, the long piece of crushed mortar was break into two. One of the pieces was spalled due to the push-through action on the bed joint.
- At 25 mm displacement, at tension side all of the loose crushed mortars were gone due to spalling. While in compression, excessive push-through of the crushed mortars from bed joint caused some spalling to occur also.


BRICK VENEER
TIE TEST
SPECIMEN TWS4
STAGE 5 CYC3A
DISPL 25 MM
MAX FORCE 0.3 KN
JUNE27, 2001

Specimen Characteristics	TWS5 - Monotonic V-Tie with horizontal wire reinforcement clipped (modified clips) V-Tie length 80 mm Type S mortar
Test Date (age)	June 21st, 2001 (55 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	2.65 kN
Displacement at Maximum Force	
Tension	0.63 mm
Failure modes	
Tension	Pull-out from mortar bed joint

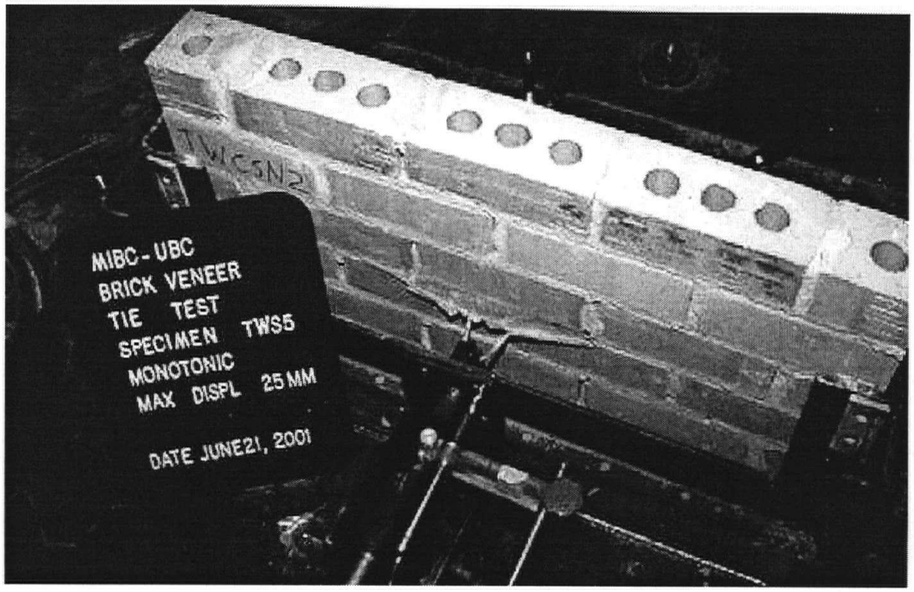
Load-Displacement Relationship



Description of Test Observations:

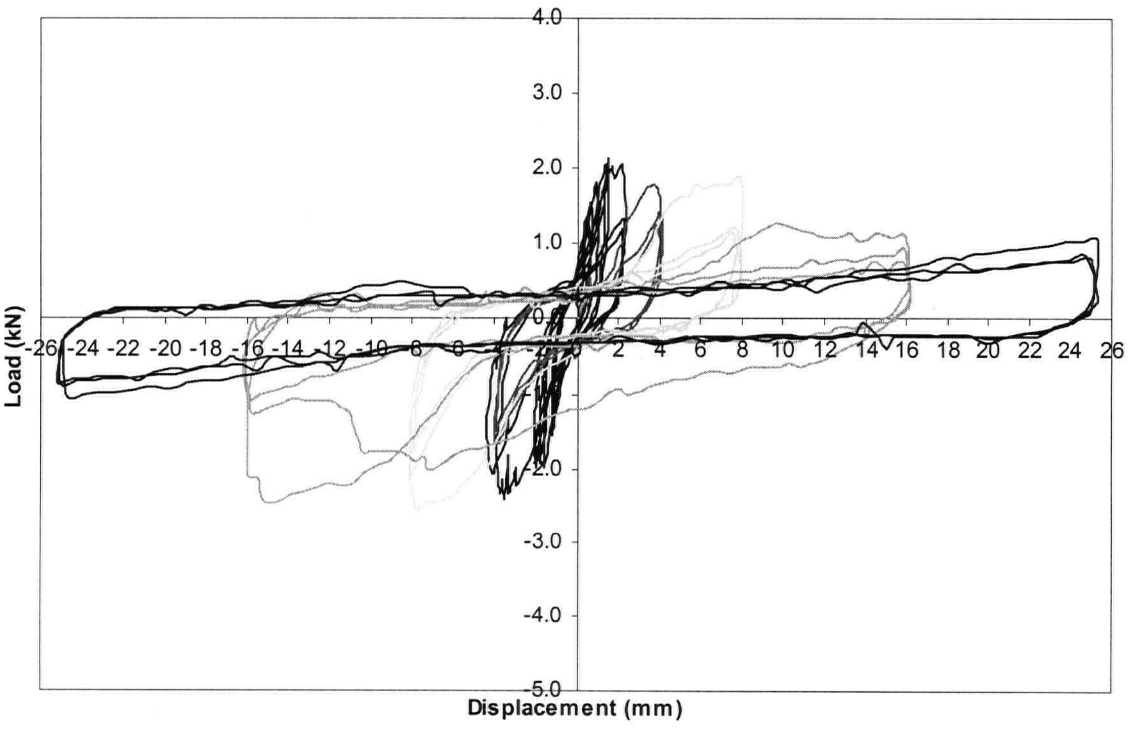
- Test was done by monotonic pull-out in tension with 15 mm as the target displacement level.
- Low peak strength was due to the disturbed specimen showing some cracks on the embedment of the tie.
- Mode of failure was a pull-out of tie from mortar bed joint with a combination of the bond failure between the mortar and the bricks, which indicated by crushed mortars pulled out from the bed joint. There was a possibility of some

deformation occurred in the horizontal wire joint reinforcement. The modified clips were still attached and relatively maintained a good connection throughout the loading test until it reached the target displacement level, which was 15 mm.

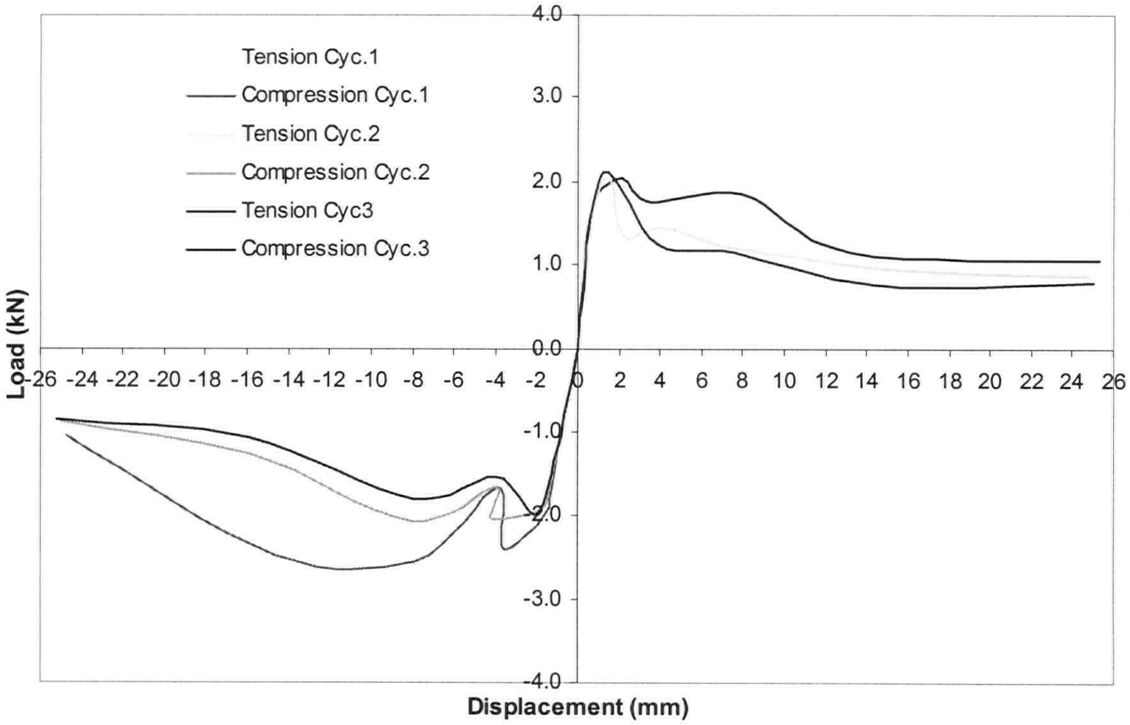


Specimen	TWS6
Characteristics	V-Tie with horizontal wire reinforcement clipped (modified clips) V-Tie length 80 mm Type S mortar
Test Date (age)	June 25th, 2001 (59 days)
Surcharge Load	4.2 kPa
Maximum Force	
Tension	2.03 kN
Compression	2.55 kN
Displacement at Maximum Force	
Tension	2.17 mm
Compression	7.84 mm
Failure modes	
Tension	Pull-out from mortar bed joint
Compression	Push-through mortar bed joint

Load-Displacement Relationship

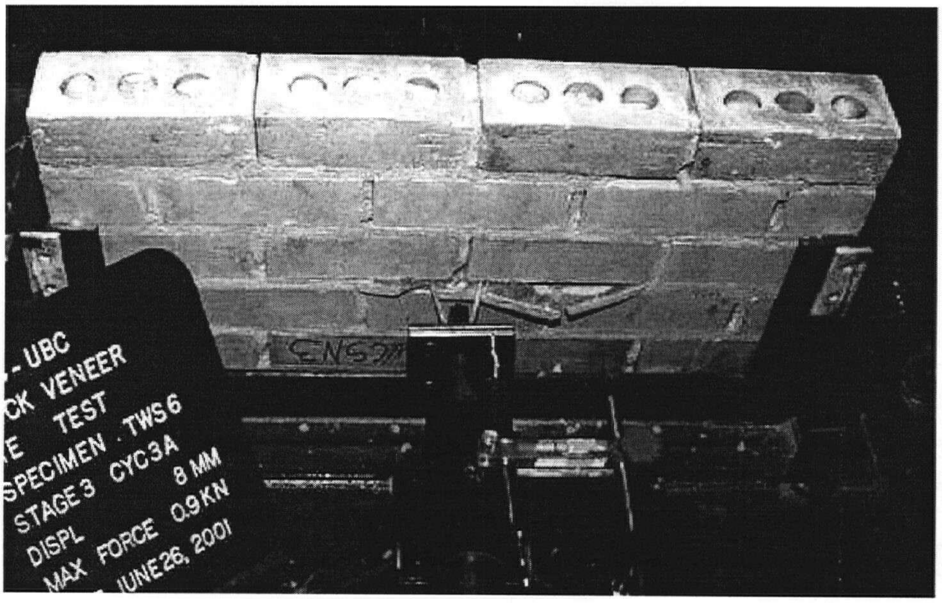


Load-Displacement Envelope Curve



Description of Test Observations:

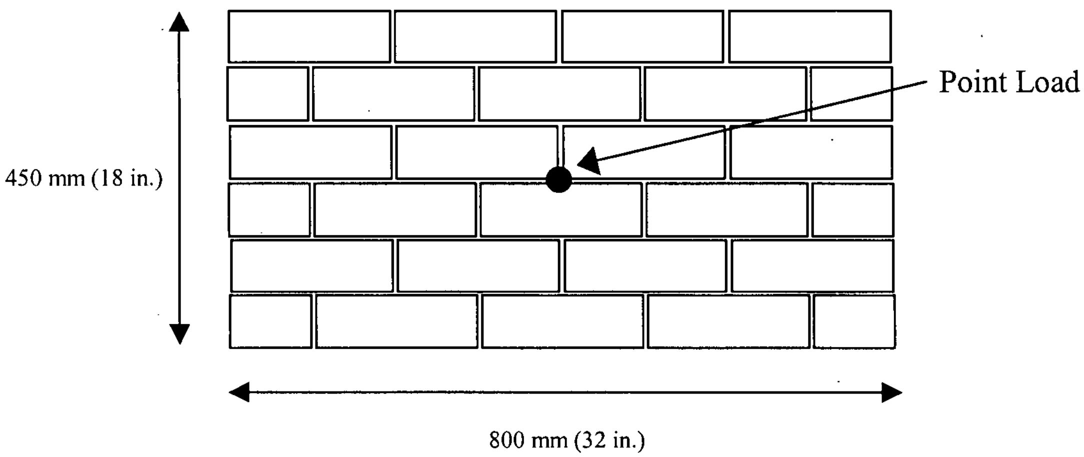
- Test was conducted using the new loading protocol.
- This specimen was turned upside-down (top course become bottom course, etc.) because of the damage suffered by the specimen.
- At the end of force-controlled part of the loading protocol, maximum loads both in tension and compression were low. This was due to the fact that the specimen already experienced severe damaged. Some cracks were already apparent at this stage of loading with some loose crushed mortars pieces already formed.
- At 4 mm displacement, pull-out of the crushed mortars in tension occurred. In compression similar thing happened with the push-through of the crushed mortars from the bed joint.
- At 8 mm displacement, excessive pull-out of crushed mortars in tension was apparent, which finally led into spalling of the crushed mortars pieces.
- At 16 mm displacement, most of the crushed mortars were gone in tension, due to spalling. In compression, more cracks were formed and some of them were spalled due to the push-through action in the bed joint.
- Final displacement was 25 mm, with crushed mortars spalled in tension, leaving a hole near the embedment location of the tie. And in compression, the specimen failed in a push-through of crushed mortars from the bed joint.


- UBC
CK VENEER
E TEST
SPECIMEN . TWS 6
STAGE 3 CYC3A
DISPL 8 MM
MAX FORCE 0.9 KN
JUNE 26, 2001

APPENDIX B

Prediction of Axial Load Capacity of Ties

First method to calculate the axial load is based on the flexural capacity of the brick specimen as specified on CSA S304.1 Engineered Masonry Design for unreinforced masonry under a point load (assuming the load from a tie).

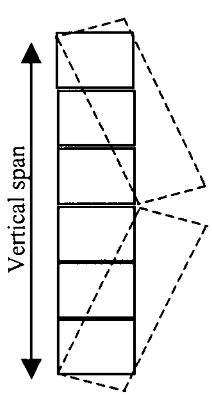


Two directions of bending action:

a) Tensile stresses are perpendicular to the bed joints – weaker. (see failure mode)
b) Tensile stresses are parallel to the bed joints – stronger.

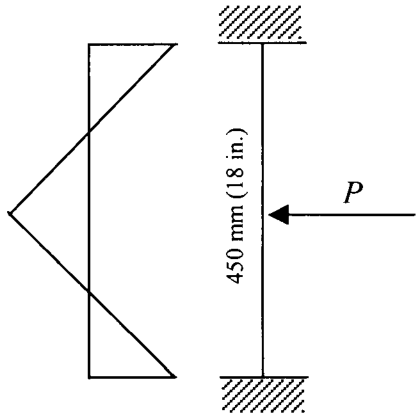
Material:
Standard clay bricks solid size: 90 mm (3.5 in.) wide x 63 mm (2.5 in.) high x 190 mm (7.5 in.) long
Mortar type: S

Flexural Capacity
a) Failure mode



Flexural tensile strength (type S mortar) f_t: 0.70 MPa (109 psi)

Assume edges are fixed:



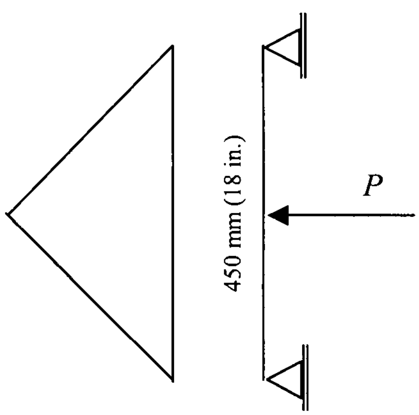
$$f_t = \frac{M}{S}, \text{ where } M = \frac{1}{8}PL$$

$$S = \frac{1}{6}bh^2$$

Maximum axial load *(P):*

$$P = \frac{8f_t bh^2}{6L} = \frac{8 \times 0.5 \times 800 \times 90^2}{6 \times 450} = 9600N = 9.6\ kN\ (2158\ lbs)$$

Assume edges are pinned:



$$f_t = \frac{M}{S}, \text{ where } M = \frac{1}{4}PL$$

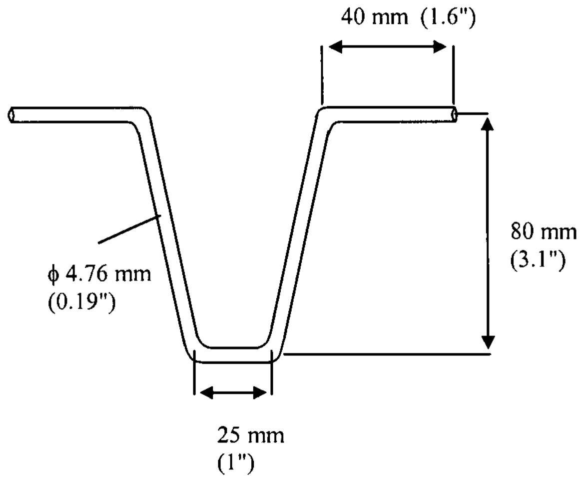
$$S = \frac{1}{6}bh^2$$

Maximum axial load *(P):*

$$P = \frac{4f_t bh^2}{6L} = \frac{4 \times 0.5 \times 800 \times 90^2}{6 \times 450} = 4800N = 4.8\ kN\ (1079 lbs)$$

For case b), due to the fact that the flexural tensile resistance parallel to the bed joints is stronger, therefore case a) always governs as the flexural capacity of the veneer wall.

Second method of calculation is based on the manufacturer's catalogue of their product, which specify of the recommended design load of the V-tie with lateral tie clip attached to each leg of the tie



Based from Fero Engineered Masonry Connectors & Accessories, 2000.

V-Tie with two lateral tie clips, one on each legs and 9-gauge wire joint reinforcement.
Design load: *0.73 kN*
Safety factor: *2.25*

Ultimate axial capacity of the tie system:
$P = 2.25 \times 2 \times 0.73 = 3285N = 3.3$ *kN (739 lbs)*

APPENDIX C

Calculation of Applied Surcharge Load

There are several sources used to compare the different method of calculating the applied surcharge load on the brick panel specimen. Due to the specimen size, which is not standard according to the code, predictions have to be used.

Based on CAN CSA A370-94 Connectors for Masonry, the surcharge load that has to be applied to the specimen is a uniformly distributed load with a value of 10 kPa.

E.F.P Burnett, M.A Postma (1995) suggested two methods of calculating the applied surcharge load to the tie from the dead weight of brick above it.

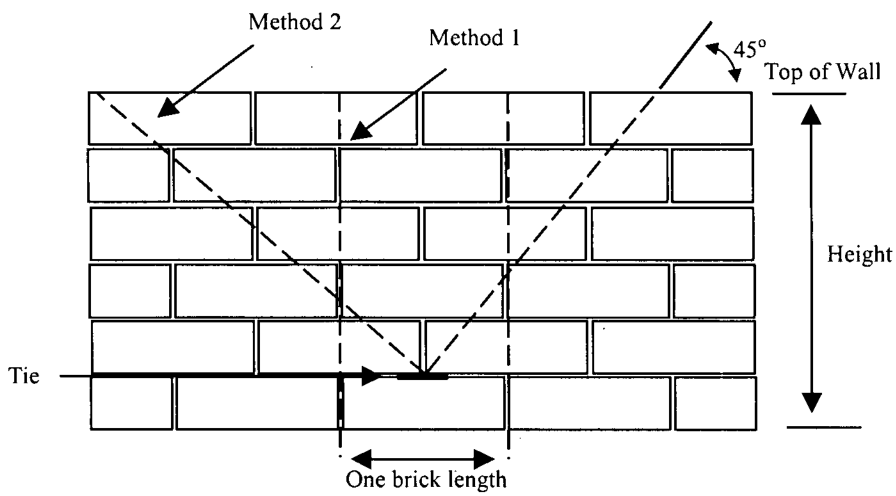


Fig. C1 – Determining the clamping force. (Adapted from ref. 9)

Method 1: Area = one length of brick x height of the specimen
Method 2: Area = weight of brickwork within a 90 degree triangle to the top level of the bricks (see figure C1)

Method 3, which this experimental study used, is a tributary area method considering the dead load from the brick above the tie and also the tensile resistance of the joints involved or engaged.

The assumption of the simulated surcharge is based on a veneer wall panel with a full height of 2800 mm.
Considering three bricks course above the tie as part of the specimen, the height of bricks that has to be simulated: 2800 – (450/2) = 2575 mm.

Comparison of the three methods results:

Method 1: Area = height x one brick length = 2575 x 200 = 515000 mm^2.
Method 2: Area = ½ x height x (2 x height) = $height^2$ = 2572^2 = 6630625 mm^2.
Method 3: Area = width of specimen x height = 800 x 2575 = 2060000 mm^2.

Veneer dead weight = 160 kg/m^2 = 1.6 kN/m^2.

Method 1: Load = 1.6 x 0.52= 0.82 kN.
Method 2: Load = 1.6 x 6.63 = 10.61 kN.
Method 3: Load = 1.6 x 2.1 = 3.36 kN.

Surcharge load for each method:

Method 1: Surcharge load = 0.82/(0.8 x 0.09) = 11.4 kN/m^2 = 11 kPa.
Method 2: Surcharge load = 10.61/(0.8 x 0.09) = 147.36 kN/m^2 = 147 kPa.
Method 3: Surcharge load = 3.36/(0.8 x 0.09) = 46.67 kN/m^2 = 47 kPa.

Tests are conducted with a total surcharge of 60 kPa, which is considered to be realistic. The surcharge load is intended to simulate the lowest row of ties that is subjected to a clamping effect from the weight of the bricks above it.