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Maximum Wall Bracing Rating that is Compatible with NZS 3604 Construction. Phase III: Testing of Room 2

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Preface

In Phase I of this project BRANZ *Study Report 162* investigated the maximum bracing rating that a bracing wall may have while still being compatible with the minimum construction stipulated by NZS 3604 for:

(a) transferring earthquake and wind horizontal load into the wall, and
(b) transferring both horizontal and racking-induced vertical load from the wall into the foundation.

A series of isolated bracing walls was constructed on top of a large foundation. The bracing loads at which failure of the connection between floor members (such as blocking-to-joists, joists-to-bearers and bearers-to-piles, and the wall bracing load at which uplift of piles occurred) were measured. The low values so determined were of concern. This stimulated the research in Phase II (BRANZ *Study Report 163*) of this project, where a complete single storey building founded on piles was racked to enable 'systems effects' to enhance the performance – as is likely to occur in a real building. It was expected that 'systems effects' would enable higher racking loads to be applied before foundation failure occurred. This work is summarised in Phase II.

Concern was also expressed about the ability of NZS 3604 type buildings to transfer the vertical loads from wall racking between top and bottom storeys of two storey buildings. This report (Phase III of the project) presents test results performed to investigate the strength of this load path.

Acknowledgments

This work was funded by the Building Research Levy. Winstone Wallboards Ltd, Auckland, New Zealand, donated the wall linings used in the testing described herein.

Note

This report is intended for standards committees, structural engineers, architects, designers and others researching this topic.

Maximum Wall Bracing Rating that is Compatible with NZS 3604 Construction. Phase III: Testing of Room 2.

BRANZ Study Report SR 164 (2007) **S.J Thurston**

Reference

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Abstract

The New Zealand standard used for non-specific design of houses specifies that wall bracing ratings are determined using the BRANZ P21 test method. This prescribes the test procedure under which isolated walls are racked and how the results are used to evaluate the wall bracing ratings. Building designers then ensure that the sum of the earthquake or wind bracing demand, as stipulated in the standard, does not exceed the sum of the bracing resistances of house walls. To do this they use manufacturer's published data giving wall bracing strengths.

The P21 test measures the ability of a wall to transmit a racking load from a wall top plate to its bottom plate. The bracing ratings derived implicitly assume that the load can get into the top plate and subsequently be carried from the bottom plate to the ground. This report attempts to determine a suitable upper limit wall bracing strength to ensure these assumptions are met for load transfer between upper and lower floors with typical NZS 3604 New Zealand house construction.

Keywords

Timber walls, earthquake, seismic, wind, racking, experimental, houses, maximum bracing.

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1. INTRODUCTION

The New Zealand standard used for non-specific design of low rise timber framed buildings (NZS 3604, SNZ 1999) specifies that wall bracing ratings, for use with this standard, are determined using the BRANZ P21 test procedure (Cooney and Collins 1979 and also King and Lim 1991). This tested isolated walls and evaluated the corresponding wall bracing ratings. Building designers then ensure that the sum of the earthquake or wind bracing demand (as stipulated in NZS 3604) does not exceed the bracing resistance of the sum of the walls actually used in the construction. Wall bracing information for proprietary systems is published by various manufacturers.

The test walls for P21 testing are fabricated the same as used in actual construction. A horizontal load is applied to the wall top plate. The test measures the ability of the wall to transmit this load to a foundation beam. The bracing rating derived implicitly assumes that the load can get into the top plate and (having been transmitted to the bottom of the wall) get out of the test wall and into the ground. To ensure this full load-path is adequate for houses designed to NZS 3604, it has been proposed that an upper limit on bracing rating be specified. This project describes racking tests on small buildings and provides theoretical calculations which were done to determine this upper limit. It consists of three investigation phases:

Phase I

Thurston (2007(a)) presented the results of an investigation which examined:

- load transfer from the ceiling into the bracing wall
- failure of the mechanical anchors which are used in practice to connect a bracing wall to a concrete foundation
- failure of floor members, such as joists or bearers in timber floors or concrete slab in concrete floors.

The Study Report also presented the results of testing single isolated bracing walls constructed on top of a large foundation. The wall racking loads at which failure of the connection between floor members (such as blocking-to-joists, joists-to-bearers and bearers-to-piles, and the wall bracing load at which uplift of piles occurred) were given.

Phase II

The low values determined from Phase I were of concern. This stimulated the Phase II investigation where a single storey building founded on piles, with ceiling but no roof, was racked to measure load transfer from bracing walls to a typical piled timber foundation (Thurston 2007(b)). A complete structure was used so that the influence of ‘systems effects’, as occurs in real buildings, could be measured. It was thought that ‘systems effects’ would enable higher racking loads to be applied to actual buildings before foundation failure occurred.

Phase III

Concern was expressed about the ability of NZS 3604 type buildings to resist the vertical loads between top and bottom storeys of two storey buildings arising from wall racking. This report presents testing results performed on a two storey room (called Room 2) to investigate this load path.

2. ROOM 2: CONSTRUCTION DETAILS

2.1 General description

The room tested was a two storey rectangular building. Two of the exterior walls were lined on the inside with plasterboard but the outside was not clad. The other two exterior walls and the two interior walls were lined with plasterboard on both sides to simulate typical interior walls.

The room had a plan size of 4.87 m x 3.49 m. The lower storey was only 1.2 m high as the interest was in load transfer <u>into</u> not <u>through</u> this storey and thus actual storey height was not relevant.

The two long sides of the room are referred to as Side 1 and Side 2, as shown in Figure 1. The two short sides are referred to as End 1 and End 2. A plan of the ceiling framing is shown in Figure 2. Cross-sectional views through the building are shown in Figure 3 and Figure 4 for the two major axis directions. It can be seen that Sides 1 and 2 of the top storey overhang the walls below but the upper two end walls are directly over the walls below. Figure 5 to Figure 8 show the framing in elevation for each side. The above drawings include the nomenclature used for discussions when specific locations on the building need to be referenced.

Construction was conventional and complied with NZS 3604. Blocking between floor members (see Figure 1) was used as required by NZS 3604.

Two 1.467 m long internal walls were added during the test series (as discussed later). The locations are shown in Figure 1. Note that the wall for Stage II testing was constructed over blocking spanning between floor joists while the wall for Stage III testing was constructed directly over a floor joist.

The ends of all bracing elements were fixed to the floor as per the Winstone Wallboards (2006) bracing wall hold-down system as illustrated in Figure 5 to Figure 8 and also Figure 13. This consisted of an end strap fixing the studs to the bottom plates and coach screws fixing the bottom plates to the floor framing. At locations where the writer wished to measure anchor forces, the coach screws were replaced with through-bolts and load cells were used, as shown in Figure 13.

2.2 Member sizes and connections

All timber was grade MSG 8.

- Floor and ceiling joists: the joists had a 140 x 45 mm cross-section and were fixed at the ends to the wall top plates with three skewed power-driven 90 x 3.15 mm Paslode JD nails as per Table 7.5 of NZS 3604 (two one side and one the other). The exterior joists are referred to as boundary joists (see Figure 3). NZS 3604 is silent on the fixing required along the length of boundary joists. The writer had discussions with several builders and examined what was used in six new Kapiti Coast houses and found the fixing used was extremely variable. The recommendations of a large building company were adopted, being a single power-driven 90 x 3.15 mm skew nail at 600 mm centres from joist-to-wall top plate along the length of the boundary joist, being placed from the outside. In addition, there were also three skew nails used at each end.. Thus, the performance of this joint under both shear and uplift load was of considerable interest.

- Fixing between internal walls and the ceiling framing above: the top plate of the internal wall for Stage II testing was nailed to each ceiling nog it crossed with a single 90 x 3.15 mm power-driven nail. However, two of these nails per nog were used for the Stage III internal wall.

- Blocking between joists: the blocking members had a 140 x 45 mm cross-section. These were fixed to each joist using two end-nailed power-driven 90 x 3.15 mm Paslode JD nails at the ends, except where blocking continuity precluded this when four power-driven 90 x 3.15 mm Paslode JD nails were driven at a skew angle.

- Flooring: the 20 mm particleboard flooring was fixed to the sub-framing with 60 x 2.8 mm ring shank galvanised nails at 150 mm centres around the sheet edges and 300 mm centres to intermediate supports as per Table 7.5 of NZS 3604. The sheets were installed in a running bond pattern with their long edges at right angles to the joists.

- Wall framing: the framing had a 90 x 45 mm cross-section. Details at windows, doors and lintels and general wall construction complied with NZS 3604:1999 for lightweight top storey construction. No wall nogs were used. A double top plate was used with the upper member being 140 x 45 mm and the lower member 90 x 45 mm.

- Ceiling: ceiling joists were 140 x 45 at 800 mm centres. Ceiling battens were 70 x 45 mm at 600 mm centres.

Wall sheathing details are identified in Figure 3 and Figure 4. The walls of Side 1 and End 1 were lined on the inside with 10 mm GIB Braceline®, fixed using the Winstone Wallboards Ltd (2006) BL1 screwed vertical sheet option. These walls were not clad on the exterior and thus simulated weatherboard cladding with no bracing capability. The walls of Side 2, End 2 and the two internal walls were lined on one side with 10 mm GIB Braceline® and on the other side with 10 mm Standard GIB®. These linings were fixed using the Winstone Wallboards Ltd (2006) BLG screwed vertical sheet option. This simulated typical interior wall construction. 13 mm GIB® standard board was used on the ceilings. All sheet joints were reinforced with paper tape and the joints plastered.

3. TEST DESCRIPTION

3.1 Overview of test programme (the three test stages)

The three test stages are referred to as Stage I to Stage III. The load application points for these are illustrated in Figure 2.

In Stage I testing, separate actuators were used to deflect the top of each of the walls on Side 1 and Side 2 to the same deflection regime. An interior wall was then added, as shown in Figure 1. Damaged areas from Stage I testing were repaired except where discussed later. The room was then picked up by crane and rotated so that End 1 and the interior wall could then be racked in Stage II testing. A cut through the entire room down to floor level was made at the room mid-length prior to Stage II testing, as shown in Figure 2, to avoid loading End 2.

After Stage II testing, a second interior wall was added, as shown in Figure 2. The room was moved again to enable End 2 and the second interior wall to be racked in Stage III testing.

3.2 Reaction frames to remove horizontal load

Unlike Phase I testing (Thurston 2007(a)) no reactions frames were required to remove horizontal load, as the lower storey walls were designed to resist these forces.

3.3 Instrumentation

The following measurements were made:

(1) Applied horizontal load (by using two load cells in series on each of the actuators). The second load cell was used as a check on the first.

(2) Horizontal deflection of each loaded wall top plate (the LVDTs on the actuators provided an approximate check on this).

(3) Horizontal deflection of the floor of the upper storey and top plate of the lower storey for each loaded wall, as shown in Figure 16. This enabled the upper storey inter-storey deflections and also the joist roll-over deflections to be determined.

(4) Vertical separation of the bottom of the stud in the top storey from the studs in the bottom storey at the boundaries of each wall opening. An example at door openings is shown in Figure 13. This was accomplished by fixing rods to the studs in the lower storey. The rods passed through slots in the flooring. A potentiometer was fixed to the top of the rods with the target attached to the upper studs. Slightly different arrangements were required at window openings (as shown in Figure 14) and wall corners (as shown in Figure 16).

(5) Vertical movement of joists from wall top plate below (Figure 15).

(6) Vertical forces in bracing wall hold-down through-bolt anchors using load cells (see Figure 13 and Figure 15).

3.4 Loading system

The actuator loads were transferred to either ceiling joists or blocking between ceiling joists using articulated steel load beams shown in Figure 10. These had pin joints either side of wall openings to allow the wall to move vertically at these locations without any restraint being imposed by the load beams. Apart from the portion of the load beam directly above wall openings, the load beam was fixed to the framing below using Tek screws to transfer the horizontal forces.

3.5 Measured end restraint

When a house wall is racked, part of the top horizontal deflection can be attributed to 'rocking'. This is due to uplift of the wall at one end and sinking at the other. This end movement, whether upward or downward, is due to the vertical wall end forces associated with racking. An equation developed for calculating wall vertical end forces due to applied racking load is given in Section 3.5 of Thurston (2007(b)) as a function of assumed end restraint and will not be repeated here. The assumed end restraint is largely due to the presence of adjacent construction. This imposes forces denoted as ER at the bracing wall uplift end and ER' at the wall compression end to resist the upward and downward movement respectively. The measured wall vertical end uplift forces were inserted into this equation to estimate the magnitude of wall continuity uplift end restraint ER.

3.6 Nomenclature

The text of this report uses a coded system to refer to specific joists and locations on walls, as shown in Figure 1 and Figure 5 to Figure 8.

4. STAGE I TESTING: RACKING TESTS ON THE TWO SIDE WALLS

4.1 General description

Roof dead load and a floor live load of 0.2 kPa were simulated by weights, as shown in Figure 10 and Figure 12.

The same deflection regime was simultaneously imposed onto the tops of both exterior long walls (i.e. Sides 1 and 2).

The loading in Stage I was imposed in three separate tests, with modifications to the construction being made after Test 2 as noted below:

(a) Test 1 consisted of three cycles to ±4 mm and one cycle to ±10 mm inter-storey movement of both Side 1 and Side 2. The upper storey plasterboard cracked at the corners of wall openings during this last cycle. As the boundary joist along End 1 lifted from the wall top plate by 9 mm during the last cycle of Test 1, the test was temporarily halted to see whether strengthening was needed. It was decided to proceed without any strengthening being done.

(b) Due to actuator malfunction, Actuator 2 moved by +20 mm inter-storey movement before commencement of Test 2. This appeared to just further lift the boundary joist along End 1 from the wall plate below and did not damage the walls. This uplift movement is shown in Figure 17. The ceiling withstood the 20 mm differential deflection between side walls without any observable damage. One cycle to ±10 mm and two cycles to ±16 mm inter-storey movement to both Side 1 and Side 2 were then imposed. This completed Test 2.

(c) The nails in the End 1 and End 2 boundary joists at both the ends and along the length were extracted and new nails were placed nearby, except that the spacing along the length was now reduced to 150 mm centres. The other joists had generally lifted less than 3 mm from their supports (see Figure 25) and thus the associated nail fixings were not replaced.

    Blocking between joists was not continuous (as shown in Figure 7, Figure 8 and Figure 9). Significant distortion was observed between blocking and joists in Test 2, at locations where there was a gap in the blocking, as shown in Figure 18. To try to avoid this distortion, new blocking was placed where there had previously been gaps.

    Test 3 consisted of one cycle to ±15 mm and two cycles to both ±25 and ±40 mm inter-storey movement. At this stage the floor joist uplifts described below precluded any increase in resisted load with greater imposed deflection. Although sheet damage at screw heads indicated the wall linings were still well short of being racked to their maximum shear strength, the test was stopped to allow Stage II and III testing to be performed on largely undamaged construction.

    At 25 mm inter-storey movement, significant distortion still occurred at the blocking, as shown in Figure 19. Figure 20 and Figure 21 show this distortion at 40 mm imposed wall deflection. The floor joists lifted off their supports, as shown in Figure 22. Floor vertical movement is illustrated in Figure 20 and Figure 21. Floor joist and blocking uplift at the End 1 corner are shown in Figure 23.

4.2 Test hysteresis loops

Figure 24 shows the generated total building load-deflection (hysteresis) loops. It can be seen that the resisted load dropped off in Test 2 due to the inadvertent prior loading of Side 1. Test 3 showed a gain in strength due to the strengthening described in Section 4.1(c).

The deflections plotted are the differential movements between wall top plate and floor, averaged between the two side walls. The load plotted is the sum of the horizontal forces measured at the two actuators. As the deflection regimes for each actuator were effectively the same, it is expected that the ceiling transferred little load.

4.3 Floor joist uplift

Floor joist uplift from the top plate of the wall below versus racking load on the corresponding top storey wall is plotted in Figure 25 for the peaks of each displacement cycle. This movement is plotted with symbols that are shaded for testing before the walls were strengthened. The same symbols are used at corresponding locations after building strengthening. From the chart it can be seen that at corresponding racking loads only a small reduction of uplift occurs for the post-strengthened state. Note: only some joists were instrumented and this did not include the boundary joists along the ends of the test room.

As the floor joist nail fixing into the lower storey top plate only penetrated approximately 40 mm, uplifts greater than 40 mm will disengage the joist hold-down. Further, it can be expected that the resistance to uplift reduces with greater joist uplift. Hence, the ability for the joists to transmit horizontal load will also decrease. Thus, failure is interpreted as being a gap between joist and top plate exceeding 20 mm.

Locations C and E show the greatest uplift. Values greater than 20 mm occurred after approximately 95 BUs/m.

The boundary joists lifted from the wall top plates at both ends of the room, but particularly at End 1. Before room strengthening the maximum gap (measured by ruler) of the End 1 boundary joist was 12 mm near both the Side 1 and Side 2 corners. This reduced to 6 mm at joist mid-length. After room strengthening, the maximum gap was 22 mm near Side 1 corner, 15 mm near Side 2 corner and 12 mm at mid-length. The maximum bracing rating along the sides was 70 BUs/m before strengthening and 113 BUs/m for the strengthened wall.

4.4 Floor joist roll-over

When horizontal load is transmitted perpendicular to a floor joist span, from the top of a joist to the bottom, the joist tends to rotate across about the base – thereby moving the top of the joist horizontally. The horizontal movement is defined as joist-rollover deflection. Measured roll-over deflections are plotted in Figure 26 versus wall racking load. The same symbol system as in Section 4.3 is used. At corresponding racking loads only a small reduction of roll-over deflection occurs for the post-strengthened state.

The roll-over deflections plotted are too small to be considered as being a failure which is assumed to be a deflection equal to the joist width. Joist roll-over is thus considered to be less critical than joist uplift. However, the movement does add to apparent wall flexibility. Extrapolation of the test data indicated that joist-rollover deflections greater than 45 mm (hence possibly leading to instability) will occur at wall bracing ratings of 120 BUs/m. This value may be lower if the bracing walls above have a lower percentage of openings as the average roll-over force per unit length would then be greater. The critical bracing values discussed above are expected to be lower if blocking is discontinuous, as currently allowed by NZS 3604.

4.5 Inter-storey uplift movement

When an upper storey bracing wall is racked, the vertical uplift force at one end of the wall must be transmitted from the upper wall to the lower wall. The vertical movement between the studs in the two walls is defined in this report as inter-storey uplift movement. To a lesser extent, vertical compression differential movement will occur at the other end of the bracing wall. Measured inter-storey uplift movements are plotted in Figure 27 and Figure 28 versus wall racking load. Only uplift movements are plotted – not compressive movement (i.e. closing). At Locations B, D, F G, I and K uplift occurred under positive racking load and at Locations A, C, E, H, J and L it occurred under negative racking loads (Figure 7 and Figure 8 show these locations). Thus,

Figure 27 and Figure 28 can use the same symbol for A and B etc, as B only plots at positive racking loads and A only plots for negative racking loads.

The same symbol system as in Section 4.3 is used to show which data corresponds to the “before and after strengthening” (i.e. the symbols for the before strengthening are shaded). At corresponding racking loads only a small reduction of inter-storey uplift deflection occurs for the post-strengthened state.

The largest movements were at F and G which (as Section 4.1 indicated) was due to uplift of End 1 due to the boundary joist lifting off the top plate. However, the inter-storey uplift movements in themselves do not constitute a failure. The movement shown is mainly due to uplift of floor joists at their support. Two other components are:

- uplift of wall studs relative to the floor (the steel straps and bolt hold-down anchorage shown in Figure 13 usually make this movement small)
- minor material stretching.

The vertical movements add significantly to apparent upper storey flexibility. If a wall of height, H, sinks by Δ_1 at one end and lifts by Δ_2 at the other end, and the distance between the two ends is L, then the contribution to horizontal deflection due to this ‘rocking’ = $(\Delta_1 + \Delta_2)H/L$. This formula has been used to derive data for Figure 29 and Figure 30 which plots the percentage of imposed inter-storey deflection which can be attributed to ‘rocking’. There is a scatter of data, some being over 100% (which indicates that the wall is not deforming as a rigid body). The overall average is 75% (i.e. only 25% of the total wall movement is due to wall shear deformation). Hence, of the 40 mm movement measured at the top of the walls, only approximately 25% x 40 = 10 mm was due to shear deformation of the wall itself.

4.6 ‘Systems effect factor’ for walls

The maximum total applied load was 35.7 kN.

Winstone Wallboards Ltd (2006) gives the wind bracing resistance of the BL1 walls as 125 BUs/m. These walls totalled 3.01 m in length. It also gives the wind bracing resistance of the BLG walls as 145 BUs/m for a 0.6 m long wall (only one such length used in the test set-up) and 150 BUs/m for a 1.2 m long wall (total length = 2.71 m). This provides a design bracing resistance of:

(125 x 3.01 + 145 x 0.6 + 2.71 x 150)/20 = 43.5 kN

‘Systems effect factor’ is defined at maximum bracing load divided by the sum of the P21 strengths of the component walls. Thus, the ‘systems effect factor’ for Stage I = 35.7/43.5 = 0.82. However, as noted in Section 4.1, the side walls were well short of their ultimate strength when testing terminated and thus this result is meaningless.

4.7 Measured end restraint

Section 3.5 discusses the method of using the measured load cell wall uplift forces to estimate the end continuity uplift restraint ER.

Figure 31 plots the theoretical relationship between wall anchor uplift force (kN) and positive wall racking loads (kN/m).

The uplift force was measured by load cells at Locations C, D, E, I, J and H (refer to Figure 7 and Figure 8 for location of these points). This data is plotted in Figure 31 at each cycle first peak upwards load.

At racking loads of magnitude less than 3.7 kN/m, the theory for ER = 9 kN predicts that there should be zero wall reaction. However, the test data in Figure 31 shows that significant wall end reactions did occur. This is attributed to the end restraint only becoming mobilised after significant wall end vertical movement has occurred.

However, at racking loads of magnitude greater than approximately 4.5 kN/m, the theory for ER = 9 kN is in reasonable agreement with the measured anchor uplift forces, although the walls have not been racked sufficiently to fully establish this.

4.8 Conclusions from Stage I testing

(1) In calculations to measure the bracing rating at which joist and bearer failure will occur, it is proposed that the vertical forces on joists and bearers be calculated using ER = 9 kN.

(2) A wall bracing rating of approximately 95 BUs/m was sufficient to separate some floor joists from their supports (i.e. to the wall top plate).

(3) Inter-storey vertical movement between upper and lower wall studs causes the walls to ‘rock’ and contributes approximately 75% of the total upper wall inter-storey movement. This is partially due to separation of the floor joists from their supports.

(4) Although the applied bracing loads were perpendicular to the room end boundary floor joists these joists still uplifted excessively. NZS 3604 is silent on the fixing required between a boundary joist and the wall below along the length of boundary joists. The first construction tested used a single power-driven 90 x 3.15 mm skew nail at 600 mm centres along the length of the boundary joist, being placed from the outside. In addition the three skew nails at joist ends, as currently stipulated were used. Failure occurred at a bracing load of approximately 70 BUs/m. The nails in the End 1 and End 2 boundary joists at both the ends and along the length were then extracted and new nails placed nearby, except that the spacing along the length was now reduced to 150 mm centres. Failure then occurred at a bracing load of approximately 110 BUs/m. It is recommended that 100 or 150 mm spacing be adopted by NZS 3604.

(5) With blocking used between every pair of floor joists along the wall support below, joist roll-over deflection was still a significant proportion of the horizontal deflection of bracing walls in the storey above. Deflections greater than 45 mm (hence possibly leading to instability) will occur at bracing ratings of 120 BUs/m and perhaps at even lower values. This critical bracing value is expected to be lower if blocking is intermittent, as currently allowed by NZS 3604. If each blocking piece is fixed to the plate below with six skewed nails, as specified in NZS 3604:1999 Table 7.6, then blocking between every pair of joists may not be necessary.

5. STAGE II: RACKING TESTS ON END 1 AND THE FIRST INTERNAL WALL

5.1 Repair after Stage 1 testing

All nails in the End 1 boundary joist were removed and the usual three end nails plus single nails at 600 mm centres placed nearby. This construction is discussed in Section 2.2. However, it was difficult to remove the nails attaching the internal floor joists to the top plates along the side walls. Even though they had been loosened by the Stage 1 testing they were not replaced.

5.2 General description of test

A plan view of the test arrangement is given in Figure 1. Note: a complete cut through all construction down to the floor level had been made at side wall mid-length, as shown in Figure 2. This structurally separated the two halves of the room to preclude upper storey lateral wall load from being transmitted from one half of the room to the other.

A general view of Stage II testing is shown in Figure 32. The same deflection regime was simultaneously imposed onto the top of both End 1 and the interior wall shown in Figure 33. A view under the floor is given in Figure 34 which shows the floor joists, blocking and instrumentation.

The internal wall for Stage II testing was constructed over blocking spanning between floor joists. This was later found to be critical as vertical movement of the blocking added significantly to wall flexibility.

Roof dead load and a floor live load of 0.2 kPa were simulated by weights, as shown in Figure 10 and Figure 12.

The loading in Stage II testing was imposed in three separate tests, with modifications to the construction being made after Test 2 as noted below:

(a) Test 1 consisted of three cycles to ±7 mm and one cycle to ±14 mm inter-storey movement. Diagonal cracks formed in the plasterboard at the End 1 door top corners during this last cycle. It was noted that very little load was being carried by the internal wall and thus the test was halted to investigate this. It was concluded that the low load resistance was due to:

   (1) the flexibility of the internal wall

   (2) ‘rocking’ motion from blocking vertical movement, and

   (3) uplift of the wall itself from the floor.

   Gauges were added to measure the vertical uplift movement of the wall from the floor at each end of the internal wall and the test was resumed. Floor joist uplift movements from the wall below at V, W, X and Y (see Figure 1) during this cycling were less than 1 mm.

(b) Test 2 consisted of one cycle to ±14 mm and one-and-a-half cycles to ±25 mm inter-storey movement. The boundary joist along End 1 lifted 16 mm at one end and 21 mm at the other during these cycles (Figure 35). The internal floor joists were now lifting excessively from their supports as shown in Figure 36 (13 mm at W, 10 mm at Y and 6 mm at V and X). Refer to Figure 1 for nomenclature. The joist beneath the End 1 wall was also lifting excessively, as shown in Figure 37. The horizontal slip between the boundary joist along End 1 and the wall below reached 9 mm. The hysteresis loops of load versus deflection had flattened or were sloping downwards with increased deflection. It was decided that the joist connections had exceeded their maximum strength and the only purpose in continuing testing was to measure the strength of the upper storey walls. Hence, the connections between the joists and the wall below were strongly strapped together and the test continued to examine the maximum racking strength the walls themselves could achieve.

(c) Test 3 consisted of two cycles to ±27 mm, one to ±40 mm and two cycles to ±62 mm inter-storey wall deflection. At this stage the hysteresis loops had flattened, as shown in Figure 41. The internal wall had ‘rocked’ considerably, sinking into the floor at one end (Figure 39) and lifting at the other, thereby creating a gap between the ceiling and wall top at the compression end (Figure 40) and tearing off the plastered joint at this location. However, there was no horizontal slip between the wall top and ceiling. The floor joist blocking was slipping vertically by approximately ±10 mm relative to the floor joists at the ends of the internal wall. The adjacent flooring was lifting by a similar amount from the supporting joists – being forced upward by the lifting blocking. The upper wall lining-to-ceiling joint had torn along part of End 1. However, an examination of the condition of the lining at the internal fasteners showed that the wall shear strength had not been exceeded.

5.3 Test hysteresis loops

Figure 41 shows the generated total building load-deflection (hysteresis) loops. It can be seen that the resisted load dropped off in Test 2 which was due to floor joist uplifts from their supports (as discussed in Section 5.2). A gain of strength was achieved in Test 3 due to the strengthening described in Section 5.2(b).

5.4 Floor joist uplift

Floor joist uplift movement from the top plate support versus racking load on the adjacent wall is plotted in Figure 42. Only the results of Tests 1 and 2 are plotted as those from Test 3 are irrelevant because the joist fixing had been strengthened. The plot indicates that racking loads 40–60 BUs/m will result in large joist uplifts. However, as discussed in Section 5.1, the joists had partially lifted from their supports in Stage I testing and hence weakened the connection. As these weakened nail connections were not replaced for this test, the results for joist end connections are rejected as being unduly conservative.

The racking load at which uplift failure of the internal floor joists occurred was influenced by the uplift failure of the boundary joist. If the boundary joist nailing connection had been increased to four times the density (as recommended later in this report), it is expected that a higher racking load would have been required before internal joist uplift failure occurred.

5.5 Floor joist roll-over and boundary joist horizontal slip

As the racking loads applied were parallel to the floor joists, joist roll-over itself is not a consideration. However, horizontal slip between the boundary joist and the lower storey top plate needs to be considered. Figure 43 plots this slip for End 1. It also plots the difference between the floor and lower-storey wall top plate horizontal deflection at both End 1 and the internal wall. The plot showed that this latter movement was small for the floor joists at the internal wall. However, the maximum racking load on the internal wall was low. At End 1 this movement was slightly greater than the boundary joist horizontal slip – as expected. A small extrapolation of the plot indicates large slip deflections would occur for racking deflections greater than 95 BUs/m. Note: this was for construction where the nails between the boundary joist and the lower storey bottom plate were at 600 mm centres.

5.6 Inter-storey uplift movement at End 1

Inter-storey uplift is defined in Section 4.5. It is plotted versus applied load in Figure 44 for points P, Q, R and S on End 1. These points are defined in Figure 5. Most of the uplift movement is due to uplift of the boundary joist from the top plate. The largest measured uplift was 16 mm at Point P at approximately 95 BUs/m i.e. at a corner. This can be taken as the failure wall bracing rating for boundary joists fixed as constructed.

Figure 45 plots the percentage of wall horizontal movement attributed to 'rocking' calculated from the vertical movements measured at these points. During the early stages of testing (i.e. in Test 1) the percentages were less than 50% as the nail pull-outs from the boundary joists were low. However, in Test 2 the percentages were generally above 80%. This was true even at low bracing ratings as the nails had loosened in the latter stages of Test 1. It is therefore not surprising that there was little damage observable at the wall lining fixings as 'rocking' limited the maximum wall loads, and hence the linings were not being subjected to large shear distortions.

5.7 Blocking vertical movement

As shown in Figure 1, blocking was used between floor joists at 100 mm from each end of the internal wall. The wall bottom plate was bolted to the blocking, as shown in

Figure 34 and Figure 51. The vertical movement of the blocking relative to the closest joist to the internal wall is plotted in Figure 46. The upward movement raised the flooring from the joists. The maximum measured blocking uplift was 13 mm which occurred at an internal wall bracing load of 5.5 kN (75 BUs/m).

5.8 Components of internal wall deflection

The percentages of wall top horizontal movement attributable to each of the three main components of 'rocking' deflection are plotted in Figure 47. Comments are:

(1) The blocking vertical movement generally accounted for between 40–60% of the top horizontal wall movement.

(2) Vertical movement of Joist 6 (see Figure 1) was only measured at Point T (measurements were made at both ends of the internal wall in Stage III testing.) In the calculations below, joist movement was assumed to be zero at U. A significant portion of the vertical movement at T (called Δ_T) and U (called Δ_U) would have been due to floor joist uplifts from their supports. The horizontal wall movement due to this deflection, as used in Figure 47, was $(\Delta_T - \Delta_U)/1.267*2.42$ where the latter two values are the spacing between points T and U and the wall height respectively. From Figure 47, 20–30% of the internal wall horizontal movement is attributable to joist vertical movement.

(3) Internal wall vertical movement from the floor was only measured in Test 2 and 3. This can be converted to a wall horizontal deflection and accounts for 15–30% of the movement.

The sum of the percentages calculated in (1) to (3) above is also plotted in Figure 47, but only for Test 2 and 3. It can be seen that these components account for approximately 100% of the internal wall horizontal movement. Thus, it is not surprising that little racking load was resisted by the internal wall as most imposed deflection was taken up by wall 'rocking'.

5.9 'Systems effect factor' for walls

The maximum magnitude of applied load was 23.1 kN on End 1 and 7.65 kN on the internal wall.

The internal wall was constructed as a Winstone Wallboards Ltd (2006) BL1 bracing element. Winstone Wallboards Ltd (2006) gives the wind bracing resistance of BL1 walls as 125 BUs/m. As the wall was 1.467 m in length the design bracing resistance = 1.467 x 125/20 = 9.16 kN. This indicates that the 'systems effect factor' = 7.65/9.16 = 0.83. However, as noted in Section 5.8, the internal wall deflection was mainly due to 'rocking' action due to the flexible foundations and the wall shear strength was well short of its ultimate strength when testing terminated. Thus, this result is meaningless.

End 1 bracing elements were constructed as Winstone Wallboards Ltd (2006) BLG bracing elements. Winstone Wallboards Ltd (2006) gives the wind bracing resistance of BLG walls as 150 BUs/m for a 1.2 m long wall. The total length of bracing walls on End 1 was 2.314 m. Thus, the design bracing resistance = 2.314 x 150/20 = 17.35 kN. This indicates that the 'systems effect factor' = 23.1/17.35 = 1.33. However, End 1 was short of its ultimate strength when testing terminated and thus this result is likely to be conservative.

5.10 Measured end restraint

Section 3.5 discusses the method of using the measured wall uplift forces to estimate the end continuity uplift restraint ER. Section 3.5 of Thurston (2007(b)) discussed the influence of axial loads W1 and W2.

Figure 48 plots the theoretical relationship between wall anchor uplift force (kN) and positive wall racking loads (kN/m).

The uplift force was measured by load cells at Locations Q and R on End 1 and T and U on the internal wall (refer to Figure 1 and Figure 5 for location of these points). This data is plotted in Figure 48 at each cycle first peak upwards load.

5.10.1 Comments on the measured restraint for End 1

At racking loads of magnitude less than 3.7 kN/m the theory for ER = 9 kN predicts that there should be zero wall reaction. However, the test data shows that significant wall end reactions did occur. This is attributed to the end restraint only becoming mobilised after significant wall end vertical movement has occurred. However, at racking loads of magnitude greater than approximately 3 kN/m, the theory for ER = 9 kN is conservative, but in reasonable agreement with the anchor uplift forces measured.

5.10.2 Comments on the measured restraint for the internal wall

This wall has no built-in end continuity, although there is some horizontal restraint on the wall plasterboard due to the plastered joint along the ceiling line. In addition, when the wall ‘rocks’ under racking loads some uplift restraint will be imposed by the ceiling framing, although the ‘soft’ foundation system (as measured in Section 5.8) is likely to make this small. The data in Figure 48 lies between the theory for ER = 0 and 4.5 kN. This shows that end continuity is required before a reliable uplift restraint of ER = 9 kN can be assumed.

5.11 Conclusions from Stage II testing

(1) In calculations to measure the bracing rating at which joist and bearer failure will occur, it is proposed that the vertical forces on joists and bearers be calculated using ER = 9 kN for walls with normal end continuity, but to use ER = 0 kN for isolated walls (i.e. those without end continuity).

(2) The connection between floor joists and their end supports failed at the internal wall bracing rating of 40–60 BUs/m. However, as these connections had already been weakened by the Stage I testing this result is discarded as being too conservative.

(3) Inter-storey vertical movement between upper and lower wall studs caused End 1 walls to ‘rock’ and contributed approximately 80% of the total upper wall inter-storey horizontal movement. This was mainly due to separation of the boundary floor joist connection from the top plate of the wall below. The boundary joist had only the normal three nails at the ends plus skew nails at 600 mm centres along the length. One boundary joist lifted by 16 mm and slipped laterally by 9 mm at approximately 95 BUs/m. It is recommended that the spacing of boundary joist skew nail fixings, as used in the test, be reduced from 600 mm to 100 or 150 mm.

(4) Vertical movement of the blocking, with respect to the adjacent joists, caused the internal wall to ‘rock’ and contributed 40–60% of the wall racking deflection e.g. an uplift deflection of 13 mm occurred at an internal wall racking load of 75 BUs/m. These large movements precluded large racking load from being introduced to the wall. It is recommended that suitable connectors (e.g. angle connectors) be used between blocking and joists where a bracing wall is bolted to the blocking and the blocking is not directly over and nailed to the framing below. The other major contributor to ‘rocking’ was uplift of the floor joists from their supports. It is recommended that mechanical connectors be considered at joist supports which are significantly loaded via a bracing wall. Without this

strengthening, very little racking load can be transferred from internal walls, which makes them ineffective as bracing walls.

(5) The ‘systems effect factor’ for End 1 was 1.33. However, it was noted that End 1 had not reached full strength.

6. STAGE III TESTING: RACKING TESTS ON END 2 AND THE SECOND INTERNAL WALL

6.1 Repair after Stage I and II testing

All nails in the End 2 boundary joist were removed after Stage I testing. The usual three end nails were added at the joist ends. However, single nails along the length of the joist were added at 150 mm rather than the usual 600 mm centres (usual nailing is discussed in Section 2.2).

It was difficult to remove the three skew nails attaching the floor joists to the top plates along the side walls and hence they were left in place. Note: they had been loosened by the Stage 1 testing. In Stage II, the weakness of these joints was the prime failure mechanism. Thus, it was decided to add two additional nails by nailing upwards through the top plate and into the floor joists, as shown in Figure 50. This connection, in itself, is considered to be the equivalent of the fixing specified in NZS 3604:1999. Thus, in conjunction with the pre-existing weakened three skew nail connection, the total nailed fixing strength provided is considered to be (slightly) in excess of that for the specified nailing in NZS 3604.

6.2 General description of test

A plan view of the test arrangement is given in Figure 1. Note that a complete cut through all construction down to the floor level had been made at wall mid-length, as shown in Figure 2. This structurally separated the two halves of the room to preclude upper storey lateral wall load from being transmitted from one half to the other.

A general view of Stage III testing is shown in Figure 49. The same deflection regime was simultaneously imposed onto the tops of End 2 and the second interior wall (Figure 51).

The internal wall for Stage III testing was constructed directly over floor joist No. 4 shown in Figure 1. This was expected to reduce the wall flexibility compared to that tested in Stage II.

Roof dead load and a floor live load of 0.2 kPa was simulated by weights, as shown in Figure 10 and Figure 12.

The loading in Stage III testing was imposed in two separate tests, with modifications to the construction being made after Test 1 as noted below:

(a) Test 1 consisted of three cycles to ±7, ±14, ±24, ±33 mm and one cycle to ±48 mm inter-storey movement. Cracks had formed in the plaster joint above the internal wall, as shown in the top photograph of Figure 52. The hysteresis loops of load versus deflection had flattened or were sloping downwards with increased deflection. The floor joists had lifted excessively (as discussed later). Consequently, it was decided that the substructure below the first floor had exceeded their maximum strength and the only purpose in continuing testing was to measure the strength of the top storey walls. Hence, the connections between the joists and the wall below (including the boundary joists) were strongly strapped together and the test continued to examine the maximum racking strength that the walls themselves could achieve.

(b) Test 2 consisted of a push to 65 mm, a pull to -55 mm and then a push to 83 mm inter-storey movement. The vertical movement of the internal wall progressively ripped the plasterboard joint between the top of the internal wall and the ceiling over the entire length, as shown in the bottom photograph of Figure 52, and the ceiling was slipping horizontally across the top plate of the internal wall with little load being transferred to the internal wall. Note: this wall was fixed to the ceiling nogs with pairs of 90 mm long power-driven nails at 600 mm centres (NZS 3604 is silent on this connection). The bottom plate failed in flexure on each side of the opening at End 2, as shown in Figure 56 and Figure 57. The boundary joist cracked at a knot, as shown in Figure 58.

6.3 Test hysteresis loops

Figure 59 shows the generated total building load-deflection (hysteresis) loops. It can be seen that the resisted load dropped off in Test 1. Test 2 showed a gain of strength due to the strengthening described in Section 6.2(a).

6.4 Floor joist uplift

The floor joist uplift movement from the top plate support versus racking load on the adjacent wall is plotted in Figure 60. The plot indicates that racking loads greater than 160 BUs/m will result in joist uplift greater than 12 mm, which is far greater than the corresponding value of 60 BUs/m which were measured in Stage II. This shows that the extra nails added in Stage III, but which were not added in Stage II, were very effective.

6.5 Floor joist roll-over and boundary joist horizontal slip

As the loads applied were parallel to the floor joists, joist roll-over itself is not a consideration. However, horizontal slip between the boundary joist and the lower storey top plate needs to be considered. Figure 61 plots this slip for the End 2 joist. It also plots the difference between the floor horizontal deflection and lower storey wall top plate deflection for both End 2 and at the internal wall.

The plot shows that this movement was small for the floor joists at the internal wall. However, the maximum racking load on the internal wall was low. At End 2 this movement was approximately twice the boundary joist horizontal slip. The plot indicates significant slip movement for racking deflections greater than 105 BUs/m, but this was still well short of failure and thus these results are deemed to be non-conclusive. Note: this was for a construction where the nails between the boundary joist and the lower storey bottom plate were at 150 mm centres.

6.6 Inter-storey uplift movement

Inter-storey uplift is defined in Section 4.5. It is plotted versus applied load in Figure 62 for points L, M, N and O on End 2. These points are defined in Figure 6. Most of the uplift movement is due to uplift of the boundary joist from the top plate of the wall below. The largest measured uplift was at Point O at approximately 105 BUs/m (i.e. at one corner). This can be taken as the maximum wall bracing rating for boundary joists fixed as constructed.

Figure 63 plots the percentage of wall horizontal movement attributed to ‘rocking’ calculated from the vertical movements measured at these points. The results varied between 62 and 89%. It is therefore not surprising that there was little observable damage at the wall lining fixings before the failure of the bottom plates as ‘rocking’ limited the maximum wall loads, and hence the linings were not being subjected to large shear distortions.

6.7 Joist vertical movement at the ends of the internal wall

The vertical movement of floor joists 3, 4, and 5 were measured at the line of each end of the internal wall (i.e. at locations Z1 and Z2 as defined in Figure 1). The results are shown in Figure 64 and Figure 65. The movement is due to joist deflection from flexural bending plus a component resulting from the floor joist uplifts from their supports. Thus, the upward movement shown in the plots is greater than the downward movement.

6.8 Components of internal wall deflection

The percentage of wall horizontal movement for Test 1, which is attributable to each of the two main components of 'rocking' deflection, is plotted in Figure 66. This does not include data from Test 2 as the horizontal slip at the top of the internal wall relative to the ceiling distorted the results. Components of deflection are described as follows:

(1) The wall horizontal deflection due to vertical movement of Joist 4 at Location Z1 and Z2 was computed (as discussed in Section 5.8). Figure 66 shows that this movement generally accounted for approximately 50% of the horizontal wall movement.

(2) Internal wall vertical movement from the floor accounted for between 27–41% of the movement.

The sum of the percentages calculated in (1) and (2) above is also plotted in Figure 66. These components account for 76–86% of the internal wall horizontal movement. Thus, it is not surprising that little racking load got transferred into the internal wall as most imposed deflection was taken up by wall 'rocking'.

The total vertical movement at the ends of the internal wall were separately measured relative to the ground. The percentage of horizontal movement was also calculated from these gauges (Figure 68). This indicates that rocking accounted for between 78-91% of the internal wall horizontal movement. This percentage dropped to below 50% in the last stages of the test when slip of the wall top plate also accounted for a significant part of the total deflection.

6.9 Separation of plates

Figure 54 shows separation of top and bottom plates – this occurred at a bracing rating of 101 BUs/m.

6.10 Bottom plate flexural failure

The bottom plate failed in flexure at the hold-down bolt locations on each side of the opening at End 2, as shown in Figure 56 and Figure 57. The timber had no knots and had regular grain near this location. The measured vertical uplift force was 10.7 kN at M and 11.0 kN at N when these two failures occurred (refer to Figure 6 for these locations).

To calculate the timber stress at failure, the model shown in Figure 67 is used. Consider the bottom plate as a free body with a downward force of A = 10.7 kN at the bolt hole. This is 78 mm from mid-width of the strap connecting the stud to the bottom plate. The wall is 1157 mm long and the distance between the mid-width of the end studs is taken as 1157 – 45 = 1112 mm. The bottom plate bending moment at the bolt hole is then given by:

10.7x 78 x (1157-78)/1157 = 778 kN mm

The failure stress, f_b^*, is determined from:

$f_b^* = 778{,}000/\{(90-12) \times 45^2/6\} = 29.6$ MPa.

This is 2.11 times the timber design stress, f_b, which is 14 MPa for grade MSG 8. The other plate failed at 2.17 times the design stress. Such failures are sometimes seen in P21 bracing tests. Clearly, plate failure is one aspect that needs careful consideration when establishing the maximum bracing rating, particularly where there is a larger lever arm between the hold-down bolt and the strap or where the bottom plate contains defects.

6.11 'Systems effect factor' for walls

The maximum magnitude of applied load on End 2 was 25.03 kN (it was 17.50 kN on the internal wall).

The internal wall was constructed as a Winstone Wallboards Ltd (2006) BL1 bracing element. Winstone Wallboards Ltd (2006) gives the wind bracing resistance of BL1 walls as 125 BUs/m. As the wall was 1.467 m in length, the design bracing resistance = 1.467 x 125/20 = 9.16 kN. This indicates that the 'systems effect factor' = 17.50/9.16 = 1.91. However, as noted in Section 6.12, the internal wall deflection was attracting significant vertical load from the ceiling at it 'rocked' upward.

End 2 bracing elements were constructed as Winstone Wallboards Ltd (2006) BLG bracing elements. Winstone Wallboards Ltd (2006) gives the wind bracing resistance of BLG walls as 150 BUs/m for a 1.2 m long wall. The total length of bracing walls on End 2 was 2.314 m. Thus, the design bracing resistance = 2.314 x 150/20 = 17.35 kN. This indicates that the 'systems effect factor' = 25.03/17.35 = 1.44. However, the End 2 lining connections were short of their ultimate strength when testing terminated, although both bracing elements had failed due to bottom plate flexural failure.

6.12 Measured end restraint

Section 3.5 of Thurston (2007(b)) discusses the method of using the measured load cell wall uplift forces to estimate the end continuity uplift restraint ER and the influence of axial loads W1 and W2.

Figure 69 plots the theoretical relationship between wall uplift anchor force (kN) and positive wall racking loads (kN/m).

The uplift force was measured by load cells at Locations M and N on End 2 and Z1 and Z2 on the internal wall (refer to Figure 1 and Figure 6 for location of these points). This data is plotted in Figure 69 at each cycle first peak upwards load.

6.12.1 Comments on the measured restraint for End 2

The test data fits closer to ER = 4.5 kN theory than ER = 9 kN theory. This is attributed to the opening being the full wall height rather than a door or window opening and thus the wall continuity effect is smaller.

6.12.2 Comments on the measured restraint for the internal wall

In Stage II the internal wall was fixed to blocking which moved vertically when subjected to the wall vertical end reactions. The connection of the floor joists to their supports had been weakened by Stage I testing. Both these construction aspects had significantly increased the flexibility of the internal wall. However, the Stage III internal wall was bolted directly to a floor joist and the floor joist connections had been strengthened (as discussed in Section 6.1). Both these construction aspects had significantly increased the stiffness of the internal wall in Stage III.

The Stage III internal wall had no adjacent construction to provide end continuity, although it had some horizontal restraint on the plasterboard along the ceiling line.

As the racking loads increased, the measured uplift forces in Figure 69 were consistent with the degree of uplift restraint increasing from ER = 0 kN to ER > 9 kN. This also is consistent with the theory in Section 3.5 of Thurston (2007(b)) who defines W2 as the axial load attracted to the uplifting wall corner. It is expected that W2 will increase with wall uplift deflection, and that uplift restraint due to W2 can be large if the internal wall is constructed on a moderately rigid foundation and the ceiling is moderately stiff.

6.13 Conclusions from Stage III testing

(1) In calculations to measure the bracing rating at which joist and bearer failure will occur, it is proposed that the vertical forces on joists and bearers be calculated using ER = 9 kN for walls with normal construction continuity. Where one end of a bracing wall is a full height opening, ER = 4.5 kN was a better fit to the data. At an internal wall founded directly on a floor joist and with a moderately stiff ceiling framing, the wall attracted a high axial load (W2) on the lifting corner. Although this wall had no continuity (i.e. was not built into adjacent walls) the effect at high bracing loads was the same as assuming W2 = 0 and putting ER > 9 kN.

(2) Inter-storey vertical movement between upper and lower wall studs caused the End 2 walls to ‘rock’ and contributed approximately 75% to the total upper wall inter-storey movement. This is partially due to separation of the boundary floor joist from the wall plate below.

(3) ‘Rocking’ of the internal wall contributed approximately 85% of the wall horizontal deflection (this added much flexibility to the wall). Vertical movement of the floor joist directly under the wall accounted for 50% of the horizontal movement with uplift of the wall from the floor accounting for 35%.

(4) The floor joists had extra nails added (as discussed in Section 6.1). Some separated from their supports by more than 16 mm at the internal wall bracing rating of 160 BUs/m. Note: they failed far earlier in Stage II testing as the joist connection was weaker (as discussed in Section 6.1). However, it is recommended that mechanical connectors be considered at joist supports which are significantly loaded via a bracing wall.

(5) NZS 3604 is silent on the fixing required between the boundary joists and the plate beneath. (It does require 18 power-driven skewed nails per 1.5 m length – i.e. spacing of 83 mm – on the plate on a foundation wall where a foundation wall is defined as a masonry or concrete wall to transfer horizontal loads to these strong elements, but this is a different situation.) It is recommended that the minimum connection be a single power-driven 90 x 3.15 mm skew nail at 100 or 150 mm centres from joist-to-wall top plate along the length of the boundary joist, being placed from the outside. In addition, the end nailing as currently stipulated should be used. Using this construction, uplift failure was found to occur at an End 2 bracing rating of 105 BUs/m.

(6) Separation of the upper and lower top plate occurred at 101 BUs/m.

(7) Two timber bottom plates failed in flexure. Based on the measured uplift force in the plate hold-down bolt and the distance (78 mm) between mid-width of the strap on the end stud and the hold-down bolt, the failure stresses were 2.25 and 2.31 times the 14 MPa design stress for the MSG 8 timber used. It is concluded that the potential for plate failure needs careful consideration when establishing maximum bracing rating.

7. CONCLUSIONS

7.1 Relationship between 'systems effect factor' and end uplift restraint

A wall bracing strength will in general be governed by either its shear strength or its 'rocking' strength. A wall with no hold-down fixing will have zero 'rocking' strength and will simply displace under lateral load by tilting on the compression corner (i.e. 'rocking'). If 'rocking' is precluded by hold-down devices or axial load then the strength will be governed by the 'shear' strength, which essentially is a function of the wall geometry and the strength of the sheathing to wall framing connections.

The P21 tests are conducted on an isolated wall as though it were cut from a house. The test arrangement incorporates an external uplift restraint which is approximately 4.5 kN at 4 mm vertical deflection. This simulates the uplift resistance which would be imposed on the wall by the adjacent construction if it had been incorporated within a house.

The wall 'systems effect factor' is defined as the ratio of the bracing strength of a wall in a house to the strength of the same wall tested in a P21 test. If the bracing strength of the wall is governed by 'rocking' then the wall 'systems effect factor' is greater than 1.0 only if the actual uplift restraint in a house is greater than simulated in the test. Similarly, if the bracing strength of the wall is governed by 'shear' then the wall 'systems effect factor' is greater than 1.0 only if the actual wall shear strength is greater than determined in the test because some aspects of the connection to adjacent construction have influenced the shear strength. Provision of plasterboard joints at the ceiling and bracing wall ends will increase the wall shear strength because it restrains the wall sheet movement.

The house 'systems effect factor' is defined as the ratio of the bracing strength of a whole house to the sum of the strength of all bracing walls used in the house if separately determined in P21 tests. Aspects which increase whole house bracing strength include walls not considered in design as bracing walls, doors, chimneys, windows, contents (which will carry horizontal load when a house is racked), portal effects of trusses and walls, and house weight resisting wall 'rocking'. If some walls are significantly stiffer than others, due to wall length or difference in flexibility of supports, then some walls may fail before others reach full strength and the house 'systems effect factor' is reduced. House twisting due to torsional response will also affect the house 'systems effect factor'.

It is proposed to increase the uplift restraint and to introduce a 'systems effect factor' in the P21 racking test procedure. Assume that the P21 uplift strength is increased by a factor X. Assume that the measured specimen strengths are factored by Y to include house 'systems effects'. Say values of X and Y are to be determined using the results of this report where the room 'systems effect factor' was found to be Z. The recommended value of X given in this report = 2.0 (as results indicated that a 9 kN end restraint as against the current P21 4.5 kN end restraint fitted the test results best). If wall strength had been governed by 'rocking' in the original P21 test, then the P21 test results would show greater strength with the enhanced end restraint. Z = room strength/(P21 wall strengths), and as the P21 wall strengths have increased, then Z would decrease. Thus, with the current value of Z, Y should be less than Z. However, as the walls used in tests described herein had a very strong hold-down system and were thus governed by shear, the value of Z as determined from the tests should be used (this is expected to be conservative). The writer's subjective assessment of the appropriate value of Z (house 'systems effect factor') is Z = 1.2.

7.2 Method of determining limiting bracing rating criteria

Generally house wall systems have performed well in past earthquakes. This is attributed to the lightweight nature and the low natural period of most houses. Also the small room sizes and excess of walls of many houses built in the past made the houses stronger than required by the Building Code. The natural periods of most houses are well below the critical period of 0.4 seconds assumed by NZS 3604, and have greater damping and are lighter than assumed in the analysis and thus are expected to attract less than the design seismic load in a design level event.

Houses lined with plasterboard on the interior and with fully taped and stopped joints are expected to be significantly stronger than construction without such joints. On the other hand, the lack of any major earthquake in New Zealand since the 1931 Napier earthquake means modern house construction has not been fully tested by a major event. However, generally reports following New Zealand and overseas earthquakes find only small house structural damage for houses comparable to that built in this country. This suggests that it is inappropriate to apply criteria which are too conservative as a measure to avoid foundation failures.

Recommending a maximum bracing rating that may be used for evaluation of P21 tests results, based on the work described herein, is very subjective. For a ductile mechanism, the values given by the author below were based on the minimum bracing rating at which failure occurred in the tests. Failure is defined as the bracing load at which a mechanism occurs where the mechanism effectively stops any greater bracing rating being achieved.

With regard to avoiding bottom plate, joist or bearer flexural failure (considered to be a more undesirable and brittle failure mechanism) it is recommended that the maximum strength be computed from twice the characteristic timber strength factored by a strength reduction factor of 0.8. Walford (2006) advised that twice the characteristic timber strength was approximately the average timber strength. Thurston 2007(a) calculates a maximum bracing rating to avoid these flexural failures based on these recommendations.

Note: both ductile and non-ductile failures may go unnoticed after an earthquake, as they both tend to close up and become difficult to detect in the ‘unload’ condition.

It is likely that the principles outlined above will be vigorously discussed and that the author’s recommendations will be modified before adoption by NZS 3604.

7.3 Criteria derived from test observations of ductile failure mechanisms

The most critical results from testing described in this report are given below:

(1) From Stage I testing, a wall bracing rating of 95 BUs/m separated floor joists from their end supports (i.e. the skew nail connection to the wall top plate connections).

(2) The joists were fixed at their ends to the wall top plates with three skewed power-driven 90 x 3.15 mm Paslode JD nails as per Table 7.5 of NZS 3604 (two one side and one the other). NZS 3604 is silent on the fixing required along the length of boundary joists. Two arrangements were tested:

    (a) single power-driven 90 x 3.15 mm skew nails at 600 mm centres from floor joist-to-wall top plate along the length of the boundary joist, being placed from the outside
    (b) as in (a) but at 150 mm centres.

    In Stage I testing, arrangement (a) failed at 70 BUs/m and arrangement (b) failed at 110 BUs/m. In Stage II testing, arrangement (a) failed at 95 BUs/m, while in Stage III arrangement (b) failed at 105 BUs/m. It is recommended that arrangement (b) be adopted by NZS 3604 or, alternatively, that the spacing be at 100 mm centres.

(3) In Stage I testing, which used blocking between each floor joists along the wall support below, joist roll-over deflection was a significant proportion of the horizontal deflection of bracing walls in the storey above. Deflections greater than 45 mm (hence possibly leading to instability) were expected to occur at bracing ratings of 120 BUs/m. This critical bracing value is expected to be lower if blocking is not required between all joists, as currently allowed by NZS 3604. If each blocking piece is fixed to framing below with six skewed nails as specified in NZS 3604:1999 Table 7.6, then intermittent blocking may be used.

(4) Blocking uplift of 13 mm occurred at a bracing rating of the internal wall of 75 BUs/m.

(5) Separation of the upper and lower top plate occurred at 101 BUs/m.

7.4 Other conclusions

(1) Based on tests on external walls with normal end continuity construction, it was concluded that the uplift restraint using the model, as discussed in Section 3.5 of Thurston (2007(b), can be approximated by the expression ER = 9 kN. If the wall terminated at a full wall height opening, the best approximation was ER = 4.5 kN.

(2) Tests were also done on internal walls with no end continuity although they had a plastered ceiling joint. A value of ER = 0 kN was found to fit the data for internal walls on a 'soft' timber floor frame. With a more rigid frame, the lifting corner attracted axial load from the ceiling, which at high bracing loads had the same effect as assuming that this axial load was zero, but that an end uplift restraint given by ER = 9 kN should be assumed.

(3) 'Rocking' accounted for approximately 75% of the deflection of the external walls. In Stage I testing, the load was perpendicular to the floor joists, and the 'rocking' was mainly due to separation of the floor joist connection from its supports. In Stage II and III testing, the load was parallel to floor joists and the 'rocking' was mainly due to vertical movement of the boundary joists.

(4) If a bracing wall is coach screwed or bolted to blocking, and if each blocking piece is not fixed to the framing below with skewed nails as specified in NZS 3604:1999 Table 7.6 (such as beneath walls on cantilevered joists or beneath upper storey internal walls without a wall directly below), it is recommended that mechanical fixing of the blocking-to-joist connection greater than the two end nails specified in NZS 3604 be used. This was particularly demonstrated in Stage II testing where the joist-to-blocking connection was so flexible that very little load could be transferred into the wall from the ceiling above.

(5) The 'systems effect factor' for wall strength varied considerably in the different test stages. In Stage I the walls did not get racked to near their ultimate strength as the substructure failure precluded greater load from being applied. Very little load could get transferred into the internal wall in Stage II due to the substructure flexibility and therefore again the factor was low. The factors were 1.33 and 1.44 for End 1 (Stage II) and End 2 (Stage III) respectively, but the lining fixings on these walls did still not reach their ultimate shear strength. The factor for the Stage III internal wall was 1.91. This latter value was influenced by the high axial load attracted from the ceiling during the 'rocking' motion of this wall.

(6) Wall deflections increase considerably due to the substructure flexibility (i.e. floor framing flexibility or the flexibility of the floor framing connections). Thus, walls founded on the more rigid parts of the substructure may consequently carry a disproportionate share of the racking load.

(7) Timber bottom plates and a boundary joist failed in flexure during the tests. The potential for these failures to occur must be considered in the determination of the maximum bracing rating.

8. REFERENCES

Cooney RC and Collins MJ. 1979 (revised 1982, 1987, 1988, 1990). 'A Wall Bracing Test and Evaluation Procedure'. BRANZ *Technical Paper P21*. BRANZ Ltd, Judgeford, New Zealand.

King AB and Lim KYS. 1991. 'Supplement to P21: An Evaluation Method of P21 Test Results for Use with NZS 3604:1990'. BRANZ *Technical Recommendation TR 10*. BRANZ Ltd, Judgeford, Wellington.

Standards New Zealand. 1992. *NZS 4203 General Structural Design and Design Loadings for Buildings*. SNZ, Wellington, New Zealand.

Standards New Zealand. 1999. *NZS 3604 Timber Framed Buildings*. SNZ, Wellington, New Zealand.

Standards New Zealand. 2004. *NZS 1170.5 Structural Design Actions. Part 5: Earthquake Actions – New Zealand.* SNZ, Wellington, New Zealand.

Thurston SJ. 1993. 'Report on Racking Resistance of Long Sheathed Timber Framed Walls with Openings'. BRANZ *Study Report 54*. BRANZ Ltd, Judgeford, Wellington.

Thurston SJ. 2003. 'Full-sized House Cyclic Racking Test'. BRANZ *Study Report SR 119*. BRANZ Ltd, Judgeford, New Zealand.

Thurston SJ. 2006. 'Racking Tests on Rooms and Isolated Walls to Investigate Uplift Restraint and Systems Effects'. BRANZ *Study Report SR 154*. BRANZ Ltd, Judgeford, New Zealand.

Thurston SJ. 2007(a). 'Maximum Wall Bracing that is Compatible with NZS 3604 Construction. Phase I: Theory, Summary and Bracing Testing of Single Walls on a Timber Piled Foundation'. BRANZ *Study Report 162*. BRANZ Ltd, Judgeford, New Zealand.

Thurston SJ. 2007(b). 'Maximum Wall Bracing that is Compatible with NZS 3604 Construction. Phase II: Testing of Room 1'. BRANZ *Study Report 163*, BRANZ Ltd, Judgeford, New Zealand.

Walford B. 2006. Private communication between the author and Bryan Walford of Scion, Rotorua, New Zealand.

Winstone Wallboards Ltd. 2006. *GIB® Bracing Systems*. Proprietary publication of wall bracing systems published by Winstone Wallboards Ltd, Auckland, New Zealand.

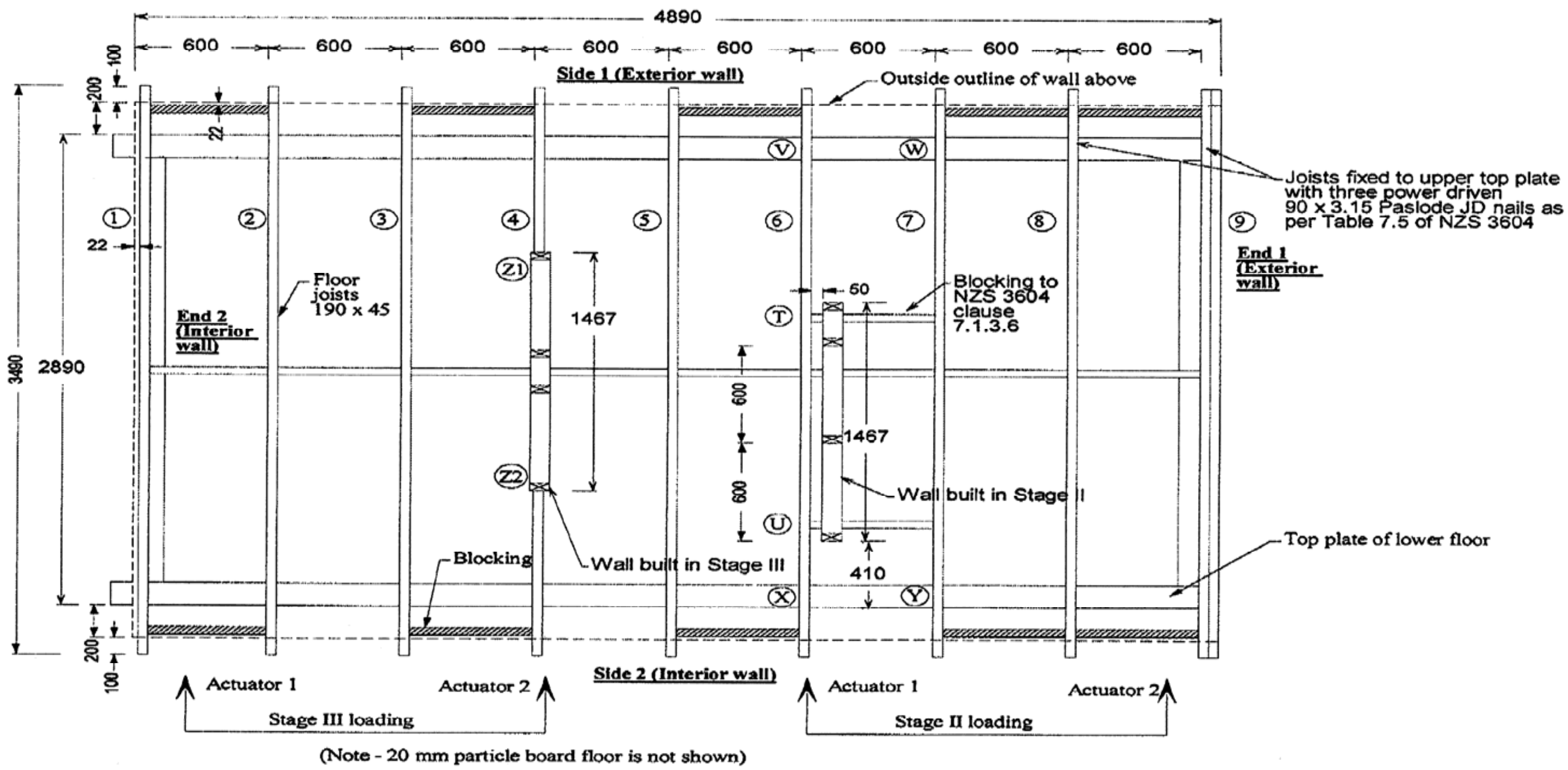


Figure 1. Plan view of floor framing

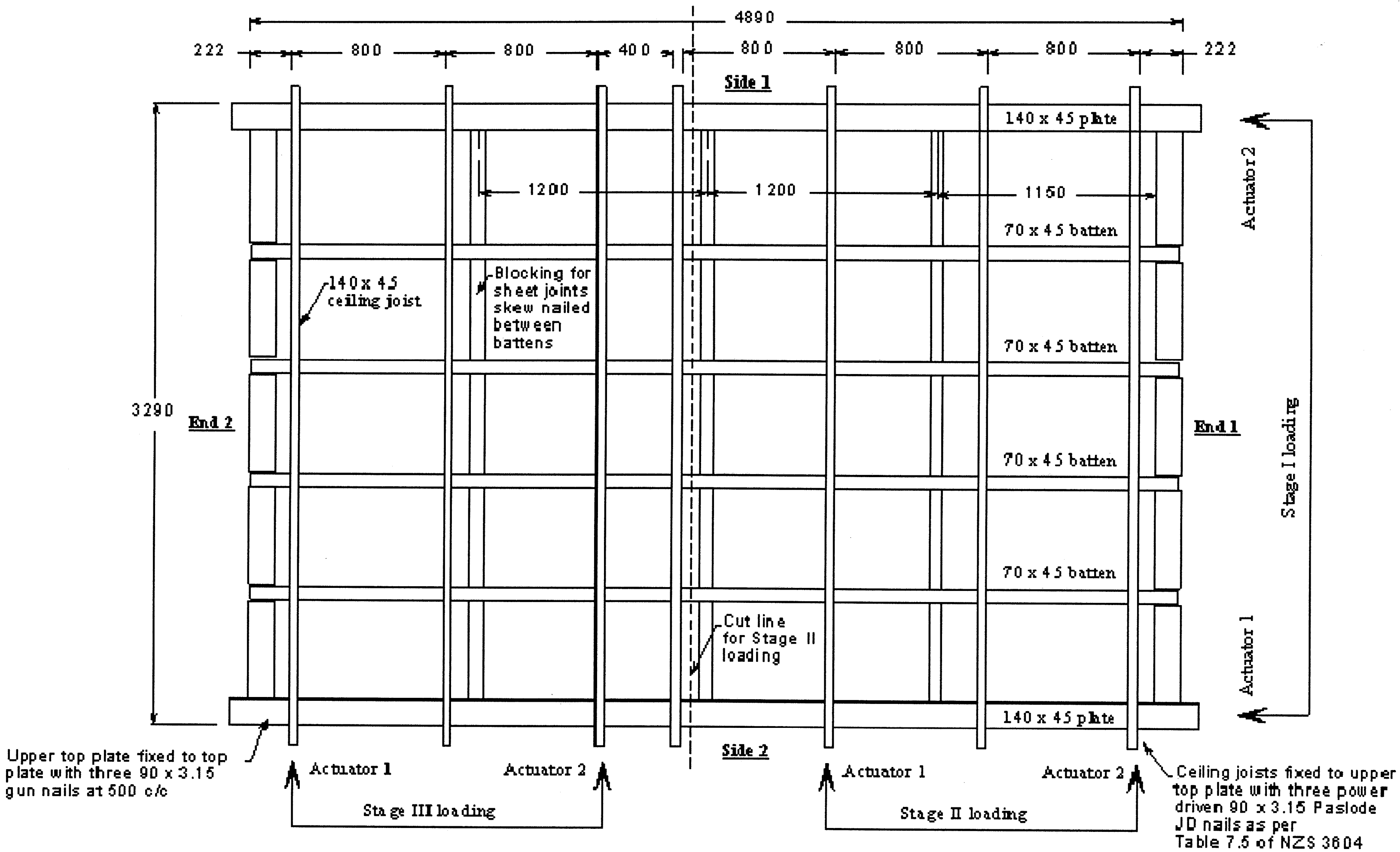


Figure 2. Plan view of ceiling framing

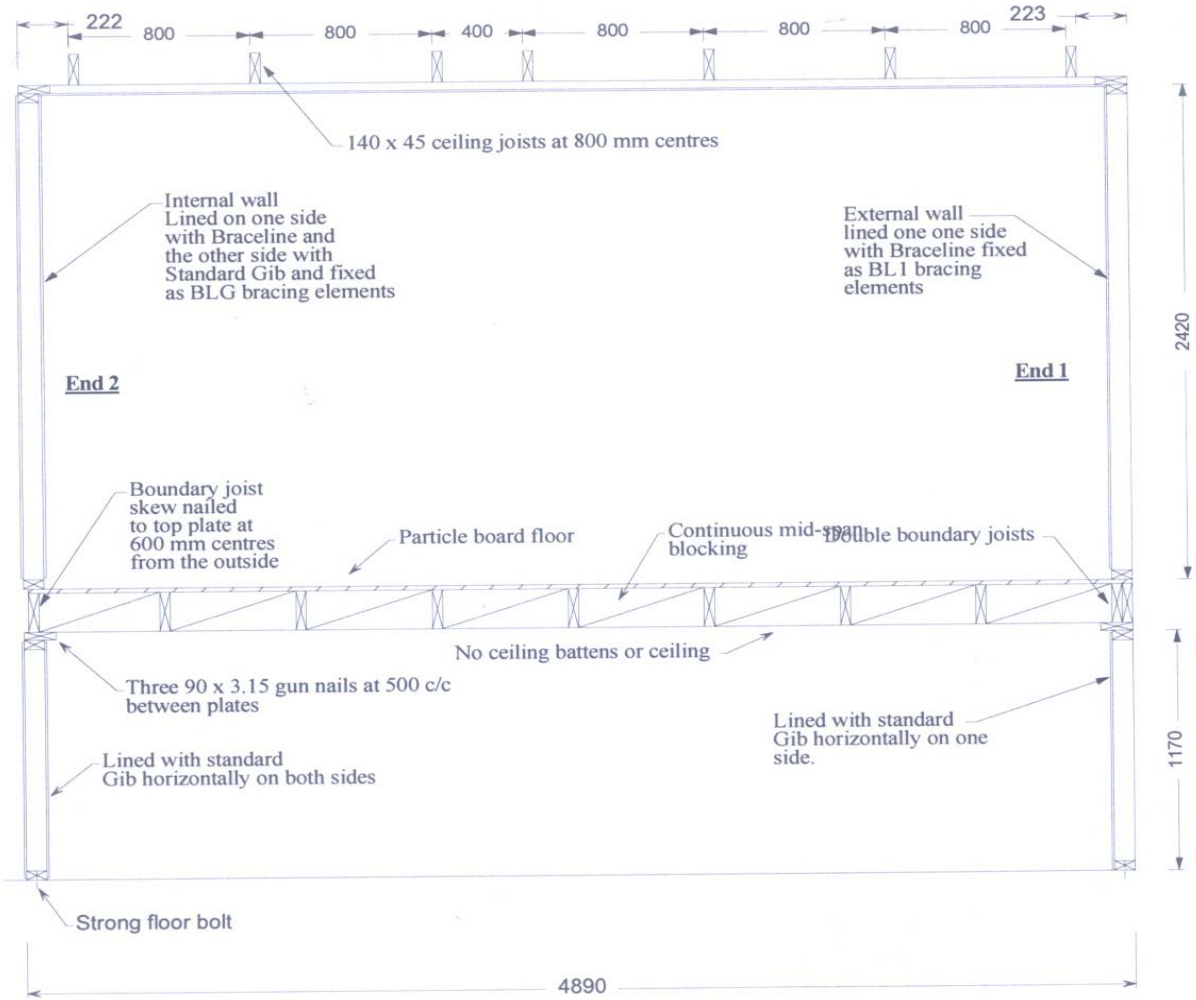


Figure 3. Cross-section of Room 2 looking towards Side 1

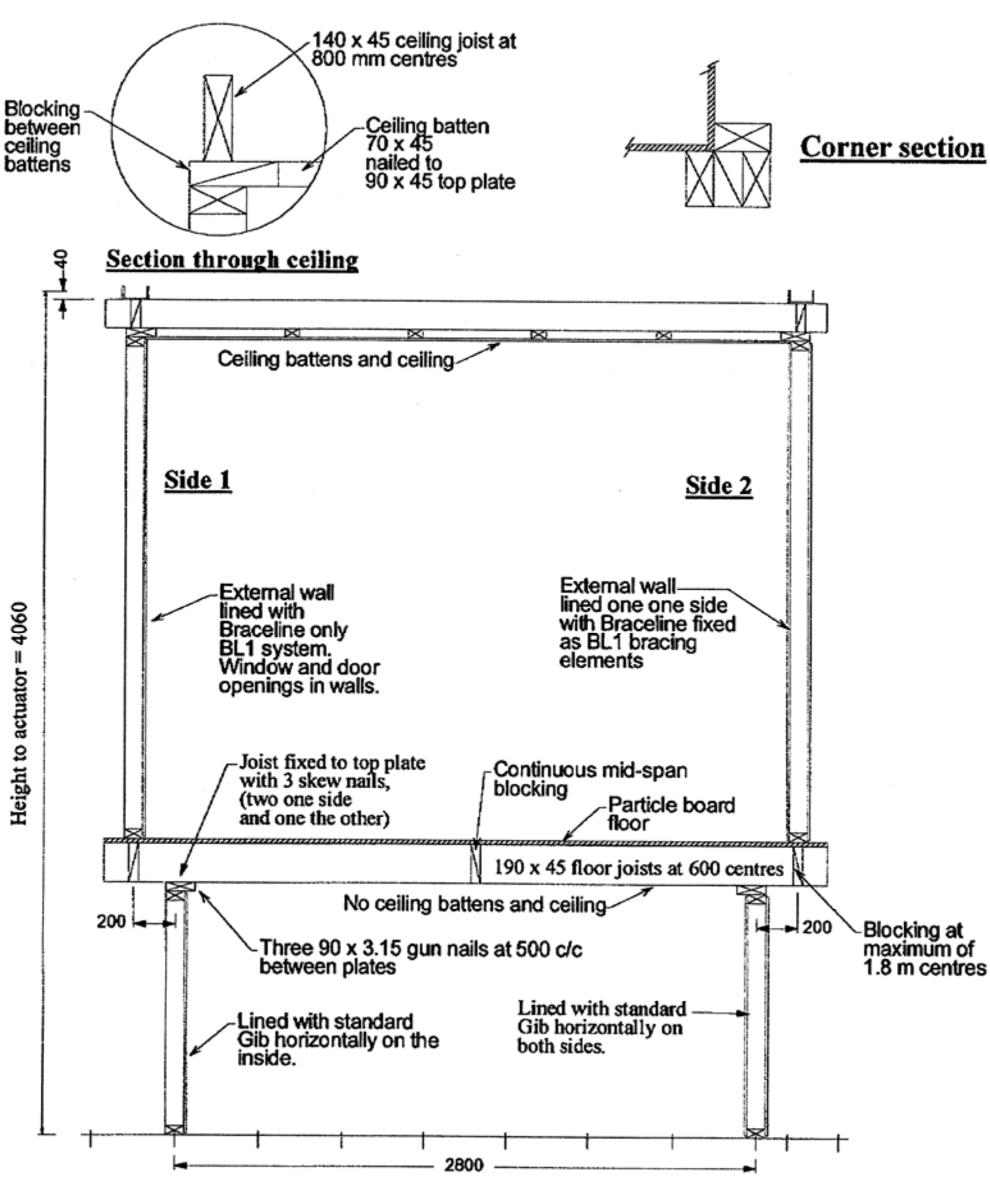


Figure 4. Cross-section of Room 2 looking towards End 1

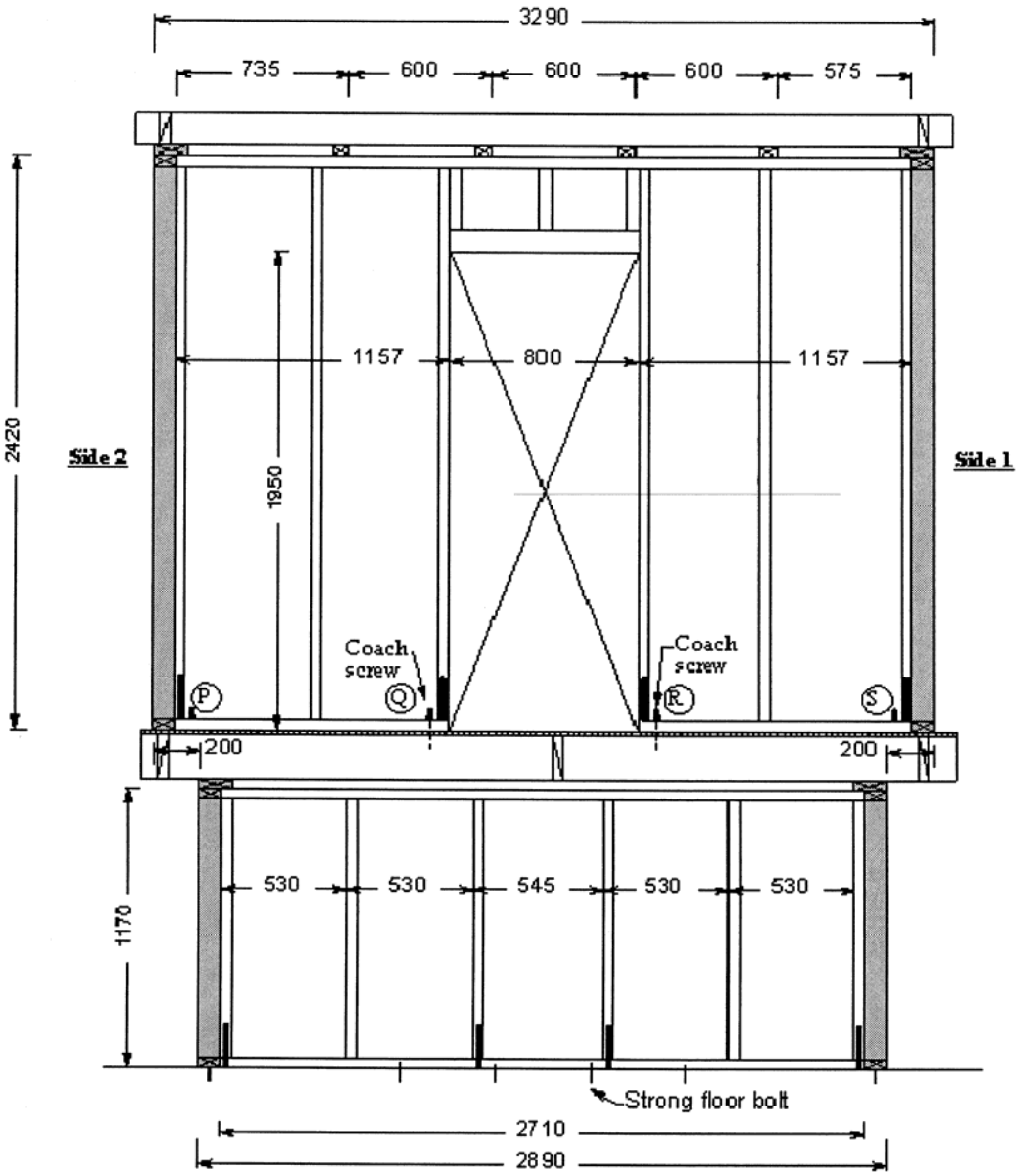


Figure 5. End 1 wall framing

(Note: greyed portion is side walls)

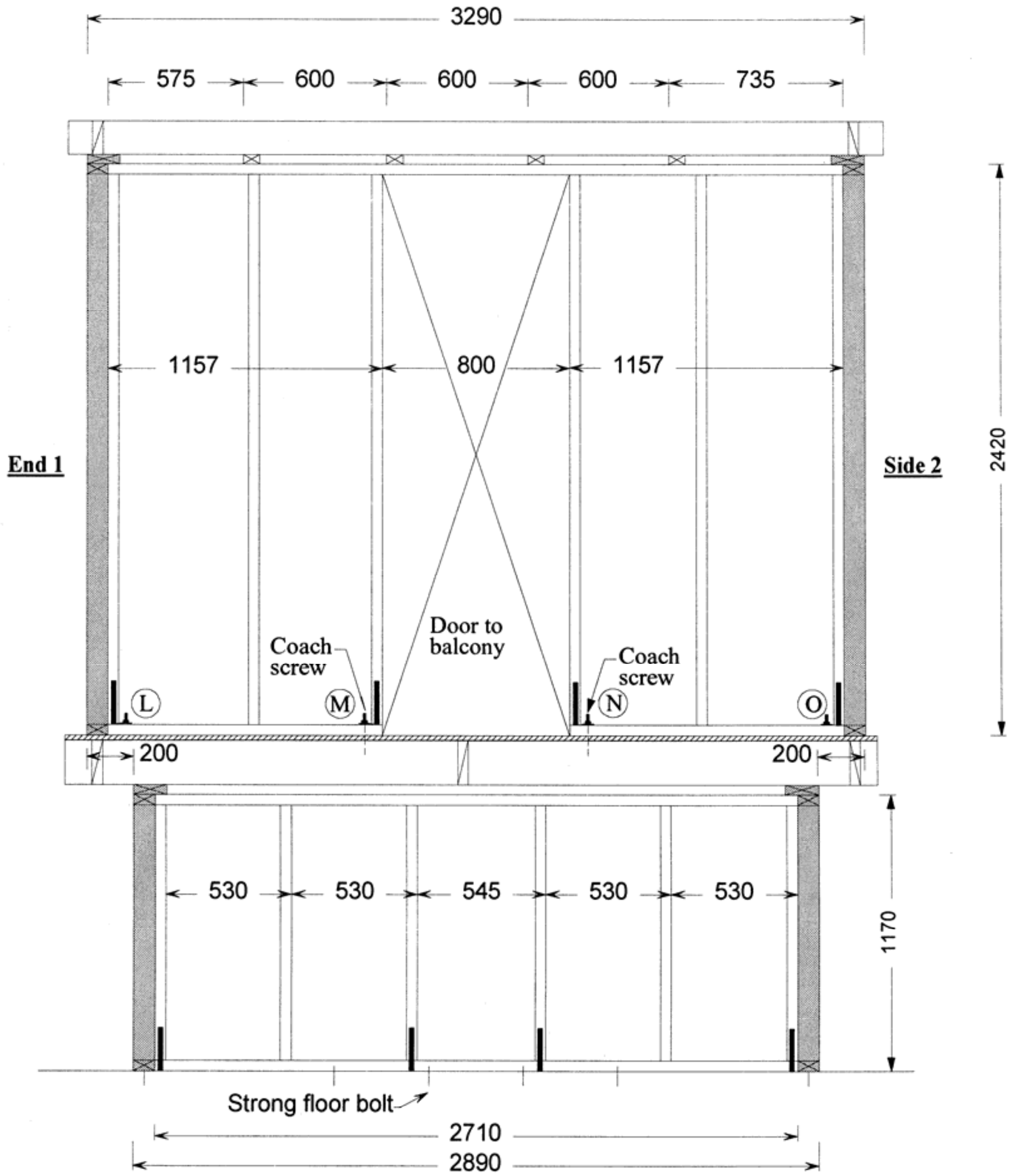


Figure 6. End 2 wall framing

(Note: greyed portion is side walls)

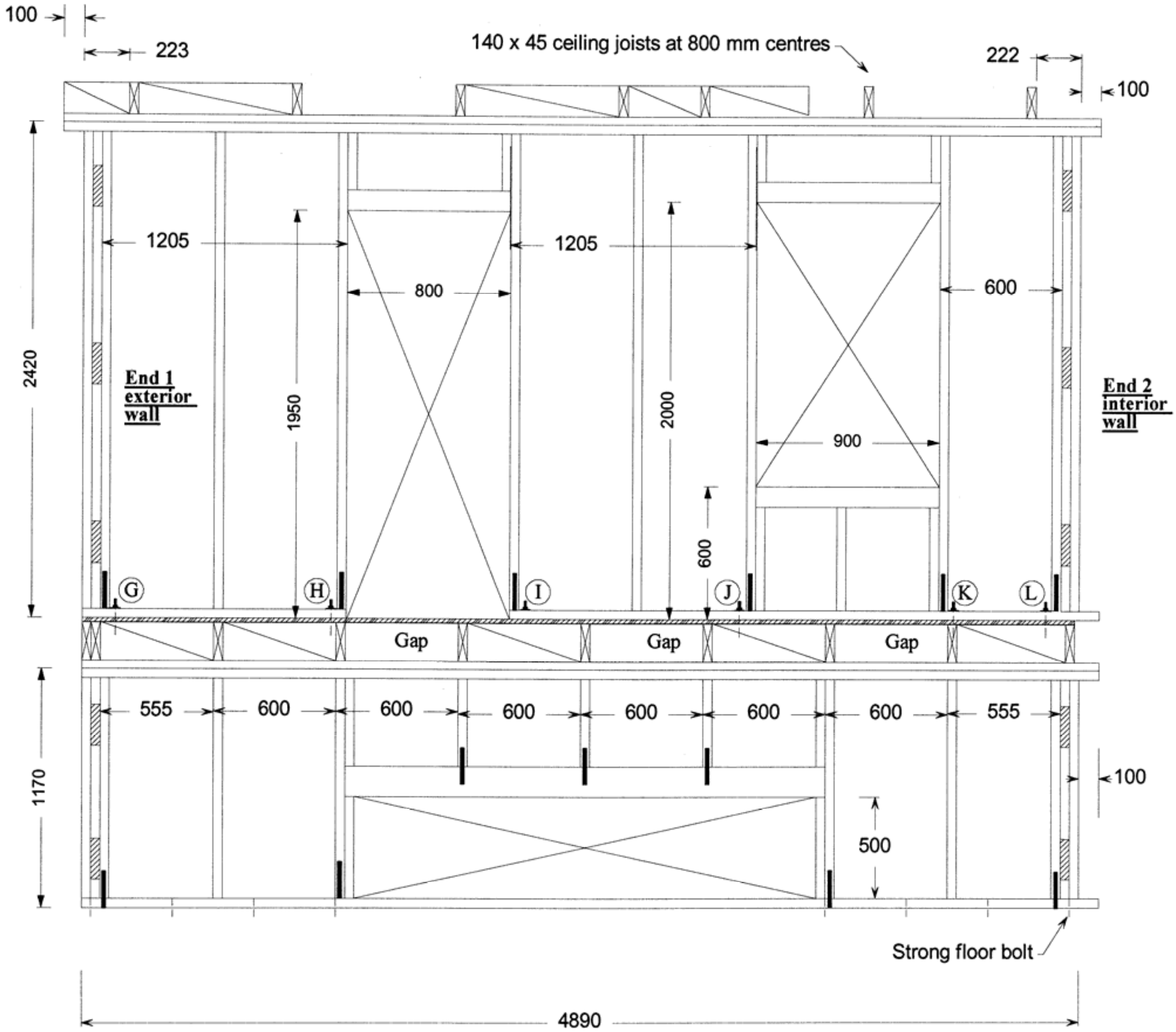


Figure 7. Side 1 framing

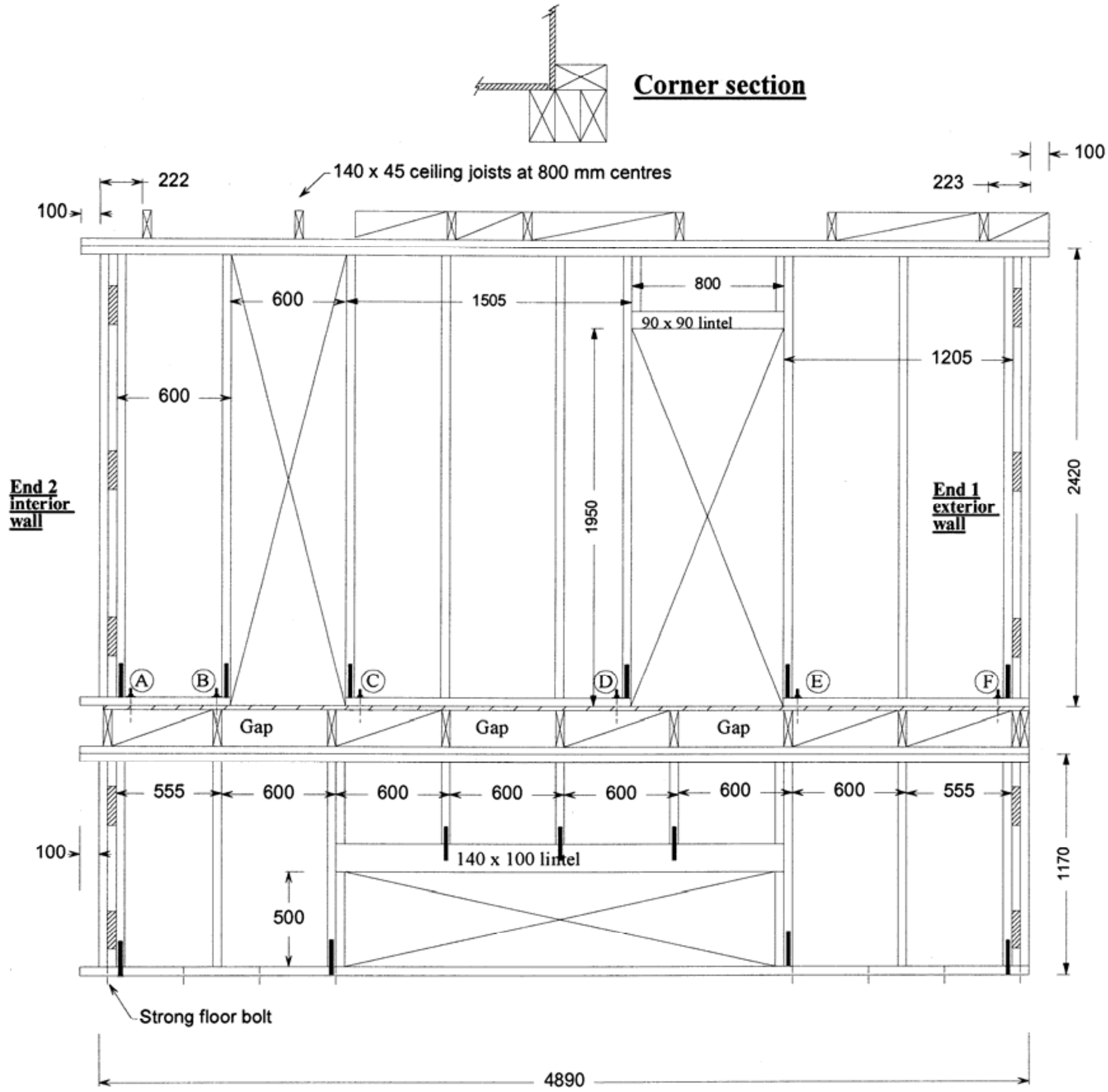


Figure 8. Side 2 framing

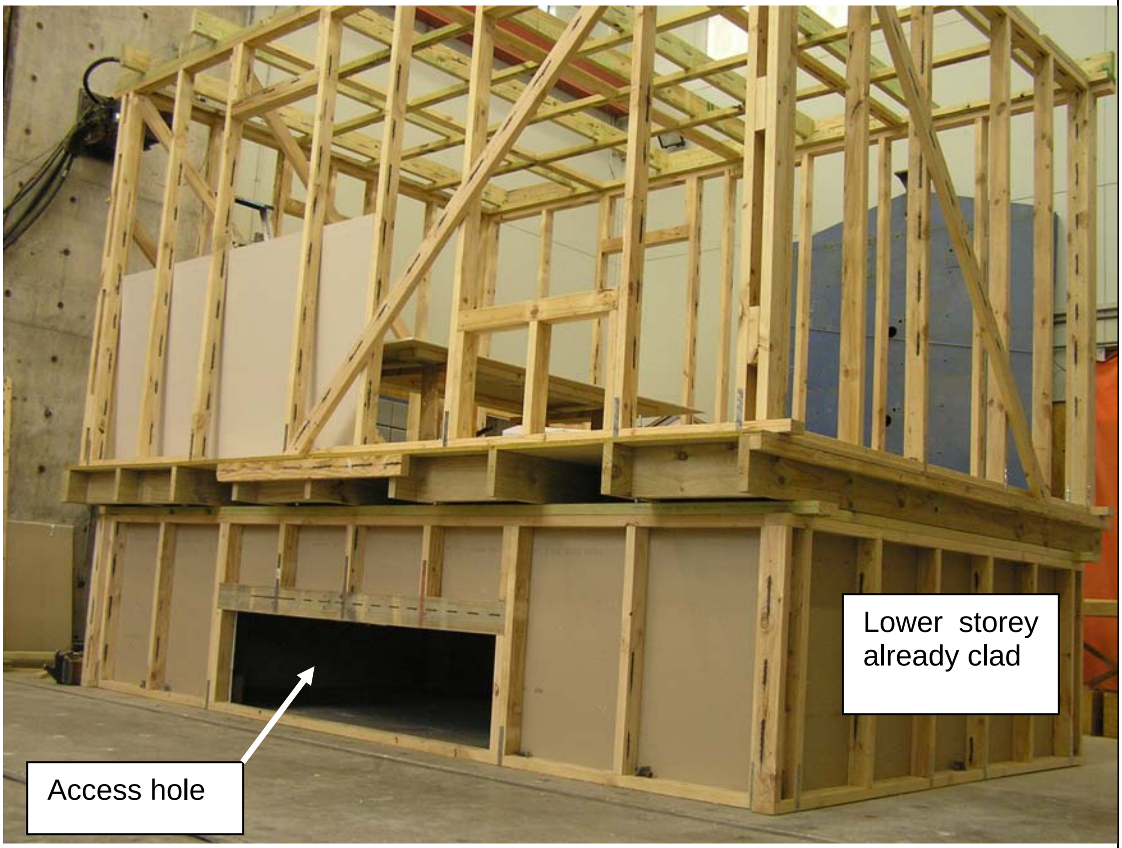


Figure 9. Photograph of framing taken during construction

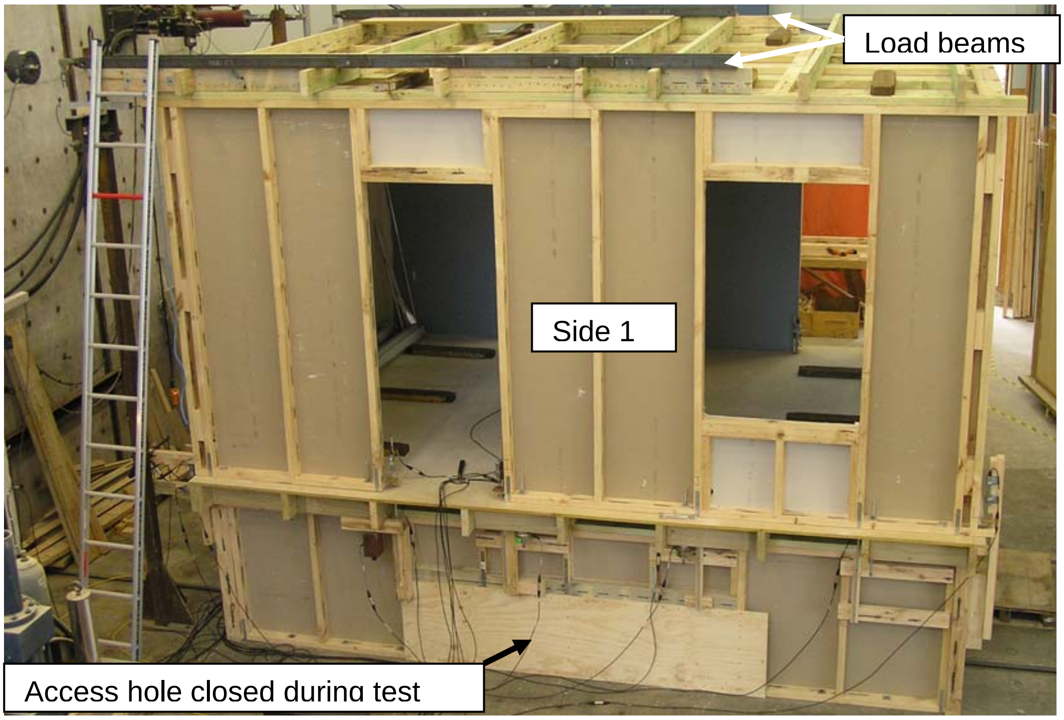


Figure 10. Photograph of test set-up for Test 1

Figure 11. Photograph of Room 2 interior plastered joints

Figure 12. Photograph of weights added to interior floor

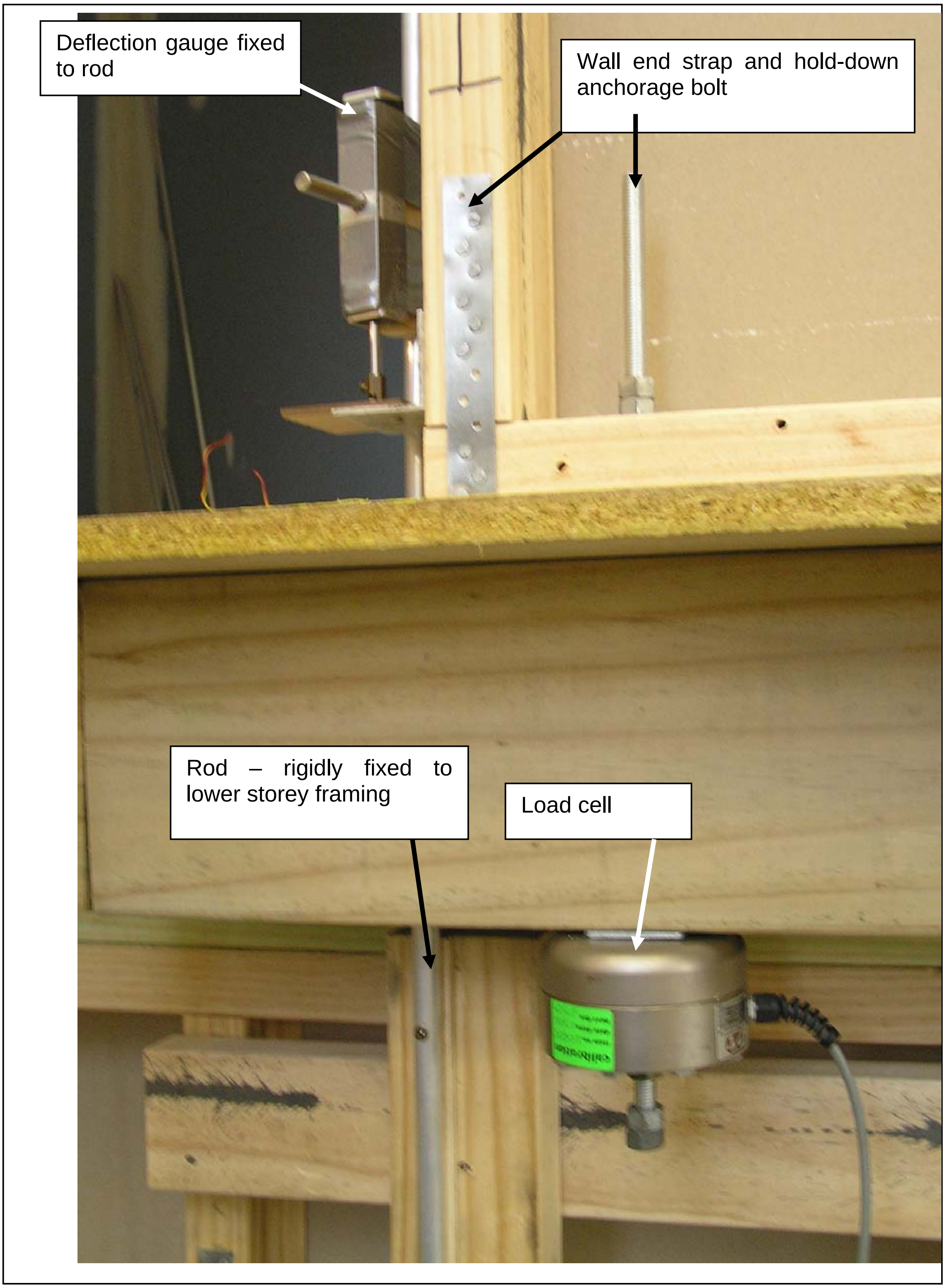


Figure 13. Measurement of inter-storey uplift and vertical load transfer

Figure 14. Measurement of inter-storey uplift

Figure 15. Measurement of floor joist uplift from the lower storey top plate

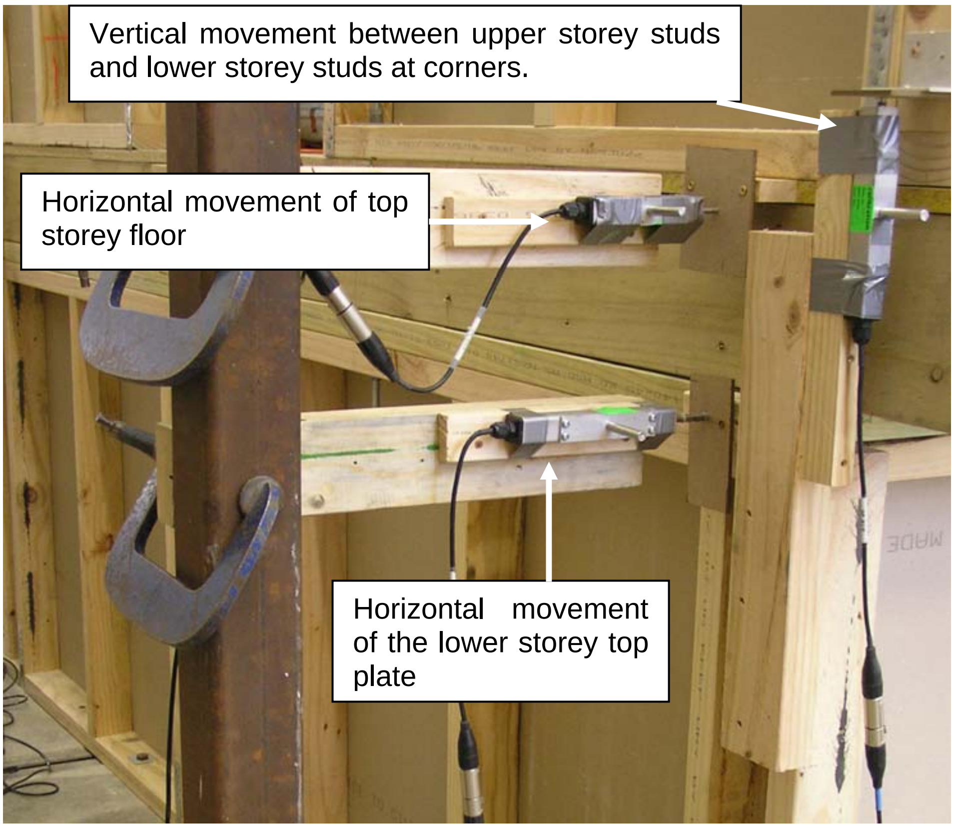


Figure 16. Measurement of horizontal deflections and inter-storey uplift of Side 1 in Stage I

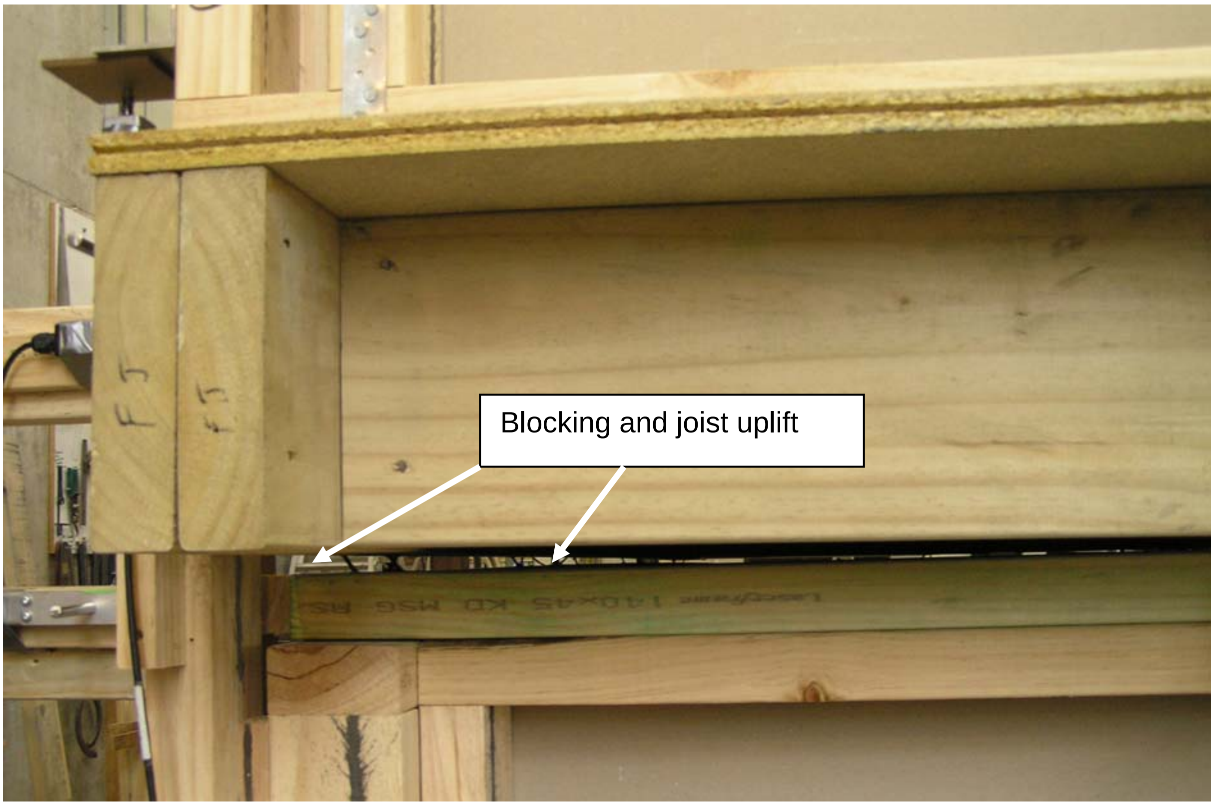


Figure 17. Boundary joist uplift from wall below at Side 1 End 1 due to actuator malfunction prior to Test 2 of Stage I

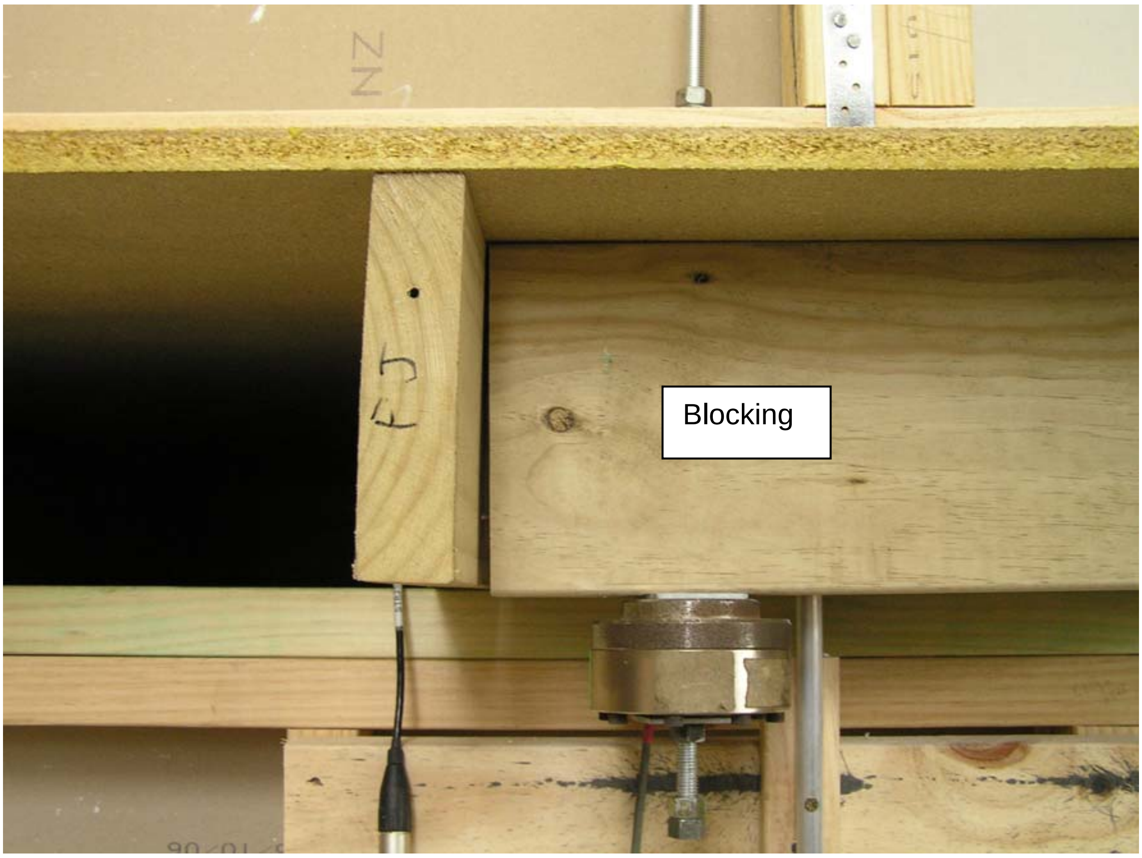


Figure 18. Distortion of blocking and joists from racking loads in Side 1 for Test 1 of Stage I at +10 mm deflection

Figure 19. Distortion of floor and blocking due to racking loads in Side 1 for Test 3 of Stage I at +25 mm deflection

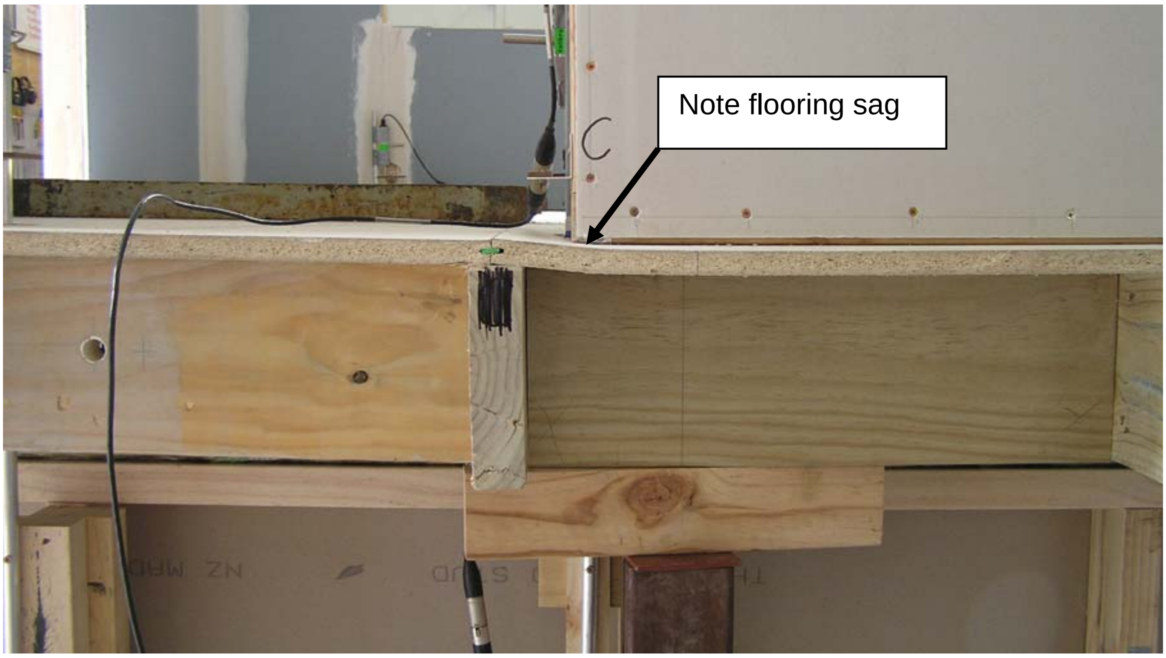


Figure 20. End of bracing element compressing floor below in Side 2 for Test 3 of Stage I at +40 mm deflection

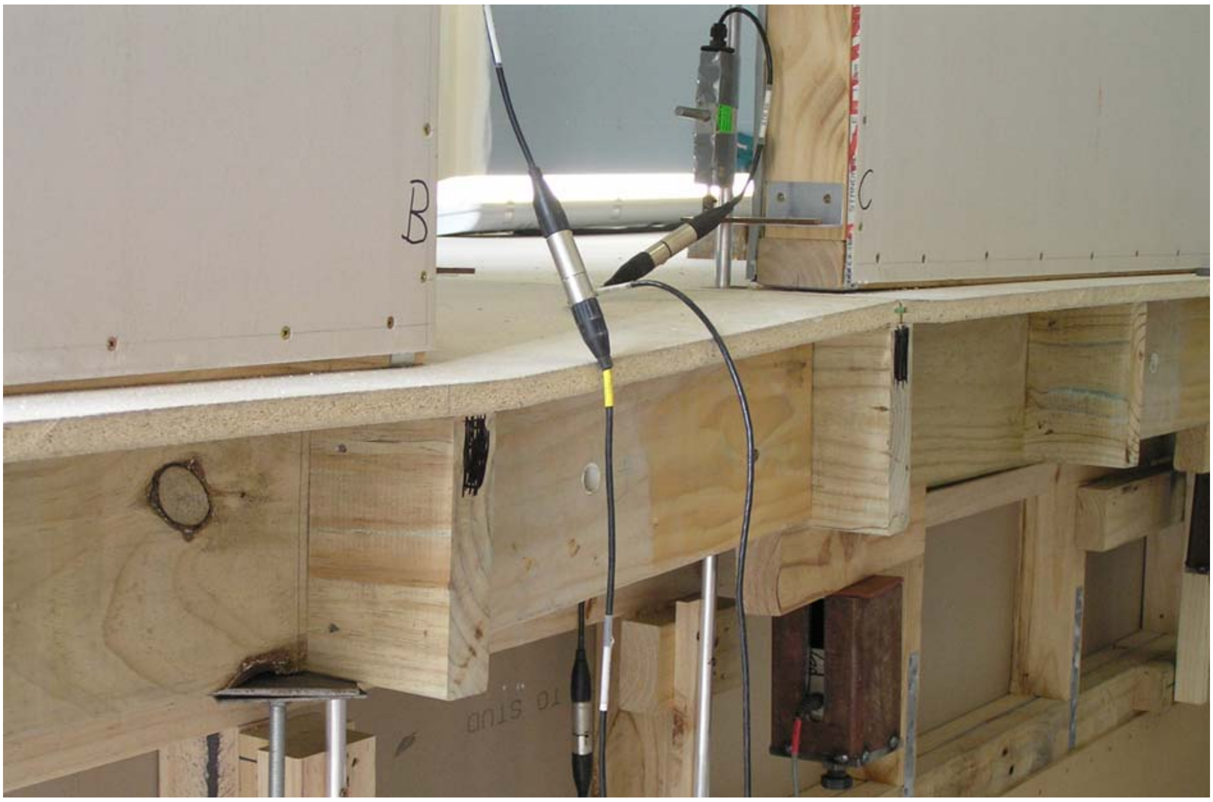


Figure 21. Distortion from both upwards and downwards loads in Side 2 for Test 3 of Stage I at -40 mm deflection

Figure 22. Joist uplift from wall below in Test 3 of Stage I at 40 mm deflection

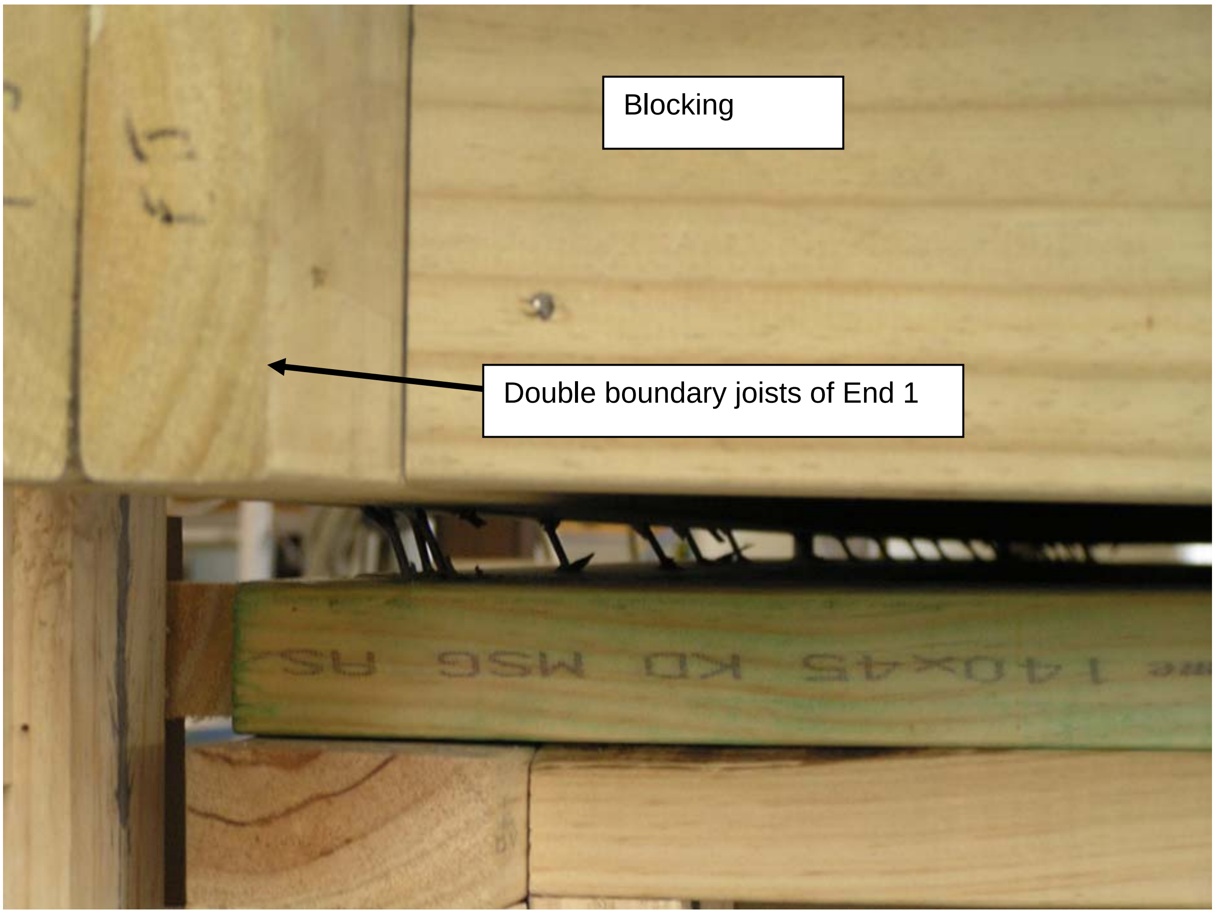


Figure 23. Close-up view of boundary joist uplift from wall below in Test 3 of Stage I at +40 mm deflection

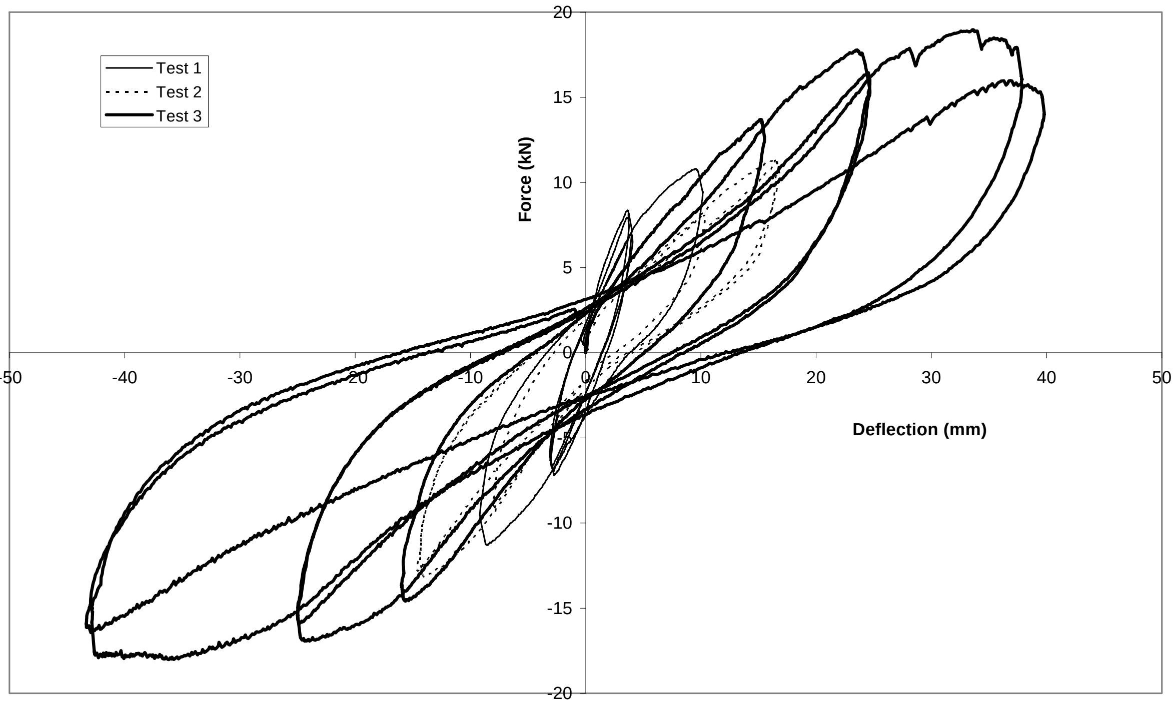


Figure 24. Total racking force versus average side wall deflections for Stage I

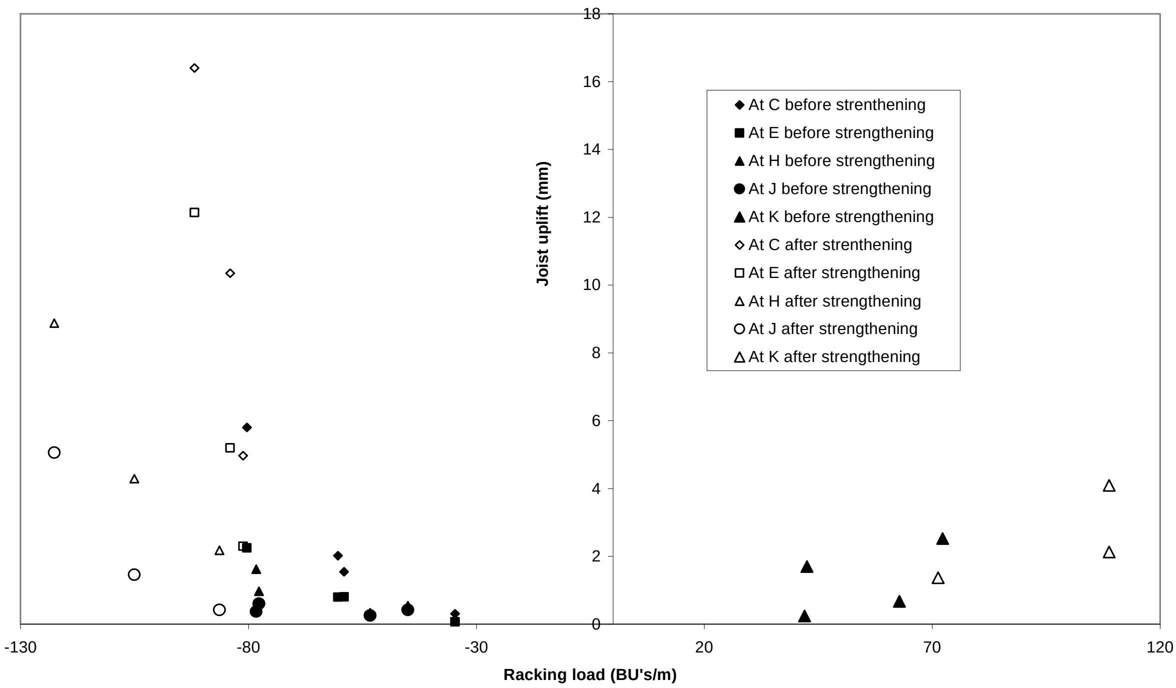


Figure 25. Joist uplift versus racking load on each side wall for Stage I

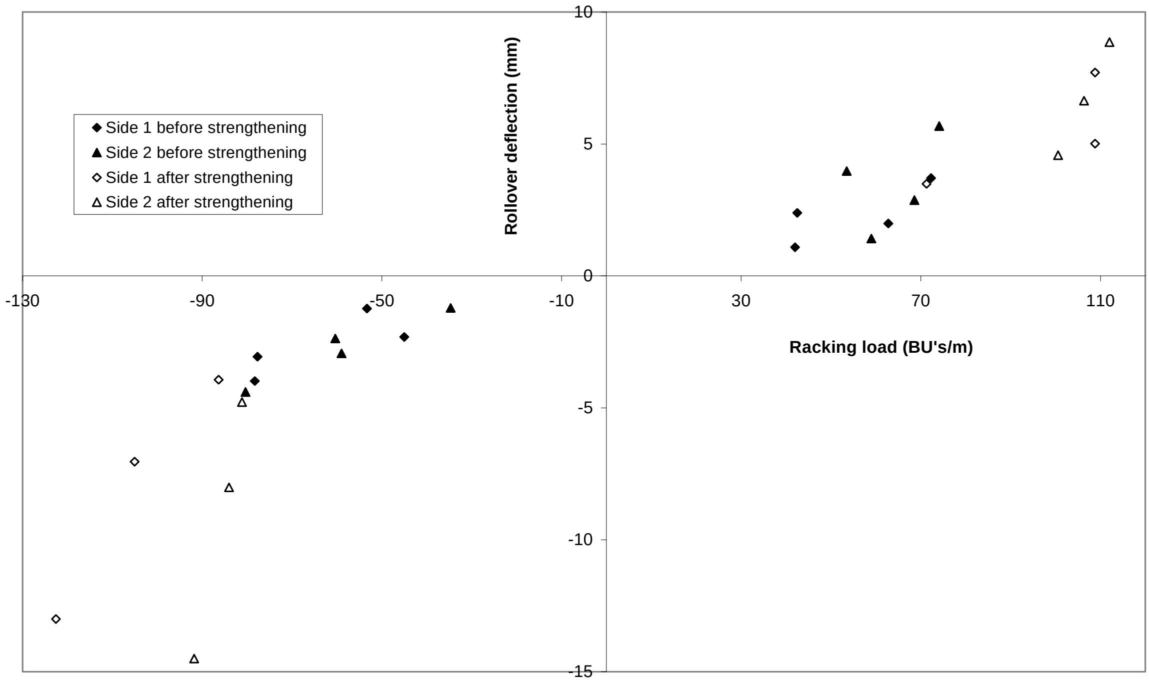


Figure 26. Joist roll-over deflections versus racking load on each side wall for Stage I

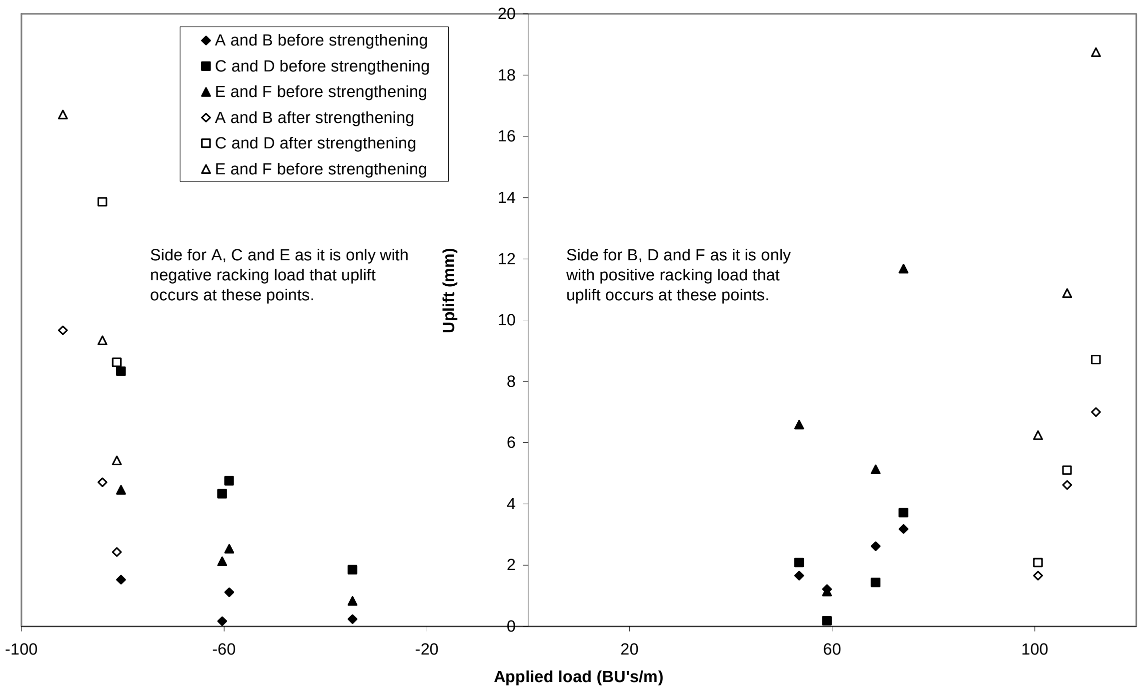


Figure 27. Inter-storey uplift movement at A, B, C, D, E and F measured during Stage I

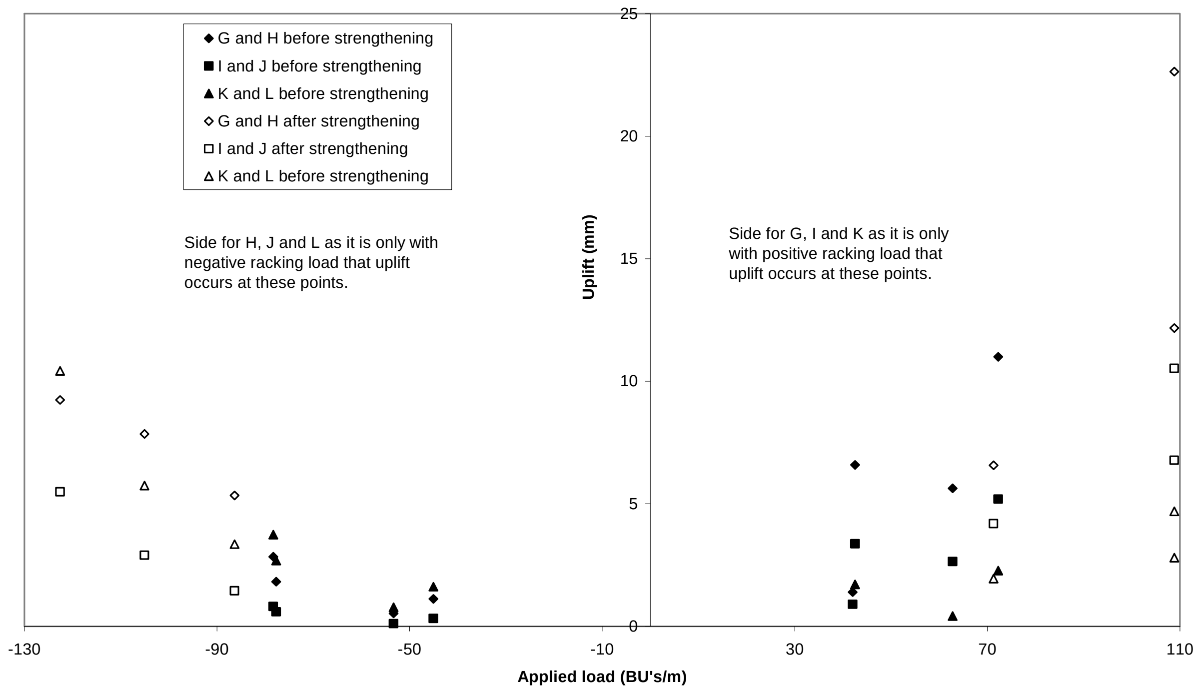


Figure 28. Inter-storey uplift movement at G, H, I, J, K and L measured during Stage I

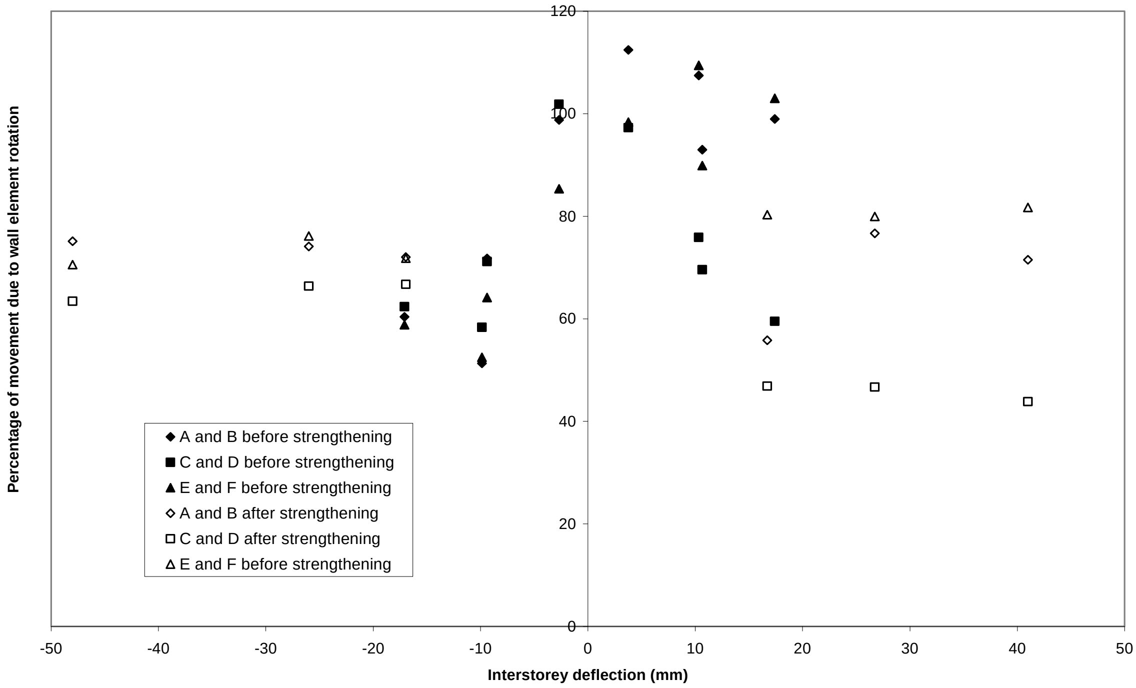


Figure 29. Percentage of wall horizontal movement due to vertical movement at A, B, C, D, E and F measured during Stage I

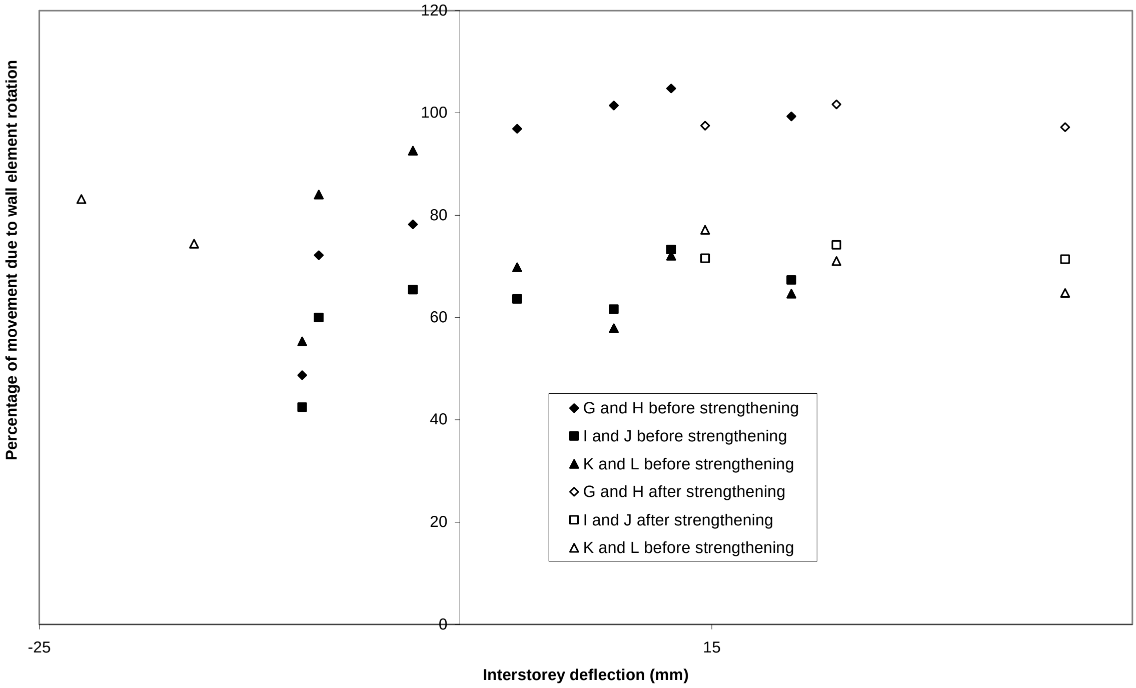


Figure 30. Percentage of wall horizontal movement due to vertical movement at G, H, I, J, K and L measured during Stage I

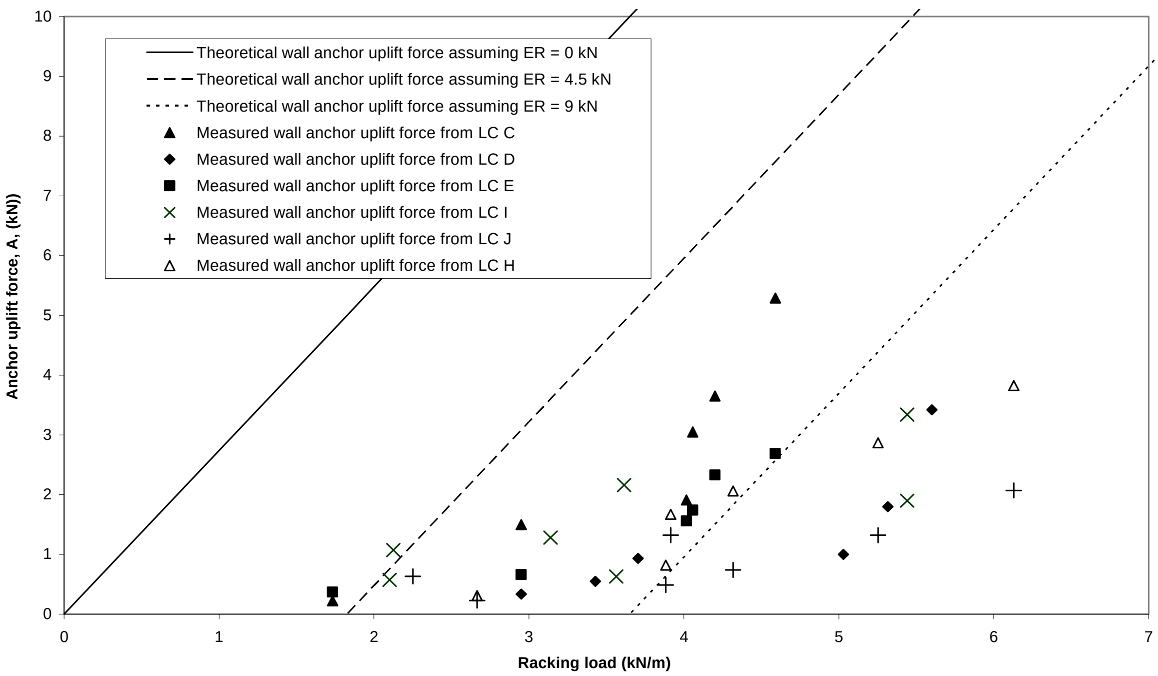


Figure 31. Comparison of theoretical and measured uplift forces on bracing walls for Stage I

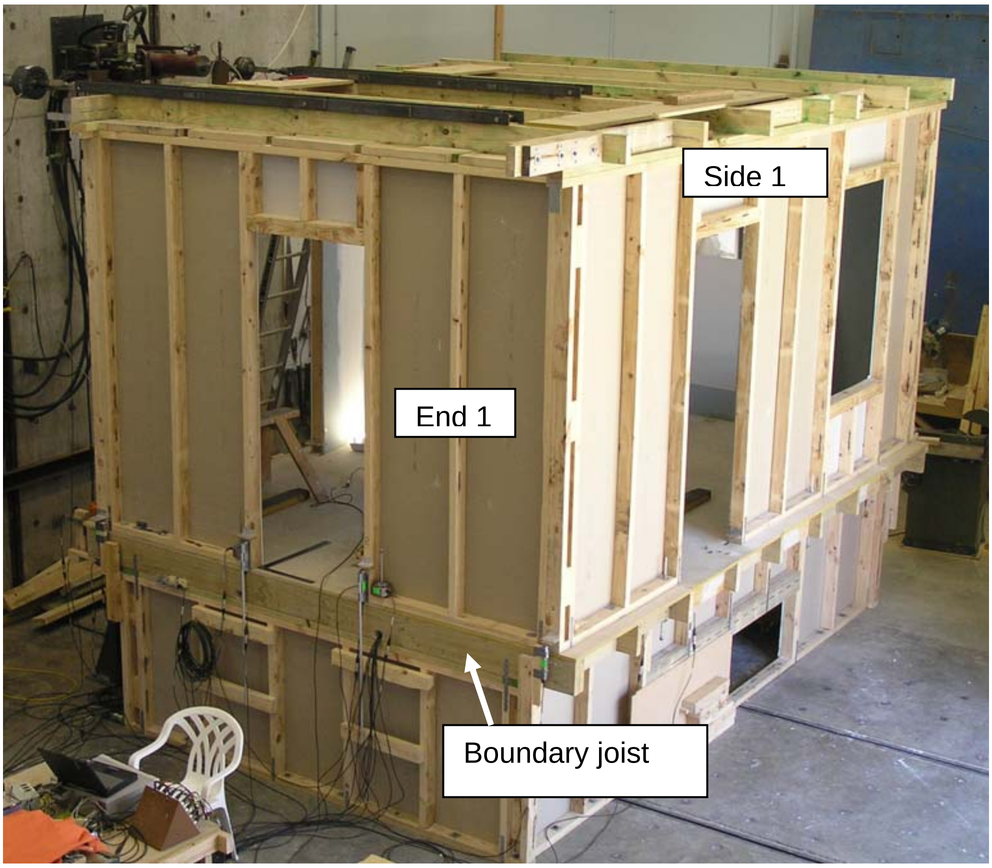


Figure 32. Test set-up for Stage II testing

Figure 33. Internal wall used for Stage II testing

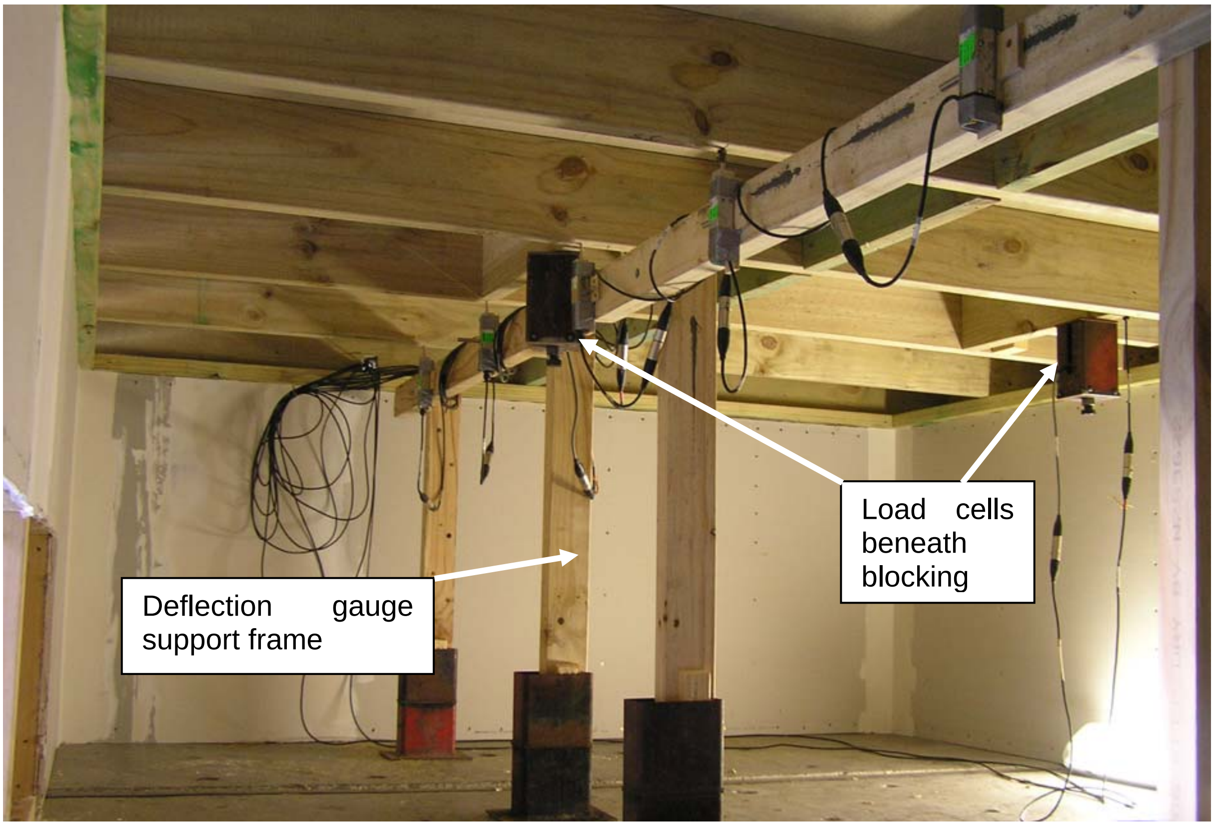


Figure 34. View under floor for Stage II testing

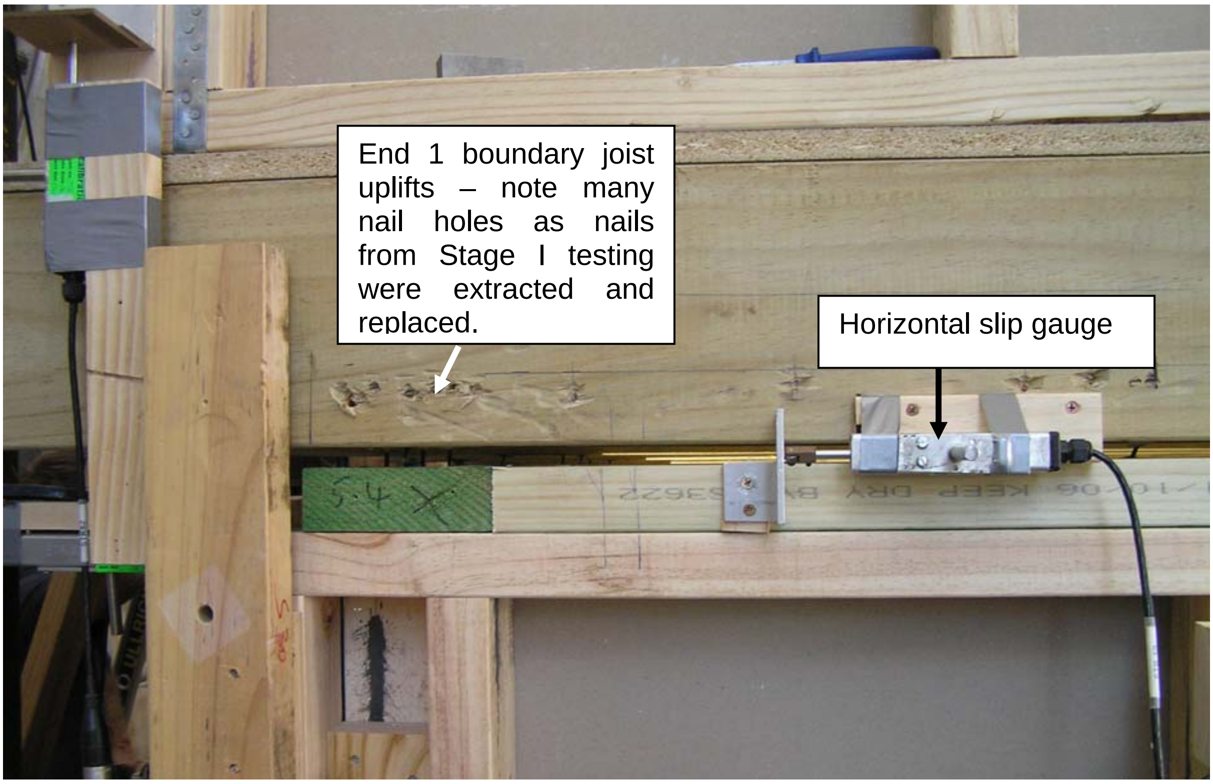


Figure 35. End 1 boundary joist uplift for Stage II testing

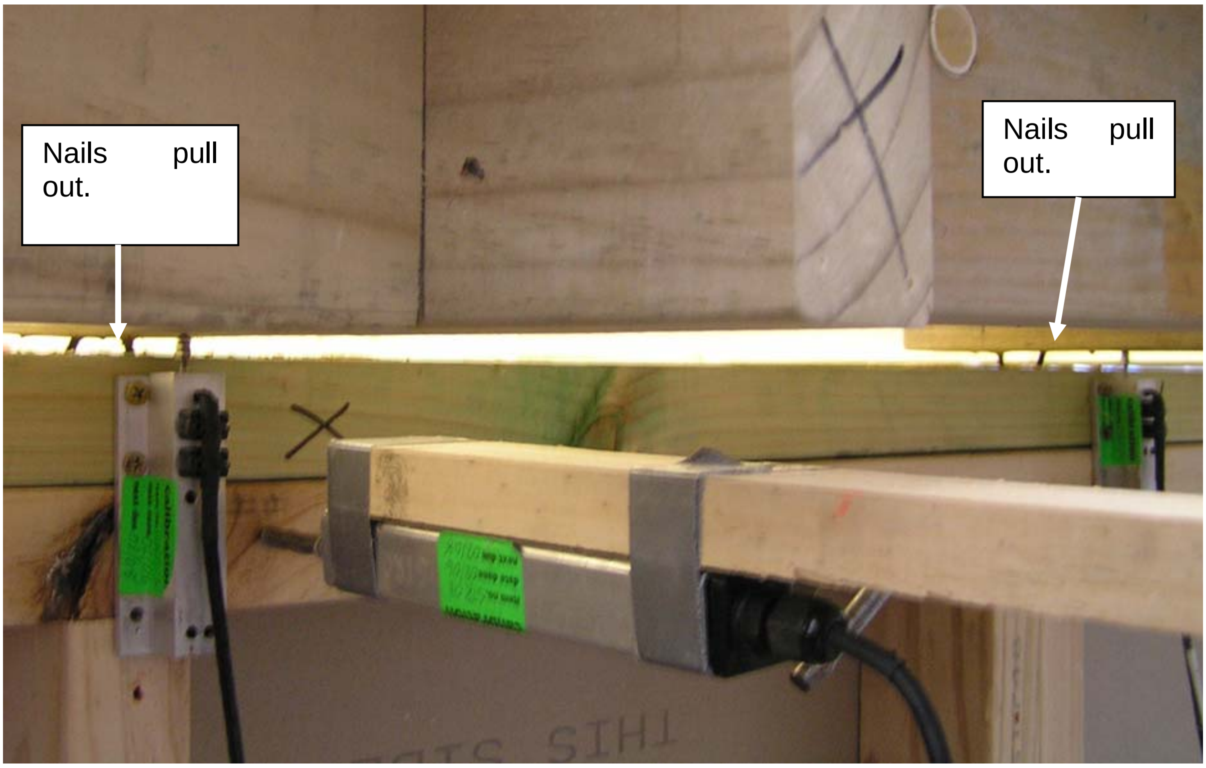


Figure 36. Joist uplift along Side 2 at joist ends for Stage II testing

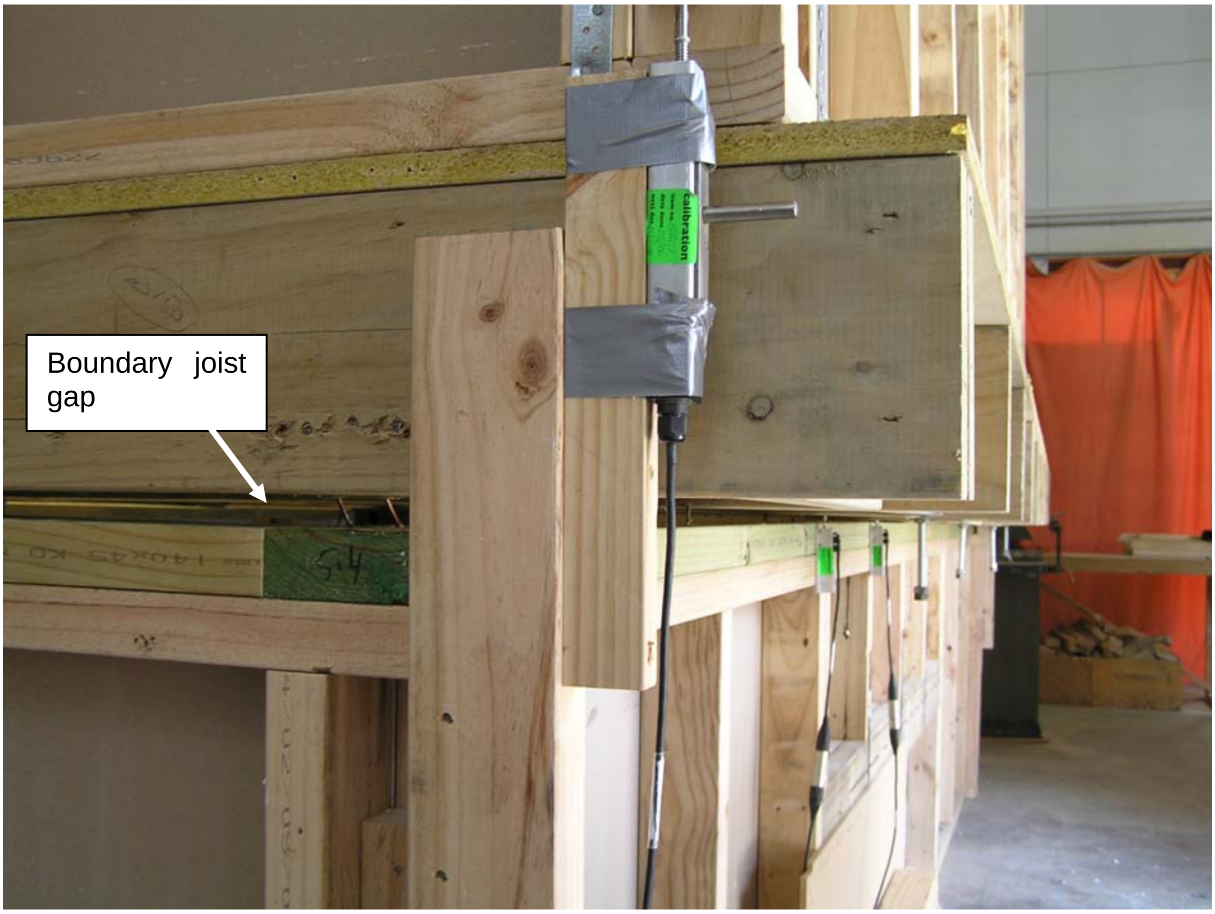


Figure 37. Joist uplift at corner for Stage II testing

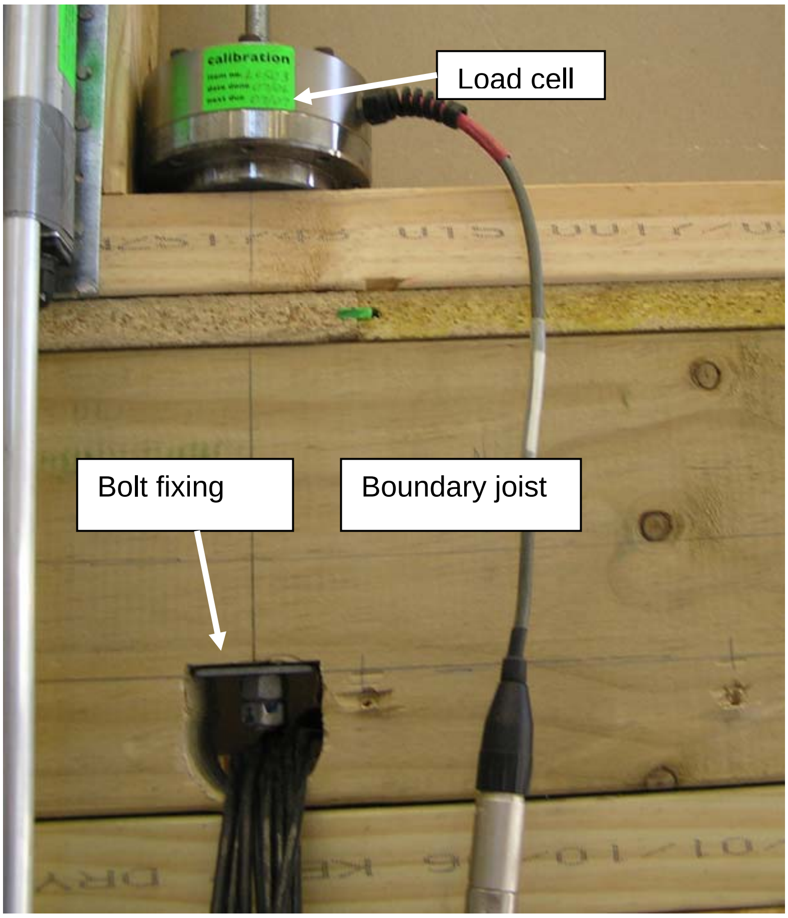


Figure 38. Measurement of axial load transferred to End 1 boundary joist for Stage II testing

Figure 39. Flooring breakage due to blocking vertical movement at the internal wall for Stage II testing

Figure 40. Separation of internal wall top corner from ceiling in Stage II testing

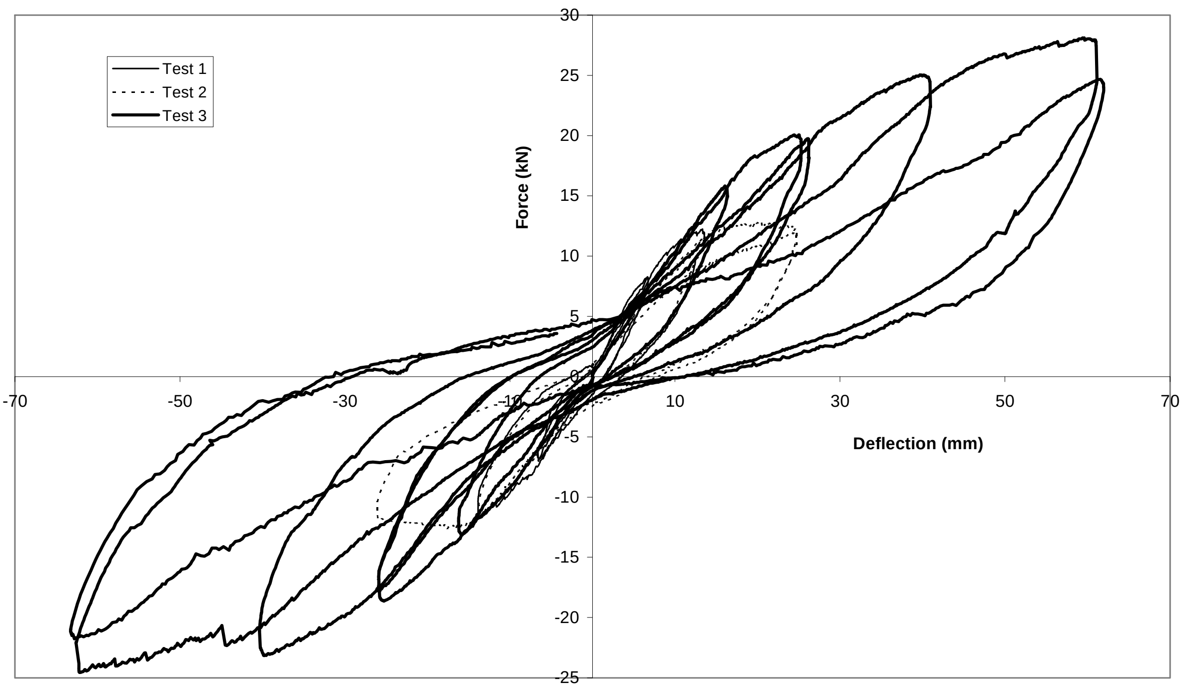


Figure 41. Total racking force versus average deflection for Stage II

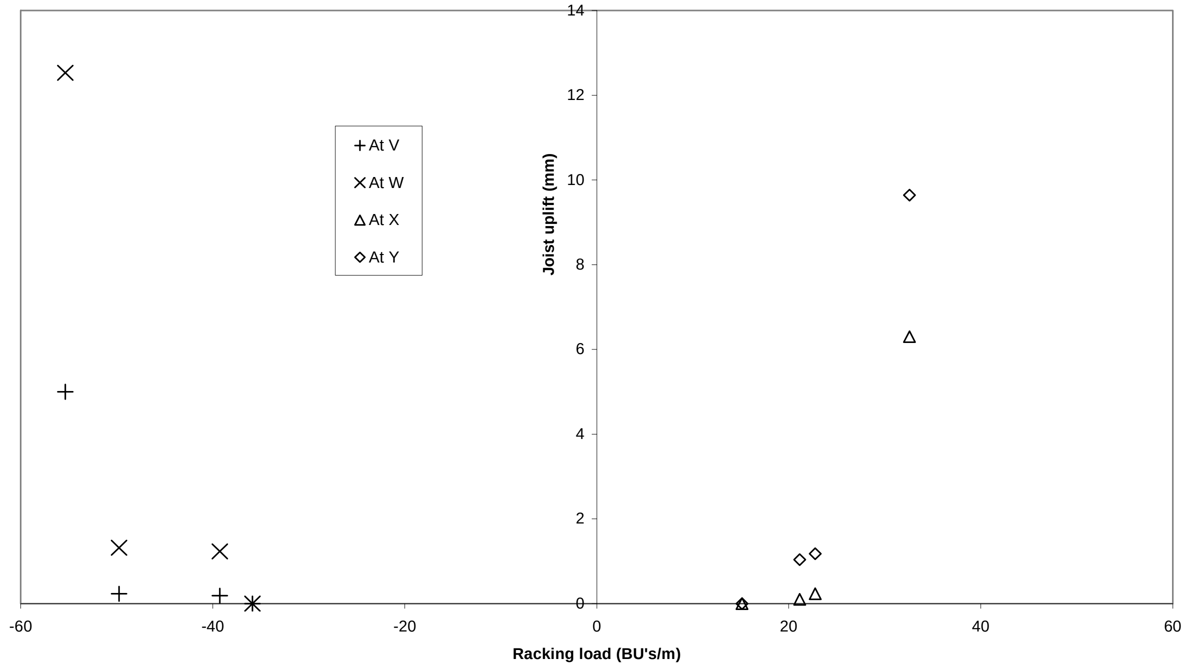


Figure 42. Joist uplift from wall below versus racking load near the internal wall for Stage II

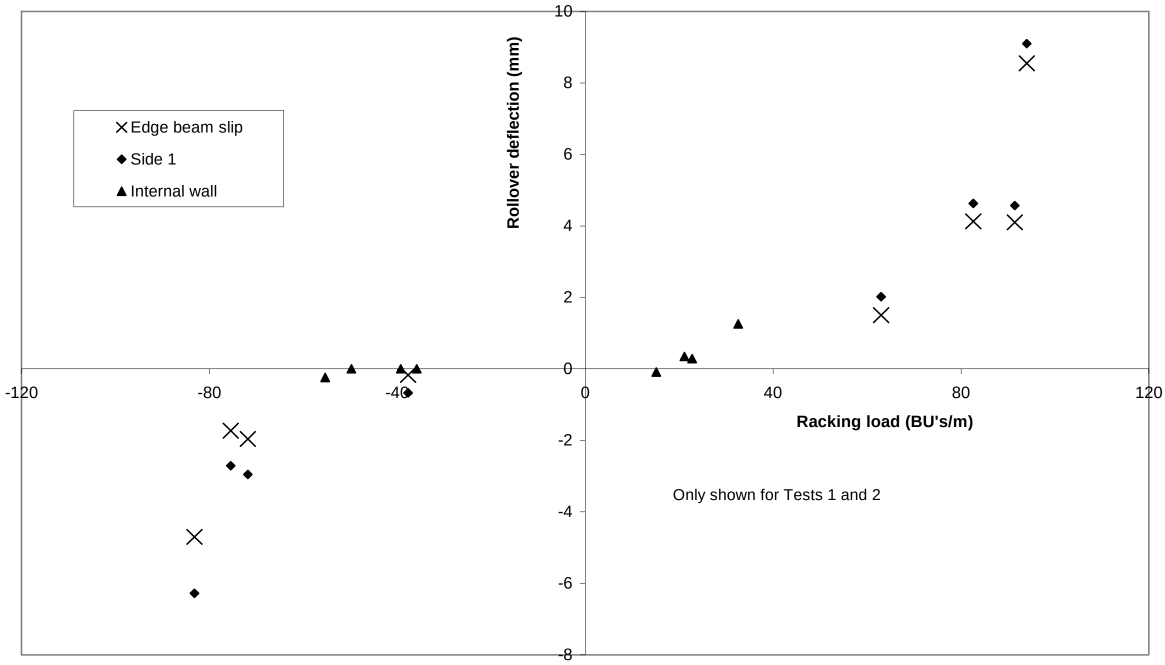


Figure 43. Joist roll-over versus racking load for Stage II

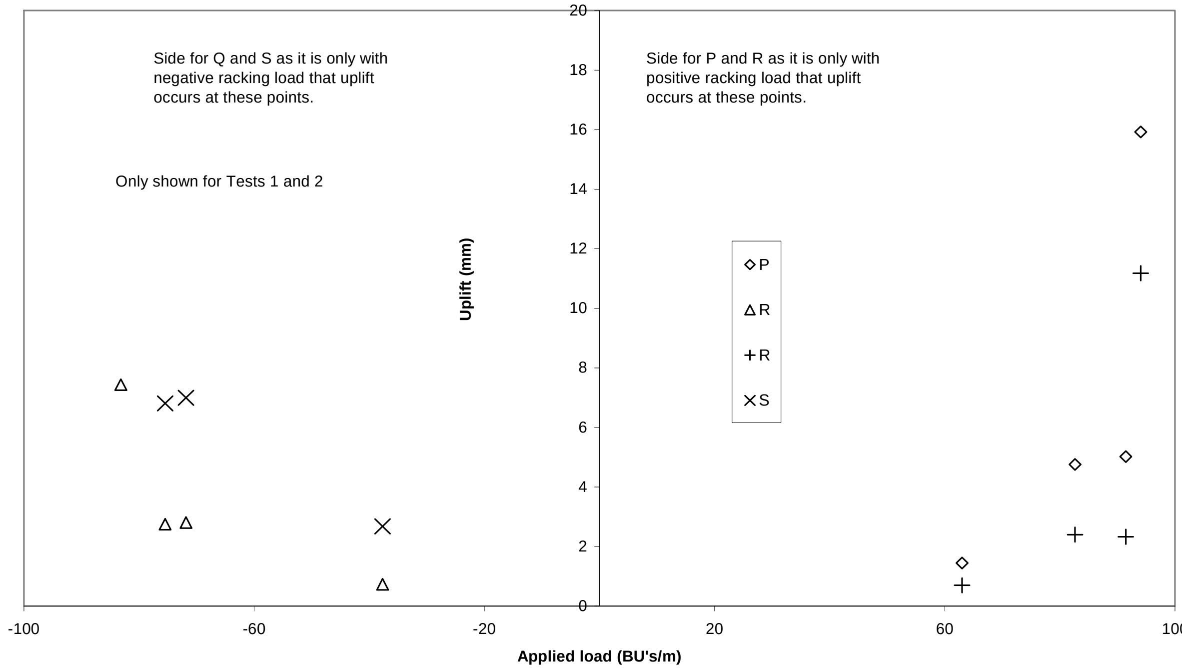


Figure 44. Uplift of upper storey studs from lower storey studs versus racking load at End 1 for Stage II

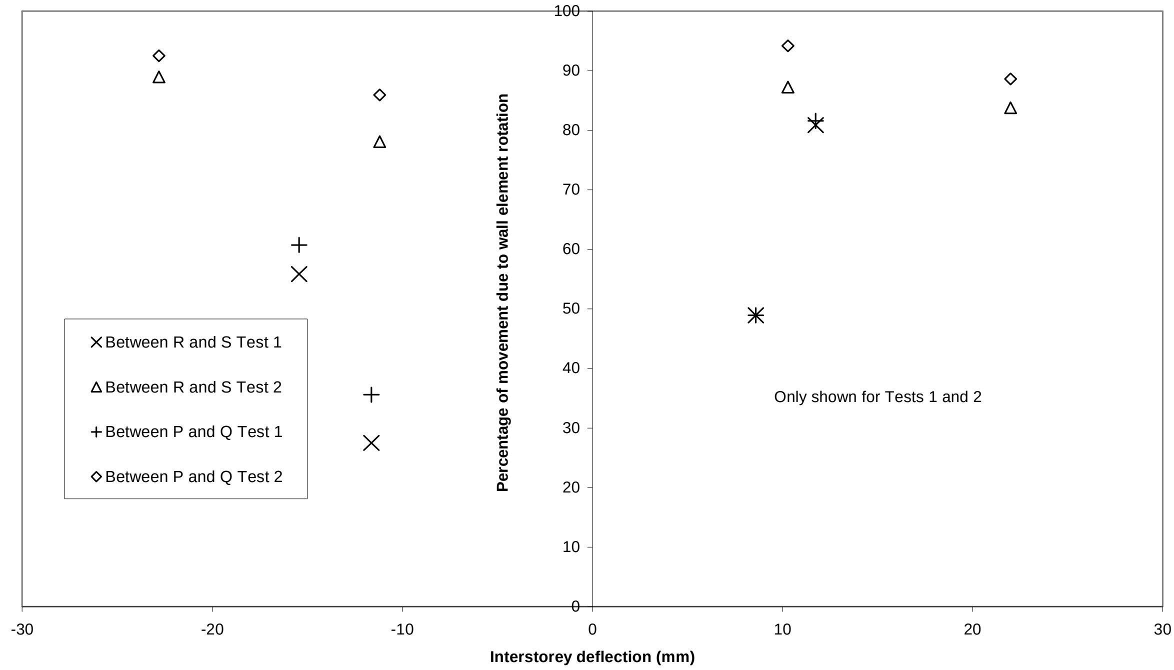


Figure 45. Percentage of deflection due to 'rocking' versus racking load at End 1 for Stage II

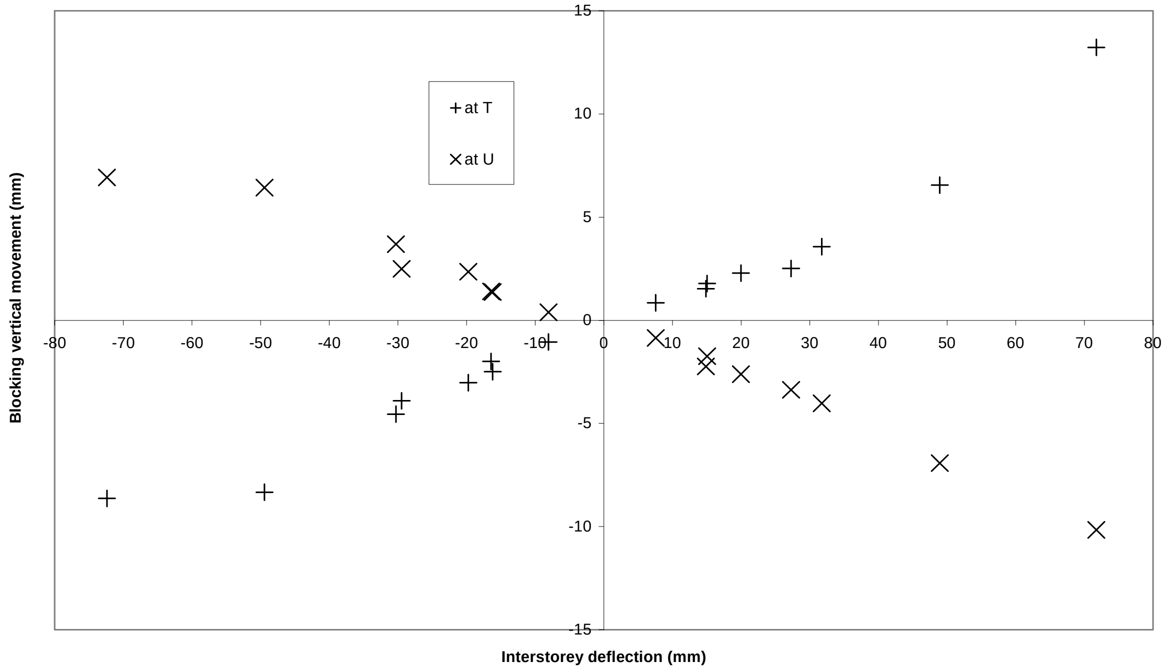


Figure 46. Blocking vertical movement versus wall deflection at the internal wall for Stage II

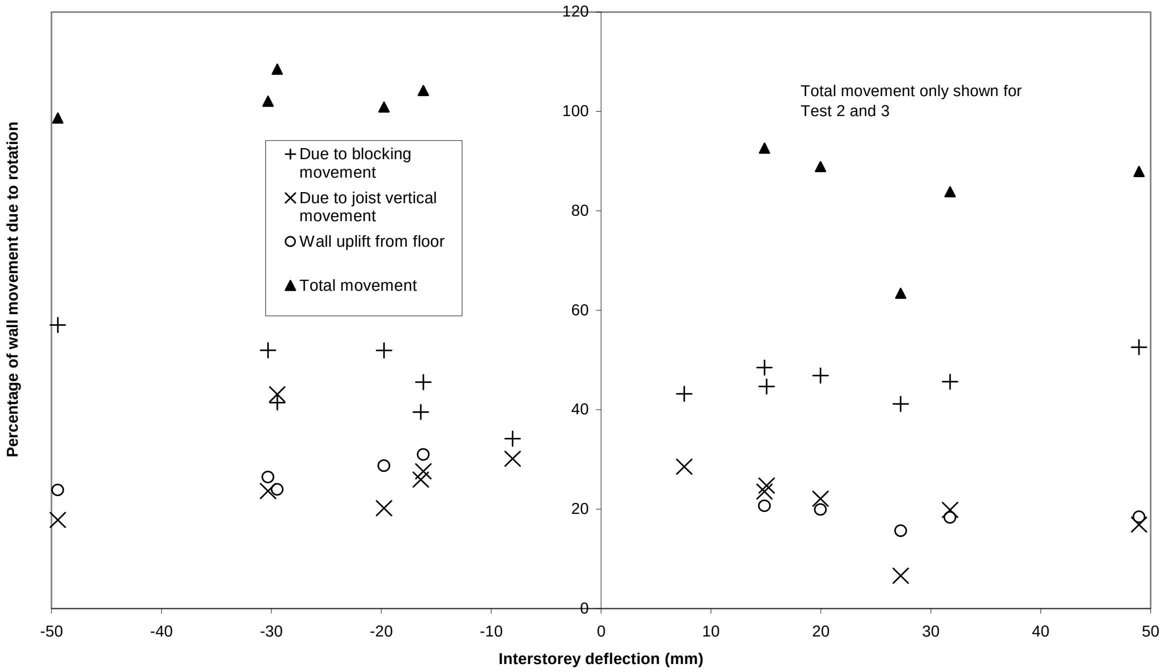


Figure 47. Percentage of deflection due to ‘rocking’ versus racking load at the internal wall for Stage II

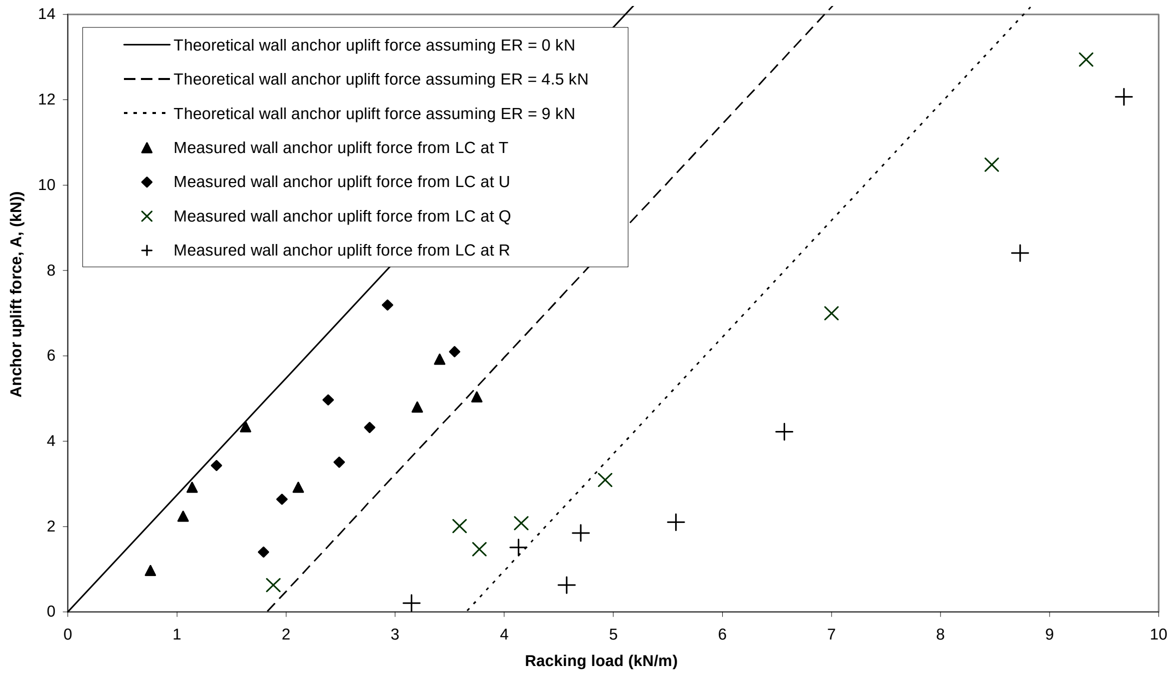


Figure 48. Comparison of theoretical and measured uplift forces on bracing walls for Stage II

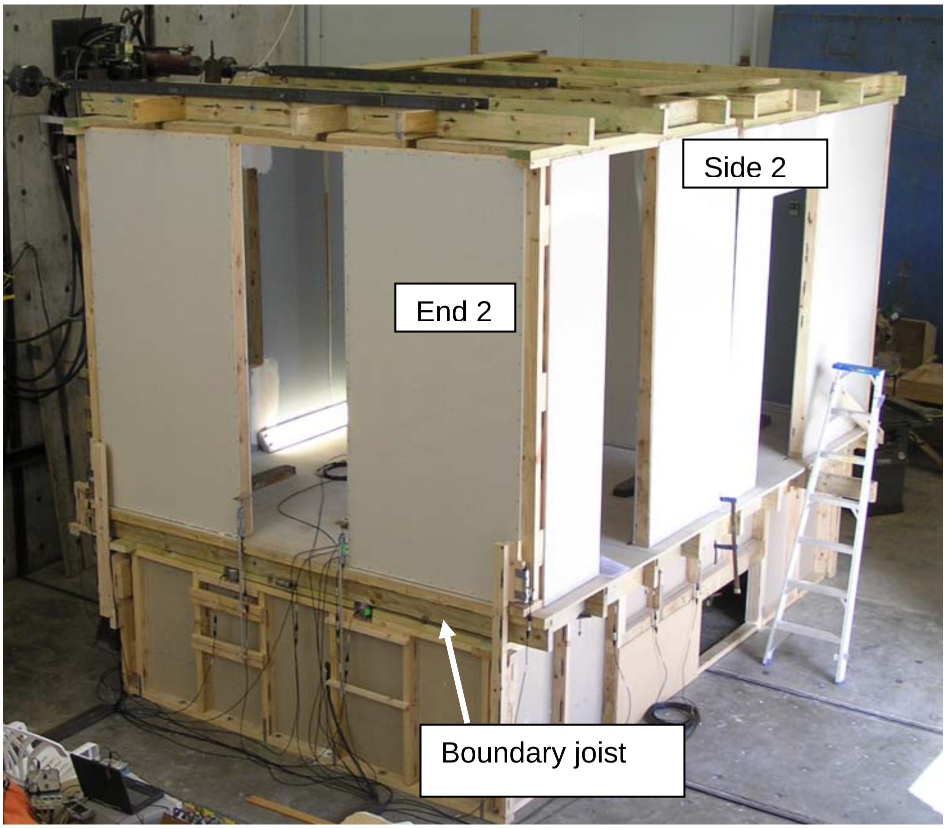


Figure 49. Test set-up for Stage III testing

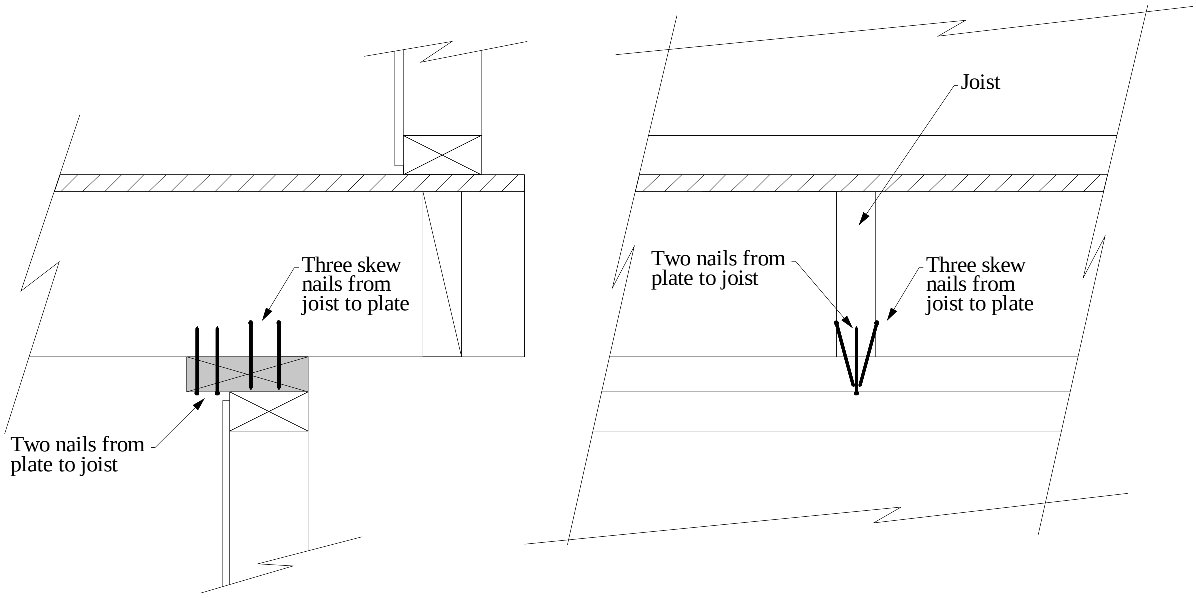


Figure 50. Nailing of internal floor joists to top plate supports used in Stage III testing

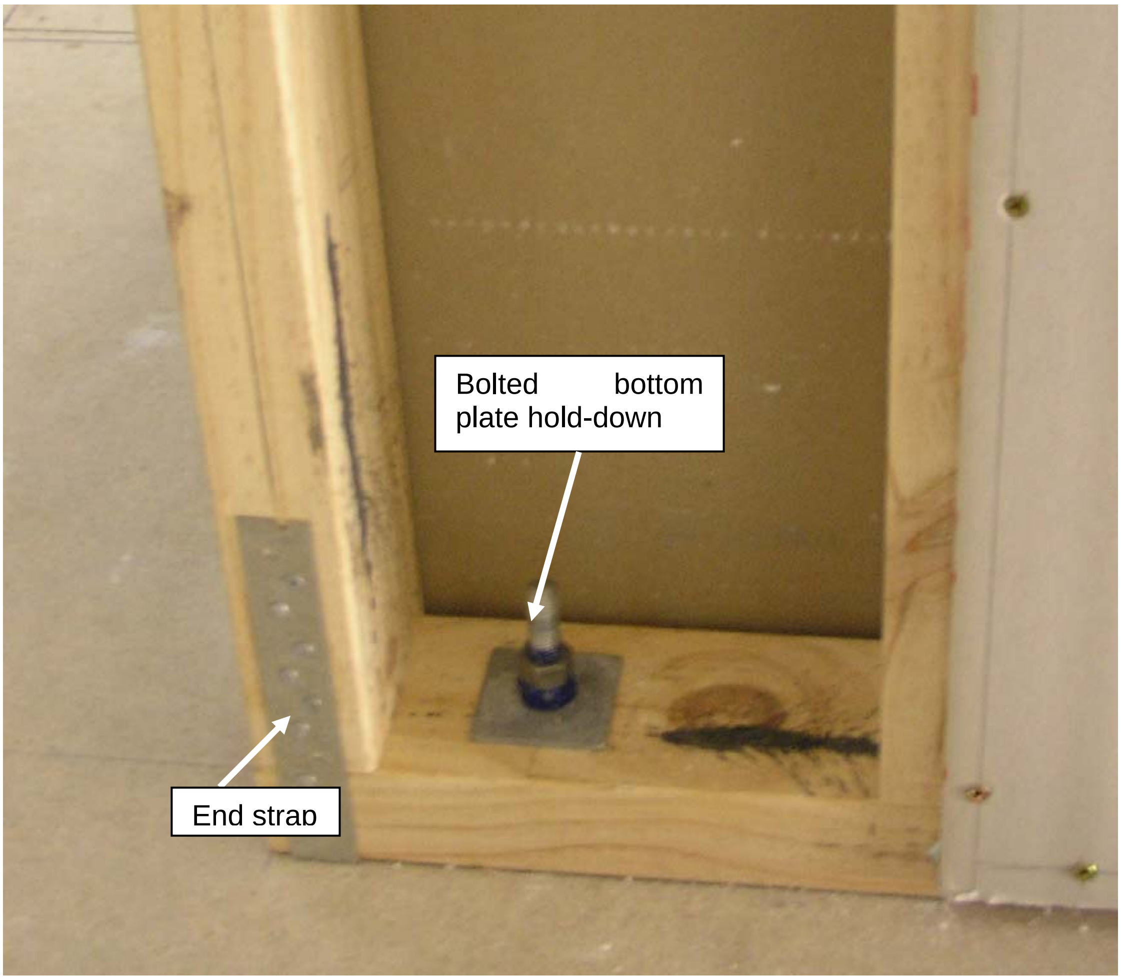
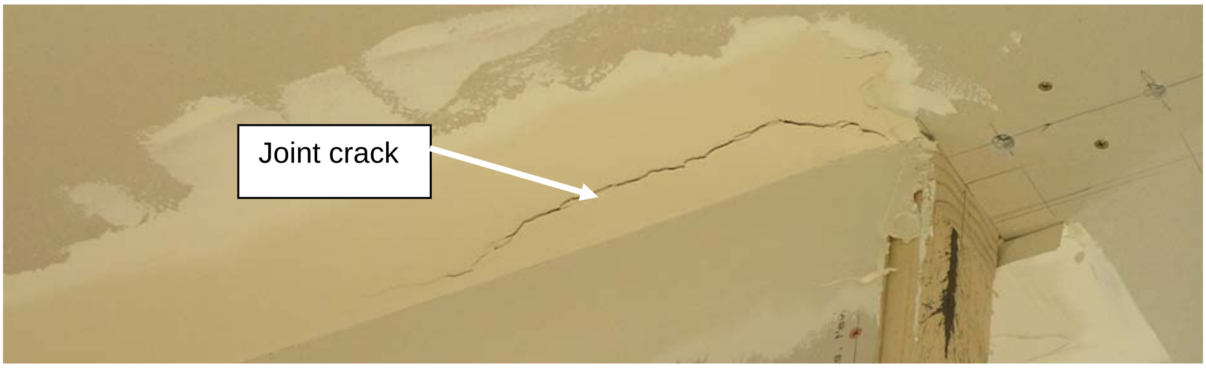


Figure 51. Internal wall used for Stage III testing



(a) Cracking from Test 1

(b) Cracking from Test 2

Figure 52. Ceiling-to-wall plasterboard joint cracks in Stage III

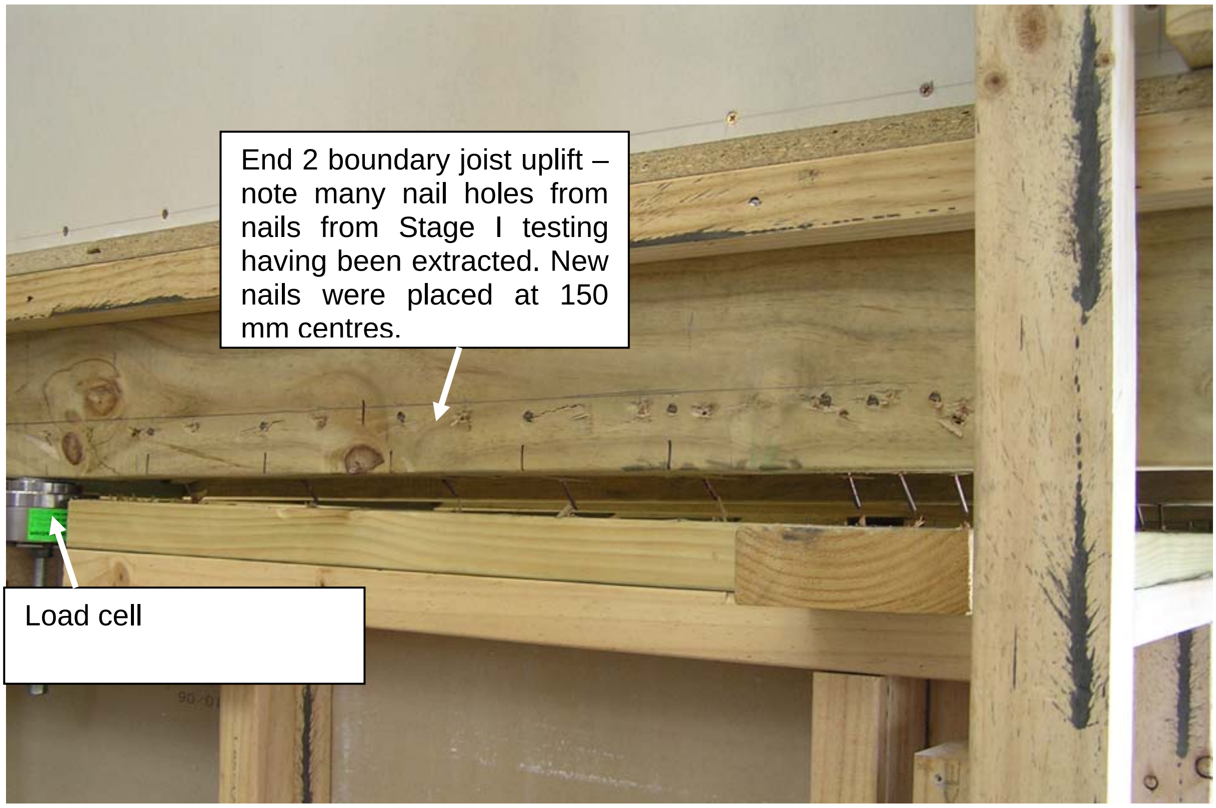


Figure 53. End 2 boundary joist uplift in latter stages of Stage III Test 1

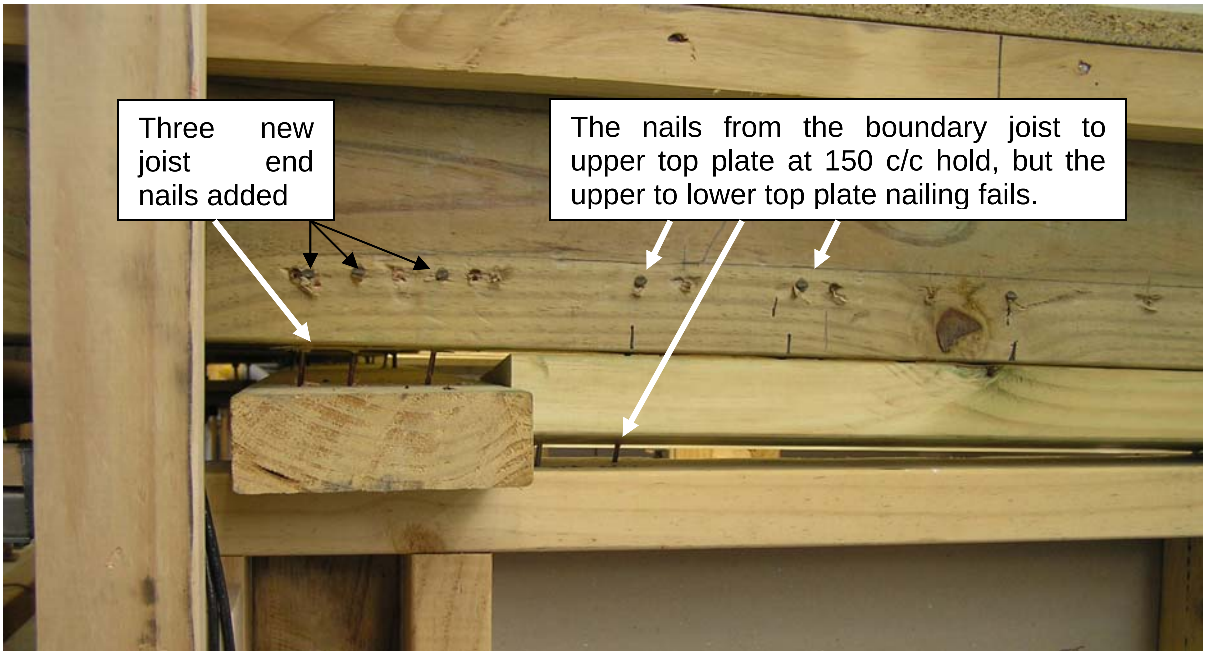


Figure 54. End 2 boundary joist uplift at other end in latter stages of Stage III Test 1

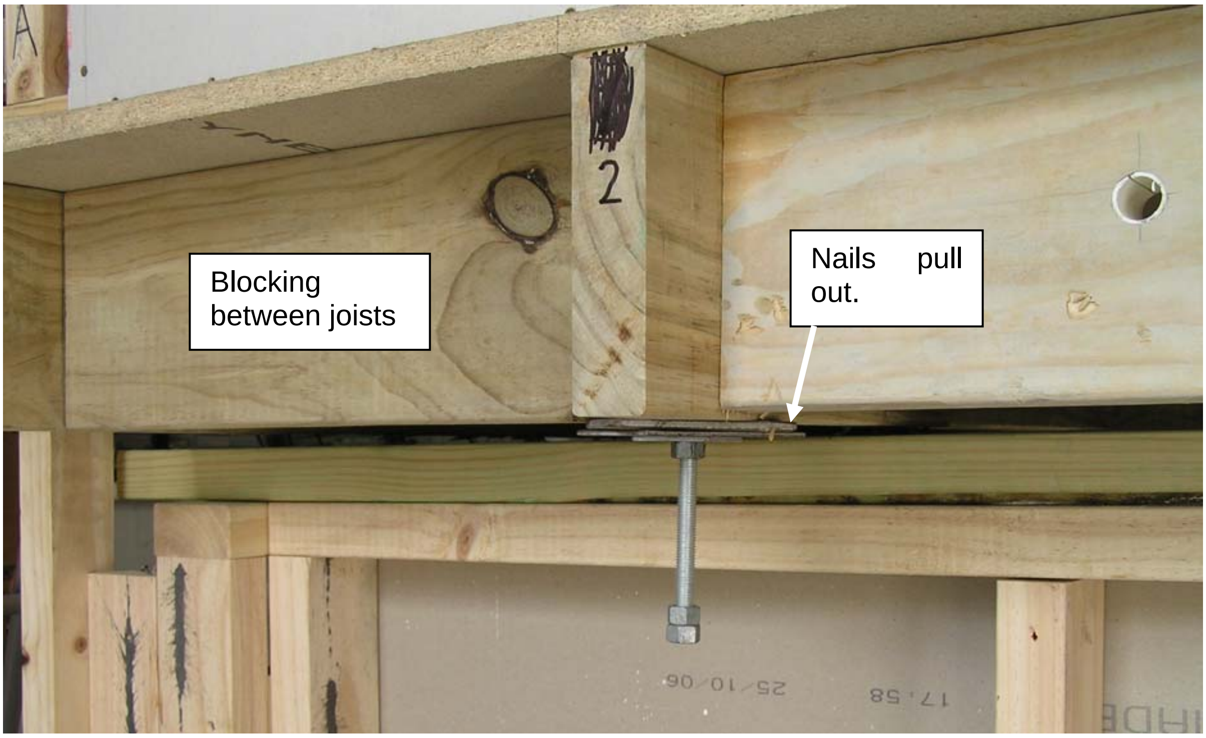


Figure 55. Joist uplift on side walls during Stage III testing

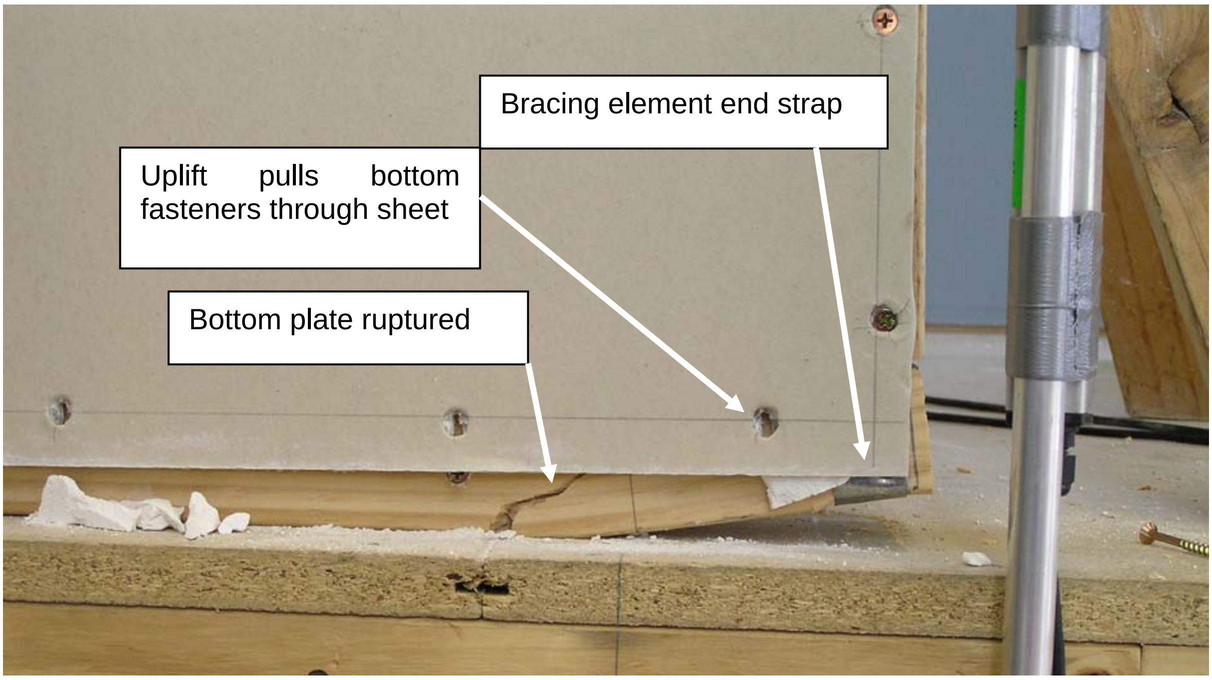


Figure 56. Bottom plate failure late in Test 2 of Stage II at End 2

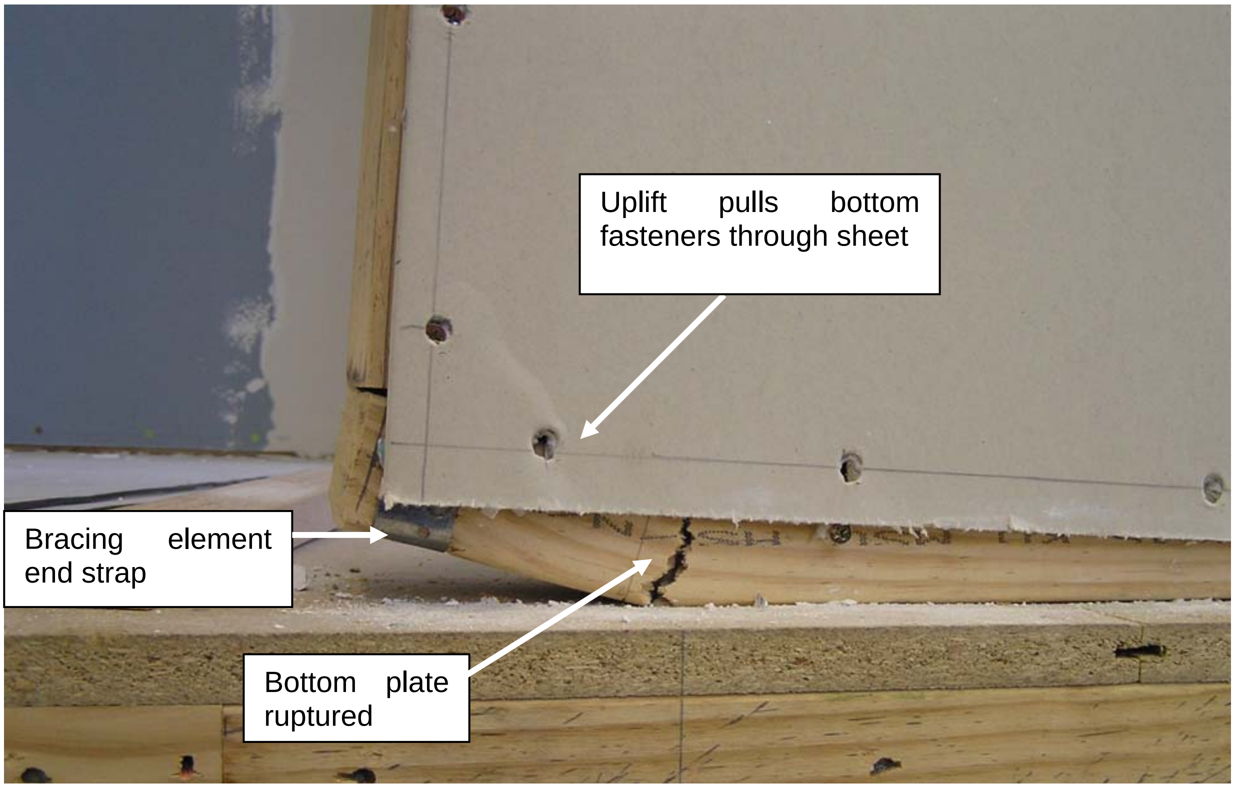


Figure 57. A second bottom plate failure late in Test 2 of Stage II at End 2

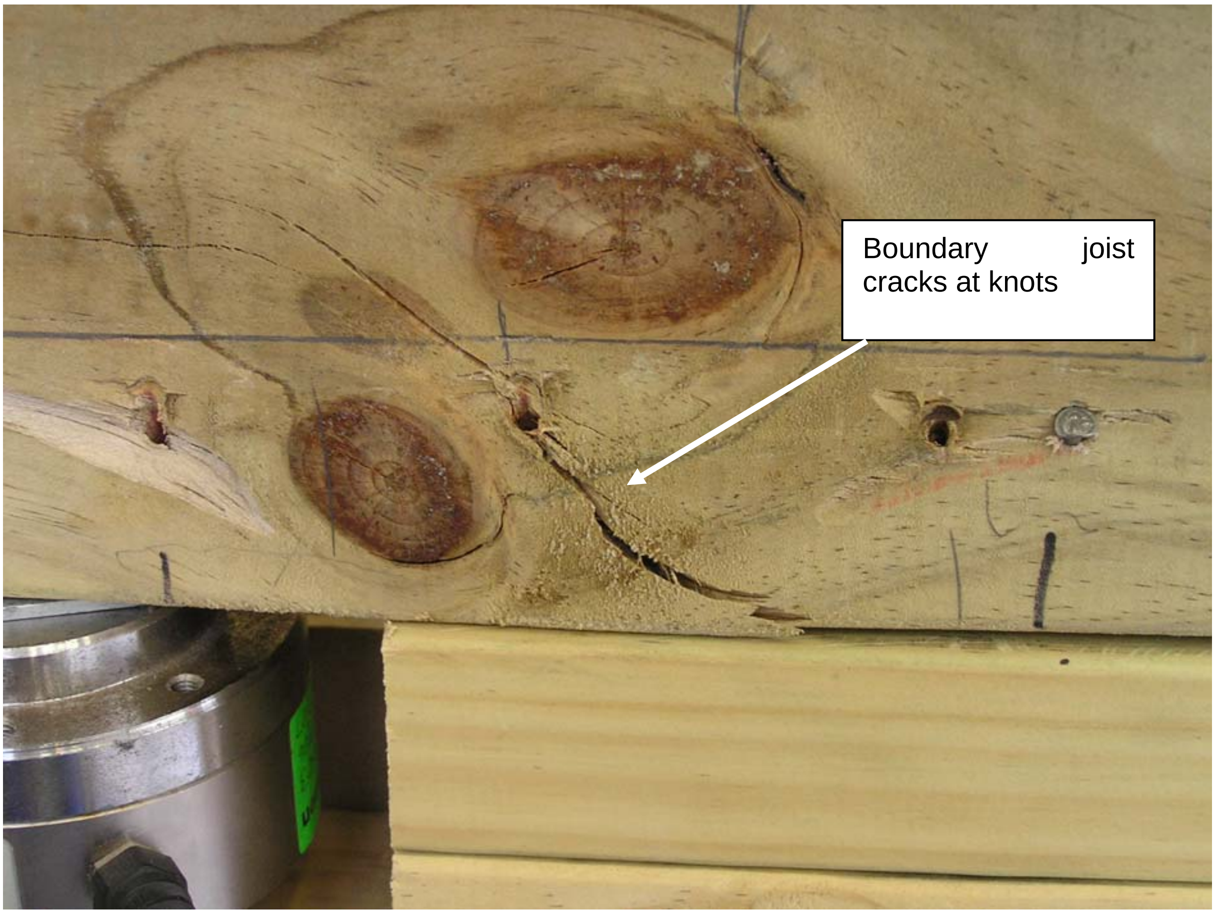


Figure 58. Crack at knots in End 2 boundary joist late in Test 2 of Stage II

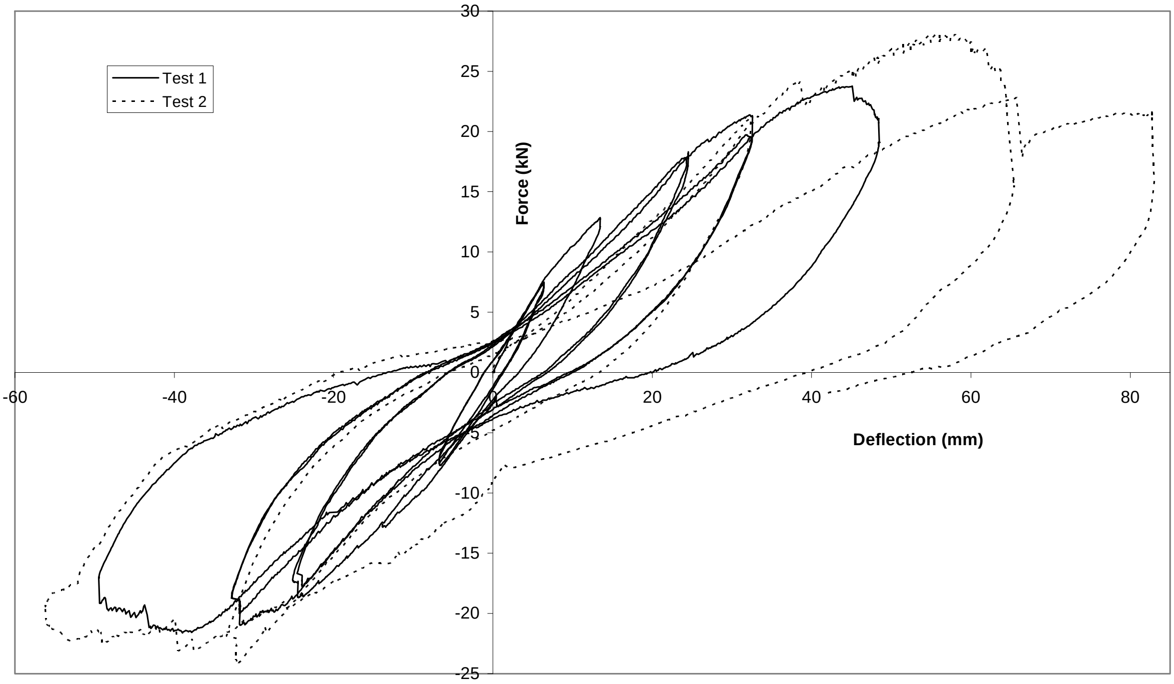


Figure 59. Total racking force versus average deflection for Stage III

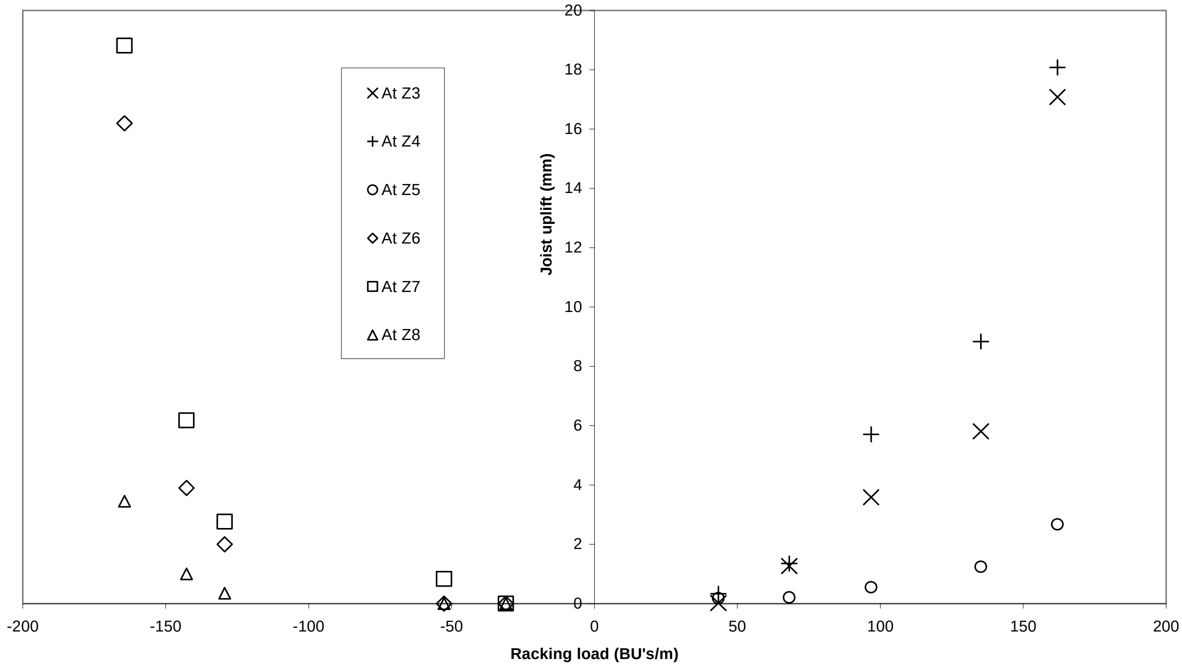


Figure 60. Joist uplift from wall below versus racking load near the internal wall for Test 1 of Stage III

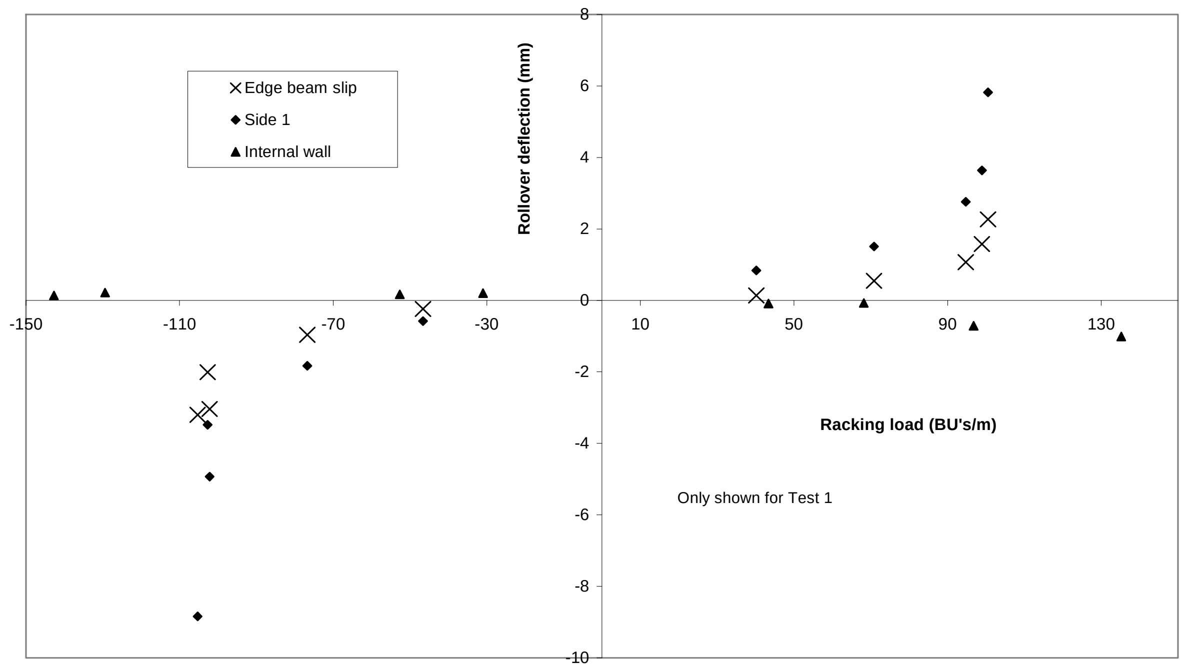


Figure 61. Joist roll-over versus racking load for Test 1 of Stage III

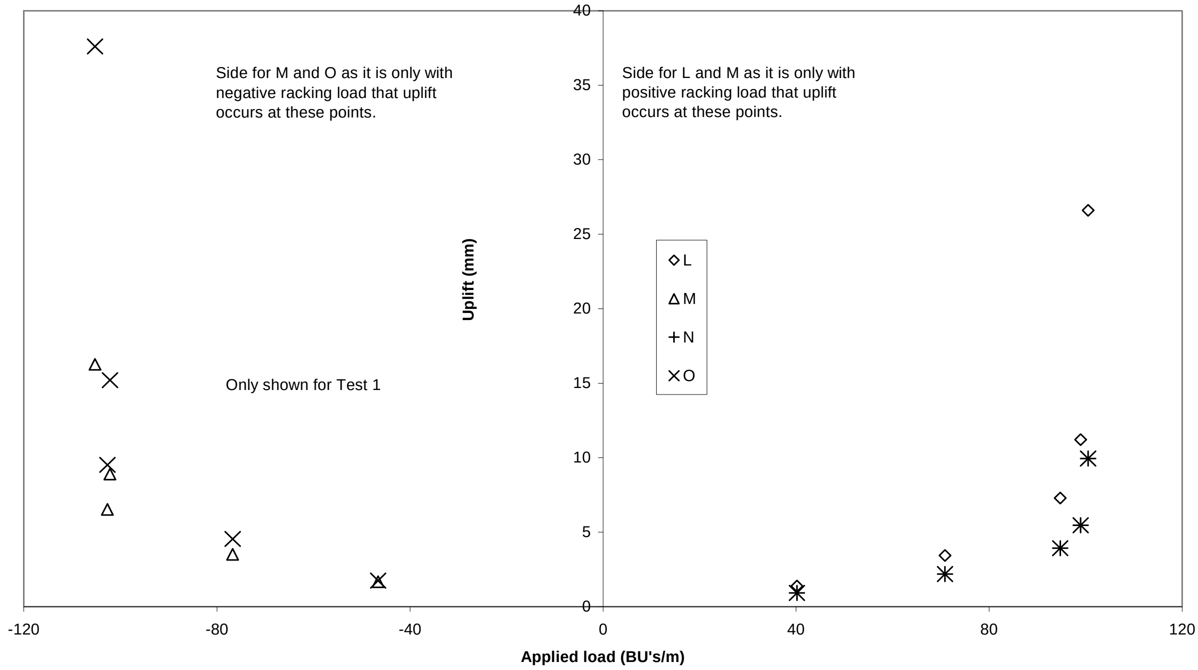


Figure 62. Uplift of upper storey studs from lower storey studs versus racking load at End 2 for Stage III

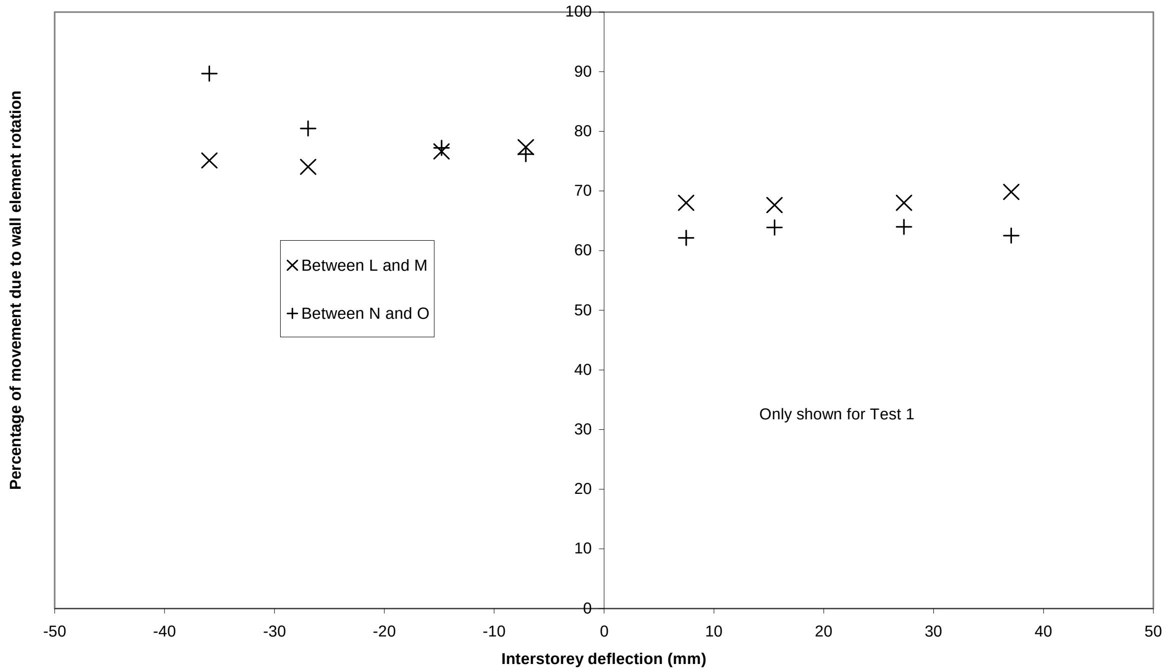


Figure 63. Percentage of wall horizontal deflection due to ‘rocking’ versus racking load at End 2 for Test 1 of Stage III

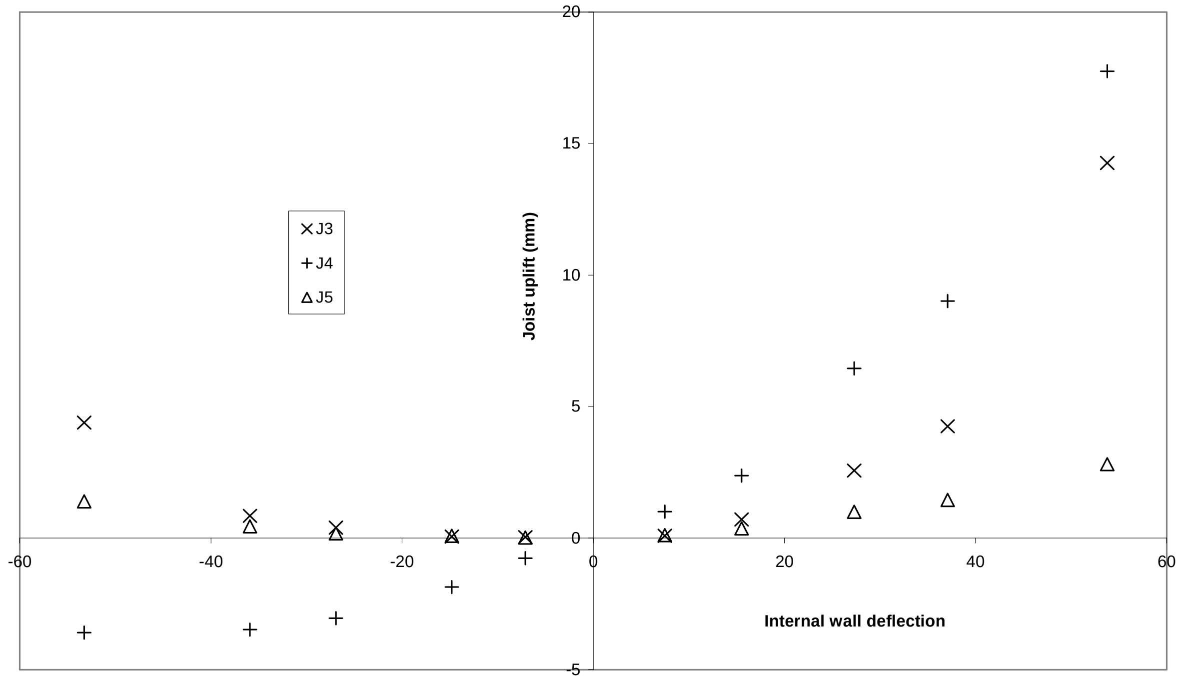


Figure 64. Floor joist deflection at Z1 for Test 1 of Stage III

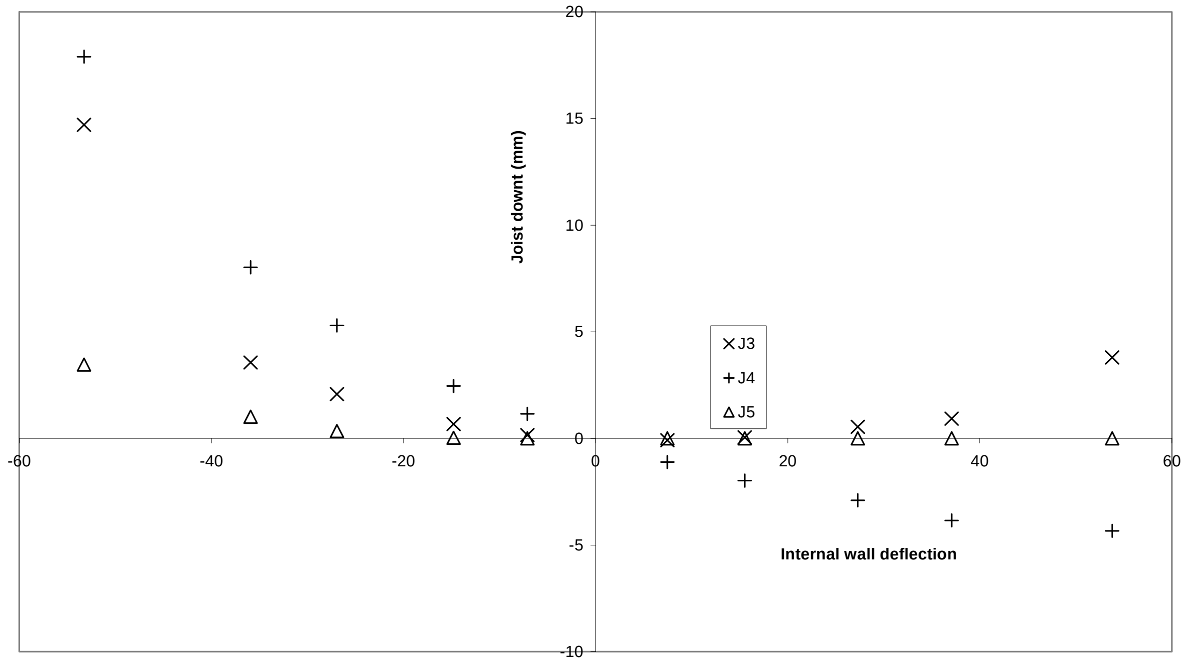


Figure 65. Joist deflection at Z2 at the internal wall for Test 1 of Stage III

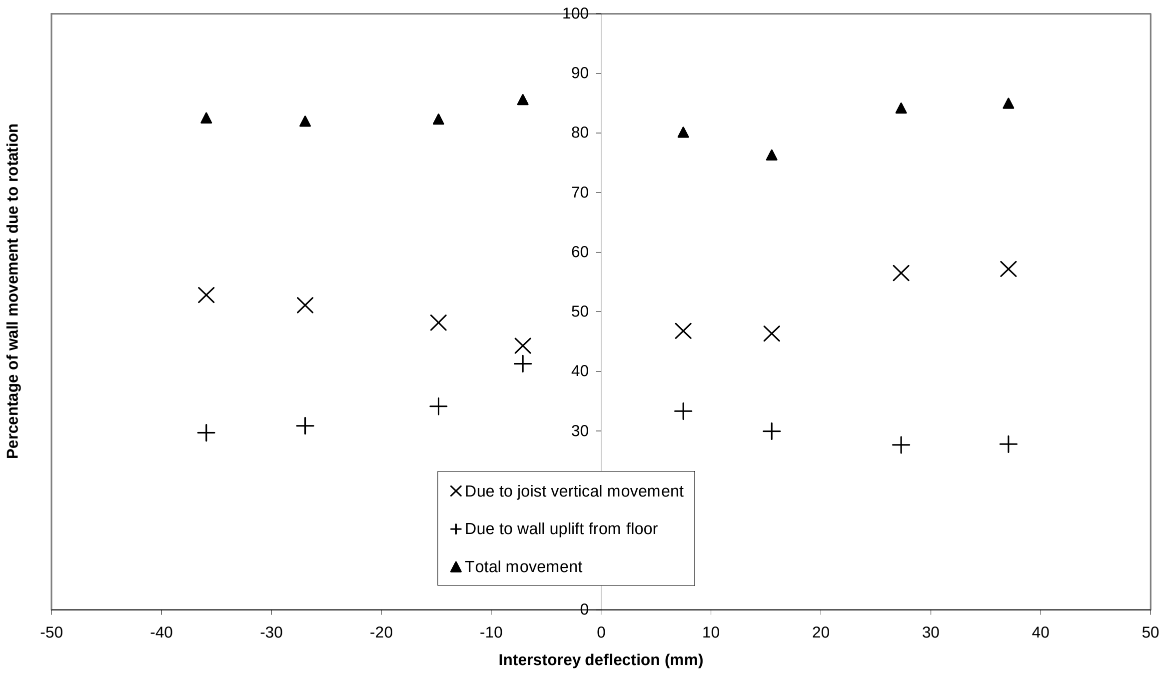


Figure 66. Percentage of deflection due to 'rocking' versus racking load at the internal wall for Stage III

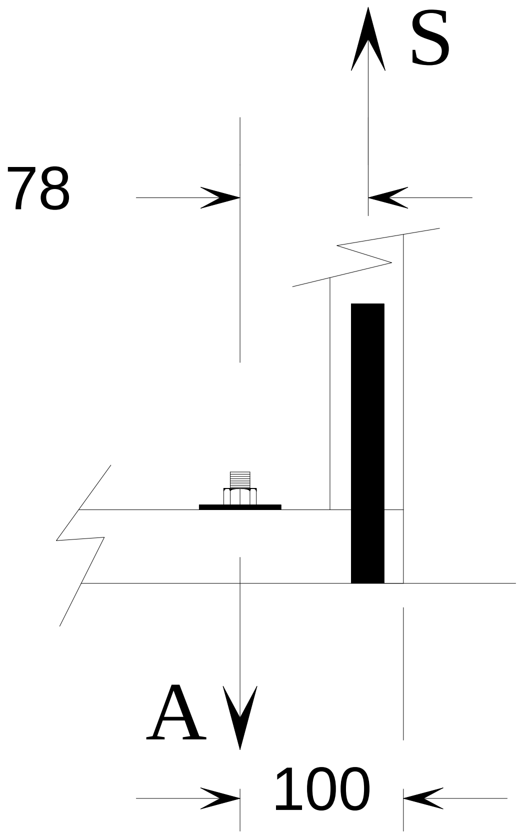


Figure 67. Flexural loading of bottom plate

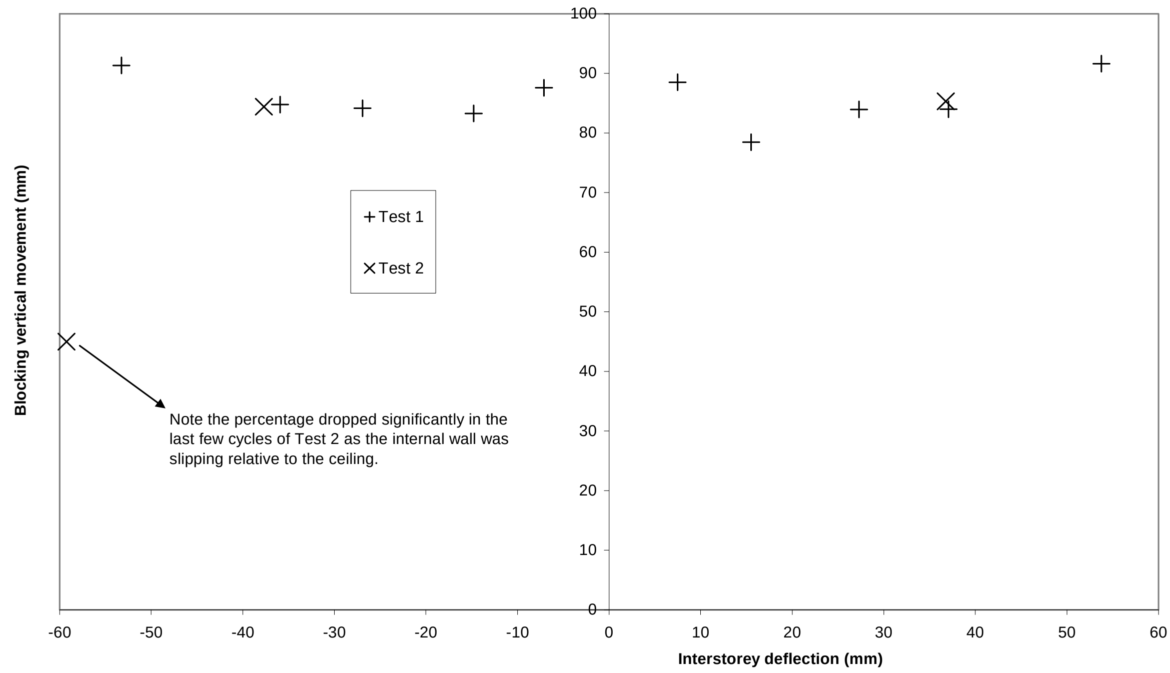


Figure 68. Percentage of deflection due to 'rocking' versus racking load at the internal wall for Stage III (from gauges relative to ground at Z1 and Z2)

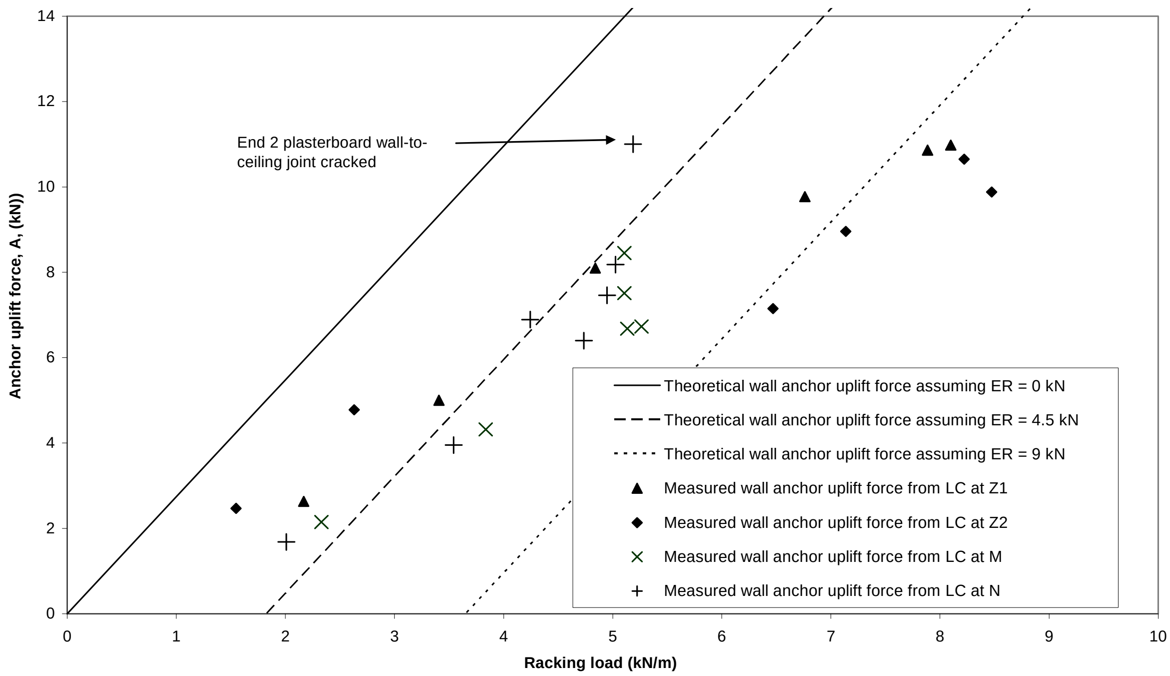


Figure 69. Comparison of theoretical and measured uplift forces on bracing walls for Stage III