James Cook University

CYCLONE TESTING STATION

AN INVESTIGATION
of
BOND BEAM TRUSS HOLD DOWN CONNECTIONS

TECHNICAL REPORT No. 49

February 2003

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SCHOOL of ENGINEERING
JAMES COOK UNIVERSITY

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By

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Leitch, Campbell John, 1952- .
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Bibliography.
Includes index.
ISBN 0 86443 697 1
ISSN 0158 - 8338

1. Buildings - Joints - Testing. 2. Girders - Joints - Testing. 3. Roof Trusses – Testing. 4. Wind-pressure - Testing. I. Cummins, Wade, 1980- . II. Henderson, David, 1967- . III. James Cook University. Cyclone Testing Station. IV. Title. (Series : Technical Report (James Cook University. Cyclone Testing Station); no. 49).

624.1772

An Investigation of Bond Beam Truss Hold Down Connections

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Cam Leitch, Wade Cummins and David Henderson

ABSTRACT

Severe wind storm events can generate large uplift reaction forces at the roof truss to wall connections of masonry housing. A common connection detail for this joint uses 50 x 5 mm steel plates (truss cleats) threaded over the top bond beam reinforcing bar with the top of each cleat bolted to the timber truss. The top connection to the timber truss has design capacities readily available, based on detailed research into the uplift strength of various bolting and over strap arrangements using different timber joint groups. However, there appears to be little information on the strength of the truss cleat to bond beam connection, the focus of this investigation.

This research program investigated the uplift strength of truss cleat to bond beam joints by undertaking uplift load tests on a range of truss cleat connections, each anchored in a small full-scale masonry test wall constructed with the commonly used 200 mm series hollow blocks. The program included joints with one or two truss cleats, one, two or three course bond beams and different arrangements of shear reinforcement in the bond beams.

Ten different truss cleat to bond beam connections were tested in the program. For nine cases, failure involved some form of "plug cracking" whereby uplift on the truss cleats caused the core fill concrete above the top horizontal bond beam bar to crack off from the underlying concrete and move upwards, relative to the face shells of the block walls. The remaining connection failed by yielding of the vertical N12 bar that was hooked over the horizontal bond beam bar between two truss cleats.

The effect of shear reinforcing on the uplift strength was also investigated. The results showed that the uplift strength for connections in double or triple bond beams without shear reinforcing can be increased by the addition of just one shear tie located between the truss cleats and anchored to a lower course bond beam. Additional shear reinforcing, located either side of the central shear tie, provides further strength gains. However, when the truss cleats are located midway between the shear ties, the connection strength is reduced, due to local failure caused by bending of the top bond beam bar.

The varying arrangements of bond beam reinforcing steel caused different failure modes and likely failure mechanisms are postulated. Indicative uplift strength design capacities have been calculated for different bond beam truss cleat connection details based on common failure modes. Recommendations for further research are also included.

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1. INTRODUCTION

1.1 General

During severe wind storm events, large uplift forces are generated on the rooves of houses. In typical masonry housing, these forces are transferred from the roof trusses to the external masonry walls and down to the slab and footings. One critical element in this load path is the connection between the truss and the top of the masonry wall. Figure 1 shows a typical connection between a normal truss and a two course masonry lintel.

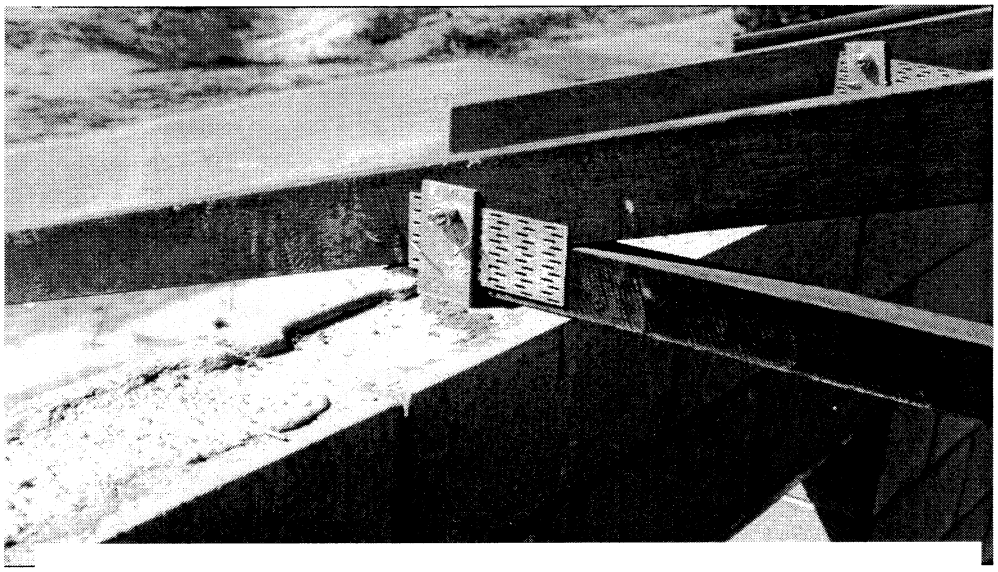


Figure 1 Typical Truss to Blockwork Lintel Connection

(Steel cleat bolted to truss and anchored in lintel)

The uplift forces to be transferred depend on both the roof tributary area and the nett uplift pressure to be supported and can be quite large. For example, a typical suburban house, located in a cyclonic area, would require a limit state uplift capacity of approximately 10 kN at each truss hold down point. For houses in more exposed locations, the roof will be subjected to larger uplift pressures and the truss supports will require larger hold down capacities.

Girder trusses, used in hip roofed houses (or houses with a complex roof shape) can have much larger tributary areas and hence require significantly larger hold down capacities. Typical strength limit state hold down capacities required lie within the range of 15 to 50 kN, but larger values are not uncommon.

These uplift forces are transferred from the truss to the cleat and then from these cleats to the bond beam at the top of the masonry wall. The Cyclone Testing Station has previously investigated the strength of the joint between the timber truss and the truss cleat (Technical Report No. 44) and provides guidance on design capacities of this joint. However, there appears to be little information on the strength of the truss cleat to bond beam connection, the focus of this investigation.

1.2 Background to Research

Engineers from the Consulting Engineers Group – Townsville, a sub-group of *The Institution of Engineers, Australia*, were concerned about the lack of information on the capacity of the truss cleat to bond beam connection. Due to their concerns, the CMAA Queensland Promotions Committee agreed to provide part sponsorship for a fourth year engineering student to undertake research into the strength of this connection, as a thesis project. The student who undertook this project is Mr. Wade Cummins and his thesis entitled "*Bond Beam Truss Hold Down Connections*", completed in late 2002, gives full details on this research.

The Cyclone Testing Station contributed significantly to this investigation by providing guidance and supervision from their research engineers, technical support and the use of CTS testing equipment. As well, the CTS undertook to produce this technical report, to ensure that the findings from this research are readily available to the building industry.

1.3 Aims of Research

The main aims of the research program were as follows:

- Conduct uplift load tests to determine the failure load for a range of the commonly used truss cleat to bond beam connections.
- Identify and comment on any typical failure mechanisms observed.
- Determine Limit State Design values for these connections (subject to the limitations of the necessarily small number of connections tested).

2. Current Practice

There are three different types of truss tie-down detailed in the CMAA (2001) Single Leaf Masonry Design Manual. Two of these methods, threaded rods either side of the truss and tie-down to a timber top plate on top of the bond beam (as detailed in Tables 5.1 and 5.2 of this manual) are rarely used. Cummins (2002) confirmed that the third approach, using 50 x 5 x 200 steel truss plates (cleats) threaded over the bond beam reinforcement and bolted to the truss (detailed in Table 5.3 of the CMAA Design Manual) is the main method used for truss tie-down for masonry houses in North Queensland. Therefore this research considered the truss plate tie-down method only.

It is interesting to note that Clause 12.6.8 of the current masonry structures design standard, *AS 3700-2001*, specifies that "*..the roof shall be independently tied to the footing..*" for roof connections to masonry walls for wind categories N3 and higher. However, the standard does not give details on how to achieve an **independent** tie down. Current practice relies on the transfer of roof uplift forces from the truss cleat to the top bond beam bar, then to vertical reinforcing that is hooked or cogged over this top bond beam reinforcement and lapped with starter bars from the footings.

3. Test Walls

3.1 Masonry Wall Details

The test walls and bond beams were constructed with the commonly used 200 mm series hollow blocks with a characteristic unconfined compressive strength of masonry unit

(f'_{uc}) of 15 MPa. These blocks are actually 190 mm wide. Full height, plain face 20.01 and 20.03 blocks were used for the lower sections of the walls and 20.20 knock out bond beam blocks were used for the bond beams.

Mortar Type M3, a 1:1:6 cement, lime and sand mix was used in this study as per the CMAA design manual and industry standards.

The core fill concrete was specified with a design characteristic compressive strength of grout (f'_{cg}) of 20 MPa.

The main horizontal and vertical reinforcing bars used were the recently introduced *AS/NZS 4671* "N" bars with a steel yield strength (f_{sy}) of 500 MPa. All vertical reinforcing steel used N12 bars, whilst both N16 and N12 bars were used for the horizontal bond beam bars. Details on the standard hook used at the top of all vertical N12 bars are provided in Appendix C.

3.2 Component Details

Standard, commercially available truss cleats, fabricated from Grade 400 Duragal steel, were used. They were nominally 200 mm long, 50 mm wide and nominally 5 mm thick. Two 20 mm diameter holes were punched in each end of the cleats.

Type "V" and "W" wall ties, nominally 4 mm in diameter, were used to locate the horizontal reinforcing bars in the bond beams

W8 shear ties (labelled DW 7.6 mm) were used in some of the two and three course bond beams. These were a single vertical stirrup with a hook each end so that the stirrup (tie) can be anchored over the top and bottom horizontal bond beam bars.

Additional details and photographs of the truss cleats, "V" and "W" wall ties and the W8 shear ties have been included in Appendix C.

3.3 Test Wall Details

The following parameters were varied in the testing program, the number of truss cleats, the number of courses in the bond beam, the shear reinforcement used and the spacing between the vertical N12 reinforcing bars.

Single truss cleats were used for tie-down details with smaller values for the estimated connection failure load. However, a double cleat which used two truss plates threaded onto the same bond beam bar and set with a clear internal width between the truss plates of 35 mm, was used when the estimated failure load was 35 kN or larger.

A range of different bond beams were tested, namely single, double and triple course bond beams. For most of these bond beams, a single N16 bar was used for the horizontal reinforcement to the top and bottom course, but Wall 10 used 2N12 bars to each course of a two-course bond beam.

Shear reinforcement (ties or fitments) were used in some of the bond beams. For Test Wall 8, a single shear tie was used and hooked over the N16 horizontal bond beam bar between the two truss cleats. For all other cases where shear reinforcement was used, the ties were horizontally spaced 200 mm apart (one tie to each "pot" of the bond beam). However, in some cases the ties were placed so that the double truss cleats were midway between the 200 mm spacing and in other cases the two truss cleats were placed either side of a tie.

The spacing of the vertical reinforcing bars, which were all N12, was set at either 1200 mm or 1800 mm, two of the commonly used spacings in practice. For Test Wall 9, three vertical N12 bars were used with the central bar hooking over the N16 horizontal bond beam bar between the two truss cleats. This bond beam cleat arrangement models the method used for tie-down of girder trusses. All of the other test walls used two vertical reinforcing bars.

A copy of elevations and sections for each of the ten test walls is shown in Figures A1 to A4 inclusive in Appendix A.

Table 1 below lists the details of the main test wall parameters. Also included in this table is the value for the estimated uplift failure capacity for each of the cleat connection details to be tested.

Table 1 Test Wall Details

Wall No.	Estimated Uplift Failure Capacity (kN)	Bond Beam Details			Truss Cleat Details			Horizontal Distance Between Vertical Wall Reo (N12 bars) (mm)
		No. of Courses	Shear Reo	Horiz Reo	No. Off	Horizontal Distance to Nearest Shear Tie (mm)	Horizontal Distance to Nearest Vertical N12 Wall Reinforcing (mm)	
1	15	1	NIL	1N16	1	N/A	600	1200
2	35	2	W8 @ 200	2N16	2	80 Nom	900	1800
3	20	2	NIL	2N16	1	N/A	600	1200
4	25	3	NIL	2N16	1	N/A	600	1200
5	55	2	W8 @ 200	2N16	2	Tie Betw. cleats	600	1200
6	55	3	W8 @ 200	2N16	2	Tie Betw. cleats	600	1200
7	45	3	W8 @ 200	2N16	2	80 Nom	900	1800
8	35	2	Single W8	2N16	2	Tie Betw. cleats	900	1800
9	70	3	NIL	2N16	2	N/A	N12 Betw. cleats	1200
10	40	2	W8 @ 200	4N12	2	Tie Betw. cleats	600	1200

3.4 Construction of Test Walls

The test walls were constructed on the 20 m x 15 m open concrete slab located at the full scale house testing facility of the Cyclone Testing Station. A summary of the construction process follows:

- The vertical N12 reinforcing bars with hooked tops were epoxied into place ensuring that the hooks were at the correct level and orientated the correct way.
- The first course of blocks was laid for each wall. This course was not filled with grout except where the vertical bars passed through. Consequently, sheet-metal block outs were placed above the pots to exclude grout from this course. Note that Test Wall 1 used two courses below the single course bond beam, all other walls used just a single unfilled course below the bond beam.
- The bond beams were then constructed above the initial course. The horizontal bond beam bars were installed and held in position using "V" or "W" wall ties.
- The top horizontal bond beam bars were installed so that they were located hard up against the inside bend of the hooked N12 vertical reinforcing bars.
- The truss cleats were threaded onto the top horizontal bond beam bars and the shear reinforcement installed where specified.
- The bond beams and pots with vertical bars were poured with grout. The truss cleats were adjusted to vertical and the double cleats set 35 mm apart.
- The grout was allowed to cure for approximately 5 weeks before the first uplift test was performed.
- Demec gauge points were glued onto the front (test surface) of the wall in the middle of each block directly under the cleats. These points allowed the vertical movement between adjacent block work courses to be accurately measured. As well, the front face and top of the walls were whitewashed so that cracking could be more easily seen.

Figure A5 in Appendix A shows schematic details of a typical test wall.

Figure 2 is a photograph showing Test Wall 1 after construction, but before the grout was placed.

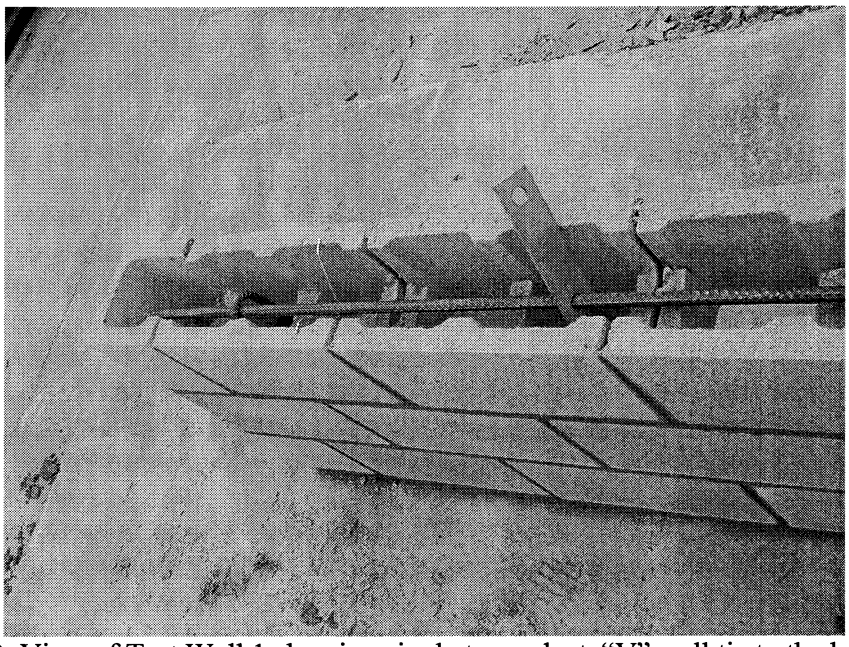


Figure 2 View of Test Wall 1 showing single truss cleat, "V" wall tie to the horizontal N16 bond beam reinforcing bar and the hooked N12 vertical bar to one end.

4. Test Apparatus and Procedure

4.1 Cleat Connection Devices

Two cleat connection devices were fabricated to allow uplift loads to be applied to the truss cleats. For the single truss cleat, the cleat connection device was fastened to the cleat using an M16 high strength bolt through steel plates either side of the truss cleat. The other end of the cleat connection device was screwed to a 2 m long, M20 high tensile steel threaded rod. By using plates to both sides of the truss cleat, the uplift load could be applied concentrically to the cleat. A view of the single cleat connection device can be seen in Figure 4(a).

For the double cleats, the cleat connection device consisted of a length of 50 mm diameter steel bar that was machined with flats each side so that it could fit between the two truss cleats. This allowed an M16 high strength bolt to be bolted through the cleats with the cleat connection device between them. The other end of this cleat connection device was screwed to the same 2 m long, M20 high tensile steel threaded rod. Again, this connection detail ensured that the uplift loads were applied concentrically to the cleats. A view of the double cleat connection device can be seen in Figure 4(b).

4.2 Loading Apparatus

Figure 3 shows the loading apparatus set up over Test Wall 1, prior to loading.

As can be seen in the photograph, the loading frame was placed directly over the cleat, perpendicular to the wall and with one leg approximately 200 mm from the back of the wall. The cleat connection device was fastened to the cleat (or cleats). A 500 kN "Doughnut" load cell was placed on top of the loading frame directly above the cleats and then a 300 kN hydraulic ram was placed on top of the load cell. The 2 m long threaded rod was passed through the central holes in the loading ram and the load cell and then screwed into the cleat connection device. A hand pump was used to drive the hydraulic ram and apply the uplift force to the truss cleats. The load applied was monitored by a digital load display connected to the load cell.

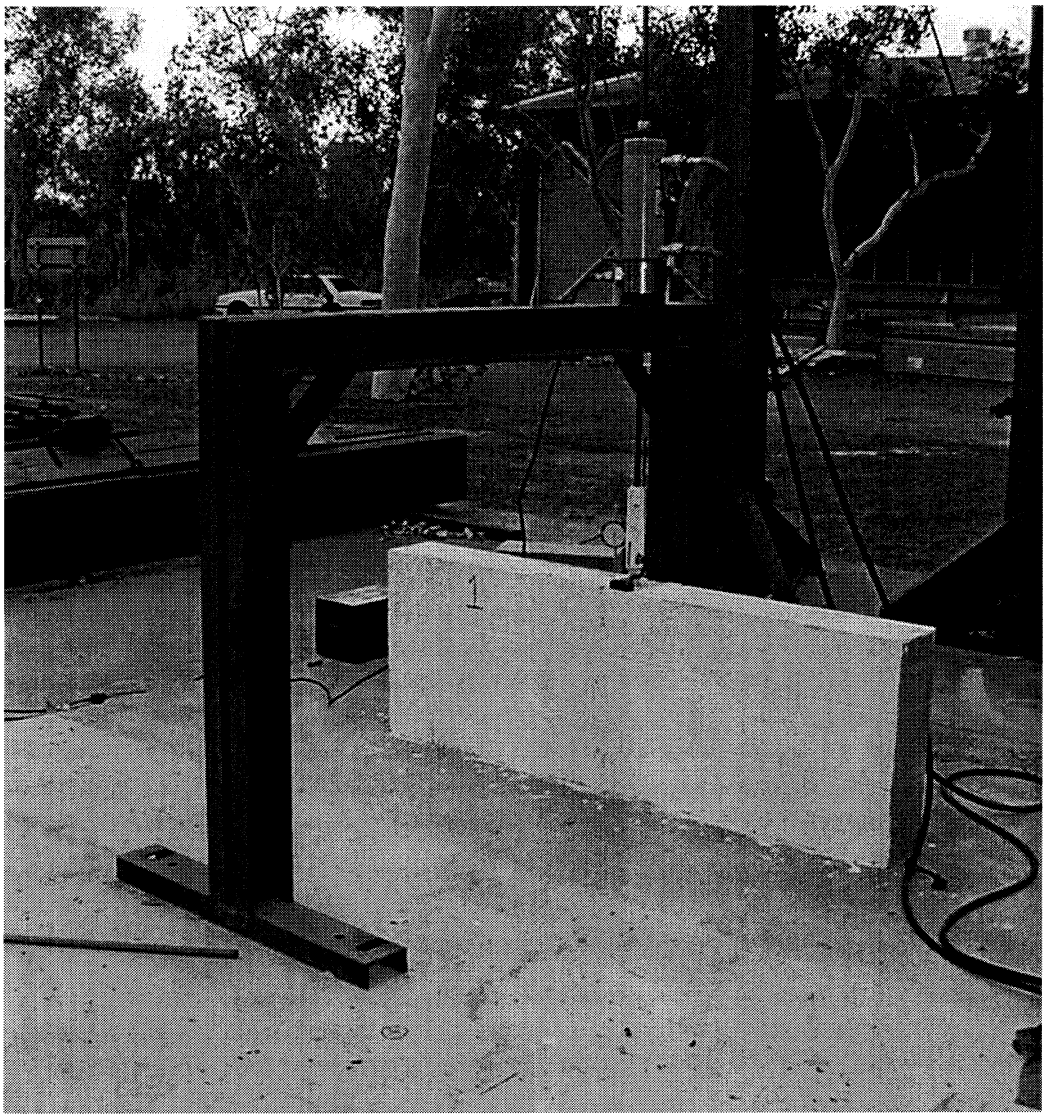
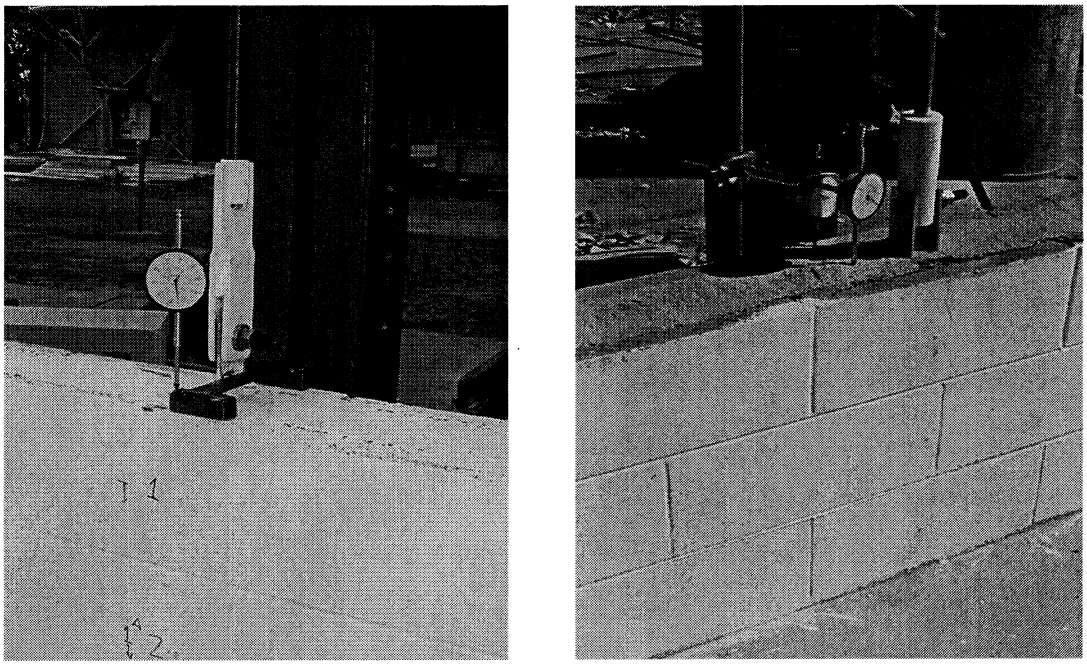


Figure 3 General View of Test Wall 1 Showing Loading Frame Set Up.

Note: Single cleat connection device used and only Dial Gauge 1 Installed.

4.3 Measurement of Vertical Movements

A dial gauge (labelled *DG1*) was set up, 75 mm to the left of the cleat, to measure the vertical movement of the centre of the concrete fill relative to the ground. During uplift loading, this corefill concrete, called the "*Plug*", was found to move separately from the immediately adjacent masonry face shells. Therefore, at a review of the testing procedure a second dial gauge (*DG2*) was installed to measure the vertical movement of the back masonry face shell relative to ground. This dial gauge was also located 75 mm to the left of the cleat. Figure 4 provides two views of different test walls that show these dial gauge positions and the two different truss cleat connection devices.



4 (a) **4 (b)**

Figure 4 Detailed Views of Dial Gauges and Truss Cleat Connection Devices
4 (a) Test Wall 1 with DG1 and the "*Masonry Block to Cleat Mark Stand*".
4 (b) Test Wall 2 with both DG1 & DG2. Note Demec gauge points below double cleat.

Figure A6 in Appendix A shows two schematic cross sections through a typical 4 course test wall which illustrate where the various vertical movement readings were taken. As shown in Figure A6, the dial gauges measure vertical movement of the top of the wall relative to the ground. Dial gauge DG1, which was always used, records the vertical movement of the centre of the concrete fill ("*Plug*") relative to the ground. Dial gauge DG2 was used for the second half of the testing program (used on Walls 2, 4, 6, 7 and 9). DG2 measures the vertical movement of the back masonry face shell (at the top of the wall) relative to ground. The vertical movement of the concrete "*Plug*" relative to the top rear masonry face shell will be equal to (ΔDG1 - ΔDG2).

Gauge points were glued to the centre of the front face of each course of blocks, directly in front of the cleats. The gauge point to the top course was labelled "A", the next course labelled "B" and so on, as shown in Figure A6. The Demec gauge points allow the vertical movement between each block course to be measured. These measurements were taken on the front face, between all of the courses, for each test wall. The vertical movement between the ground and the first (bottom) course could not be measured using these gauge points. However, if the vertical movement between ground and the first course is assumed to be negligible and there is no cracking between the top of the front masonry shell and the top Demec gauge point (Point A), then the sum of the Demec gauge point movements will be equal to the vertical movement of the front top masonry face shell. Provided that the vertical movements of the top front and top rear masonry shells (DG2) are similar, then the sum of the Demec gauge point movements should be approximately equal to the DG2 values.

The vertical movement of the truss cleat relative to the top of the front and rear face shells of the immediately adjacent block was measured relative to a horizontal line drawn on the cleat across the top of the "*Masonry Block to Cleat Mark Stand*", refer to Figure A6.

4.4 Test Procedure

The loading apparatus was set up above the test wall to be loaded. The truss cleats were loaded with a tensile (uplift) force using the hand pump to apply a steady increase in load to the hydraulic ram. At either 5 or 10 kN load increments hand pumping of the ram was paused to allow vertical movement readings to be taken. These readings involved measuring the vertical movements at the dial gauges, the vertical movement of the cleat relative to the face shell of the block and the movement between the Demec gauge points. The wall was also examined for cracks and these were drawn on a cracking map. The test wall cracking patterns at failure, are detailed in Cummins (2002).

The loading was steadily increased until the truss cleat connection failed. When failure appeared to be imminent, the dial gauges were removed to ensure that they were not damaged.

After load testing was complete, some test walls had sections of the masonry face shells removed to allow inspection of the cracking patterns in the core fill concrete and to examine the steel reinforcing for signs of failure.

5. Results

5.1 General Test Wall Results

A copy of the measured test results is provided in Tables B1 to B10 inclusive in Appendix B. These tables list the vertical movements readings measured and two calculated values of vertical movement, for each load increment.

The first calculated value is the sum of the Demec gauge point vertical movements and this value will be approximately equal to the vertical movement of the top of the rear masonry shell, which is the DG2 reading, as noted in Section 4.3. The second calculated value in Appendix B is the approximate vertical movement of the truss cleat relative to ground. It is calculated by adding the vertical movement of the cleat, measured by the "*Masonry Block to Cleat Mark Stand*", to the sum of the Demec gauge point movements and has been rounded to the nearest 0.5 mm. When calculating these values for double truss cleats, the average truss movement was used.

Also included below each table in Appendix B is the "*Top Surface Length of Plug*" (defined in Figure 5), and both the estimated and actual failure loads for the bond beam truss hold down connection on each wall. Note that the maximum (peak) load was recorded as the cleat connection failure load for most walls. However, as noted in Section 6.6, the local failure load (less than the peak load) was adopted as the cleat connection failure load for Walls 2 and 7.

Failure of the bond beam cleat connection was always associated with cracking of the core fill concrete and/or the masonry face shells. In most cases, the top (horizontal) bond beam bar was lifted upwards by the truss cleats. This caused the corefill concrete above this bar to crack off from the underlying concrete and also move upwards, relative to the face shells of the block wall. This cracking of the corefill concrete continued horizontally for some distance until further away from the truss cleats there was little or no upwards movement of the horizontal bond beam bar. At this point, the cracking extended diagonally upwards to the top surface of the bond beam. The upwards movement of this portion of the corefill concrete was called "*plug cracking*" and it is shown in Figure 5 below.

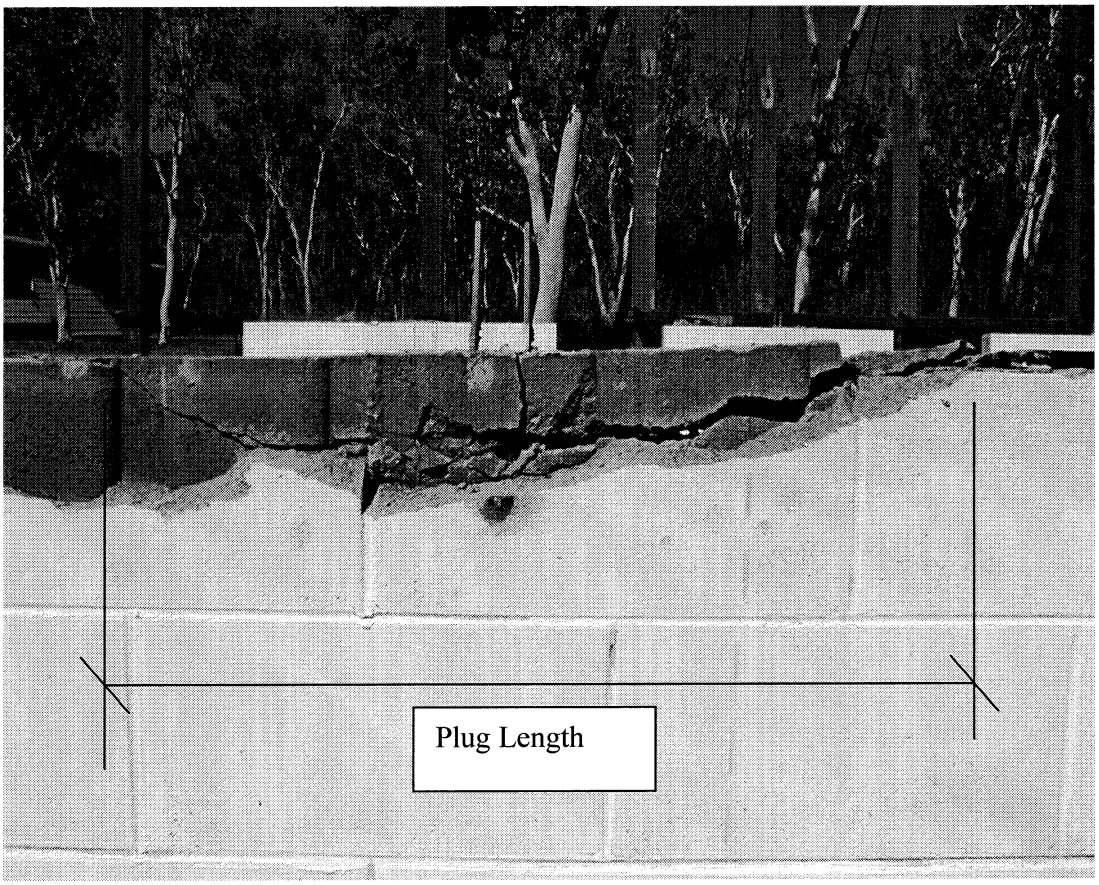


Figure 5 Typical Plug Cracking

More detailed descriptions and comments on the different failure mechanisms for the cleat connections are provided in Section 6 of this report.

A summary of the failure loads with their associated vertical cleat movements and calculated vertical wall and cleat movements for all ten test walls is provided in Table 2.

Table 2 Summary of Test Results and Calculated Wall and Cleat Movements

Wall No.	Cleat Connection Uplift Failure Loads (kN)	Vertical Movement of Cleats relative to Face Shell of Block (mm)		Calculated Values (mm)	
		Left Cleat or Single Cleat	Right Cleat	Sum of Demec Gauge Point Movements	Approximate Average Movement of Cleats Relative to Ground
1	19.6	9	Not Applicable	1.90	11
2	38.0 #	10	9.5	2.62	12.5
3	27.8	14	Not Applicable	0.75	15
4	23.2	13	Not Applicable	0.24	13
5	50.1	3	Not measured	1.79	5
6	47.0	16	15	0.60	16
7	35.2 #	7.5	9	0.55	9
8	40.1	14.5	5.5	1.60	11.5
9	72.0	3	2	1.40	4
10	37.2	12	8	0.98	11

Note: Local failure loads adopted for Wall Nos. 2 and 7 because truss cleats (and core fill concrete plug) moved vertically upwards approximately12 mm, see Tables B2 and B7.

5.2 Approximate Vertical Movements

The vertical movement measurements can be used to give some general comments on the overall patterns of movement of various components of the bond beam cleat connections.

During uplift loading on the truss cleats, the face shells of the masonry blocks adjacent to the cleats tended to move upwards for most of the walls tested. A general indication of these movements can be gained by examining the Demec gauge point movements, as tabulated in Appendix B. In many cases there was relatively more vertical movement between the courses immediately below the lower course of the bond beam than between the courses of the double or triple bond beams. However, this trend is not apparent for Walls 3 and 4 where the top course of the bond beam was not connected to the lower courses by shear ties. This is consistent with the observation that the lack of shear ties in Walls 3 and 4 meant that the lower courses did not contribute very much to the strength of the truss cleat connection.

Apart from Wall 9, the corefill concrete adjacent to the cleats tended to move vertically upwards relative to the immediately adjacent masonry face shells. This was called "plug cracking" and it was noticeable for most walls, see Figures 5 and 10 for example.

The truss cleats used in this study had an oversize (20 mm diameter) hole that was threaded over the horizontal bond beam bar (N16 bars except for Wall 10 which used N12 bars). When the walls were poured, the cleats sat on top of the horizontal bond beam bar which caused a nominal 4 mm void between the bottom of the bar and the bottom of the cleat hole. Consequently, there was a potential for the cleats to move upwards relative to the "plug" of corefill when the uplift loading was applied to the cleats. However, there was no evidence of this relative movement during the testing performed in this study.

5.3 Component Strength Results

Tensile testing of typical truss cleats, W8 shear ties and one N12 reinforcing bar was performed to determine their yield and failure capacities and the results are included in Appendix C.

6. Modes of Failure

6.1 General

For these ten test walls, the truss cleat connection uplift strength was controlled by failure in the bond beams. The varying arrangements of the bond beam reinforcing steel caused different failure mechanisms. Cleats with similar failure modes have been grouped together and are discussed below.

6.2 Comparison to Bond Beam Design Strengths

Cummins (2002) undertook calculations, (using both *AS 3600-2001* and *AS 3700-2001*), to determine the maximum uplift point loads that should be able to be supported at the cleats. He assumed that the bond beams were simply supported beams and calculated the maximum limit state central uplift point loads that could be supported if the full design strengths of the bond beams in both shear and bending were utilized. These calculations showed that the truss cleat failure loads were significantly less than the values needed to utilize the full design strengths of the bond beams. Thus these cleat connection uplift strengths were controlled by more critical "local" failures rather than the overall strength of the bond beams.

6.3 Walls 1, 3 and 4

These three walls used single, double and triple bond beams, all without shear ties and the connections all failed in a similar manner. The single truss cleat "lifted up" the top (horizontal) bond beam bar that it was threaded over and caused failure in the corefill concrete above this bar. Figure 6 shows a photograph of Wall 1, after failure of the cleat connection.

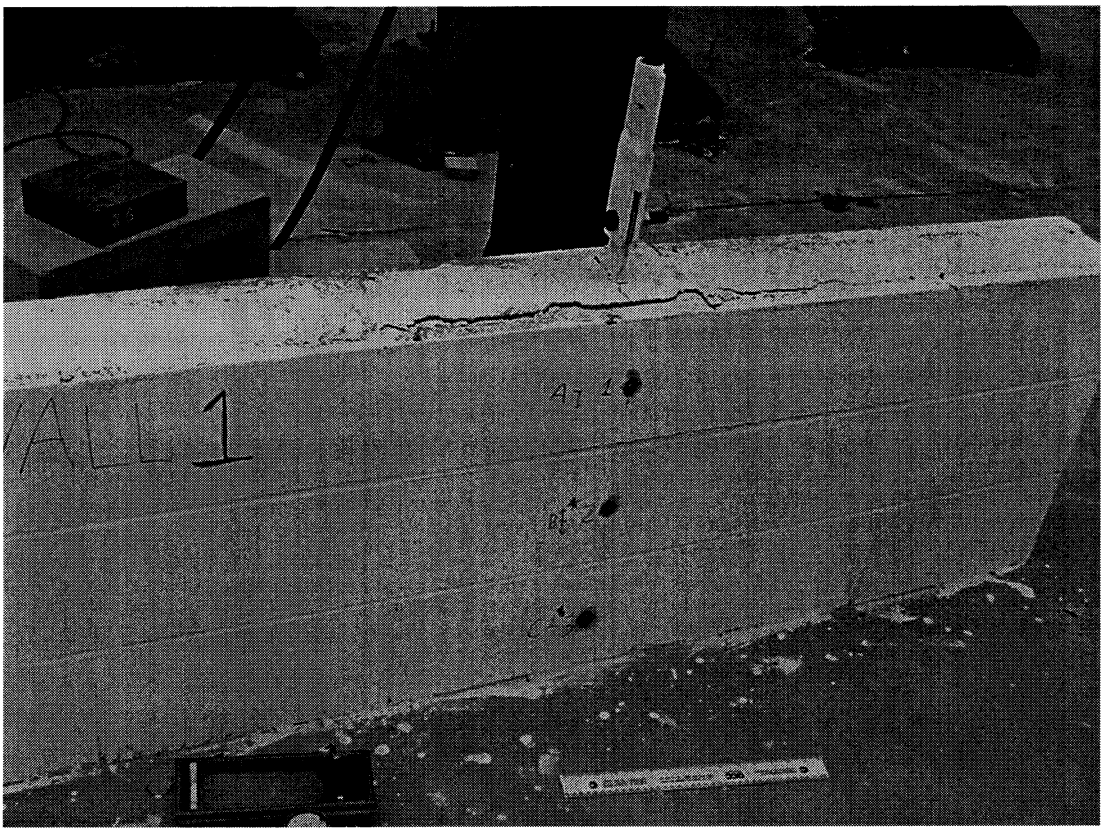


Figure 6 General View of Wall 1 after failure

The likely failure mechanism for these connections is postulated to be as follows:

At low loads the upward movement of the top bond beam bar was prevented by the restraint from the corefill concrete in the top course. However, as the uplift load on the bar was increased, local cracking across a horizontal plane through the bar occurred when the tensile strength of the corefill concrete was exceeded. This caused the corefill concrete and cleat to move vertically upwards relative to the face shells of the immediately adjacent block. This is called "*plug cracking*".

The relative movement between the corefill concrete (the "plug") and the block face shells can be clearly seen in Figure 6 and is a maximum at the truss cleat and reduces as the horizontal distance away from the cleat is increased.

Figure 7 shows a view of Wall 1 with the block face shells removed where the plug cracking is clearly visible.

The three course bond beam for Wall 4 had a lower cleat failure load (23.2 kN) than that for the two course Wall 3 connection (27.8 kN). These values are essentially the same, within the bounds of likely experimental variation and this is likely to be caused by the failure mechanism for both walls being dominated by the plug cracking failure in the top course. In other words, the additional capacity possible from the third course of Wall 4 is not utilized because the truss cleat connection is not anchored into it.

Both Walls 3 and 4 had larger connection failure loads than that measured for Wall 1 (19.6 kN), so it appears that the second course adds some additional strength to the connection. However, the lower failure load for Wall 1 could also be due to experimental variation.

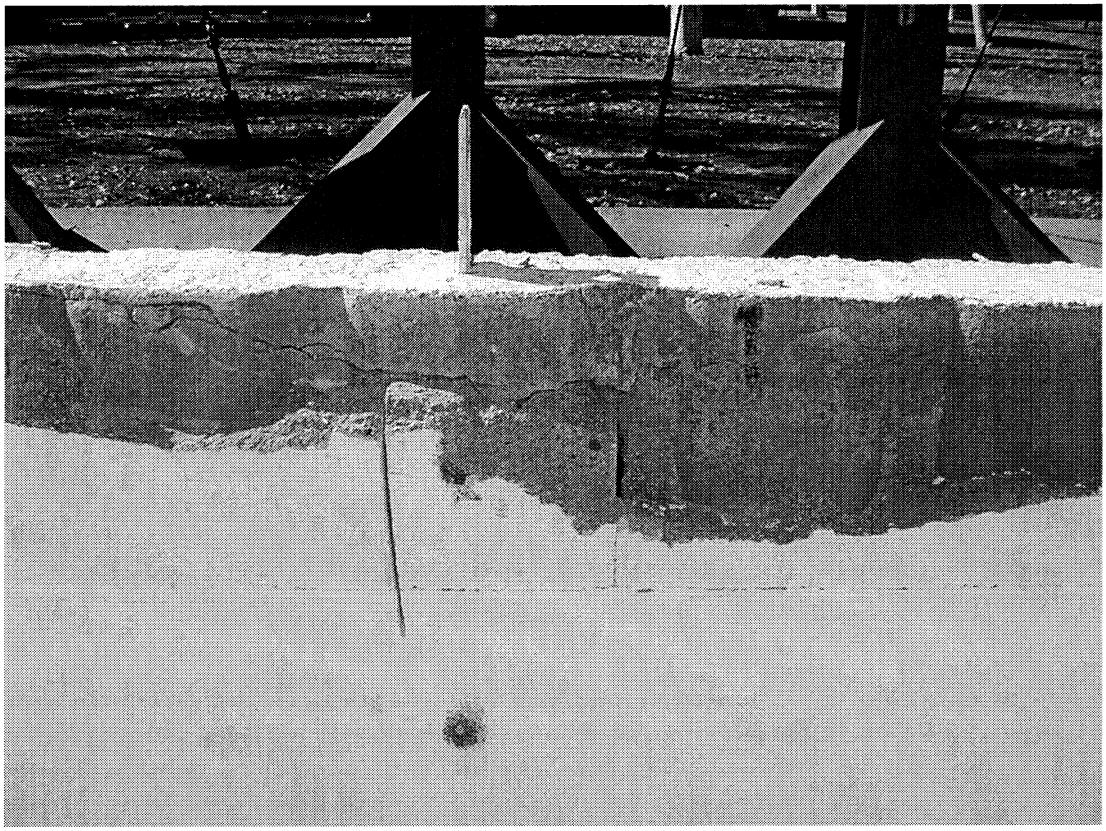


Figure 7 View of Wall 1 After Failure, with Face Shells Removed.

6.4 Walls 8 and 10

A connection failure load of 40.1 kN was measured for Wall 8 which used a two course bond beam (2N16) with a single W8 shear tie located between the two truss cleats and anchored to the lower N16 bar. This is significantly more than the failure load for the similar connection to Wall 3 where there were no shear ties, so the addition of the shear tie does increase the connection strength. The Wall 8 connection failure load is also significantly more than the W8 shear tie measured yield capacity of approximately 27 kN. The likely failure mechanism is postulated to be as follows:

> The initial uplift loading is resisted by a combination of support from the concrete plug, as described in Section 6.3 above and "hold down" by the W8 tie anchored in the lower course of the bond beam. As the uplift load is increased, the W8 bar carries more of the load until it starts to yield. As the uplift load is increased further, the W8 shear tie continues to yield and deform plastically, until the upward

movement of the cleats (and the top bond beam bar) causes plug cracking in the top course of the bond beam resulting in the failure of the connection.

Figure 8 is a photograph of Wall 8 after the failure of the connection and the typical plug cracking is clearly visible.

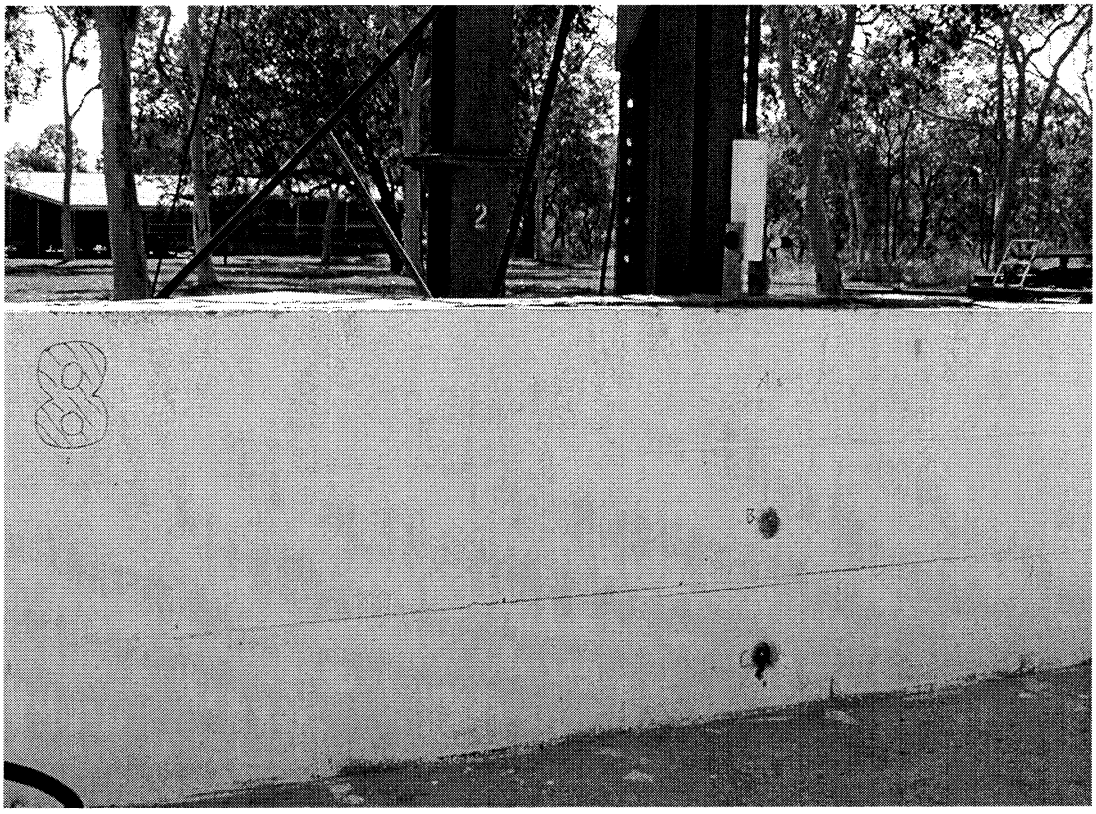


Figure 8 General View of Wall 8 After Failure.

The double cleat connection for Wall 10 failed at a load of 37.2 kN, approximately 8% less than the value for Wall 8. Wall 10 also used a double bond beam but used 2N12 bars in each course and was fitted with W8 shear ties placed on alternate N12 bars spaced at 200 mm centres along the wall. Both truss cleats were threaded onto the one N12 bar with the central shear tie placed between the cleats. However, this shear tie arrangement meant that the next shear tie located on the same top bond beam bar as the truss cleats, was approximately 400 mm away from the cleats. It is postulated that the shear ties that were 200 mm away from the truss cleats could not contribute to the connection strength because they did not directly anchor the bond beam bar. The next closest shear ties located 400 mm away and on the same N12 bar as the cleats, also did not contribute because they were more than the *effective depth* (d = 325 mm approx as defined in *AS 3700-2001*) of the two course bond beam from away the truss cleats. Consequently, for Wall 10, it is postulated that only the central shear tie between the cleats was effective. Therefore the connections for Walls 10 and 8 would be expected to have a similar failure mode, and this appeared to be the case.

When the top bond beam bars of Walls 8 and 10 were exposed there was some necking of the shear ties that were placed between the cleats. This confirms that these shear ties had undergone plastic deformation during yielding.

Figure 9 is a detailed view of Wall 10 after the masonry shells and part of the top course have been removed and shows how the shear ties were fitted on alternating N12 bond beam bars.

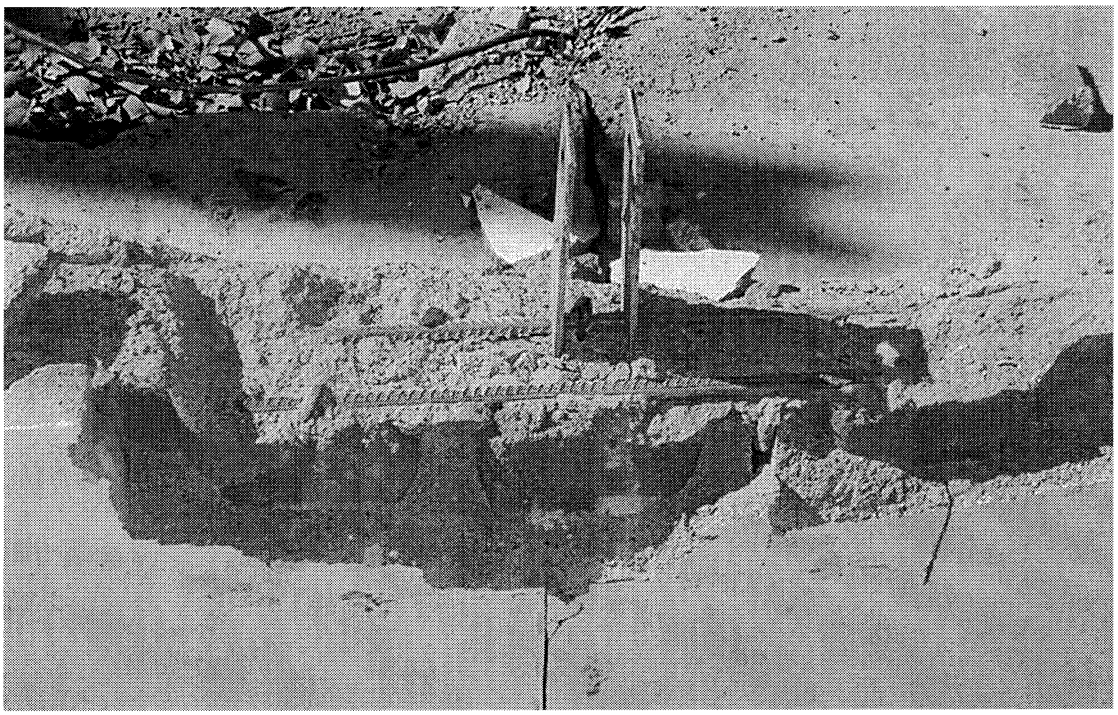


Figure 9 Detailed View of Wall 10 After Failure, with Masonry Shells Removed.

The truss cleats for Wall 10 were located asymmetrically (closer to Side B face) across the wall and were also anchored on an N12 bar rather than an N16 bar. Both of these factors would be expected to cause the connection failure load for this wall to be less than that for Wall 8. A small difference in the failure loads, following this trend was observed. However, with only one result for each wall and the relatively small difference in values, this difference may be solely within the range of experimental variability.

6.5 Walls 5 and 6

A connection failure load of 50.1 kN was measured for Wall 5 which used a two course bond beam (2N16) with W8 shear ties at 200 mm centres along the bond beam including a central shear tie located between the two truss cleats. This failure load is approximately 25% more than the value for the similar connection to Wall 8 where there was just one shear tie, so the additional shear ties do increase the connection strength. However, the Wall 5 connection failure load is still significantly less than the values necessary to utilize the full design shear and bending strengths of the bond beam, so a "local failure" at the

cleat connection is still occurring. This "local failure" was observed as a plug cracking failure mode and can be seen in Figure 10.

The likely failure mechanism for the Wall 5 connection (similar to that proposed for Wall 8) is postulated to be as follows:

> The initial uplift loading is resisted by a combination of support from the concrete "plug" and "hold down" by the central W8 tie anchored in the lower course of the bond beam, as described in Section 6.4 above. As the uplift load is increased, the central W8 bar carries more of the load until it starts to yield. The adjacent W8 ties, a nominal 200 mm either side of the truss cleats then provide some additional "hold down" to the "plug" and ensure that the connection is stronger than was the case for Wall 8. However, with increasing uplift load, the central W8 shear tie continues to yield and deform plastically until the upwards movement of the cleats (and the top bond beam bar) causes "plug cracking" in the top course of the bond beam resulting in the failure of the connection.

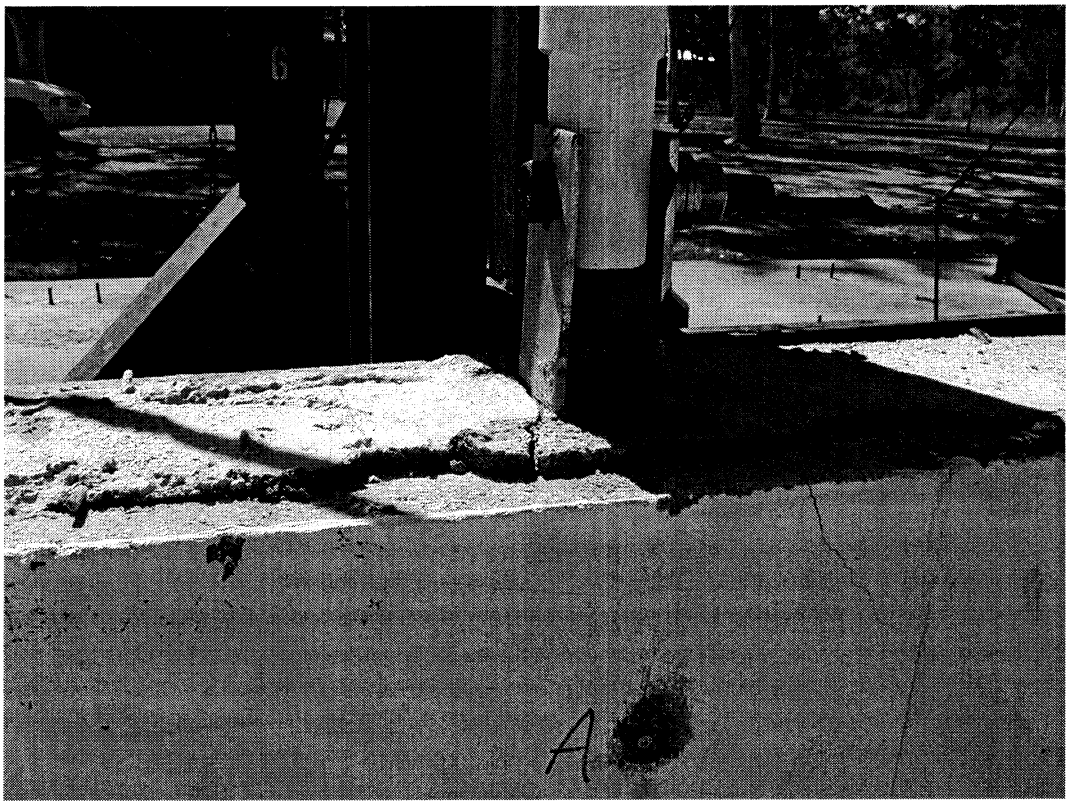


Figure 10 Detailed View of Wall 5 After Failure

Figure 11 is a detailed photograph taken after the connection failure of Wall 5 to allow an examination of the reinforcing steel. As can be seen, the hook of the central W8 tie is approximately 5 mm above the top of the horizontal beam bar, which indicates that this tie has increased in length, due to plastic deformation during yielding. This was confirmed by the necking of this shear tie that was visible when the top bond beam bar was exposed.

The double cleat connection for Wall 6 failed at a load of 47.0 kN, approximately 6% less than the value for Wall 5. Wall 6 used exactly the same reinforcing steel as Wall 5, but used a three course bond beam and so the shear ties were longer. However, the mode of failure for both walls was the same and the central shear tie between the cleats of Wall 6 also elongated plastically in a similar manner to that observed for Wall 5. Therefore it is postulated that the failure mechanism is the same for both walls. This implies that the W8 shear ties either side of the truss cleats contribute to the strength of the connection but it is the tensile strength of the central shear tie, located between the truss cleats, that has the major controlling influence on the failure load. Again there was evidence of necking in the central shear tie and this confirms that it had yielded.

Figure 11 Detailed View of Wall 5 After Failure, with Masonry Shells Removed

6.6 Walls 2 and 7

The connections for Walls 2 and 7 had similar failure loads of 38.0 kN and 35.2 kN respectively. Both used bond beams with W8 shear ties at 200 mm centres along the wall, but the double truss cleats (approximately 40 mm apart) were placed midway between two shear ties such that the each cleat plate was approximately 80 mm away from the nearest shear tie.

The failure mechanism for both connections is postulated to involve a two-stage process:

Firstly, local upward bending of the top bond beam bar spanning a nominal 200 mm between the adjacent W8 shear ties allowed the truss cleats to move upward and initiate plug failure adjacent to the truss cleats. For both walls, this "local failure"

was deemed to be the failure load for the connection. Then as the upward deflection on the cleats was increased further, a second stage to the failure mechanism was observed. This appeared to involve the transfer of uplift forces by the downward component of the tension in the top bond beam bar (now bent) to the W8 ties either side of the cleats. This transfer of load provided additional uplift resistance to the connection. The relatively severe local upward bending of the top bond beam bar, after the second stage failure, can be seen in the photograph of Wall 7 reproduced as Figure 12.

The increase in load between the first stage "local failure" of 38.0 kN and the subsequent second stage failure (41.0 kN) was relatively small for Wall 2, presumably because the cleats were anchored to a two course bond beam. However, as can be seen in Table B7, the difference between the "local failure" value of 35.2 kN and the second stage failure load of 49.2 kN is much larger for the three course bond beam used in Wall 7.

Figure 12 Detailed View of Wall 7 after Masonry Shells Removed.

It is clear that the shear tie arrangement used for Walls 2 and 7 provides less support to the top bond beam bar than when a shear tie is placed between the cleats and this is confirmed by the significantly larger failure loads observed for Walls 5 and 6 (both used a tie between the cleats).

6.7 Wall 9

Wall 9 used an N12 vertical bar located between the two truss cleats and hooked over the top bond beam bar. The double cleat connection to this wall failed at a load of 72.0 kN. Even after failure, there was very little upwards movement of the corefill concrete as can be seen in the photograph in Figure 13.

The vertical movements of the top of the wall and the cleats started to increase significantly when the applied uplift load reached 65 kN, see Table B9. As noted in Appendix C, a tensile strength test on one N12 bar showed an initial yield load of 64 kN and a maximum tensile load of 78.5 kN. Therefore the failure mechanism for this connection can be attributed to yielding of the N12 bar in tension. The N12 vertical bar started to yield at approximately 65 kN and then the uplift load resisted by the connection increased to a maximum of 72 kN as the N12 bar strain hardened during plastic deformation. There did not appear to be any failure in the horizontal bond beam bar that spanned between the vertical N12 bar and the two cleats.

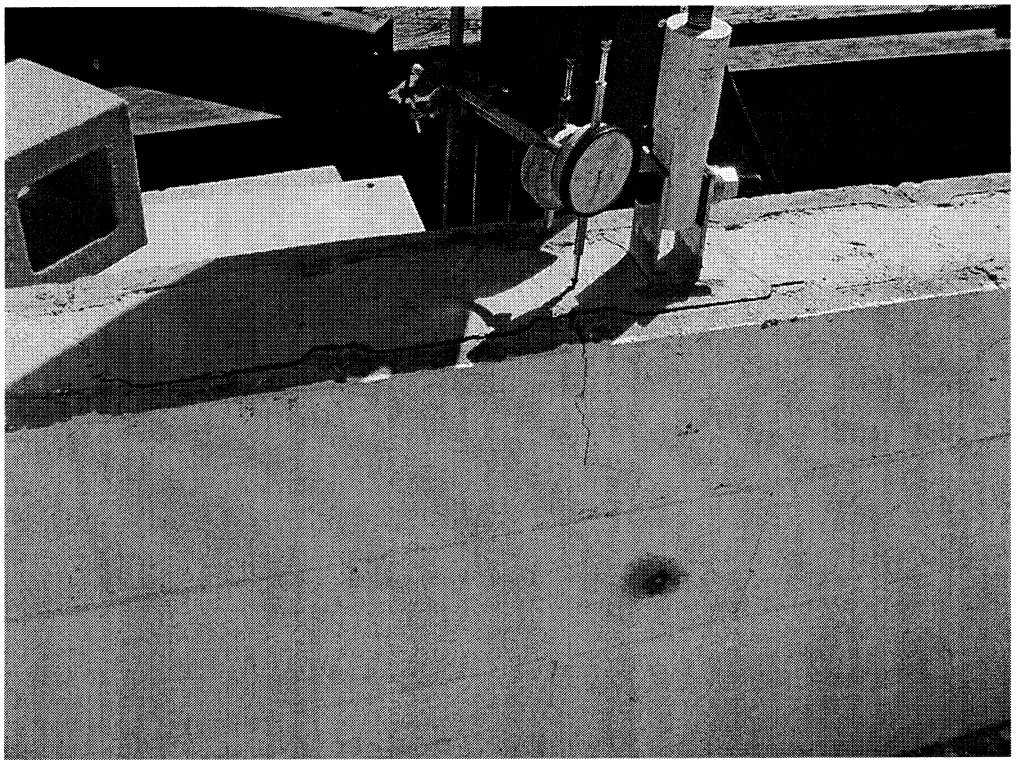


Figure 13 View of Wall 9 after Failure. (Vertical N12 bar yielded.)

In summary, the truss connection detail for Wall 9 was able to develop the full tensile capacity of the vertical N12 bar.

7. Design Capacities

The failure loads can be used to determine strength limit state design uplift capacities for the different connections tested. As there were no repeats of the tests, some design capacities will necessarily be based on one test result (eg Wall 9). Other test results from different bond beam configurations have been grouped together because they failed in a similar manner. However, all of the design capacities calculated will be subject to the limitations of the small number of test results available.

Appendix B3, "*Prototype Testing*" from *AS/NZS 1170.0:2002* has been used to determine strength limit state uplift capacities for the cleat connections. Clause B3.2 of specifies that the design capacity should be the **minimum** of the test results divided by the *Factor to Allow for Variability of Structural Units* (k_t), as given in Table B1 of this standard. The value of (k_t) reduces as the number of units tested increases and is also smaller for lower values of the *Coefficient of Variation of Structural Characteristics* (V_{sc}). Clause B3.4 of the standard notes that the value of (V_{sc}) for the parent population should be established "*..taking into account variation due to fabrication and material.*" In the absence of more accurate information, a value of 10% has been adopted for (V_{sc}).

To allow the determination of strength limit state uplift capacities, walls that had the truss connection fail in a similar manner were grouped together. The separation into these groups is the same as that adopted in Section 6 except that the single course bond beam, for Wall 1, has been considered separately. Table 3 details how the walls were grouped together and summarizes the strength limit state uplift capacity for each group of truss cleat connections.

Table 3 Failure Mode Groupings and Strength Limit State Uplift Capacities For Truss Cleat Connections

Wall No.	Uplift Failure Load (kN)	Common Failure Mode of Truss Cleat Connection	Minimum Uplift Failure Load (kN)	No. in Group	(k_t)	Strength Limit State Uplift Capacity (kN)
1	19.6	"Plug Cracking" in single course	19.6	1	1.46	13.4
3	27.8	"Plug Cracking" but second course appears to add strength	23.2	2	1.38	16.8
4	23.2					
8	40.1	Yielding of single W8 tie between cleats, then "Plug Cracking"	37.2	2	1.38	27.0
10	37.2					
5	50.1	Yielding of single W8 tie between cleats, then additional support by W8 ties at 200 centres	47.0	2	1.38	34.0
6	47.0					
2	38.0	Double cleat causes local bending of top bond beam bar between shear ties	35.2	2	1.38	25.5
7	35.2					
9	72.0	Yielding of N12 vertical bar hooked over bond beam bar between cleats	72.0	1	1.46	49.3

The strength limit state uplift capacities from Table 3 must then be multiplied by the *Capacity Reduction Factor* (ϕ) to obtain the **design** capacities. A value of $\phi = 0.75$ was adopted from Table 4.1 of *AS 3700-2001* by considering the truss cleat connections to be connectors (Item (c), (iii) in the table). Table 4 lists the strength limit state **design** uplift capacities for each group of bond beam truss cleat connections. Also included in this table is a general description of the bond beam cleat connection details and the minimum failure load used to calculate each group's design capacity.

Table 4 Recommended Strength Limit State **Design** Uplift Capacities for Bond Beam to Truss Cleat Connections

Test Wall Numbers	Bond Beam Truss Cleat Connection Details	Minimum Connection Uplift Failure Load (kN)	Recommended Strength Limit State **Design** Uplift Capacity (kN)
1	Single bond beam with single truss cleat	19.6	10
3, 4	Double or triple bond beams without shear reinforcing	23.2	13
2, 7	Double or triple bond beams with W8 shear ties at 200 and double truss cleats located centrally between the shear ties	35	19
8, 10	Double bond beams with just one W8 shear tie between double truss cleats hooked to the lower bond beam bar	37	20
5, 6	Double or triple bond beams with W8 shear ties at 200 including one W8 shear tie between double truss cleats hooked to the lower bond beam bar	47	26
9	N12 vertical bar hooked over top horizontal N16 bond beam bar and anchored to foundation	72	37

Note: The recommended strength limit state design capacities include the effect of the capacity reduction factor ($\phi = 0.75$) as specified in Table 4.1, Item (c) (iii) of AS 3700-2001.

The recommended **design** capacities listed in Table 4 are less than some of the truss cleat uplift capacities specified in Table 5.3 of the Single Leaf Masonry Design Manual, CMAA (2001). The CMAA Design Manual values become larger as the joint group strength of the timber trusses increases to reflect the increasing strength of the truss cleat to timber connection. However, the recommended design capacities from this research suggest that the strength of the connection to the bond beam will be critical in some instances. Therefore it appears that some of the CMAA Design Manual values are too large.

8. Conclusions

8.1 Summary of Findings

This research demonstrates the importance of using reinforcing steel to anchor the truss cleats down to the lower courses of blockwork. When the top bond beam bar was not restrained by shear ties, the truss cleat connections failed by "plug cracking".

As expected, the addition of a single W8 shear tie between the cleats, anchored to a lower course bond beam, increased the uplift strength of the truss cleat connection significantly. The connection strength then appeared to be controlled by the yield strength of the shear tie.

Providing additional shear ties at 200 mm centres on the same bond beam bar, either side of the single W8 shear tie between the cleats, did give some extra uplift capacity to the connection. However, the connection strength still appeared to be controlled by yielding of the central W8 shear tie.

For connections where double cleats were threaded onto a horizontal bond beam bar, both midway between shear ties (Walls 2 and 7), local upward bending of this bond beam bar, spanning between the shear ties, controlled the failure load. This indicates the importance of locating shear ties or vertical reinforcing steel reasonably close to the truss cleats, preferably between the cleats, to allow a more direct load path to transmit the uplift forces.

The double truss cleat connection that used an N12 vertical bar hooked over the top horizontal bond beam bar between the two truss cleats was able to develop the full yield strength of this bar. Note however, that the N12 bar was installed in a single length with the lower end epoxied into the "foundation slab" and so no tensile lap joints were tested.

The results of this study suggest that the uplift strength capacity of a truss cleat connection is likely to be increased by using stronger (or additional) shear ties hooked over the top horizontal bond beam bar and located between the cleats.

8.2 Comments on Design Capacities

The recommended design uplift capacities listed in Table 4 should be takes as indicative only, as the sample size for each group of truss cleat connection details was very small. Note that these capacities will only be valid for cleat connections that conform to the parameters adopted for these tests. Further investigations should be undertaken to verify these values, as noted in Section 8.3.

It should be noted that this investigation examined only direct uplift loading on the truss cleat connections. However, in practice, most truss cleats would be subjected to both uplift and lateral loads and these combined actions are likely to be more critical than just uplift loading alone. As well, all of the cleat uplift loads were applied concentrically in this study, whereas they are often loaded eccentrically in practice.

8.3 Recommendations for Further Research

As already noted, this research was planned with just a single test on a fairly wide range of bond beam connection types to determine overall failure modes and indicative uplift strength capacities. Therefore, it is recommended that the following additional research be performed:

- Perform repeat testing on identical connections so that the experimental variability can be measured. This will allow for additional confidence in the uplift strength values obtained and may also provide larger design capacities.
- Perform some tests using stronger (or additional) shear ties hooked over the top horizontal bond beam bar and located between the truss cleats.
- Consider possible effects of combined lateral and uplift loads on the truss cleat connections.

9. Acknowledgments

This research project was supported in part by the CMAA Queensland Promotions Committee. Masonry blocks were donated by Boral and Besser. The authors are very grateful for this support.

10. References

AS/NZS 1170.0:2002 *Structural Design Actions Part 0: General Principles*. Standards Australia/Standards New Zealand

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Cummins W. (2002) *Bond Beam Truss Hold Down Connections.* Thesis prepared as a partial requirement for the Bachelor of Civil Engineering, James Cook University.

TR44 (1996) *An Investigation of Truss Hold Details.* Technical Report No. 44 issued by Cyclone Structural Testing Station, James Cook University. Written by: G. Reardon and D. Henderson.

Appendix A

Test Wall Drawings

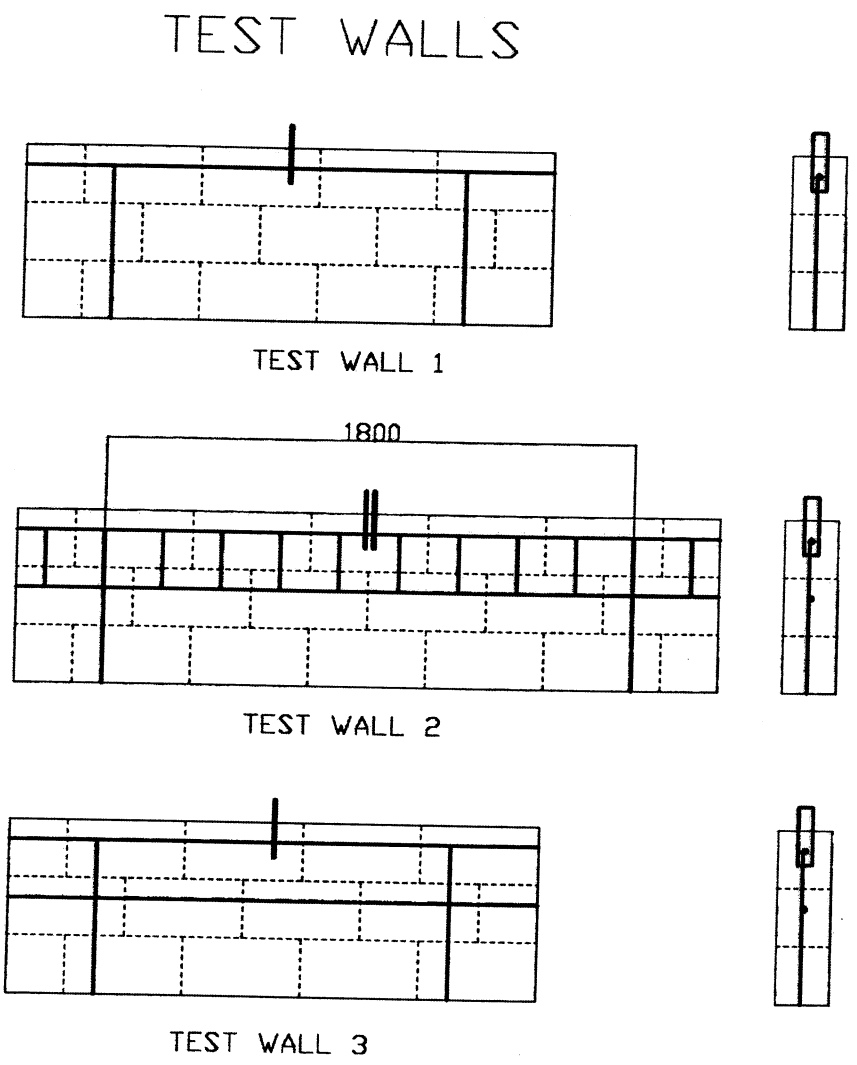


Figure A1 Elevations and Sections of Test Walls 1, 2 and 3, from Cummins (2002)

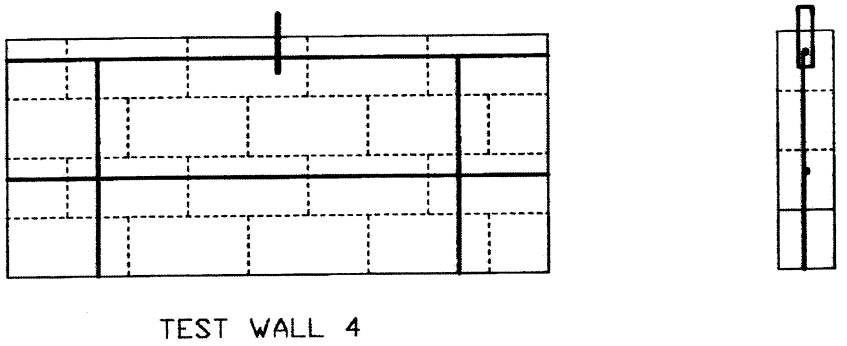
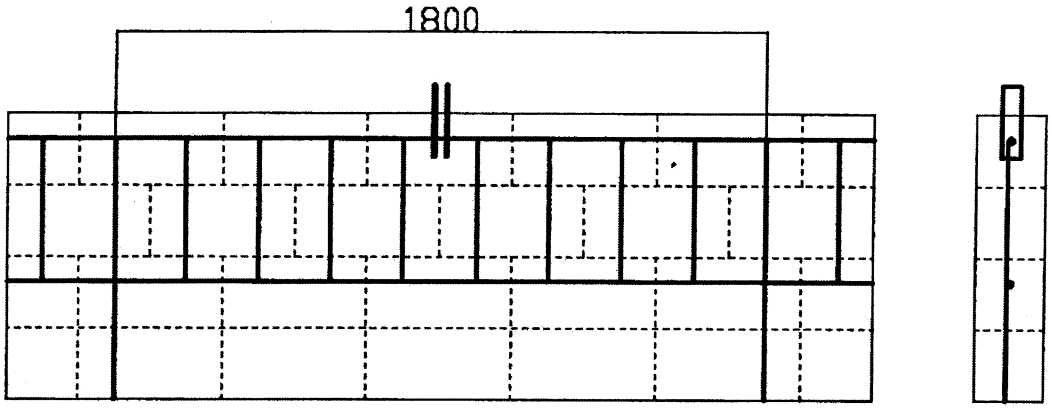
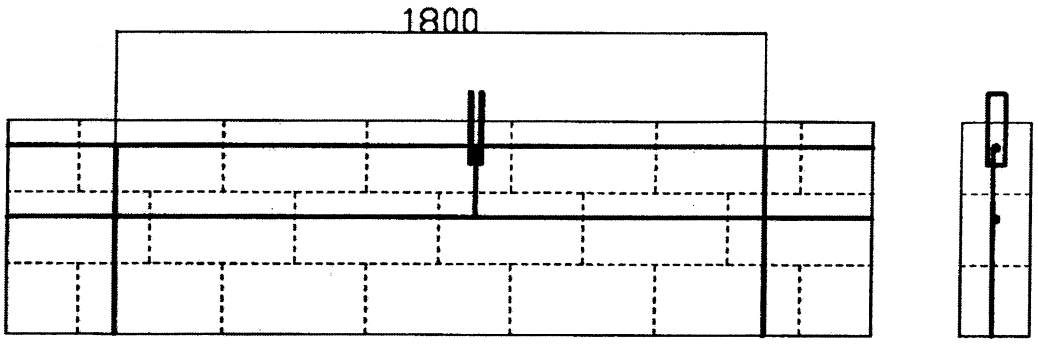


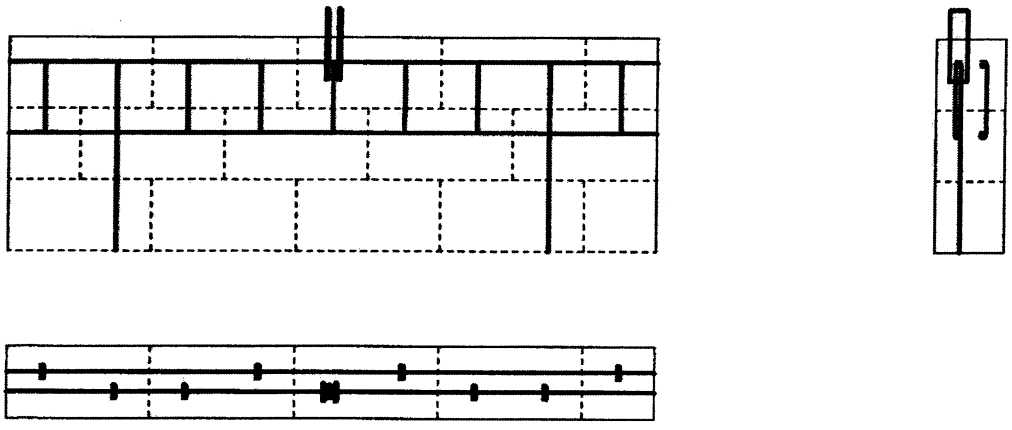
Figure A2 Elevations and Sections of Test Walls 4, 5 and 6, from Cummins (2002)



TEST WALL 7



TEST WALL 8



Top View

TEST WALL 10

Figure A3 Elevations and Sections of Test Walls 7, 8 and 10, from Cummins (2002)

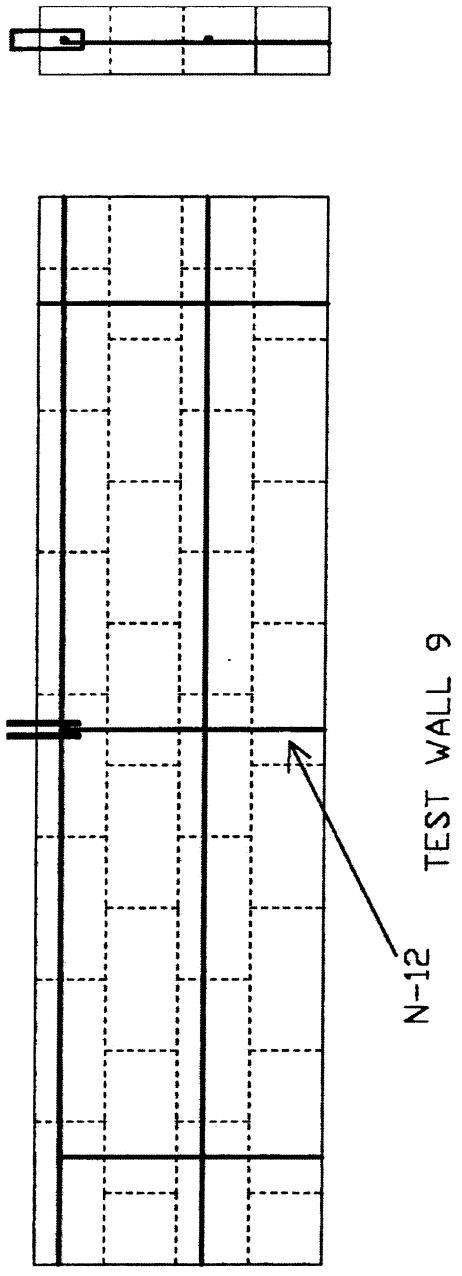


Figure A4 Elevations and Sections of Test Wall 9, from Cummins (2002)

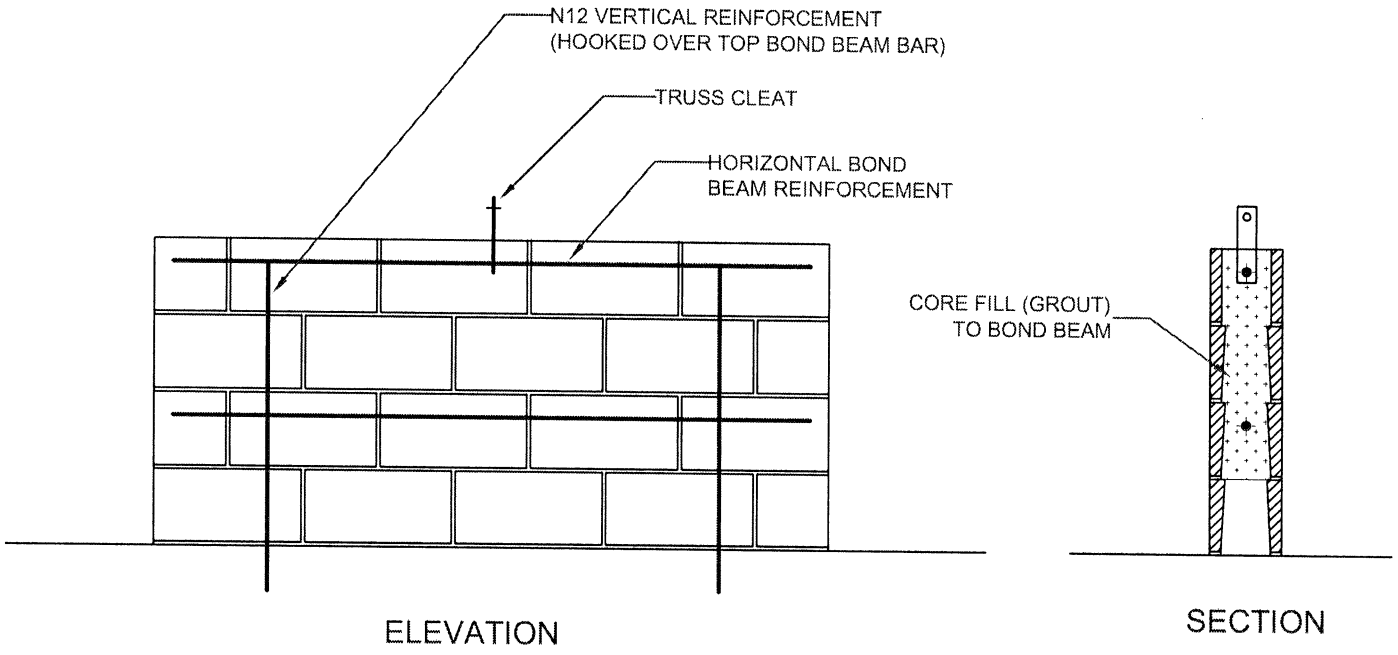


Figure A5 Elevation and Section for a Typical Test Wall (Wall 4 shown)

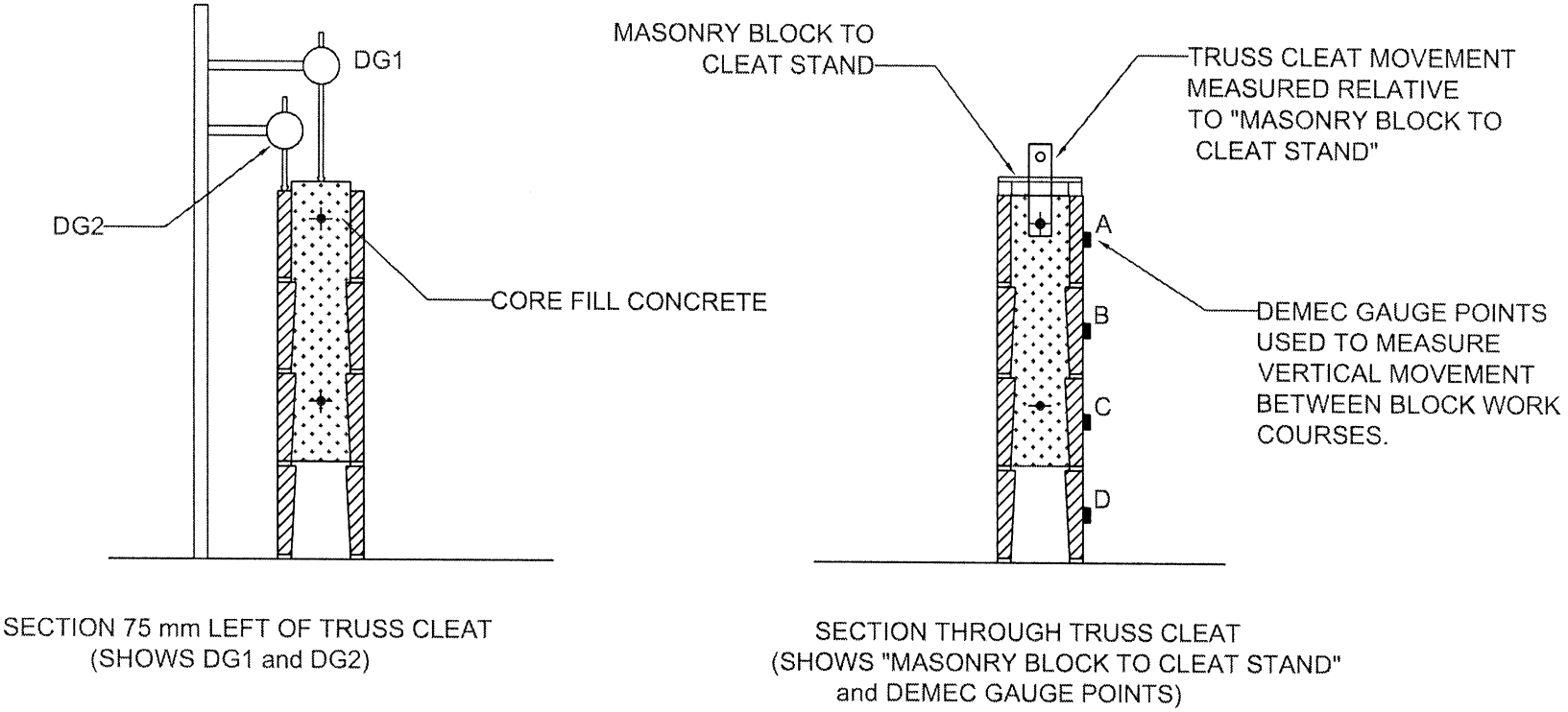


Figure A6 Schematic Sections Through Typical Test Wall
(Defines the vertical movement parameters)

Appendix B

Tables of Results and Calculated Vertical Movements

(Tables B1 to B10 inclusive)

Table B1 Measurements and Calculated Cleat Movement Test Wall 1

(Single truss cleat to single course bond beam. No shear ties)

Measured Values					Calculated Values	
Applied Uplift Loading (kN)	Vertical movement of concrete plug at Dial Gauge 1 (mm)	Vertical movement of cleat relative to face shell of block (mm)	Difference in Demec gauge points (mm)		Sum of Demec gauge point Movements (mm)	Approximate Vertical Movement of cleat relative to ground (mm) (See Note 2)
			From A-B	From B-C		
0	0	0	0	0	0.00	0.0
5	0	0	0.00	-0.01	-0.01	0.0
10.1	0	0	0.06	-0.01	0.05	0.0
15	0.13	0.5	0.12	0.00	0.12	0.5
16.5	0.37	1.5	0.48	0.01	0.48	2.0
19	1.88	3	1.16	0.13	1.29	4.5
19.6	Gauge Removed	9	1.71	0.19	1.90	11.0

Notes: 1. Positive values indicate an increase in separation between the Demec gauge points.
2. The calculated approximate vertical movement of the truss cleat relative to ground is rounded to the nearest 0.5 mm.

Top Surface Length of Plug: 675 mm
Estimated Failure (Target) Load: 15 kN
Failure Load: 19.6 kN

Table B2 Measurements and Calculated Cleat Movement Test Wall 2

(Two truss cleats to two course bond beam. W8 shear ties at 200, double cleat between shear ties.)

Measured Values							Calculated Values	
Applied Uplift Loading (kN)	Vertical movement of concrete plug at Dial Gauge 1 (mm)	Vertical Movement of masonry face at Dial Gauge 2 (mm)	Vertical movement of cleats relative to face shell of block (mm)		Movement of Demec gauge points (mm)		Sum of Demec gauge point Movements (mm)	Approximate Average Vertical Movement of cleats relative to ground (mm) (See Note 2)
			Left	Right	From A-B	From B-C		
0	0	0	0	0	0.00	0.00	0.00	0.0
10.1	0.12	0.09	0	0	0.00	0.03	0.03	0.0
20.3	0.29	0.26	1	1	-0.01	0.20	0.19	1.0
25	0.46	0.41	1	1	0.00	0.31	0.31	1.5
30	2.16	1.41	2.5	2.5	0.11	1.18	1.29	4.0
35	3.51	1.96	3	2.5	0.13	1.63	1.76	4.5
38	9.42 (See Note 3)	1.64	10	9.5	0.16	2.46	2.62	12.5
41	Gauge Removed	Gauge Removed	14	13.5	0.18	2.78	2.95	17.0

Notes: 1. Positive values indicate an increase in separation between the Demec gauge points.
2. The calculated approximate average vertical movement of the truss cleats relative to ground is rounded to the nearest 0.5 mm
3. Adopt failure load of 38 kN because the truss cleats moved vertically upwards 12 mm approx.

Top Surface Length of Plug: 820 mm
Estimated Failure (Target) Load: 35 kN
Failure Load: 38 kN

Table B3 Measurements and Calculated Cleat Movement Test Wall 3

(Single truss cleat to two course bond beam. No shear ties.)

Measured Values					Calculated Values	
Applied Uplift Loading (kN)	Vertical movement of concrete plug at Dial Gauge 1 (mm)	Vertical movement of cleat relative to face shell of block (mm)	Movement of Demec gauge points (mm)		Sum of Demec gauge point Movements (mm)	Approximate Vertical Movement of cleat relative to ground (mm) (See Note 2)
			From A-B	From B-C		
0	0	0	0.00	0.00	0.00	0.0
5	0	0	0.00	0.01	0.01	0.0
11.5	0	0	0.00	0.00	0.00	0.0
15	0.05	0	0.00	0.01	0.00	0.0
20	0	0	0.00	-0.02	-0.01	0.0
25	-0.02	1	0.00	-0.02	-0.02	1.0
27.5	-0.02	2	0.04	-0.01	0.03	2.0
27.8	14.53	14	0.78	-0.04	0.75	15.0

Notes: 1. Positive values indicate an increase in separation between the Demec gauge points.
2. The calculated approximate vertical movement of the truss cleat relative to ground is rounded to the nearest 0.5 mm

Top Surface Length of Plug: 980 mm
Estimated Failure (Target) Load: 20 kN
Failure Load: 27.8 kN

Table B4 Measurements and Calculated Cleat Movement Test Wall 4

((Single truss cleat to three course bond beam. No shear ties.)

Measured Values							Calculated Values	
Applied Uplift Loading (kN)	Vertical movement of concrete plug at Dial Gauge 1 (mm)	Vertical Movement of masonry face at Dial Gauge 2 (mm)	Vertical movement of cleat relative to face shell of block (mm)	Movement of Demec gauge points (mm)			Sum of Demec gauge point Movements (mm)	Approximate Vertical Movement of cleat relative to ground (mm) (See Note 2)
				From A-B	From B-C	From C-D		
0	0	0	0	0.00	0.00	0.00	0.00	0.0
5.1	0	0.01	0	0.00	0.00	0.00	0.00	0.0
10.1	0	0.04	0	0.00	0.00	0.01	0.01	0.0
15.3	0	0.04	0	0.02	0.00	0.01	0.03	0.0
20.2	0.04	0.07	0	0.03	0.00	0.02	0.06	0.0
23.2	14.55	0.05	13	0.02	0.00	0.23	0.24	13.0

Notes: 1. Positive values indicate an increase in separation between the Demec gauge points.
2. The calculated approximate vertical movement of the truss cleat relative to ground is rounded to the nearest 0.5 mm

Top Surface Length of Plug: 780 mm
Estimated Failure (Target) Load: 25 kN
Failure Load: 23.2 kN

Table B5 Measurements and Calculated Cleat Movement Test Wall 5

(Two truss cleats to two course bond beam. W8 shear ties at 200 with one shear tie between truss cleats.)

Measured Values					Calculated Values	
Applied Uplift Loading (kN)	Vertical movement of concrete plug at Dial Gauge 1 (mm)	Vertical movement of one cleat relative to face shell of block (mm)	Movement of Demec gauge points (mm)		Sum of Demec gauge point Movements (mm)	Approximate Vertical Movement of one cleat relative to ground (mm) (See Note 2)
			From A-B	From B-C		
0.0	0	0	0.00	0.00	0.00	0.0
6.2	0	0	-0.03	0.05	0.02	0.0
10.6	0	0	-0.12	0.07	-0.06	0.0
20.2	0.05	0	0.00	0.09	0.09	0.0
25.1	0.06	0	-0.05	0.10	0.05	0.0
30.1	0.06	0	-0.06	0.26	0.21	0.0
36.0	0.32	0	0.06	0.53	0.59	0.5
40.4	0.88	0.5	0.13	0.71	0.83	1.5
45.2	1.8	1.5	0.22	0.72	0.95	2.5
50.1	4.45	3	0.29	1.50	1.79	5.0
38.0	Gauge Removed	11	0.23	1.15	1.37	12.5

Notes: 1. Positive values indicate an increase in separation between the Demec gauge points.
2. The calculated approximate vertical movement of the truss cleat relative to ground is rounded to the nearest 0.5 mm

Top Surface Length of Plug: 715 mm
Estimated Failure (Target) Load: 55kN
Failure Load: 50.1 kN

Table B6 Measurements and Calculated Cleat Movement Test Wall 6

(Two truss cleats to three course bond beam. W8 shear ties at 200 with one shear tie between truss cleats)

Measured Values								Calculated Values	
Applied Uplift Loading (kN)	Vertical movement of concrete plug at Dial Gauge 1 (mm)	Vertical Movement of masonry face at Dial Gauge 2 (mm)	Vertical movement of cleats relative to face shell of block (mm)		Movement of Demec gauge points (mm)			Sum of Demec gauge point Movements (mm)	Approximate Average Vertical Movement of cleats relative to ground (mm) (See Note 2)
			Left	Right	From A-B	From B-C	From C-D		
0.0	0	0	0	0	0.00	0.00	0.00	0.00	0.0
10.5	0	0.04	0	0	0.00	-0.01	0.03	0.02	0.0
16.0	0	0.13	0	0	0.00	-0.01	0.04	0.04	0.0
20.9	0	0.19	0	0.5	0.02	0.00	0.04	0.06	0.5
25.4	0	0.24	0	0.5	0.02	0.00	0.06	0.08	0.5
30.2	0.02	0.28	0	0.5	0.03	0.01	0.06	0.10	0.5
35.2	0.45	0.56	0	0	0.19	0.01	0.26	0.46	0.5
40.2	1.02	0.95	1.5	0.5	0.31	0.03	0.40	0.73	1.5
45.2	4.82	2.31	5.5	4.5	0.42	0.03	0.40	0.85	6.0
47.0	Gauge Removed	Gauge Removed	16	15	0.30	0.02	0.28	0.60	16.0

Notes: 1. Positive values indicate an increase in separation between the Demec gauge points.

2. The calculated approximate average vertical movement of the truss cleats relative to ground is rounded to the nearest 0.5 mm

Top Surface Length of Plug: 825 mm
Estimated Failure (Target) Load: 55 kN
Failure Load: 47 kN

Table B7 Measurements and Calculated Cleat Movement Test Wall 7

(Two truss cleats to three course bond beam. W8 shear ties at 200, double cleat between shear ties.)

Measured Values								Calculated Values	
Applied Uplift Loading (kN)	Vertical movement of concrete plug at Dial Gauge 1 (mm)	Vertical Movement of masonry face at Dial Gauge 2 (mm)	Vertical movement of cleats relative to face shell of block (mm)		Movement of Demec gauge points (mm)			Sum of Demec gauge point Movements (mm)	Approximate Average Vertical Movement of cleats relative to ground (mm) (See Note 2)
			Left	Right	From A-B	From B-C	From C-D		
0.0	0	0	0	0	0.00	0.00	0.00	0.00	0.0
10.7	0	0.04	0	0	0.01	-0.08	0.01	-0.05	0.0
15.2	0.05	0.08	0	0	0.05	0.00	0.01	0.07	0.0
20.3	0.13	0.15	0	0.5	0.06	0.01	0.03	0.10	0.5
25.5	0.23	0.25	0	0.5	0.05	0.01	0.04	0.10	0.5
30.5	0.38	0.38	0	0.5	0.05	0.02	0.05	0.12	0.5
35.2	11.97 (See Note 3)	0.58	7.5	9	0.08	0.01	0.46	0.55	9.0
49.2	Gauge Removed	Gauge Removed			0.06	0.00	0.21	0.27	Not measured

Notes: 1. Positive values indicate an increase in separation between the Demec gauge points.
2. The calculated approximate average vertical movement of the truss cleats relative to ground is rounded to the nearest 0.5 mm
3. Adopt failure load of 35.2 kN because the truss cleats (and core fill concrete plug) moved vertically upwards 12 mm approx.

Top Surface Length of Plug: 735 mm
Estimated Failure (Target) Load: 45 kN
Failure Load: 35.2 kN

Table B8 Measurements and Calculated Cleat Movement Test Wall 8

(Two truss cleats to two course bond beam. Single W8 shear tie between cleats.)

Measured Values						Calculated Values	
Applied Uplift Loading (kN)	Vertical movement of concrete plug at Dial Gauge 1 (mm)	Vertical movement of cleats relative to face shell of block (mm)		Movement of Demec gauge points (mm)		Sum of Demec gauge point Movements (mm)	Approximate Average Vertical Movement of cleats relative to ground (mm) (See Note 2)
		Left	Right	From A-B	From B-C		
0.0	0	0	0	0.00	0.00	0.00	0.0
10.6	0.01	0	0	0.04	0.00	0.04	0.0
15.7	0.11	0	0	0.04	0.11	0.15	0.0
20.4	0.13	0	0	0.04	0.16	0.20	0.0
25.6	0.28	0	0	0.04	0.29	0.33	0.5
30.4	0.44	0	0	0.05	0.30	0.34	0.5
33.4	0.79	2	0	0.07	0.84	0.91	2.0
37.0	3.05	3.5	1	0.14	1.38	1.52	4.0
40.1	gauge removed	14.5	5.5	0.18	1.43	1.60	11.5

Notes: 1. Positive values indicate an increase in separation between the Demec gauge points.

2. The calculated approximate average vertical movement of the truss cleats relative to ground is rounded to the nearest 0.5 mm

Top Surface Length of Plug: 927 mm
Estimated Failure (Target) Load: 35 kN
Failure Load: 40.1 kN

Table B9 Measurements and Calculated Cleat Movement Test Wall 9

(Two truss cleats to three course bond beam. N12 vertical bar between truss cleats. No shear ties.)

Measured Values								Calculated Values	
Applied Uplift Loading (kN)	Vertical movement of concrete plug at Dial Gauge 1 (mm)	Vertical Movement of masonry face at Dial Gauge 2 (mm)	Vertical movement of cleats relative to face shell of block (mm)		Movement of demec gauge points (mm)			Sum of Demec gauge point Movements (mm)	Approximate Average Vertical Movement of cleats relative to ground (mm) (See Note 2)
			Left	Right	From A-B	From B-C	From C-D		
0	0	0	0	0	0.00	0.00	0.00	0.00	0.0
11	0	0	0	0	-0.04	-0.06	0.00	-0.10	0.0
20	0	0.02	0	0	0.01	-0.05	-0.04	-0.08	0.0
30	0.01	0.02	0	0	0.03	-0.01	0.01	0.03	0.0
40	0.02	0.08	0	0	0.03	-0.01	0.00	0.02	0.0
45	0.02	0.07	1	0	0.04	0.00	0.02	0.06	0.5
50	0.05	0.12	1	0	0.10	0.00	0.07	0.18	0.5
55	0.18	0.3	1	0.5	0.19	0.00	0.10	0.30	1.0
60	0.51	0.54	1.5	0	0.30	0.02	0.13	0.46	1.0
65	0.96	1.01	2	0	0.43	0.11	0.21	0.74	1.5
70	1.35	1.47	2	2	0.58	0.17	0.28	1.03	3.0
72	4.63	2.48	3	2	0.78	0.27	0.35	1.40	4.0

Notes: 1. Positive values indicate an increase in separation between the Demec gauge points.

2. The calculated approximate average vertical movement of the truss cleats relative to ground is rounded to the nearest 0.5 mm

Top Surface Length of Plug: 960 mm
Estimated Failure (Target) Load: 70 kN
Failure Load: 72 kN

Table B10 Measurements and Calculated Cleat Movement Test Wall 10

(Two truss cleats to two course bond beam 4N12 bars. W8 shear ties on alternate N12 bars with one tie between double cleat.)

Measured Values								Calculated Values	
Applied Uplift Loading (kN)	Vertical movement of concrete plug at Dial Gauge 1 (mm)	Vertical movement of cleats relative to face shell of block (mm)		Side A Movement in demec gauge points (mm)		Side B Movement in demec gauge points (mm)		Sum of Demec gauge point Movements Side B (mm)	Approximate Average Vertical Movement of cleats relative to ground (mm) (See Note 2)
		Left	Right	From A-B	From B-C	From A-B	From B-C		
0.0	0	0	0	0.00	0.00	0.00	0.00	0.00	0.0
10.3	0	0	0	-0.06	0.01	0.01	0.20	0.21	0.0
20.6	0.02	0	0	-0.05	0.02	-0.02	0.26	0.24	0.0
25.2	0.03	0	0	-0.06	0.07	0.01	0.30	0.31	0.5
30.2	0.17	0	0	-0.05	0.19	0.03	0.38	0.41	0.5
35.3	1.67	1	1	0.13	0.60	0.19	0.77	0.96	2.0
37.2	9.98	12	8	0.10	0.51	0.20	0.77	0.98	11.0

Notes: 1. Positive values indicate an increase in separation between the Demec gauge points.

2. The calculated approximate average vertical movement of the truss cleats relative to ground is rounded to the nearest 0.5 mm

Top Surface Length of plug: 885 mm
Estimated Failure (Target) Load: 40kN
Failure Load: 37.2 kN

Appendix C

Component Details

Appendix C Component Details

C1 Truss Cleats

Standard, commercially available truss cleats (Duragal, Grade 400) were used. They were nominally 200 mm long, 50 mm wide and nominally 5 mm thick. The measured thickness varied between 4.7 and 4.8 mm. Each cleat had two 20 mm holes, one used to thread the cleat onto the bond beam reinforcement and the other for connecting to the truss. The centre of each hole was located on the centreline of the cleat and a nominal 28 mm in from each end of the cleat. Therefore, the hole centres were nominally 144 mm apart. Figure C1 is a photograph showing these truss cleats.

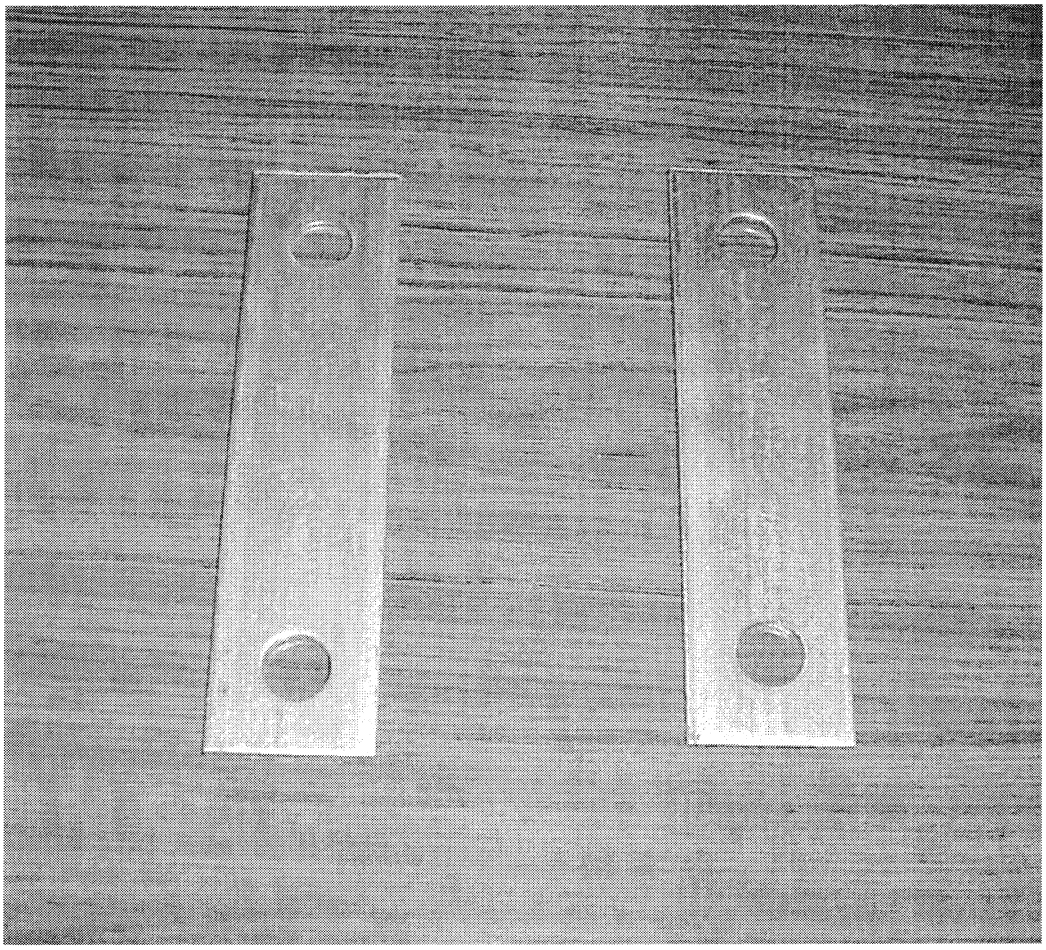


Figure C1 View of Truss Cleats

Three truss cleats were tested in tension. The average yield load was 42 kN and the minimum failure load was 50 kN.

C2 W8 Shear Ties

W8 shear ties (labelled DW 7.6 mm) were used in some of the two and three course bond beams. They were nominally 7.6 mm in diameter with a specified yield strength (f_{sy}) of 500 MPa. Each tie consisted of a single vertical stirrup with a hook to both ends so that it could be anchored over the top and bottom horizontal bond beam bars. The hooks had a 20 mm nominal clear gap between the main vertical leg and the hooked return leg. The overall length of the hook from the outside of the hook to the free end of the hook was a nominal 50 mm. The actual overall lengths of these hooks was measured on 6 ties and varied between 44 and 51 mm. The overall length of the two course ties, from outside of hook to outside of hook was measured for 3 ties and varied between 236 and 238 mm. The same dimension was measured for 3 of the three course ties and varied between 434 and 436 mm. Figure C2 is a photograph of a two course shear tie.

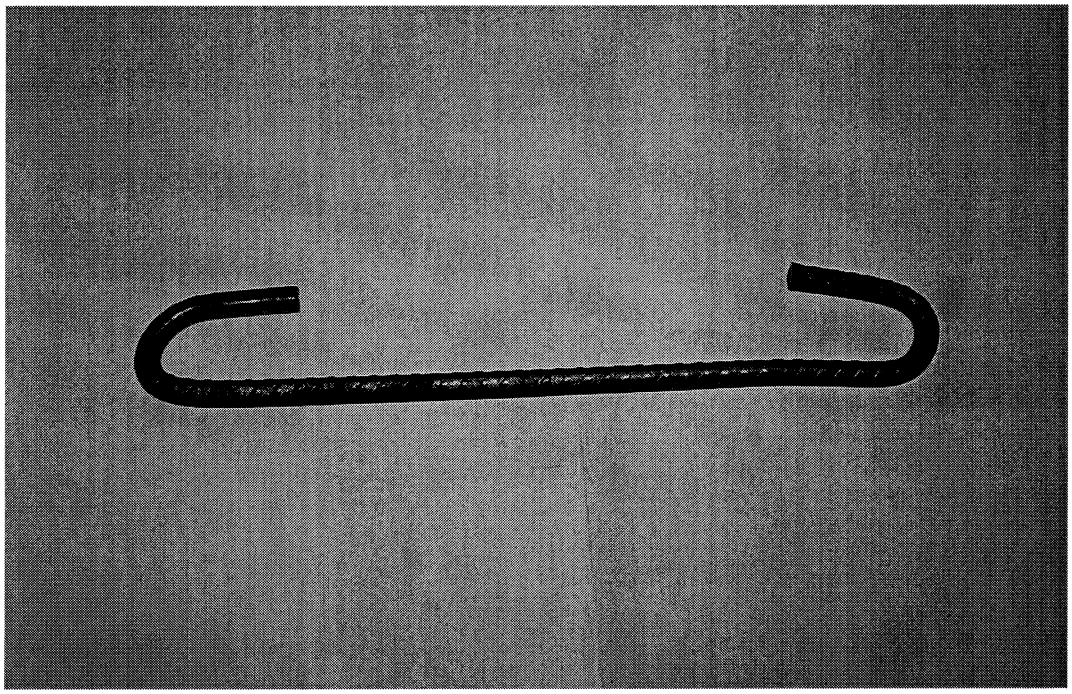


Figure C2 W8 Shear Tie Used for Two Course Bond Beams

Three W8 shear ties were tested in tension. The average yield load was 27 kN and the minimum failure load was 29 kN.

C3 N12 Vertical Bars

The vertical N12 bars were epoxied into the concrete slab supporting the test walls. The top of these bars were hooked and fitted over the top of the horizontal bond beam bars. Figure C3 is a general view of the testing site before the test walls were built and shows the hooked top end of these vertical N12 bars.

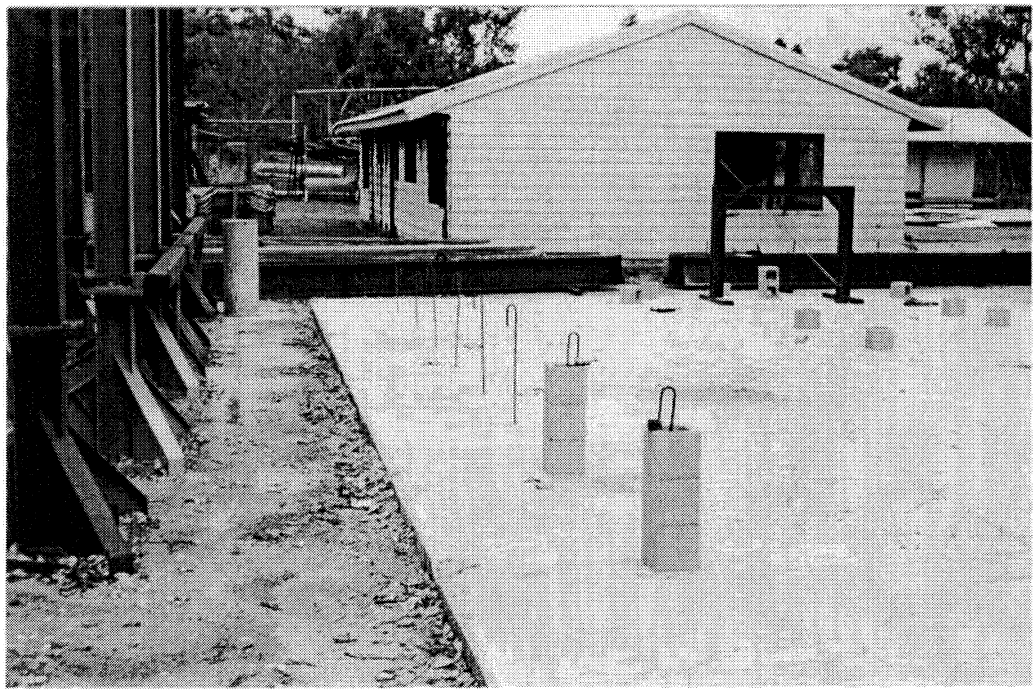


Figure C3 General View of Testing Site Showing Hooked Vertical N12 Bars

The hooks were standard hooks with a 40 mm nominal clear gap between the main vertical leg and the hooked return leg. The overall length of the hook from the outside (top) of the hook to the free end of the hook was a nominal 150 mm. The actual lengths of these hooks was measured on 4 bars and varied between 144 and 147 mm.

Tensile testing was performed on one N12 reinforcing bar to determine its yield and failure loads. The yield load was 64 kN and the failure load was 78.5 kN.

C4 "V" and "W" Bond Beam Wall Ties

"V" wall ties, nominally 4 mm in diameter, were used to locate the single horizontal reinforcing bars in the bond beams. They are designed to position the bond beam bar midway across the bond beam. The overall measured height of these wall ties was 76 mm and so they will locate the bottom of the horizontal bond beam bar a nominal 68 mm below the top surface of the block.

"W" wall ties, again nominally 4 mm in diameter, were used when two horizontal reinforcing bars were installed in the same course of the bond beams. They locate these two horizontal bars a nominal 55 mm apart and keep them positioned symmetrically about the mid-point of the 190 mm wide block width. The overall

measured height of these wall ties was 79 mm and so they will locate the bottom of the two horizontal bond beam bars a nominal 71 mm below the top surface of the block. Figure C4 shows the "V" and "W" wall ties that were used.

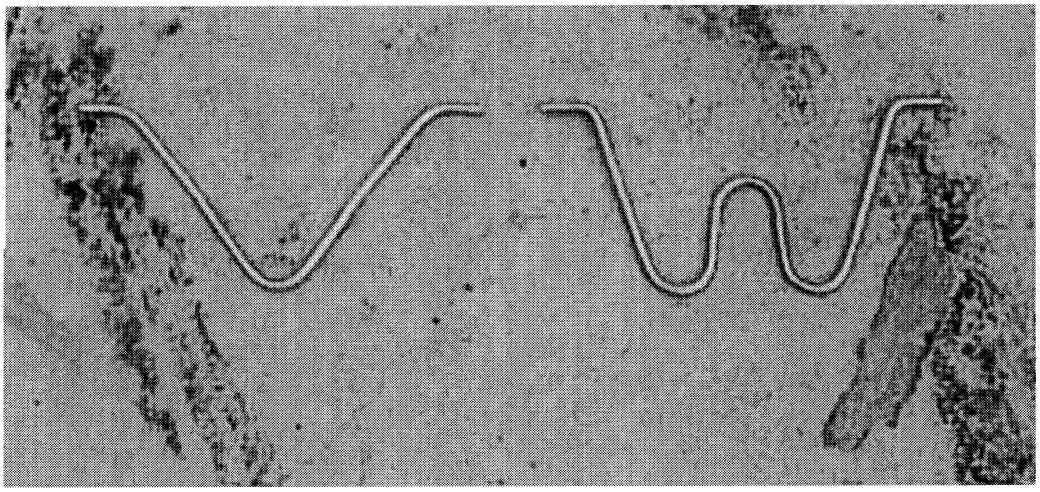


Figure C4 "V" and "W" Wall Ties Used to Support Horizontal Bond Beam Bars

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