STUDY REPORT

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Maximum wall bracing rating that is compatible with NZS 3604 Construction Phase II – Testing of Room 1

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Preface

In Phase I of this project (BRANZ *Study Report 162*) investigates 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 were 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) was measured. The low values so determined were of concern. This stimulated the research reported herein (Phase II of this project), where a complete structure 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.

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. In Phase III of this project BRANZ *Study Report 164* presents test results performed to investigate the strength of this load path.

Acknowledgements

This work was jointly 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 into this topic.

MAXIMUM WALL BRACING RATING THAT IS COMPATIBLE WITH NZS 3604 CONSTRUCTION. PHASE II – TESTING OF ROOM 1

BRANZ Study Report SR 163 (2007) **SJ Thurston**

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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 bracing strength to ensure these assumptions are met with typical New Zealand house construction.

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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 tests 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 report describes racking tests of a small single storey building founded on piles (called Room 1) which were done as part of an investigation to determine this upper limit.

There are many potential limitations to the required load transfer. The most critical of these will be used to determine the recommended maximum bracing rating. BRANZ *Study Report 162* (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.

BRANZ *Study Report 162* (Thurston 2007(a)) also presented the results of testing single isolated bracing walls 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) was given. The low values determined were of concern. This stimulated the research presented in this report whereby a complete structure was racked so that the influence of "systems effects", as occurs in a 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.

Concern was also expressed about the ability of NZS 3604 type buildings to resist the vertical loads arising from wall racking between top and bottom storeys of two storey buildings. BRANZ *Study Report 164* (Thurston 2007(b)) presents testing results performed to investigate this load path.

2. ROOM 1 CONSTRUCTION DETAILS

2.1 General description

The room tested was a single storey building founded on piles. It had a plan size of 6.49 m x 3.29 m. The foundation consisted of joists supported on bearers which themselves were supported on piles. Blocking between these floor members was used as required by NZS 3604 (SNZ 1999). A general view of the foundation framing is shown in Figure 1. The joists were single span rather than continuous because it enabled the applied vertical force on the joists to be estimated from the measured deflections.

The room was bounded by walls on all four sides. The two long sides are referred to as Side 1 and Side 2 (as shown in Figure 2). The two short sides are referred to as End 1 and End 2. Drawings showing the framing in elevation for each side are shown in Figure 3 to Figure 6 respectively. The ceiling framing is shown in Figure 7. Construction was conventional and complied with NZS 3604.

An internal wall was added part-way through the test program at the location shown in Figure 7. This is described in Section 6.

The ends of all bracing elements were fixed as per the Winstone Wallboards (2006) bracing wall hold-down system as illustrated in Figure 8. This consisted of an end strap fixing the studs to the bottom plates and coach screws or through-bolts fixing the bottom plates to foundation framing. Load cells were used to measure the anchor force as shown in Figure 8.

2.2 Member sizes and connections

All timber, except that for the piles, was grade MSG 8.

- Piles: standard New Zealand house piles of rough sawn H4 timber and cross-section 125 x 125 mm.

- Bearers: 90 x 90 mm (formed from two 90 x 45 pieces spiked together) placed symmetrically over the piles. Bearers were fixed to the piles with two 4 m "wire dogs" plus two power-driven 90 x 3.15 mm Paslode JD nails placed on a skew angle (as noted in Section 6.5.2(b) of NZS 3604). The "wire dogs" were diagonally opposite each other.

- Joists: the joists had a 145 x 45 mm cross-section and were fixed to the bearers 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).

- Blocking between joists: the blocking had a 145 x 45 mm cross-section. It was fixed to each joist using two end-nailed power-driven 90 x 3.15 mm Paslode JD nails at one end and four power-driven 90 x 3.15 mm Paslode JD nails placed skewed at the other. Where blocking was above bearers it was also fixed to the bearers using three power-driven 90 x 3.15 mm Paslode JD nails on each side, placed at a skew angle.

- Flooring: the 20 mm particle board 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: 90 x 45 mm cross-section. Lintels and details at windows and doors and wall construction complied with NZS 3604:1999 for lightweight single storey construction. No nogs were used. A double top plate was used with the upper member being 140 x 35 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 35 mm at 600 mm centres.

The walls were initially lined on the inside with 10 mm GIB Braceline®, donated by Winstone Wallboards (2006), fixed using their BL1 screwed vertical sheet option. They were not clad on the exterior. 13 mm GIB® standard board was used on the ceilings. All sheet joints were reinforced with paper tape and the joints plastered. In some tests modifications were made to wall sheathing. This is described in the text prior to the description of subsequent testing.

3. TEST DESCRIPTION

3.1 Overview of test programme (the four test stages)

Four tests were performed (referred to as Stage I to Stage IV).

In Stage I testing, a separate actuator was used to deflect the top of each of the walls on “Side 1” and “Side 2” respectively to the same deflection regime. This is illustrated in Figure 2. Damaged areas were then repaired and the room was picked up by crane and moved so that End 1 could then be racked in Stage II testing. A cut through the ceiling lining and framing, at the location shown in Figure 7, was made in an attempt to isolate End 1 before Stage II testing. This was only partially successful and thus the cut was extended to the full depth of the walls and Stage II testing recommenced.

An interior wall was added for Stage III of the test program as shown in Figure 7. A cut line for the full depth of the room (as shown in Figure 7) was made prior to testing.

Finally, the room was moved again to enable End 2 to be racked in Stage IV testing.

3.2 Blocking to remove horizontal load

The testing was designed to determine the effectiveness of the construction to transfer the vertical forces (induced from the horizontal racking loads) to the foundations. The foundation system used was not designed to resist horizontal loads. Thus, the horizontal load was removed at joist level by end blocking in Stage I (as shown in Figure 3, Figure 4 and Figure 9) and at bearer level by end blocking in other test stages (as shown in Figure 5 and Figure 6).

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 the test specimen loaded wall top plate (the LVDT on the actuators provided an approximate check on this).

(3) Horizontal deflection of the floor, taken at the joist level (Figure 11) or particle board floor level).

(4) Pile uplift (Figure 12)

(5) Vertical separation of the bearers from the piles (Figure 13).

(6) Vertical separation of joists from bearers (Figure 14).

(7) Vertical movement of blocking from the joists (Figure 15).

(8) Vertical deflection of bearers and joists over their length at member mid-span. These used a reference stick pinned to the bearer (or joists) at both bearer (or joist) supports. A gauge measured the relative vertical movement between bearer (or joist) and the reference stick (see Figure 16).

(9) Load cells to measure the vertical force in bracing wall hold-down anchor through-bolts (see Figure 15 and Figure 16).

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 9. 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. Cuts were also made in the perimeter ceiling framing directly above the edges of wall openings (as illustrated in Figure 32) to remove restraint from wall vertical movement.

3.5 Measured end restraint

The forces on a bracing element being racked are shown in

Figure 17. This is drawn for a 1.25 m long bracing wall and for a distance from the wall hold-down bolt to the end of the bracing element of 0.125 m. These values were selected as they represented an average of those used in the construction.

It is assumed that the bracing element has a self-weight of W1. When it rocks as shown in

Figure 17 it is assumed that it carries an axial load of W2 at the uplift corner. This is likely to be greater than the axial load which would be present at wall mid-length if there was no rocking action as the uplift motion attracts axial load which would otherwise be carried by other walls etc. The racking also induces an upward reaction 'R' on one end of the wall and a hold-down anchorage force 'A' near the other end of the wall. It is assumed that adjacent construction imposes a restraining force of ER at the tension end and ER' at the compression end. Forces from any nails holding down the bottom plate are ignored.

If the forces ER and ER' are known then A and R can be determined and then the substructure below the floor, in particular the joists and bearers, can then be designed.

Taking bending moments about the compression corner the following equation can be written:

$$B \times 1.25 \times 2.465 = A \times 1.125 + ER \times 1.25 + W1 \times 1.25/2 + W2 \times 1.25$$

$$\text{Hence } A = 2.739\,B - 1.111\,ER - 0.556W1 - 1.111W2 \quad .... (1)$$

Equation (1) shows that W1 and W2 reduce the hold-down anchorage force 'A'.

Thus, by considering the vertical force balance the following equation can be written:

$$R + ER' = ER + A + W1 + W2 \quad ......(2)$$

If ER ≈ ER' then this equation shows that R will always be greater than A if W1 and W2 are significant and hence more likely to result in joist failure than the uplift force. However, in the following calculations it is assumed that the bracing element is an upper storey wall beneath a light roof and that W1 and W2 are relatively small and can be considered to be zero.

Substituting using equation (1),

$$R = 2.739B - 0.111\,ER - ER' \quad ..... (3)$$

From equation (3) it can be seen that the influence of ER on the value of R is small (the coefficient being 0.111). If ER is put = ER' it should not affect R greatly. Equation (3) then becomes:

$R = 2.739B – 1.111\ ER'$ ..… (4) (i.e. similar to equation (1)).

Taking bending moments about the tension corner the following equation can be written:

$B \times 1.25 \times 2.465 + A \times 0.125 = (R + ER') \times 1.25 - W1 \times 1.25/2$

Hence $R = 2.465\ B – ER' + 0.5W1 + 0.1A$ .... (5)

Equation (5) shows that the downward reaction R is independent of W2.

3.6 Nomenclature

The text of this report uses a coded system to refer to specific piles, bracing elements, joists and locations on walls. The system used is identified in Figure 18 to
Figure 20.

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

4.1 General description

The same deflection regime was simultaneously imposed onto the tops of both exterior long walls i.e. Sides 1 and 2. Horizontal load was removed from the construction at joist level as shown in Figure 9 and Figure 11 so that only the vertical component of bracing load was transmitted through the foundations. Roof dead load and a floor live load of 0.2 kPa was simulated by weights as shown in Figure 9.

The loading in Stage I testing was imposed in three separate tests, with modifications to the construction being made in each test:

- Test 1 terminated when it was realised that part of the loading arrangement on the Side 1 wall had been incorrectly installed. This resulted in a sudden 15 mm differential movement between the two loaded sides. The ceiling appeared to tolerate this without any damage. This was rectified for Test 2.

- Test 2 terminated with the failure of bracing element W3-4 of the Side 2 wall (see Figure 18). Reaching wall failure, rather than foundation failure, had been considered likely and was the desired outcome. The results are used to provide an indication of wall "system factor" in Section 4.9. The following repairs were then done before Test 3 commenced to avoid wall failure and thereby force failure into the foundations. Badly damaged areas of the bracing elements were replaced. Fastener spacing was reduced and joints repaired. Plywood and MDF sheets were added on the exterior of the room long sides (i.e. Side 1 and Side 2).

- Test 3 terminated when it was decided that sufficient failures had occurred in the foundations. The "wire dogs" and skew nailing in all observed areas of damage were replaced in preparation for Stage II testing described in Section 5.

4.2 Test hysteresis loops

Figure 21 to Figure 23 show the generated total building load-deflection (hysteresis) loops for Test 1 to Test 3 respectively.

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 Measured distribution of racking load resisted by wall bracing elements

If all wall elements were equally stiff there would have been the same racking load (kN/m) in each wall element. However, as the walls were of different lengths and boundary conditions they had different stiffnesses. In particular, wall bracing element W_{3-4} (see Figure 18) attracted a disproportionate share of the load mainly because each end of the wall was founded almost directly over piles and there was little vertical movement between joists-bearer-pile. At the other locations vertical joist deflections (and later in the test joist-bearer-pile gaps) significantly increased wall bracing element flexibility.

From simple statics, an isolated bracing element of length L and height H which is racked at B kN/m will have an end vertical reaction = B x H – i.e. independent of L. Thus, in order to estimate the distribution of bracing loads amongst the wall bracing elements, bracing element loads per unit length (kN/m) were assumed to be proportional to the vertical reactions. These were directly determined either from load cells, or indirectly from joist deflections, using the method outlined in Section 4.9. Backbone curves of the racking load resisted by the wall elements from Test 2 (based on these calculations) are shown in Figure 24 and Figure 25 for Side 2 and Side 1 respectively. It can be seen that the load in bracing wall element W_{3-4} is disproportionately high. When the bracing element W_{3-4} failed near the end of the test, load was transferred to other elements as can be seen in the plot. The load in each bracing wall element, so calculated, is hereby defined as the "determined" wall bracing element loading. In contrast, the "average" load is defined:

$$\text{“average” load} = \frac{\text{the applied load per wall}}{\text{sum of the bracing element lengths in that wall.}}$$

4.4 Load sharing between joists

Vertical load from bracing element end reactions is carried by not only the joists directly below the wall but also by adjacent joists. In calculations to determine the risk of joist failure the degree of load sharing must be estimated. This can be determined from the tests, as described below, if it is assumed the joist deflection is proportional to the vertical load carried. Figure 26 and Figure 27 plot the first interior joist deflection versus the exterior joist deflection and the second interior joist deflection versus the exterior joist deflection respectively. There is a scatter of results. The linear regression best fit curves are plotted and provide ratios of 0.145 and 0.036 respectively. Thus, as a pair of joists was used on the exterior, and single joists elsewhere, the load in the exterior joists is theoretically reduced due to load sharing by:

$$1 - \frac{2}{2 + 0.145 + 0.036} = 8\%$$

4.5 Pile uplift

Weights were fixed to each pile to simulate the weight of soil which would uplift if a "normal" pile pulled out of soil. At an "average" bracing load of 103 BU's/m, in Test 1, Pile P33 and Pile P32 uplifted 3 mm and 2 mm respectively (see

Figure 20 for locations of these piles). All piles were then bolted to the strong floor and were kept in this condition for the remainder of the testing to assess the strength of the rest of the foundation.

4.6 Joist uplift from bearer

Figure 28 plots the joist uplift from the bearer at each pile position as measured in Test 3. Notes on this plot list the movement monitored in previous tests. These show that the prior testing caused an increase in the deflections monitored in subsequent tests. As loading increased, load redistribution clearly occurred as otherwise no increase in total applied load would have been possible once uplift started.

The most critical uplift location is clearly at Pile P33. Here the joist pulled away from the bearer by 4.2 mm at an "average" of 114 BU's/m in Test 2 and 6.6 mm at 111 BU's/m in Test 3.

As large gaps opened in Test 3, the following joist-to-bearer joints were strengthened by straps as follows:

- at Pile P33 after loading to an "average" of 132 BU's/m
- at Pile P30 after loading to an "average" of 170 BU's/m.

Selecting a maximum bracing rating from these results is clearly very subjective.

4.7 Bearer uplift from piles

Figure 29 plots the bearer uplift from the piles. These connections were all formed using two "wire dogs" and two skew nails, except at Pile P32 where four skew nails were used. From the plot, the bearer-to-pile connection was deemed to have failed at 190 BU's/m at Pile P30 and 170 BU's/m at Pile P32. This connection is thus considered to be less critical than the joist-to-bearer connection.

The following bearer-to-pile connections were strengthened by straps in Test 3:

- at Pile P33 after loading to an average of 170 BU's/m
- at Pile P30 after loading to an average of 206 BU's/m.

The four skew nailed connections (without "wire dogs") at Pile P32 did not fail. However, such a joint would be expected to fail at a low bracing rating if it was used at a corner.

4.8 Joist behaviour

The vertical load from the bracing walls was mainly transferred to pairs of joists. No joist flexural failures occurred. The bracing load at which this failure would be expected is examined below.

Equation (1) of Section 3.5 calculates the vertical anchor force transmitted to joists directly below for the bracing wall arrangement shown in

Figure 17.

If the joist failure was to occur at twice the characteristic flexural design stress (14 MPa), then the bracing load to induce failure can be calculated and is summarised in Table 1, using simple statics and joist dimensions.

It can be seen that joist failure was unlikely in the test series reported herein (two joists case) as failure is not expected before 203 BU's/m, whereas it is expected in the proposed tests on an internal wall (one joist case) at a wall bracing of 138 BU's/m. Note: that the value of 8% load sharing is only for the specific construction described in Section 2 and far greater load sharing is likely at internal walls.

Table 1. Bracing load to induce joist failure (BU's/m)

(Joist size 145 x 45, failure flexural stress = 14 MPa)

Load sharing	Number of joists beneath bracing wall	Assumed value of ER (End restraint – see Figure 17)	
		4.5 kN	9 kN
None	1	97	133
	2	157	193
8% spread to adjacent joists	1	101	138
	2	166	203

4.9 "Systems effect factor" for walls

In Test 2, the maximum total applied load was 46.9 kN. On the next cycle the internal lining on the wall bracing element $W_{3\text{-}4}$ partially dislodged from the framing and the total resisted load dropped. As the total length of bracing walls was 7.48 m, the average maximum bracing resistance was therefore = $\frac{46.9 \; x \; 20}{7.48}$ = 125 BU's/m.

Winstone Wallboards Ltd (2006) gives the bracing resistance to wind of the tested walls as 125 BU's/m, which is the same as the average measured and indicates that the "systems effect factor" = 1.0. However, as noted in Section 4.3, the intensity of bracing resisted varied considerably (mainly due to the support conditions), with bracing element $W_{3\text{-}4}$ resisting a peak of 183 BU's/m which is 46% greater than the Winstone Wallboards Ltd (2006) published rating for this configuration. If all the walls had been founded on a concrete foundation a more uniform distribution of load resistance would have been expected.

4.10 Measured end restraint

When a house wall is racked it may "rock" as shown in

Figure 17. The adjacent construction imposes forces denoted as ER at the uplift end and ER' at the compression end to resist the upward and downward movement respectively. Section 3.5 provides equations relating bracing load, the wall end reactions and ER/ER'. This section uses the measured reactions and bracing loads to estimate the magnitude of ER/ER'.

The upper right quadrant of Figure 30 plots the theoretical relationship between wall anchor uplift force A (kN) and positive wall racking loads (kN/m) as calculated using equation (1) in Section 3.5. Three lines are plotted for the anchor uplift force associated with values of ER = 0, 4.5 and 9 kN respectively. Note: these correspond to what BRANZ assumes to be no end restraint, the P21 "3-nail" end restraint and a "6-nail" end restraint.

The lower left quadrant Figure 30 plots the theoretical relationship between wall compressive downward end reaction, R, and negative wall racking load for the same assumptions, but for ER' not ER.

The uplift force was measured by load cells at Point V14 on Side 1 and Point P4 on Side 2. These locations are shown on Figure 18. The data is plotted as solid symbols in Figure 30 at each hysteresis first peak upwards load i.e. if the room is deflected to 3 cycles to 5, 10 and 15 mm, three points are extracted being at the first loading at +5, +10 and +15 mm deflection. Note: that the load cells only register the vertical load for uplift (tension) which (for the load cell locations used) correspond to positive racking deflections. The racking load plotted is the "determined" racking load as defined in Section 4.3 rather than the "average" load.

Joist vertical deflections were measured at Points V11, V13 and V14 on Side 1 and Points P2 and P5 on Side 2. The elastic stiffnesses of all joists were measured before construction of the room. The wall reaction (i.e. vertical force on the joists) was deduced from the joist deflections using separate analysis and the force distribution shown in Figure 31 and assuming an 8% redistribution to adjacent joists as discussed in Section 4.4. This data is plotted as a "+" symbol in Figure 30 for each hysteresis first peak load (positive and negative). Agreement between loads calculated from the deflection gauge and load cell at Point V14 was within 20%.

At racking loads of magnitude less than 3.7 kN/m the theory for ER = 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 5 kN/m the theory for ER = ER' = 9 kN predicts a greater end reaction than measured. Hence, it is concluded that joist and bearer strength can conservatively be checked assuming ER = ER' = 9 kN.

4.11 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 the model shown in
2. Figure 17, with ER = ER' = 9 kN.

3. For construction with double joists on a building exterior and single joists subsequently located at 600 mm centres it is recommended that 8% of the vertical load on the joist may be assumed to be redistributed to adjacent joists.

4. A wall bracing rating of more than 100 BU's/m may induce pile uplift in single storey NZS 3604 construction.

5. A wall bracing rating of more than 140 BU's/m may separate joist-bearer skew nail connections.

6. A pile-bearer connection of two "wire dogs" and two skew nails is likely to be less critical than the joist-bearer connection.

5. STAGE II: RACKING TESTS ON END 1

5.1 General description

A general view of the test set-up is shown in Figure 32. All of the foundation fixings beneath the end walls were extracted and replaced before commencement of Stage II testing.

The full width of the ceiling was cut at 1.7 m from End 1 as shown with dashed lines in Figure 7 and Figure 18. This was at the window openings in the side walls. The cut went through the ceiling battens, blocking between joists and wall double top plates, but not the side walls themselves. The purpose was to isolate End 1 to prevent horizontal load being transmitted by ceiling diaphragm action to End 2.

Loading was applied using an articulated beam with pins at the edges of doorway as shown in Figure 33. The edge ceiling joist was also cut above the doorway. This was to allow the wall to lift at doorway openings as shown in Figure 33.

The force in wall hold-down bolts was measured at two locations by openings as shown in Figure 8 and Figure 34. The floor deformation and wall uplift due to bracing wall end reactions can be seen in Figure 8.

End 1 was lined on the room interior with GIB® Braceline (Winstone Wallboards Ltd 2006), but not clad on the exterior. The testing terminated when foundation failure occurred. Although vertical cracks developed at sheet joints above the doorways, an inspection of the plasterboard at screw head locations indicated that the bracing walls still had some reserve strength at test completion.

At ±72 mm imposed End 1 racking deflection, a slip between adjacent ceiling edges of ±30 mm was observed to occur along the cut line at 1.7 m from End 1. This showed that the desired isolation was moderately effective.

It was considered that some horizontal load was still being transmitted across the cut line. Hence, the cuts were extended to the full depth of the side walls so that the end 1.7 m of the room nearest to End 1 was completely isolated down to the floor level as shown in Figure 18. End 1 was then racked again. At the maximum imposed wall deflections the measured slip across the cut was approximately ±70 mm which was similar to the differential deflection between End 1 top plate and the floor showing that the cut was now fully effective at isolating End 1. The peak resisted loads were 72% (push) and 74% (pull) (i.e. average of 73%) of the resisted peak loads before the cut was extended down to floor level. Thus, conservatively, all measured loads from End 1 testing without extended cuts presented in this report have been factored by 0.73 to give the horizontal load actually carried by the End 1 wall.

5.2 Test hysteresis loops

Figure 35 shows the generated load-deflection (hysteresis) loops for racking the isolated End 1 wall. The deflection plotted is the differential movement between wall top plate and floor. The load plotted in Figure 35 is the measured racking load factored by 0.73 as discussed in Section 5.1.

At peak resisted load the wall deflections for this plasterboard lined wall were approximately ±72 mm, which are far greater than measured in a P21 test (normally about 20 mm). This was attributed to foundation flexibility. Walls founded on the more rigid parts of the foundation may consequently carry a disproportionate share of the racking load.

5.3 Pile uplift

Weights were fixed to each pile to simulate the weight of soil which would uplift if a "normal" pile pulled out of soil. At an "average" bracing load of 50 BU's/m, Pile P33 uplifted 1 mm (see

Figure 20 for location of this pile). All piles were then bolted to the strong floor and were kept in this condition for the remainder of the testing to force failure into other parts of the foundation.

5.4 Joist uplift from bearer

Figure 36 plots the measured joist uplift from the bearer versus “average” bracing load for each of the seven joists labelled in Figure 18. The most critical uplift location is clearly at Joist J1. Here, the joist pulled away from the bearer by 10 mm when the bracing load was an “average” of 93 BU’s/m. Joist J7 pulled away from the bearer by 6.9 mm at an “average” bracing load of 90 BU’s/m.

Greater imposed wall deflections just resulted in increased bearer-to-joist gaps with little increase in resisted load (as shown in Figure 13). Thus, all joist-to-bearer joints were then strengthened by straps or “wire dogs” to allow the test to continue.

5.5 Bearer uplift from piles

Figure 37 plots bearer uplift from its support pile versus “average” bracing load. These connections were all formed using two “wire dogs” and two skew nails. From the plot, the bearer-to-pile connection at Pile P33 was deemed to have failed at 139 BU’s/m. Thus, this connection is considered to be less critical than the joist-to-bearer connection.

5.6 Blocking-to-joist connection

The vertical slip between the blocking-to-joist connection was measured at each joist. The slip was in all instances less than 0.5 mm, except for the blocking-to-joist connection at Joist J4 on the side closest Joist J3. This slipped 4.2 mm at an average racking load of 128 BU’s/m and 6.5 mm at an average racking load of 139 BU’s/m. As this load is higher than the failure at the joist-to-bearer connection, this connection is considered to be less critical than the joist-to-bearer connection.

5.7 Bearer flexural failure

Figure 38 illustrates the vertical forces (compression R and tension A) transmitted to the End 1 bearer. The analysis below assumes R = -A as load cells only monitor the load for positive values of A. This sub-section considers the bearer bending moments and deflection of the bearer due to such forces. The bearer did not fail in flexure in the test. The bracing load at which this failure would be expected is also examined below.

Vertical deflections at bearer mid-span and upwards vertical loads at bolt hold-down locations were measured on the End 1 bearer during the repeat racking tests for construction where the cuts had been fully extended to the bottom of the walls.

The bearers were analysed using the computer model of the static loads shown in Figure 39 using the measured uplift forces given in Table 2. In the model, the uplift forces are assumed to be at the hold-down bolt locations (i.e. at the load cells), denoted at Loc1 and Loc2 in Figure 39. The downward forces, denoted at locations Loc3 and Loc4 in Figure 39, were assumed to be at the mid-width of the end studs. The theoretical stiffnesses and strength of the bearer are:

- moment of inertia, $I = 90^4/12 = 5.47E4\ mm^4$
- elastic modulus, E = 8 GPa from NZS 3603
- ultimate moment, $M_u = 14 \times 90^3/6 \times 1.0E\text{-}6 = 1.70$ kNm.

The values subsequently measured for the bearer in a four point bending test were:

- elastic modulus = 8.9 GPa
- failure moment = 5.17 kNm (3.0 times greater than M_u calculated above).

The most critical bending moment was computed to be at Loc2 under "Push" loading, where the calculated bending moment was 3.71 kNm. This is 2.19 times the design value of 1.70 kNm given above, but less than the measured flexural failure.

A comparison of the predicted and measured mid-span deflections is given in Table 2. It can be seen that the measured deflections are significantly lower than the computed values. This may partially be due to the bearer having some vertical movement restraint – i.e. imposed on the bearer by the walls above on the end cantilevered portion.

Table 2. Computed and measured bearer deflections

Load direction	**Location where force measured**	**Upwards force (kN)**	**Span A**			**Span B**		
			Comp. (mm)	**Meas. (mm)**	**Ratio**	**Comp. (mm)**	**Meas. (mm)**	**Ratio**
Pull	Loc1	8.94	9.33	7.39	1.26	10.86	7.58	1.43
Push	Loc2	11.29	9.18	4.64	1.98	13.52	8.85	1.52

Legend:

Comp. = computed mid-span deflection using the static load model in Figure 39
Meas. = measured mid-span deflection at the same force used in the static load model.

5.8 "Systems effect factor" for walls

The maximum total applied load was 16.91 kN although, as noted in Section 5.1, this was probably short of the ultimate bracing strength of the walls. As the total length of bracing walls was 2.293 m, the average maximum bracing resistance was therefore:

$$\frac{16.91\,x\,20}{2.293} = 147 \text{ BU's/m.}$$

Winstone Wallboards Ltd (2006) give the bracing resistance to wind of the tested walls as 125 BU's/m which indicates that the "systems effect factor" = 147/125 = 1.18. However, this is expected to be conservative as the 0.73 factor applied is expected to be on the low side and the walls did not appear to have reached their ultimate strength.

5.9 Measured end restraint

Figure 40 plots the theoretical relationship between wall uplift anchor force A (kN) and positive wall racking loads (kN/m) as discussed in Section 4.10.

The uplift force was measured by load cells at L21 and L22 (refer to Figure 19). Figure 34 shows the load cells. This data is plotted in Figure 40 at each hysteresis first peak upwards load.

Points are plotted for the main test and also for the repeat loading when the cut was extended to include the full height of the walls. The results for the repeat loading show similar results except that L22 at high racking loads gave greater uplift forces. This is consistent with the observation of a horizontal ceiling crack part-way across wall element $W_{22\text{-}23}$, which would have reduced the "systems effects" on this wall element.

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 4.5 kN/m, the theory for ER = 9 kN is in reasonable agreement with the anchor uplift forces measured. Hence, it is concluded that joist and bearer strength can be checked assuming ER = 9 kN.

5.10 Conclusions from Stage II testing

1. A wall bracing rating of more than 95 BU's/m may separate joist-bearer skew nail connections.

2. Blocking-to-joist connection as specified in NZS 3604 is likely to be less critical than the joist-bearer connection.

3. Wall deflections increase considerably due to foundation flexibility. Walls founded on the more rigid parts of the foundation may consequently carry a disproportionate share of the racking load.

4. The "systems effect factor" for this construction was greater than 1.18.

6. STAGE III TESTING: RACKING TESTS ON THE INTERNAL WALL

6.1 General description

A general view of the Stage III internal wall tested is shown in Figure 41 and a view at large wall deformations in Figure 42. A plan view showing the construction, framing and dimensions is given in Figure 43. Figure 44 shows the layout on the floor and the naming conventions used.

The internal wall was an "I" shape in plan view with the "web of the I" being parallel to the applied load. The "web of the I" was lined on one side with 10 mm thick GIB Braceline® , and on the other side with 10 mm GIB® Standard board, with the system being fixed as per a Winstone Wallboards Ltd (2006) BLG bracing element using screws. The "flanges of the I" (usually referred to as return walls) were lined with 10 mm GIB® Standard board being fixed as per a Winstone Wallboards Ltd (2006) GS2 bracing element using screws. It was recognised that the large flanges would help resist rocking motion of the wall. Upwards movement of the flanges would be resisted by the presence of the ceiling structure which would impose a downwards load on the flanges – i.e. a large value of "W2" would occur as discussed in Section 3.5.

The full width of the ceiling was cut at 1.7 m from both End 1 and End 2 as shown with dashed lines in Figure 7 and Figure 18. This was at the door openings. The cuts went through the entire room down to floor level. The purpose was to isolate the internal end wall to prevent horizontal load being transmitted by ceiling diaphragm action to other parallel walls.

Loading was applied using a steel articulated beam with pins at each end of the internal walls as shown in Figure 41. To resist the horizontal load at foundation level, the ends of the bearer were blocked to rigid supports as shown in Figure 41 so that only the vertical component of bracing load was transmitted through the foundations. A floor live load of 0.2 kPa was simulated by weights. No roof loading was used.

Joist-to-bearer vertical movement, blocking-to-joist vertical movement and joist deflections were extensively monitored. The force in wall hold-down bolts was measured at the two ends of the internal wall as shown in Figure 15 and Figure 16.

The foundation connections were in the "as-new" condition except that the pile-to-bearer joints had been strengthened by straps. Testing terminated when a joist directly below one end of the tested wall failed in flexure as shown in Figure 45.

6.2 Test hysteresis loops and description of damage

Figure 46 shows the generated load-deflection (hysteresis) loops from racking the isolated internal wall. The deflections plotted are the differential movement between wall top plate and floor.

At the peak resisted load the wall deflections for this plasterboard lined wall were approximately 105 mm in the "push" direction and 142 mm in the "pull" direction, which is far greater than usually measured in a P21 test (normally about 20 mm). This was attributed to foundation flexibility.

For most of the test the only damage noted was when the flanges of the internal wall lifted from the foundation. At peak push load (Figure 46) one flange wall tore from the ceiling (see Figure 42). At peak pull load (Figure 46) Joist J2 failed in flexure directly under the compression end of the internal wall (Figure 45).

6.3 Joist uplift from bearer

Figure 47 plots the measured joist uplift from the bearer versus racking load for the most critical uplift locations from monitored joints (Points C, D, E, F, G and H labelled in Figure 44). From the plot the most critical location for joist uplift is clearly where the return walls are directly over a joist i.e. Points C and D. At Point C, the joist pulled away from the bearer by 7.4 mm when the bracing load was 234 BU's/m. This is well above the failure loads from other testing on this room and is therefore deemed to be not critical.

6.4 Blocking movement from joists

At Point A (Figure 44) the blocking had moved vertically by 4.4 mm at a bracing load of 244 BU's/m. The measured uplift force was 9.27 kN. The movement increased to 16.7 mm at 251 BU's/m. This is well above the failure loads from other testing and is deemed to be not critical. The movement at Point I was less than the movement monitored at Point A.

6.5 Load sharing between joists

The mid-span deflections of the joists were determined from the vertical movement from a "reference stick" to the joist. The "reference stick" was a straight piece of timber attached at each end to a joist as shown in Figure 16. As the joist deflected, the reference stick retained its original position. The measured joist deflections versus applied bracing load are shown in Figure 48. If the joists remain elastic, it would be expected that the deflection of the joists is proportional to axial load imposed on the joists. Thus, Figure 49 was plotted which is the joist deflection versus the measured uplift force. As expected, this provided a more linear plot. The deflection profile of the floor across the seven joists (J1 to J7) is plotted in Figure 50.

For joist design it is useful to know the proportion of the bracing wall imposed vertical load that is carried by the joists closest to the loading points which, from Figure 43, are Joists J2 and J5.

If the load carried by a joist is proportional to its deflection then the proportion of axial load carried by Joist J2 can be estimated to be:

ProportionJ2 = $(\Delta_2)/(\Delta_2 + \Delta_3 + \Delta_4)$ ….. (6) Note: Δ_1 is the opposite sign as Δ_2 and was thus omitted from the equation.

Similarly, for Joist J5:

ProportionJ5 = $(\Delta_5)/(\Delta_5 + \Delta_6 + \Delta_7)$ ….. (7) Note: Δ_7 is the same sign as Δ_5.

These proportions are plotted in Figure 51. It can be seen that the Proportion factor varies with applied load and is different for Joists J2 and J5. The average value of the Proportion factor is 0.59. The inverse of this = 1.70. Hence, it is recommended that for calculations of the maximum vertical point load that can be carried by a particular joist from racking of internal walls perpendicular (to joists below) it is assumed that the load is shared between 1.7 joists.

6.6 Joist behaviour

As the presence of the flanges would resist "rocking motion" and hence W2 defined in Section 3.5 would be large, it was expected that the joist failure was most likely occur under downward loading from the bracing wall rather than upward loading as the transmitted vertical force is expected to be greater. This is discussed further in Section 3.5.

Joist J2 failed in flexure at a peak pull load of 22.9 kN (316 BU's/m). This was when this joist was subjected to downward loading from the bracing wall. This section investigates the loading on the joist at the time of failure. Joist J5 did not fail. Refer to Figure 43 for joist locations.

In the following analysis the following joist timber properties were assumed:

- moment of inertia, $I = 45 \times 140^3/12 = 10.29E6\ mm^4$
- elastic modulus, E = 8 GPa from NZS 3603
- ultimate moment, $M_u = 14 \times 45 \times 140^2/6 \times 1.0E\text{-}6 = 2.06$ kNm.

The values subsequently measured for Joist J5 in a four point bending test were:

- elastic modulus = 8.8 GPa
- failure moment = 5.01 kNm (2.4 times greater than M_u calculated above).

At the actuator peak pull load of 22.9 kN, when Joist J2 failed under compressive (downward) load at Point B (refer to Figure 44), the measured upward load at Joist J5 was 11.45 kN. The downward load at Joist J2 is assumed to be 11.45 + W2 kN and is assumed to be transmitted as a point load at joist mid-span. The joists were 2 m long. Thus, taking into account the 1.70 distribution factor derived above, the failure joist bending moment at Joist J2 is given by:

(11.45 + W2) x 2/4/1.70 = 3.37 kNm + 0.294W2.

The strength of Joist J5 (measured later) was 5.01 kNm. If Joist J2 also failed at 5.01 kNm then W2 = (5.01-3.37)/0.294 = 5.8 kN.

Table 3 compares the measured deflections of Joist J2 and J5 with the calculated deflections for the last few load cycles of the test. As only the upward load was measured the downward load was put equal to the upward load for this calculation. However, the downward load is expected to be greater due to the presence of W2 and thus the measured downward deflections are expected to be greater than the calculated deflections. Some joist stiffening is expected due to the presence of the flooring. The joists were assumed to remain elastic. It can be seen that measured deflections are generally greater than the predicted deflections, particularly under downward load.

As a sample calculation, the predicted joist deflection given in the bottom RHS (right hand side) cell of Table 3 (using the 1.7 factor for load sharing derived above) is:

$$\Delta = \frac{PL^3}{1.7\ x\ 48EI} = \frac{11.45\ x\ 2000^3}{1.7\ x\ 48\ x\ 8\ x\ 10.29E6} = 13.64\ mm$$

Table 3. Deflections of joists

Load direction	Racking force (kN)	Uplift force (kN)	Deflections (mm)		
			Measured Joist J2	Measured Joist J5	Predicted
Pull	17.6	7.37	-12.7	10.00	8.78
Push	18.4	9.30	8.84	-13.78	11.08
Pull	22.9	11.45	-19.4	15.06	13.64

Upward deflections shown as +ve

6.7 "Systems effect factor" for walls

The maximum total applied load was in the "pull" direction and was 22.94 kN. As the overall length of walls was 1.45 m, the average maximum bracing resistance is:

$$\frac{22.94\ x\ 20}{1.45} = 316 \text{ BU's/m.}$$

Winstone Wallboards Ltd (2006) gives the bracing resistance to wind of the tested walls as 150 BU's/m, which indicates that the "systems effect factor" = 316/150 = 2.11. This large value shows that when a wall has a plastered joint at both the ends and ceiling, a large "systems effect factor" can be achieved.

Collier (2005) came to the same conclusion. He compared the strength of two test walls with full rocking restraint – one being similar to the internal wall tested herein and the other having no ceiling and end wall continuity. Using the ratio of the strengths of these walls he also obtained a "systems effect factor" exceeding 2.0.

<u>Wall racking failure is usually due to shear failure or "rocking failure".</u> The presence of the flanges would have prevented wall "rocking" by attracting axial load from the ceiling.

6.8 Measured end rocking restraint

The uplift force was measured by load cells at the through bolt locations shown in Figure 43. This data is plotted in Figure 52 at each hysteresis first peak upwards load.

The measured wall uplift forces were low because of the large axial load (W2) attracted to the wall uplift corners. This follows from Equation (1). Figure 52 indicates that the effective end restraints are far greater than found in the other test stages. However, this should be interpreted as the sum of end restraints plus W2 was far greater.

6.9 Conclusions from Stage III testing

1. Wall deflections increase considerably due to foundation flexibility. Walls founded on the more rigid parts of the foundation may consequently carry a disproportionate share of the racking load.

2. The "systems effect factor" for this construction was 2.06, which was attributed to the plastered joints at the wall ends and ceiling level. Consequently, the effective uplift end restraint from wall continuity plus attracted axial load W2 was far greater than found in other test stages. Hence, as expected the wall bracing rating at which the joist-bearer skew nail connection separated was far higher than measured in tests on exterior walls of this

room. The failure of the blocking connection also only occurred at a very high bracing rating.

3. From considerations of joist deflections it was concluded that the vertical loads from racking tests on internal walls perpendicular to joists can be assumed to be carried by 1.7 joists. However, the measured joist deflections were greater than predicted when the 1.7 factor given above was used. This suggested that this factor was too high.

4. Joist J2 failed in flexure at an extremely high bracing rating (316 BU's/m). The very high "systems effect factor" for internal walls with fully taped and plastered joints to return walls means that loads on the foundation elements for a given wall bracing rating are low and member and joint failures are unlikely at normal wall bracing levels.

5. It is expected that under high bracing loads that the internal wall attracted a high axial load W2 at the uplift corner which resisted the rocking action. Based on the Joist J2 failure load, and calculations herein, this was estimated to be 5.8 kN.

7. STAGE IV TESTING: RACKING TESTS ON END 2

7.1 General description

An elevation of the End 2 wall is shown in Figure 6 and nomenclature is shown in Figure 19. Loading was applied using an articulated beam with pins at the edges of doorways as shown in Figure 53. The ceiling joist was cut above each doorway vertical edge as shown in Figure 53. This was to allow the wall to lift at doorway openings. Figure 54 and Figure 55 show the construction used at the bottom of bracing element $W_{25\text{-}26}$ and $W_{27\text{-}28}$ respectively.

Vertical cracks developed at sheet joints above the doorways as shown in Figure 56. The test terminated when the lining fell off bracing element $W_{25\text{-}26}$ (refer to Figure 19).

7.2 Test hysteresis loops

Figure 57 shows the generated load-deflection (hysteresis) loops from racking the End 2 wall. The deflections plotted are the differential movement between wall top plate and floor.

At peak resisted load the wall deflections for this plasterboard lined wall were approximately 95 mm in the "push" direction and 65 mm in the "pull" direction, which is far greater than measured in a P21 test (normally about 20 mm). This was attributed to foundation flexibility.

7.3 Pile uplift

Weights were fixed to each pile to simulate the weight of soil which would uplift if a "normal" pile pulled out of soil. At an "average" bracing load of 111 BU's/m, Pile V30 uplifted 5.1 mm (see

Figure 20 for location of this pile). This pile was then bolted to the strong floor and was kept in this condition for the remainder of the testing to enable greater bracing loads to be applied. At an "average" bracing load of 132 BU's/m, Pile P30 uplifted 5.9 mm and Pile S30 uplifted 5.1 mm. These piles were then bolted down.

7.4 Joist uplift from bearer

Figure 58 plots the measured joist uplift from the bearer versus "average" bracing load for each of the seven joists labelled in Figure 19. The most critical uplift location is clearly at Joist J7.

Here, the joist pulled away from the bearer by 7.1 mm when the bracing load was an "average" of 111 BU's/m. Joist J1 pulled away from the bearer by 8.1 mm at an "average" bracing load of 158 BU's/m. These two joist-to-bearer joints were then strengthened by straps to allow the test to continue.

7.5 Bearer uplift from piles

Figure 59 plots the bearer uplift from the piles versus "average" bracing load. These connections were all formed using two "wire dogs" and two skew nails. The bearer uplifted from the pile by between 5-6 mm at 132 BU's/m at Pile P30 and Pile V30. As these racking loads were less than measured in Section 7.4, this connection is considered to be less critical than the joist-to-bearer connection.

7.6 Bearer flexural failure

Figure 38 illustrates the vertical forces (compression R and tension A) transmitted to a bearer below a wall bracing element. The analysis below puts R = -A as load cells only monitor the load for positive values of A. This sub-section considers the bearer flexural bending moments and deflection of the bearer due to such forces. The bearer did not fail in flexure in the test. The bracing load at which this failure would be expected is also examined below.

Upward vertical loads at bolt hold-down locations were measured on the End 2 bearer during the racking test. The mid-span deflections of the bearer were also measured and were taken as the vertical movement from a reference stick to the bearer. The reference stick was a straight piece of timber attached at each end to the bearers at the bearer supports as shown in Figure 15.

The bearers were analysed using the computer model shown in Figure 60 using the measured uplift forces given in Table 1. In the model, the uplift forces are assumed to be at the hold-down bolt locations (i.e. at the load cells), denoted as Loc1 and Loc2 in Figure 60. The downward forces, denoted at locations Loc3 and Loc4 in Figure 60, were assumed to be at the mid-width of the end studs. The theoretical stiffnesses and strength of the bearer are:

- moment of inertia, $I = 90^4/12 = 5.47E4\ mm^4$
- elastic modulus, E = 8 GPa from NZS 3603
- ultimate moment, $M_u = 14 \times 90^3/6 \times 1.0E\text{-}6 = 1.70$ kNm.

The values subsequently measured for the bearer in a four point bending test were:

- elastic modulus = 10.5 GPa
- failure moment = 4.40 kNm (2.6 times greater than M_u calculated above).

The most critical bending moment was computed to be at Loc2 under "push" loading, where the calculated bending moment was 3.62 kNm. This is 2.13 times the design value of 1.70 kNm given above, but less than the measured flexural failure moment.

A comparison of the predicted and measured mid-span deflections is given in Table 4. It can be seen that the measured deflections are significantly lower than the computed values. This may partially be due to the bearer having some vertical movement restraint i.e. imposed on the bearer by the walls above on the end cantilevered portion.

Table 4. Computed and measured bearer deflections

Load direction	Location where force measured	Upwards force (kN)	Span A			Span B		
			Comp. (mm)	Meas. (mm)	Ratio	Comp. (mm)	Meas. (mm)	Ratio
Push	Loc2	11.15	12.88	7.93	1.62	-11.21	-7.99	1.40
Pull	Loc1	8.28	-9	-4.38	2.05	6.86	6.72	1.02

Legend:
Comp. = computed mid-span deflection using the computer model in Figure 39
Meas. = measured mid-span deflection at the same force used in the computer model.

7.7 "Systems effect factor" for walls

The maximum total applied load was 12.44 kN. As the total length of bracing walls was 1.385 m, the average maximum bracing resistance was therefore:

$$\frac{12.44\ x\ 20}{1.385} = 180 \text{ BU's/m.}$$

Winstone Wallboards Ltd (2006) gives the wind bracing resistance of the tested walls as 125 BU's/m, which indicates that the "systems effect factor" = 180/125 = 1.44. This large value is not surprising as each short bracing wall length tested had one end and the top ceiling joint fully taped and plastered.

7.8 Measured end restraint

Figure 61 plots the theoretical relationship between wall anchor force A (kN) and positive wall racking loads (kN/m) as discussed in Section 4.10.

The uplift force was measured by load cells at L26 and L27 (refer to Figure 19). This data is plotted in Figure 61 at each hysteresis 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 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 predicts a greater end reaction than measured. Hence, it is concluded that joist and bearer strength can conservatively be checked assuming ER = 9 kN.

7.9 Conclusions for Stage IV testing

1. A wall bracing rating of more than 110 BU's/m may induce pile uplift in single storey NZS 3604 construction.

2. A wall bracing rating of more than 110 BU's/m may separate a joist-bearer skew nail connection.

3. Calculations of the bending moment in the bearer based on the measured upwards loads indicated that the bearer was stressed to 2.1 times its design strength. However, the bearer did not fail and the measured bearer deflections were significantly less than predicted. The restraint on the cantilevered portion of the bearer from the side walls, and the reality that the loading was from several joists rather than a single point load at each bolt position, would have stiffened/strengthened the bearer.

4. The “systems effect factor” for this construction was 1.44.

8. CONCLUSIONS

8.1 Discussion on 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 and the small room sizes and excess of walls of many houses built in the past. The natural periods of most houses are well below the critical period of 0.4 seconds assumed by NZS 3604 (SNZ 1999) 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, this historical data does suggest that it is inappropriate to apply criteria which is too conservative as a measure to avoid foundation failures.

Recommending a maximum bracing rating that may be used from 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 occurred which effectively stopped any greater bracing rating being achieved.

With regard to avoiding joist or bearer flexural failure (considered to be a more undesirable and brittle failure mechanism) it is recommended that the 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 joist and bearer flexural failure based on these recommendations.

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

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

8.2 Criteria derived from test observations of ductile failure mechanisms

Results from Stage II testing were not used as these were considered to be too conservative for reasons discussed in Section 5. However, from the ductile failure mechanisms observed in other tests it is hereby concluded that:

1. A wall bracing rating of more than 100 BU’s/m may induce pile uplift in single storey NZS 3604 construction. As this is not an undesirable mechanism it is proposed that this not be used to limit the maximum wall bracing rating.

2. A wall bracing rating of more than 110 BU’s/m may separate joist-bearer skew nail connections.

3. A pile-bearer connection of two “wire dogs” and two skew nails is likely to be less critical than the joist-bearer connection. However, it is recommended that the current option of using four skew nails be removed from the standard based on tests by Thurston (2007a).

4. Blocking-to-joist connections as specified in NZS 3604 are likely to be less critical than the joist-bearer connections.

8.3 Other conclusions

1. 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 the model shown in

2. Figure 17, with ER = ER' = 9 kN. This is used with a "systems effect factor" of 1.0 as is assumed in the method used to derive the effective values of ER/ER' in the various sections of this report.

3. For construction with double joists on a building exterior and single joists subsequently located at 600 mm centres, it is recommended that 8% of the vertical load on the joist may be assumed to be redistributed to adjacent joists. However, for internal walls supported by single joists, and at right angles to the joists, it was concluded the vertical forces directed to one joist are in fact shared between 1.7 joists, although there was some evidence that this value may be too high.

4. The "systems effect factor" for wall strength varied considerably in the different tests. Stage III testing clearly showed that paper-taped and stopped joints at wall corners and ceilings altered the resistance mechanism of the bracing walls and allowed increases in strength of over 100% to occur i.e. "systems effect factor" greater than 2.0. On the other hand, Stage I testing gave a "systems effect factor" of only 1.0. This low value was attributable to one of the wall elements taking a disproportionate share of the applied load as it was located on a stiffer foundation than the other wall elements. The "systems effect factor" in Stage II testing was 1.18, although this was noted to be conservative. In Stage IV it was 1.44. Based on this work and that by Thurston 1993, 2002 and 2006 it is concluded that a "systems effect factor" of 1.2 is appropriate.

5. Wall deflections increase considerably due to foundation flexibility. Thus, walls founded on the more rigid parts of the foundation may consequently carry a disproportionate share of the racking load.

6. As a bracing wall "rocks" it will attract a resistive axial load at the uplift corner. This is likely to be greater at wall returns where plastered joints give continuity at the corner. This will reduce bracing wall anchor hold-down forces, but not the compressive reaction at the other end of the bracing wall. Therefore, the required joist and bearer strength will not be reduced by this action. However, where the resistive axial load is greater than in the construction tested the failures summarised in Section 8.2 will occur at greater bracing loads.

9. REFERENCES

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Figure 1. General view of foundation framing.

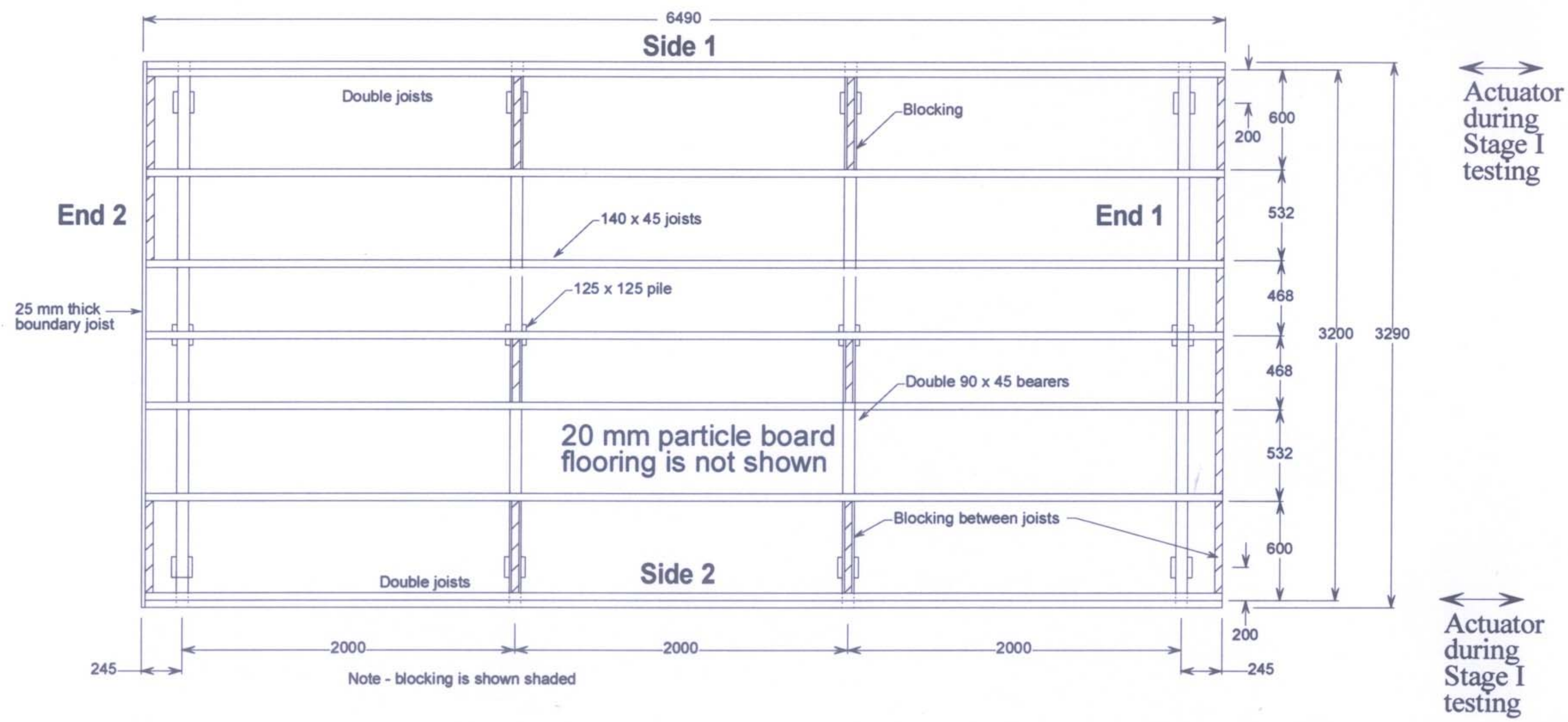


Figure 2. Plan view of floor framing.

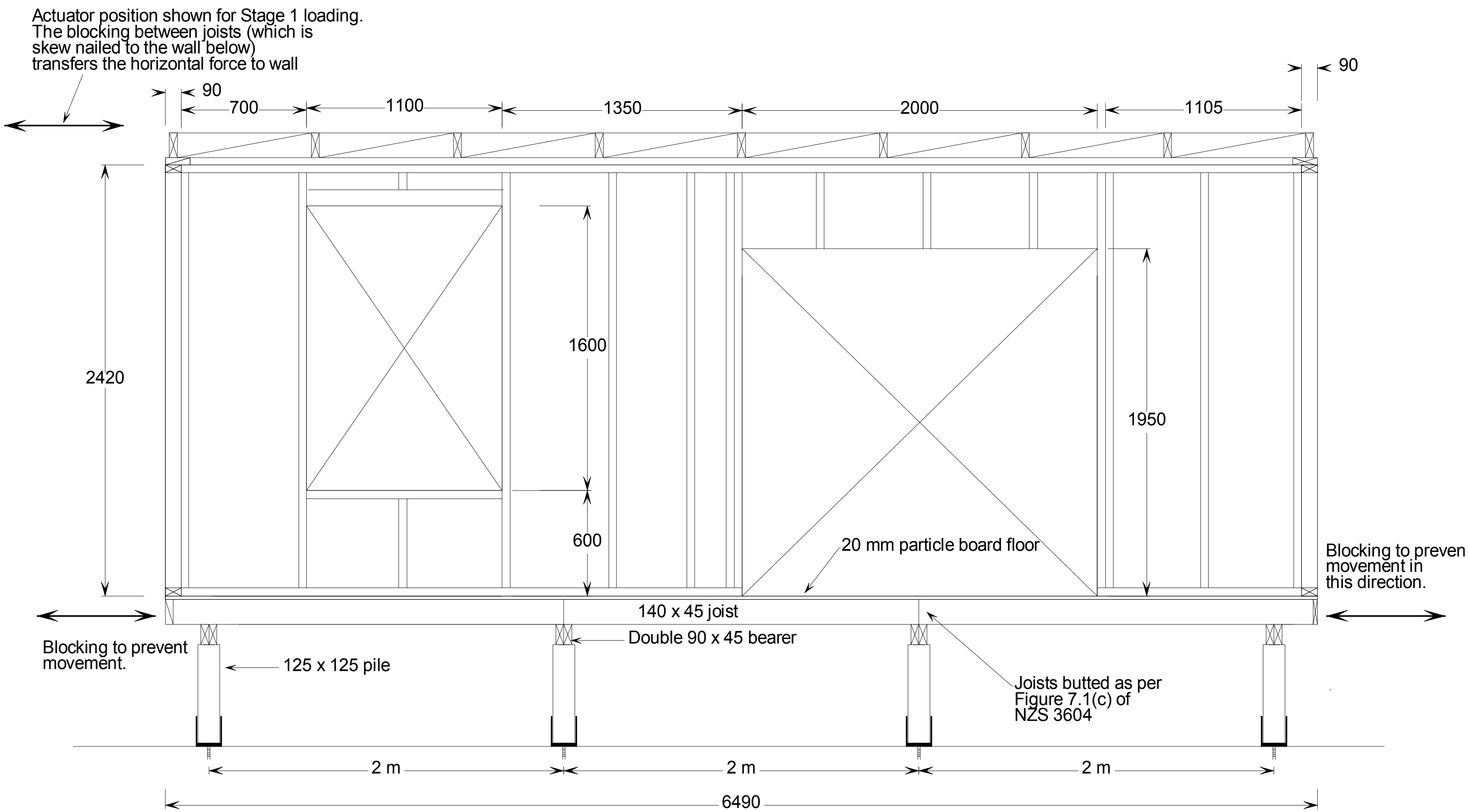


Figure 3. Side 1 framing

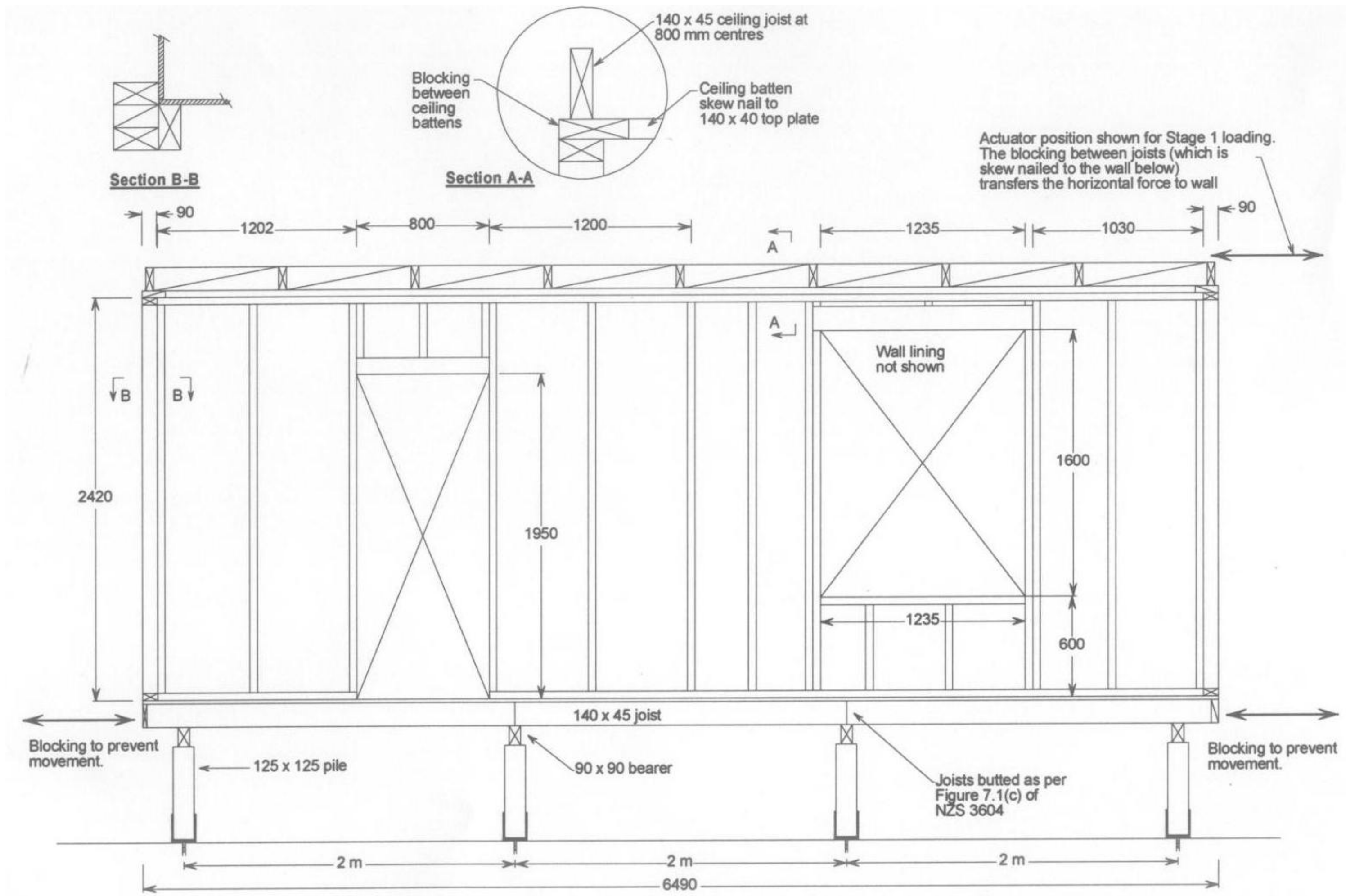


Figure 4. Side 2 framing.

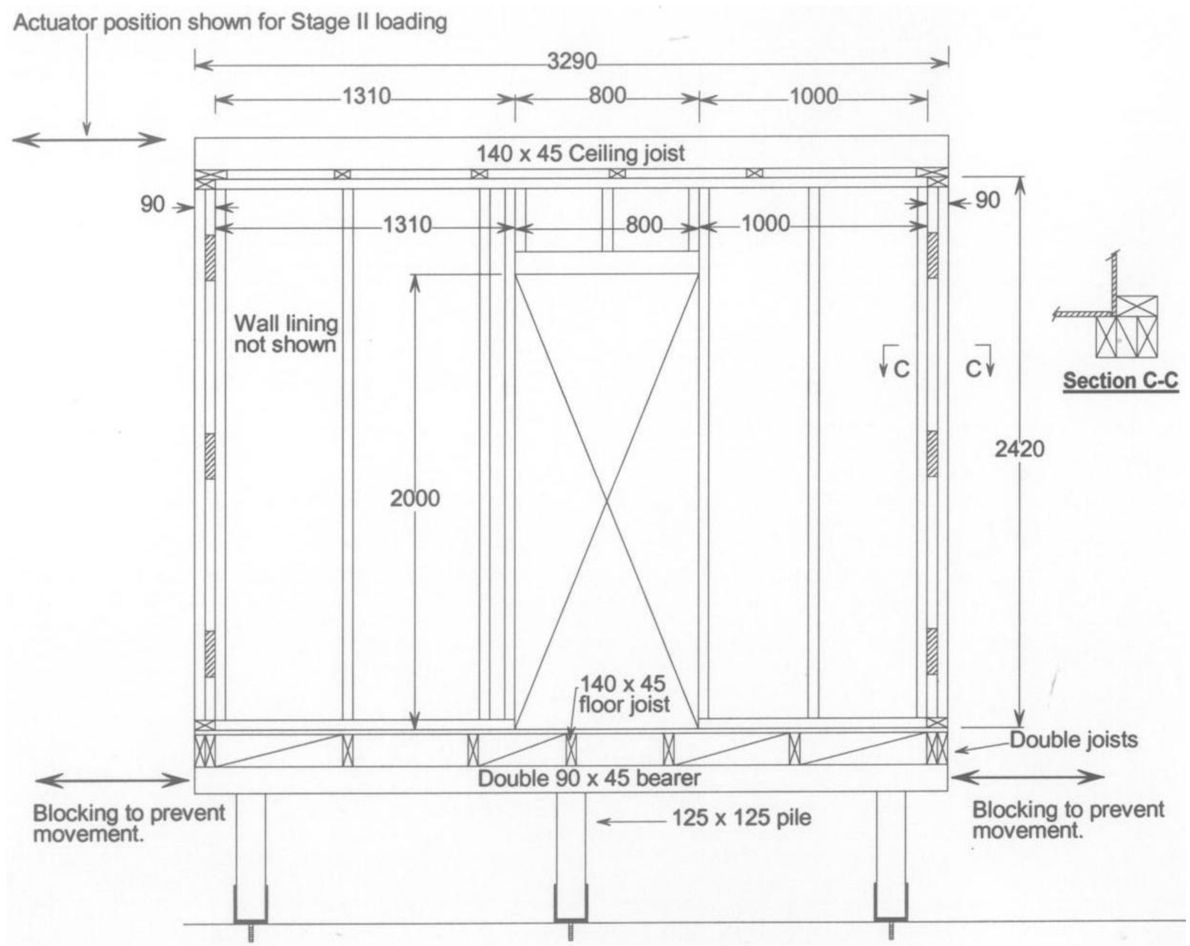


Figure 5. End 1 framing.

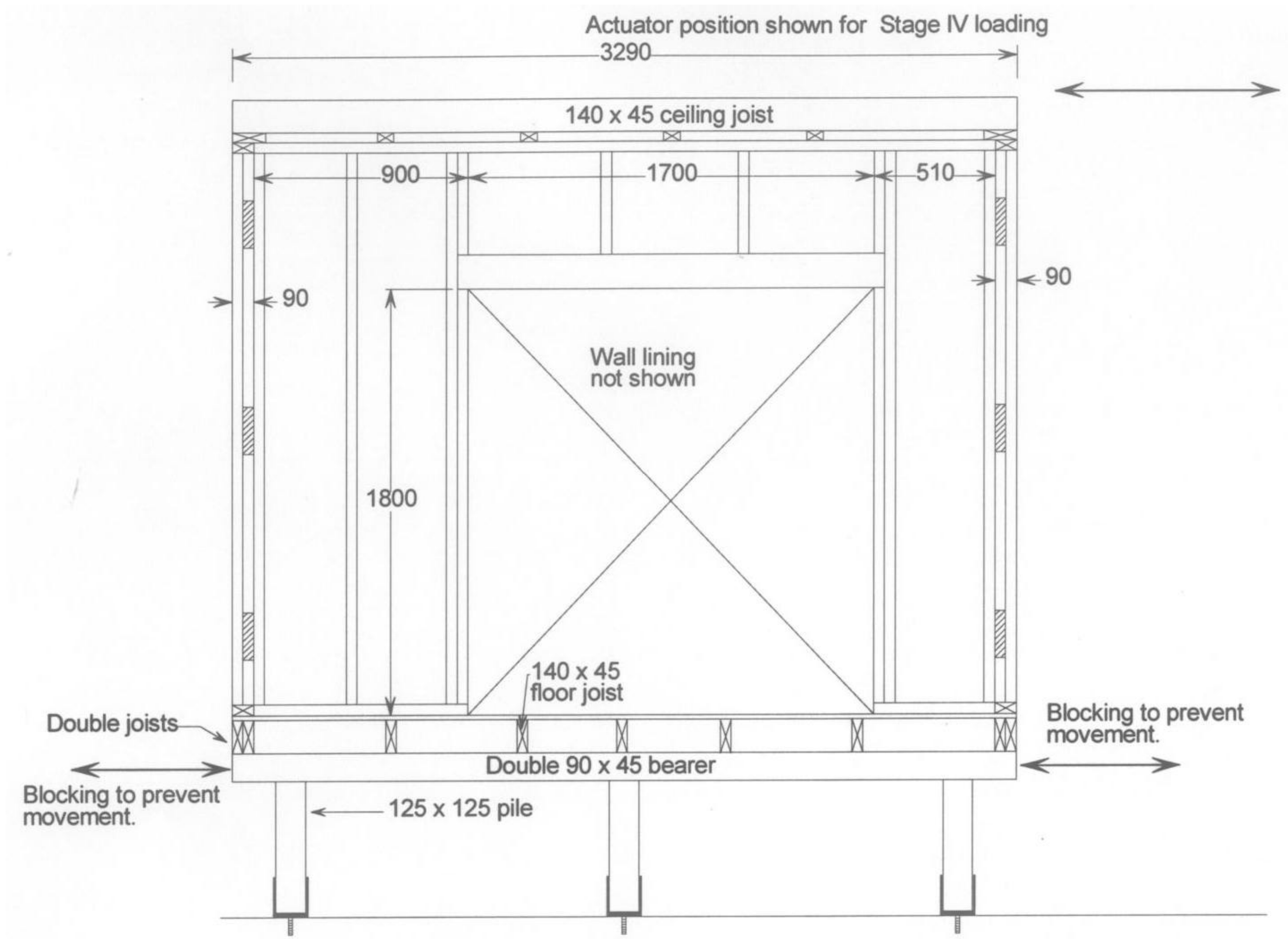


Figure 6. End 2 framing.

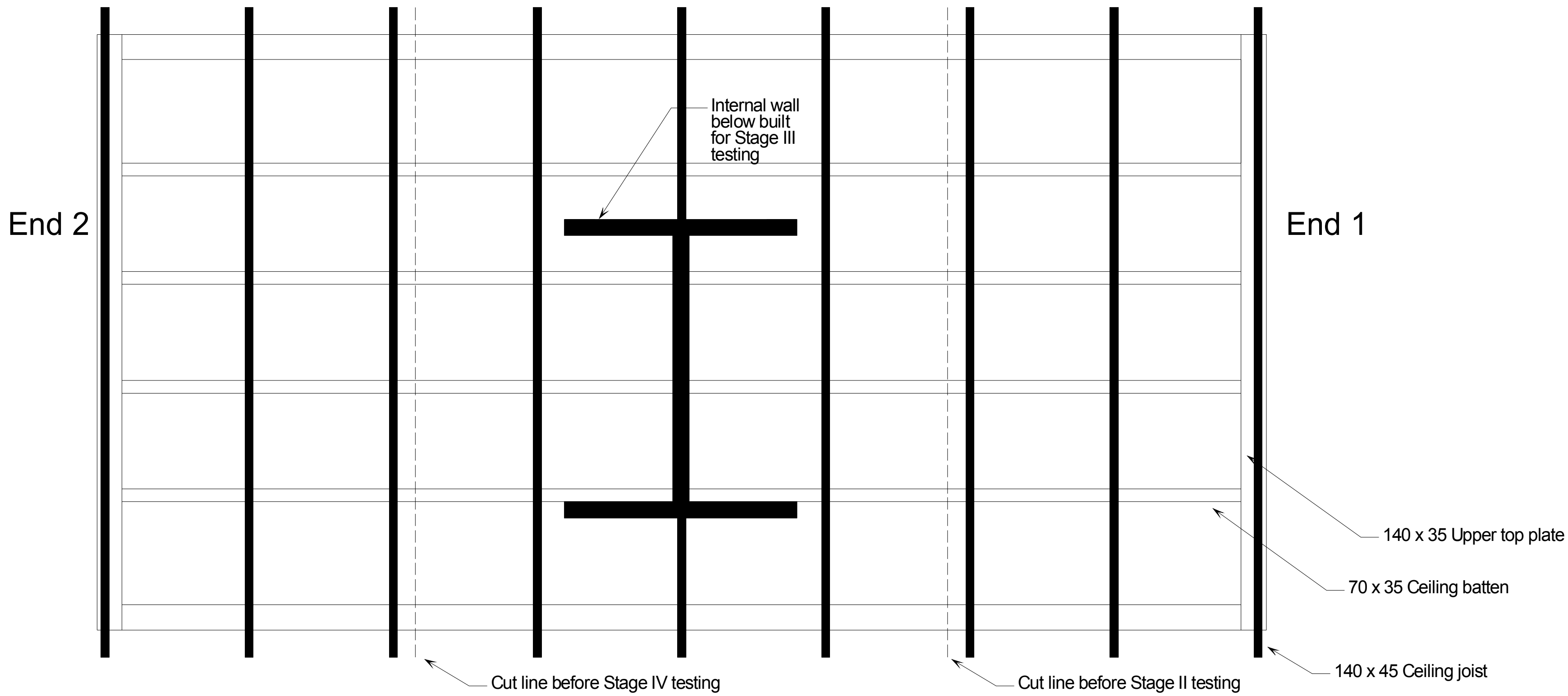


Figure 7. Ceiling framing.

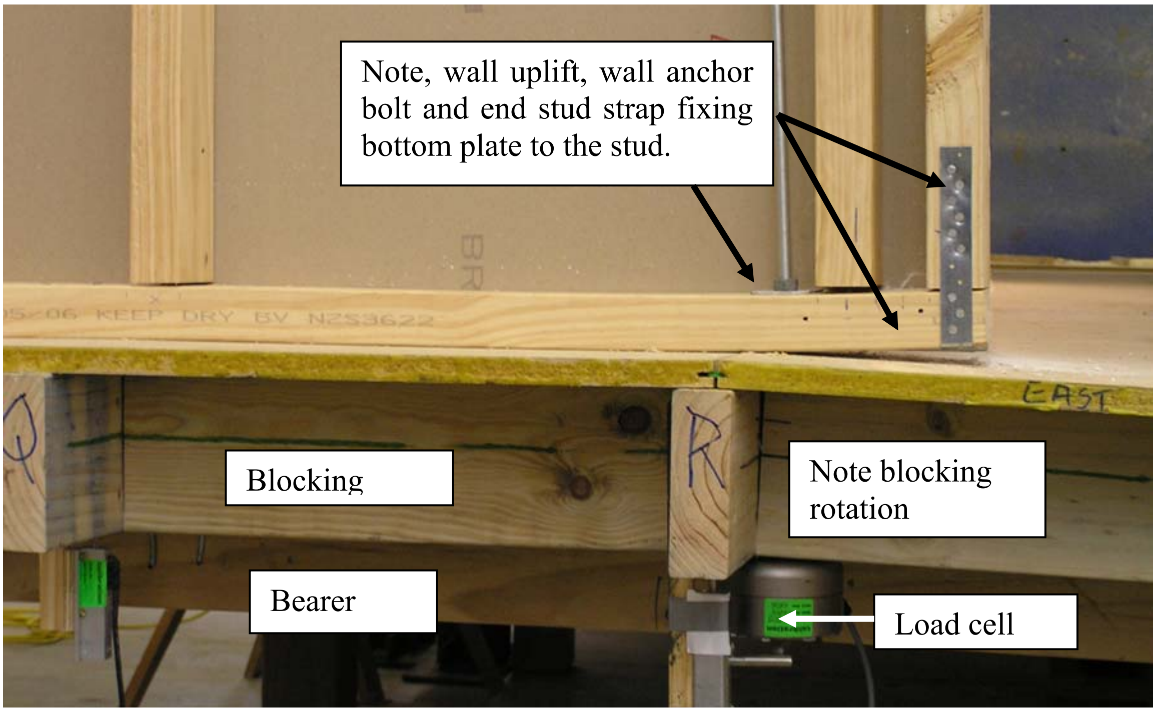


Figure 8. Wall uplift from floor during pull load in Stage II.

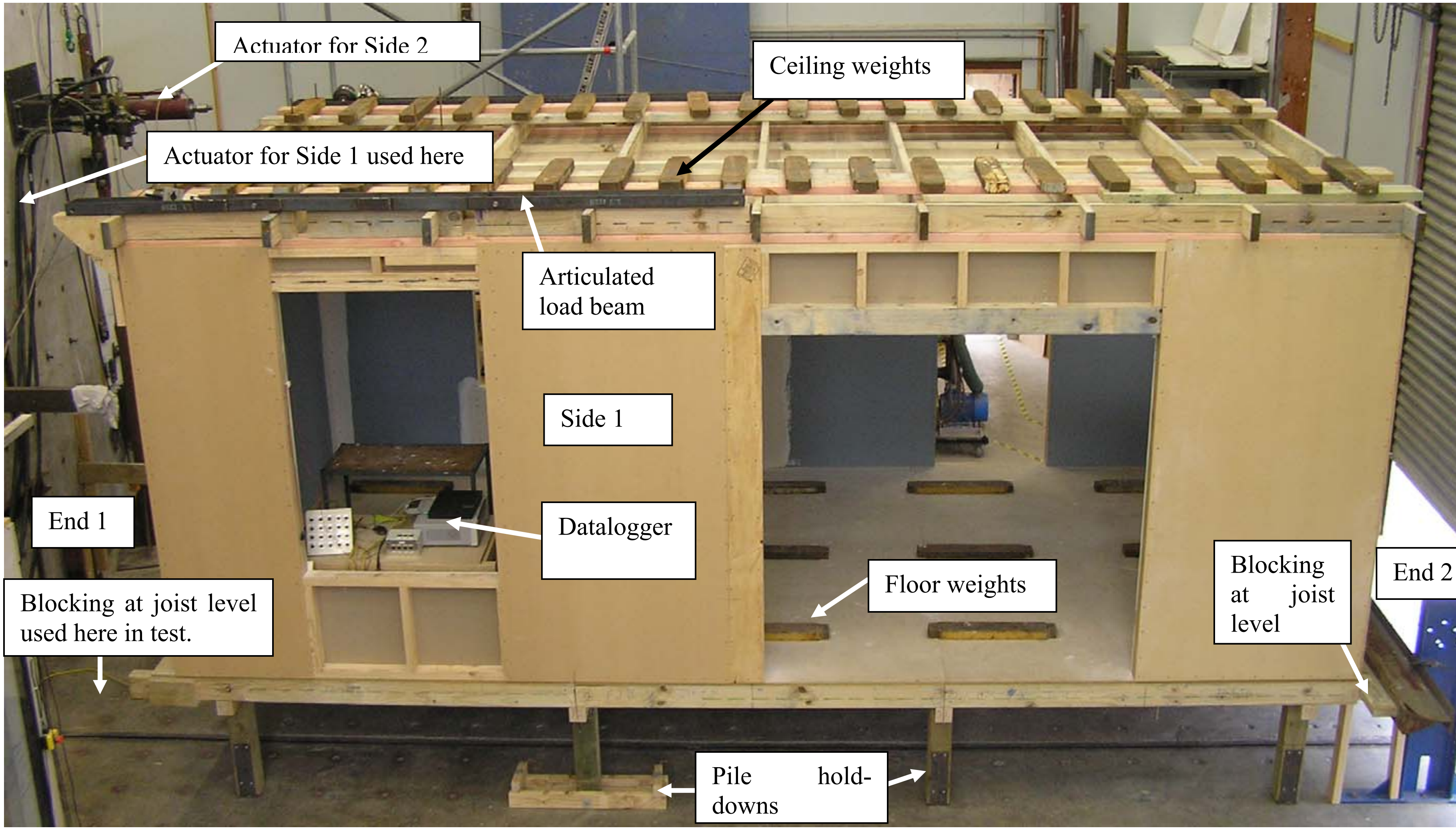


Figure 9. Set-up for Stage I testing.

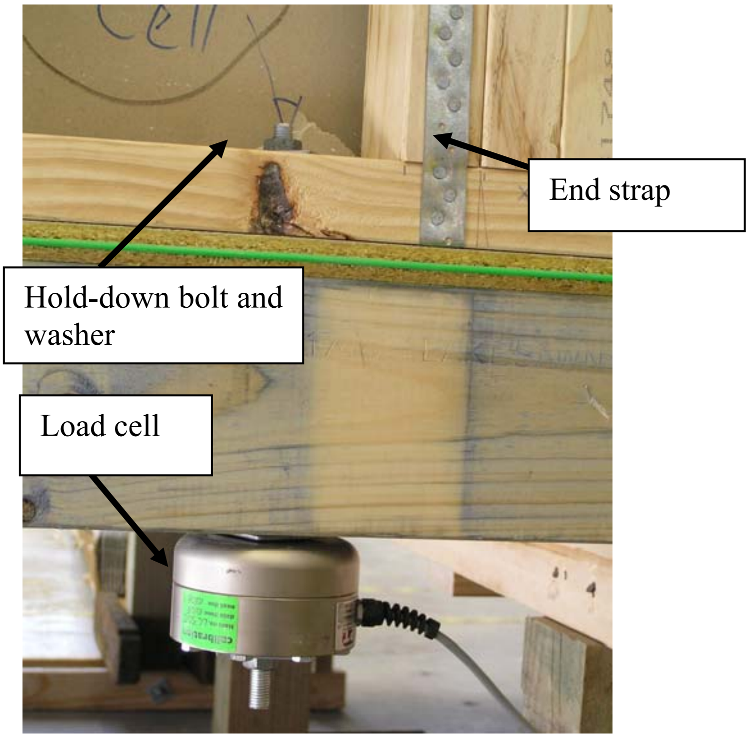


Figure 10. Load cell to measure wall anchor force in Stage I.

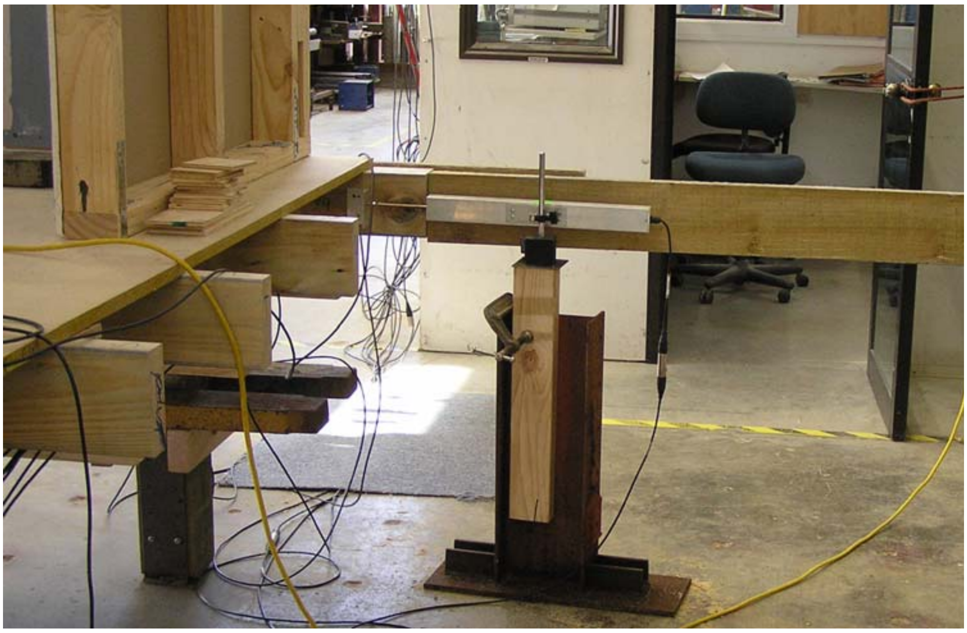


Figure 11. Joist blocking at End 1 in Stage I.

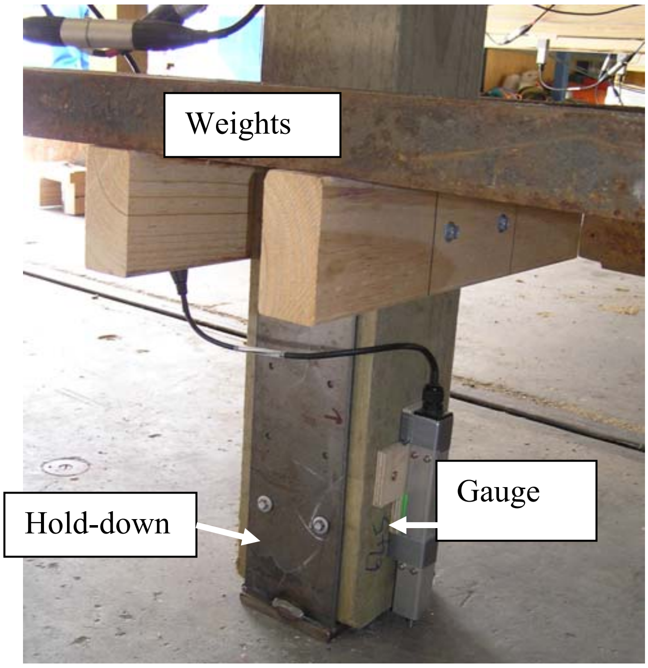


Figure 12. Pile uplift measurements, pile weights and pile hold-down in Stage I.

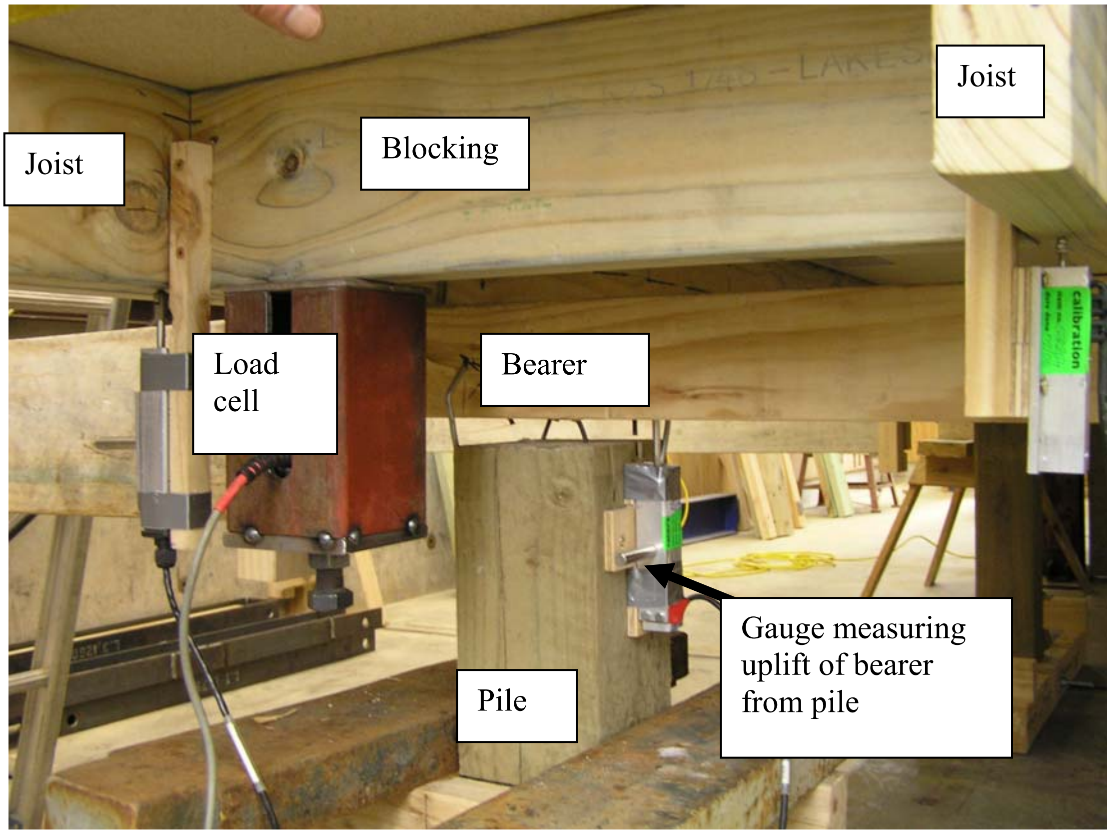


Figure 13. Bearer uplift from the piles during Stage II testing.

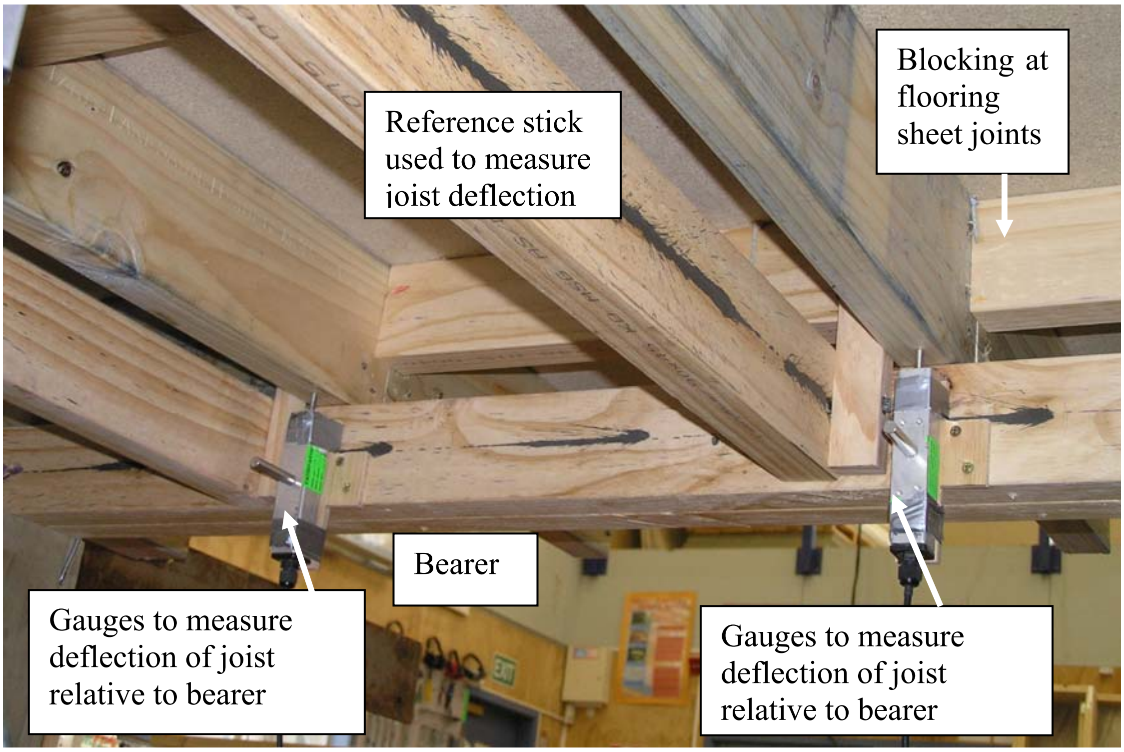


Figure 14. Measurement of vertical deflection of joist relative to bearer during Stage IV.

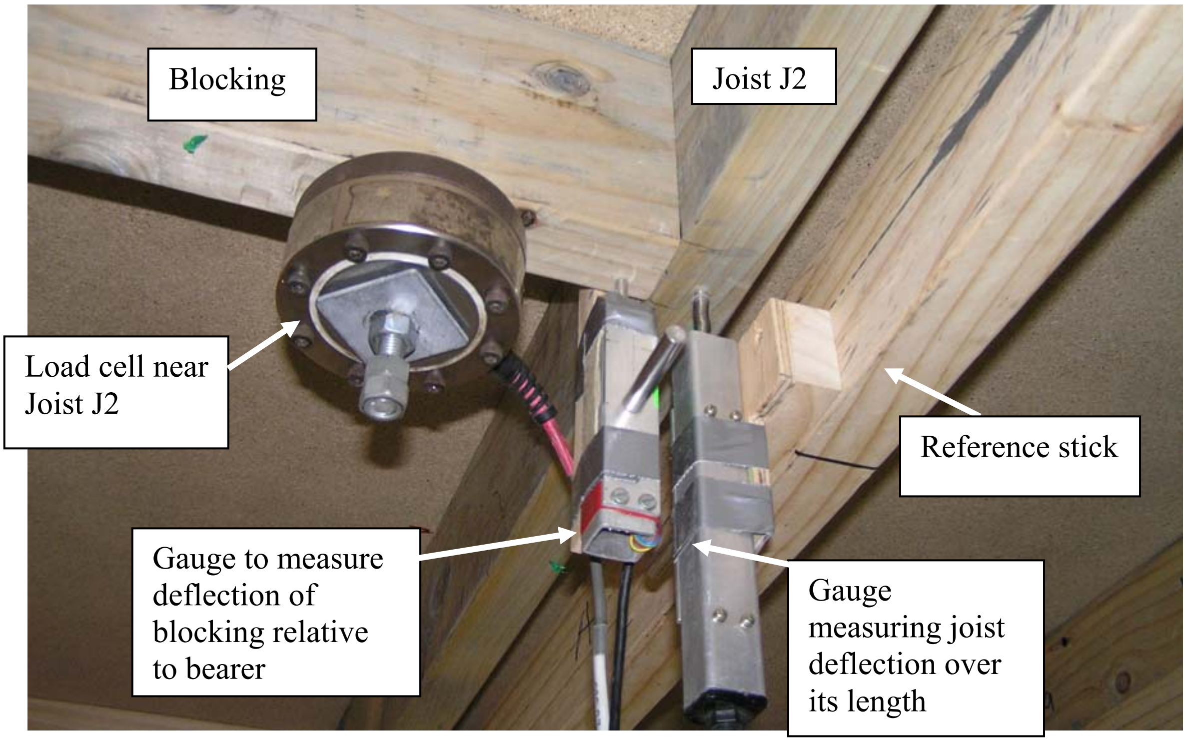


Figure 15. Foundation deflection measurements at Joist J2 during Stage III testing.

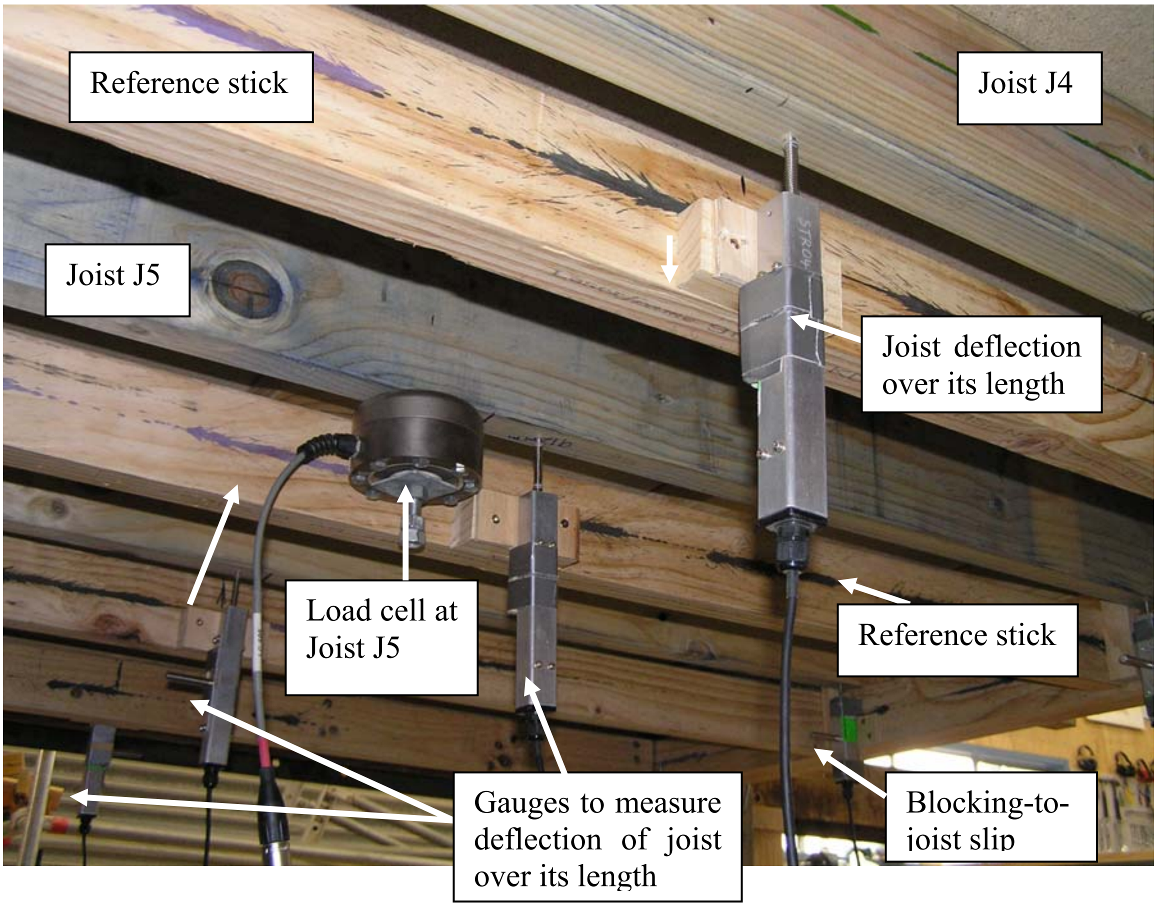


Figure 16. Gauges measuring joist deflection during Stage III testing.

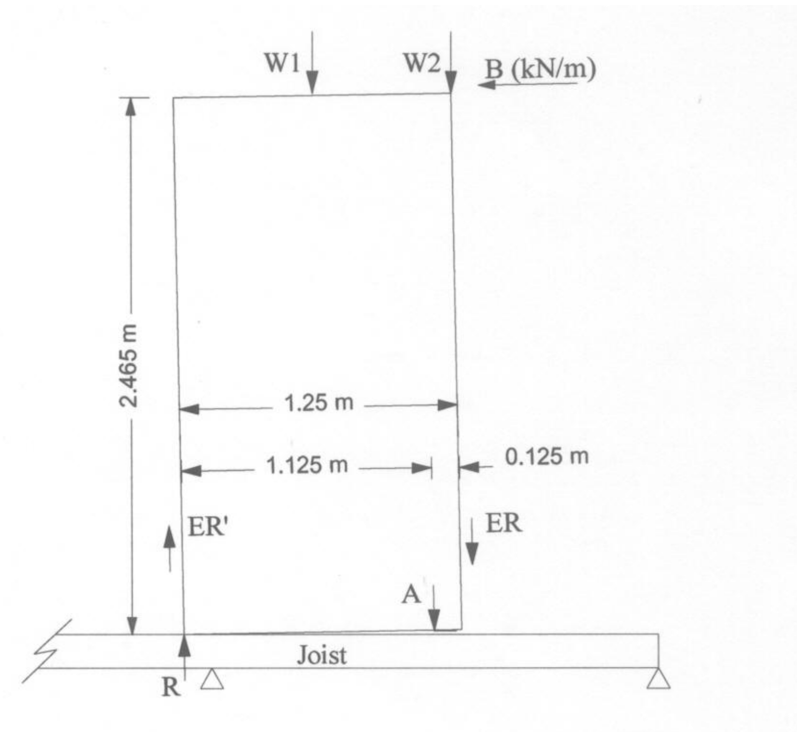
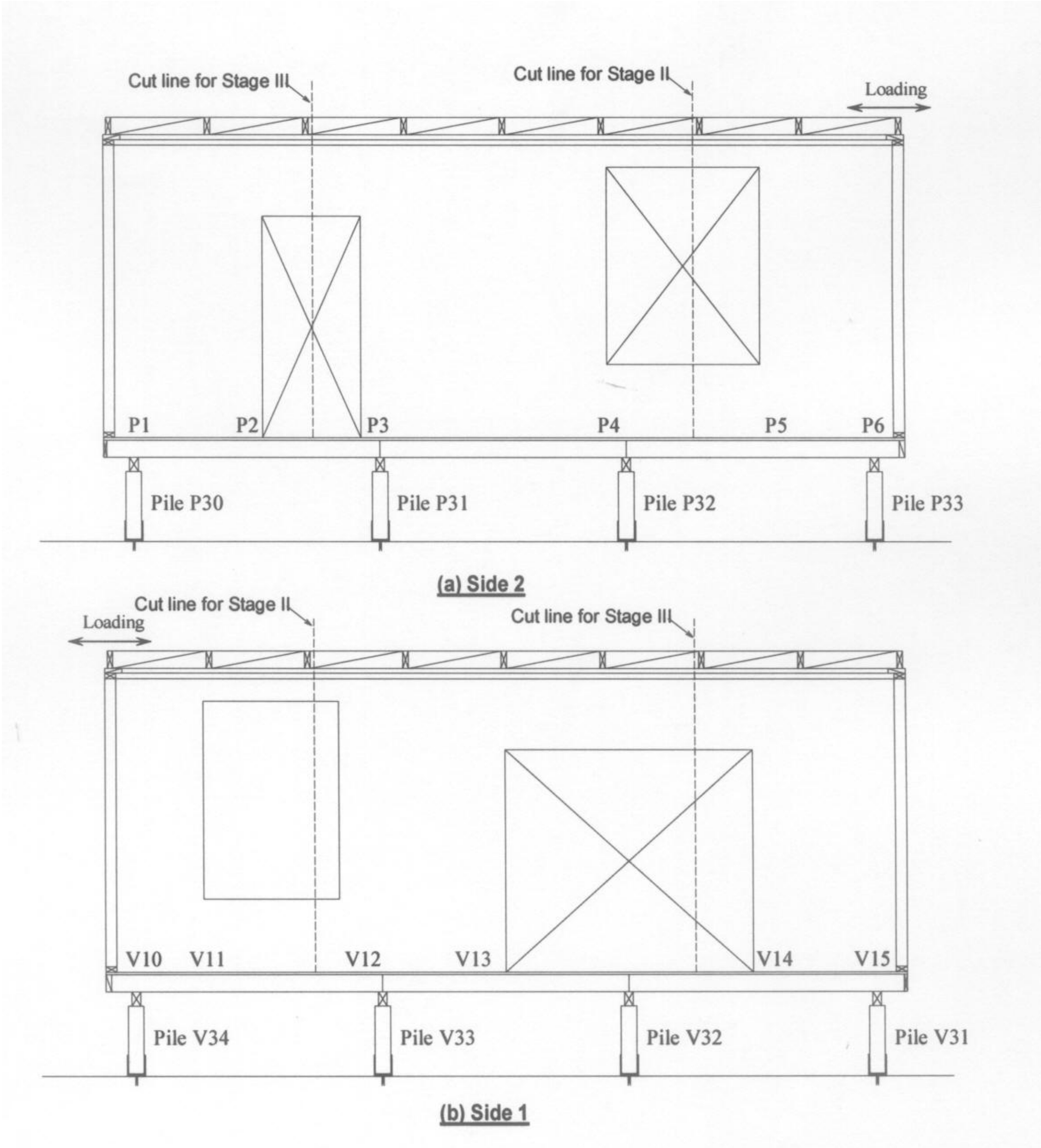
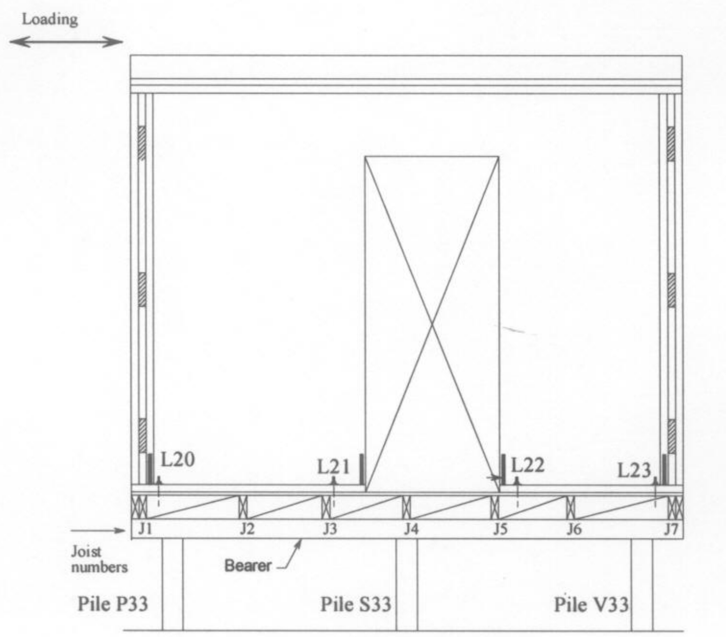


Figure 17. Forces on a bracing wall.

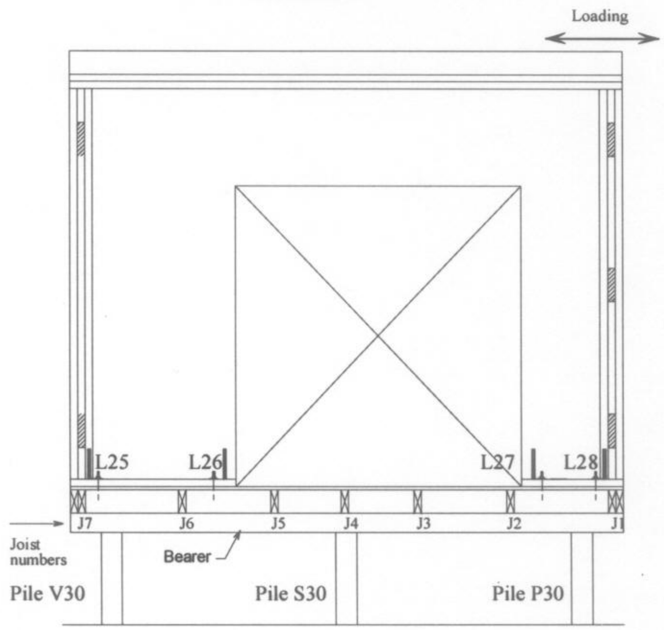


<u>Nomenclature:</u> The length of wall between wall openings or corners is referred to as a wall bracing element. Thus, $W_{3\text{-}4}$ refers to the wall bracing element between locations "P3" and "P4" shown on the drawing above.

Figure 18. Elevations of side walls showing nomenclature.



(a) End 1



(b) End 2

Nomenclature: The length of wall between wall openings or corners is referred to as a wall bracing element. Thus, $W_{20\text{-}21}$ refers to the wall bracing element between locations "L20" and "L21" shown on the drawing above.

Figure 19. Elevations of end walls showing nomenclature.

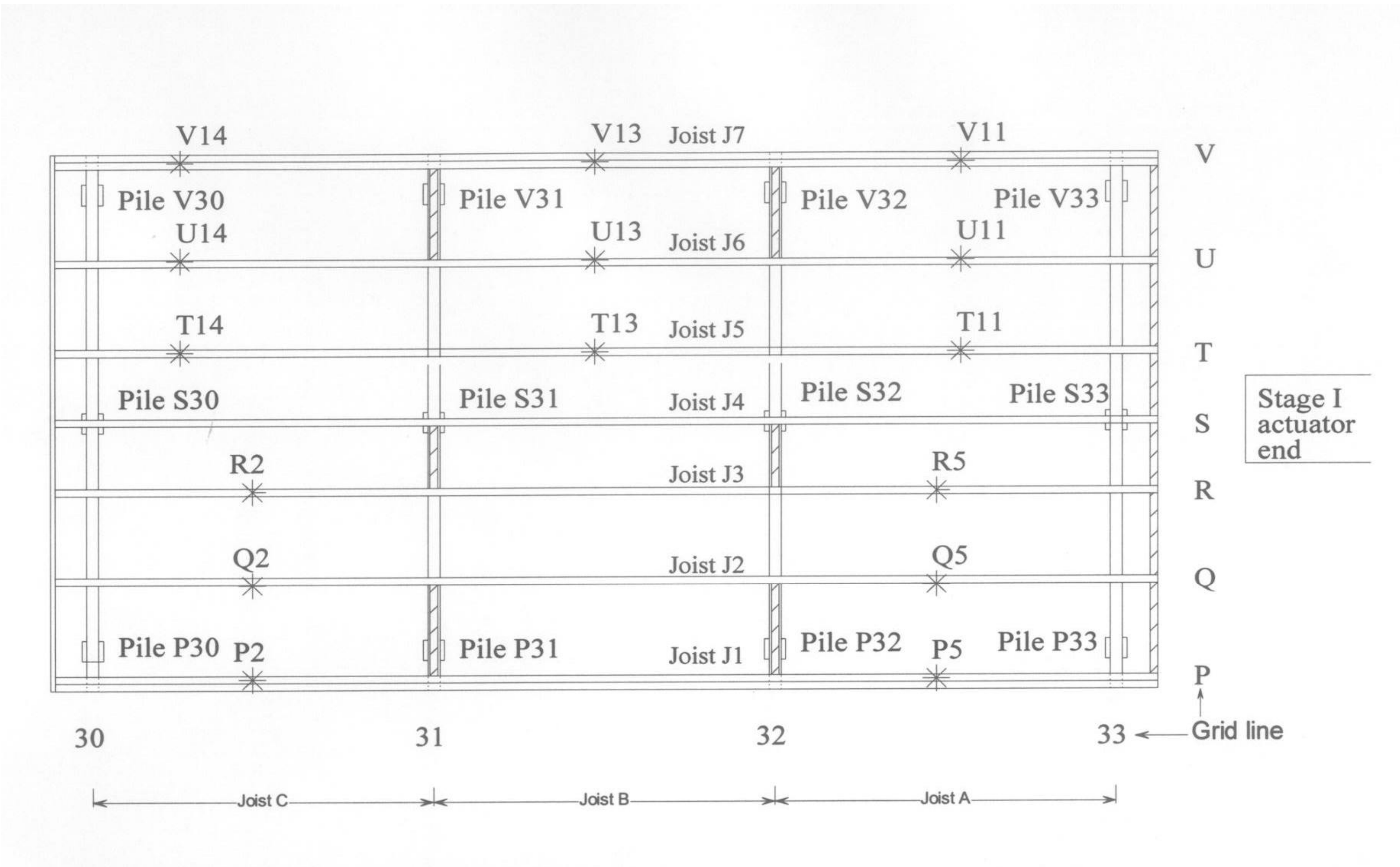


Figure 20. Plan view of test room showing locations of joist deflection measurements and nomenclature.

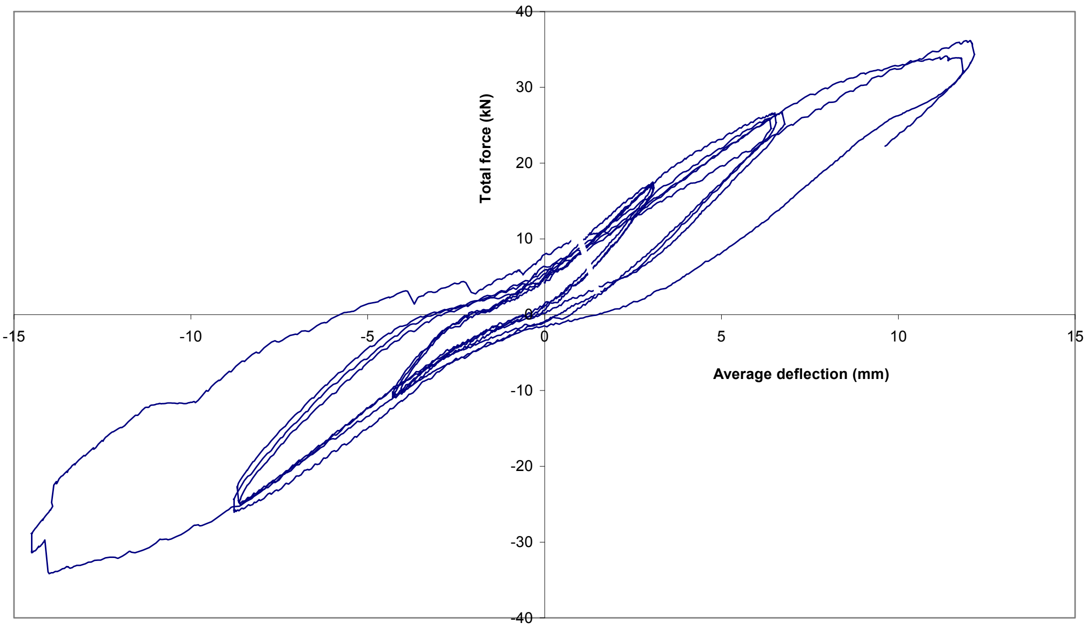


Figure 21.Total racking force versus average side wall deflection for Stage I Test 1.

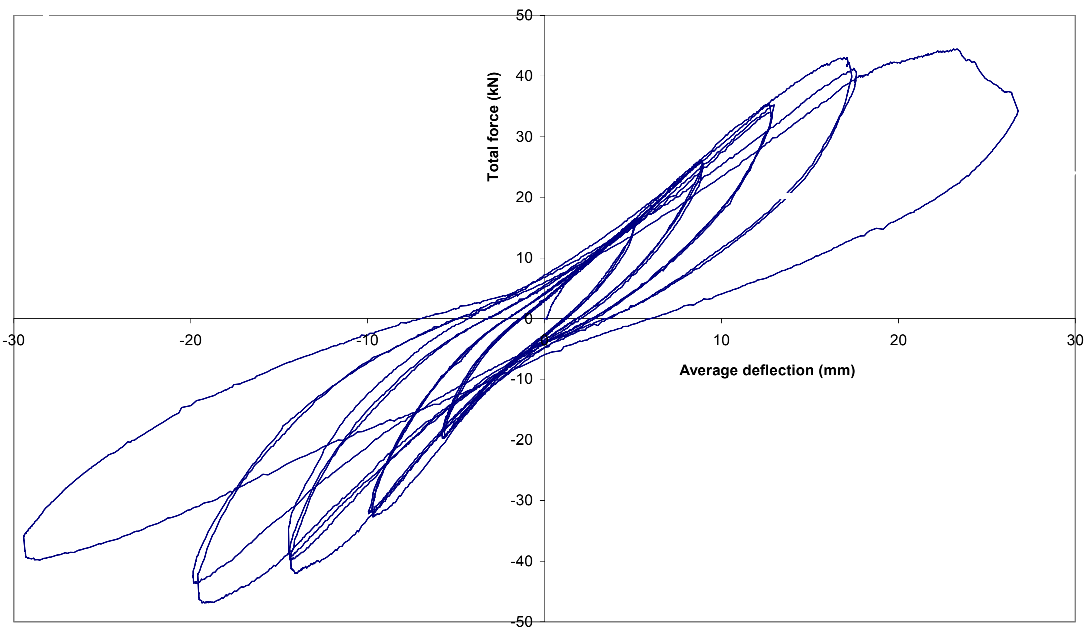


Figure 22. Total racking force versus average side wall deflection for Stage I Test 2.

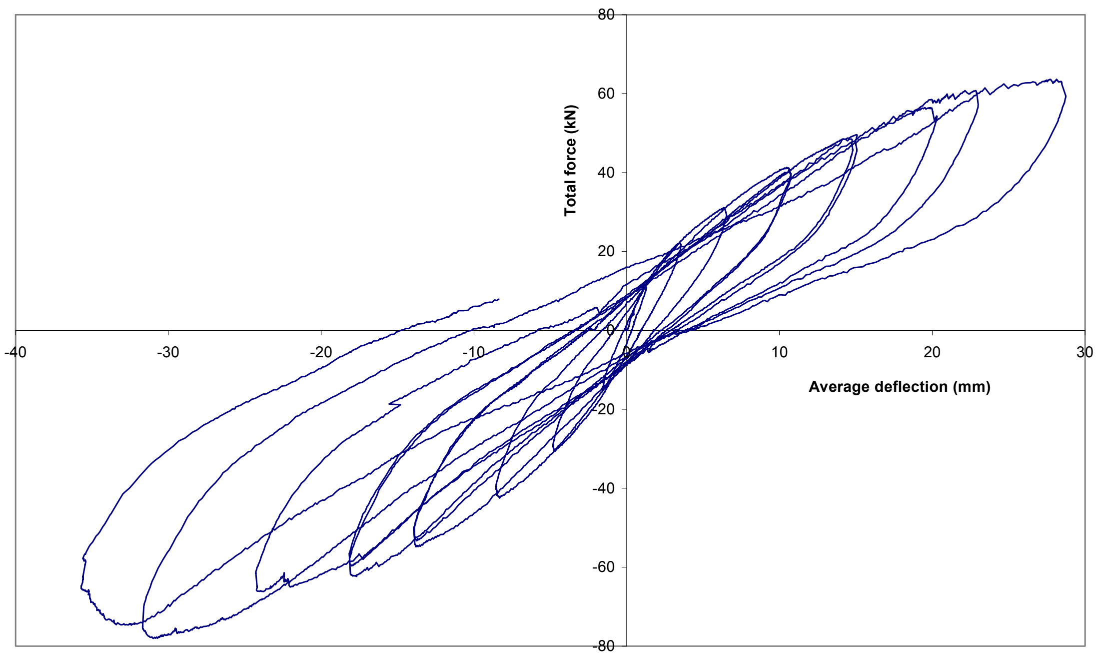


Figure 23. Total racking force versus average side wall deflection for Stage I Test 3.

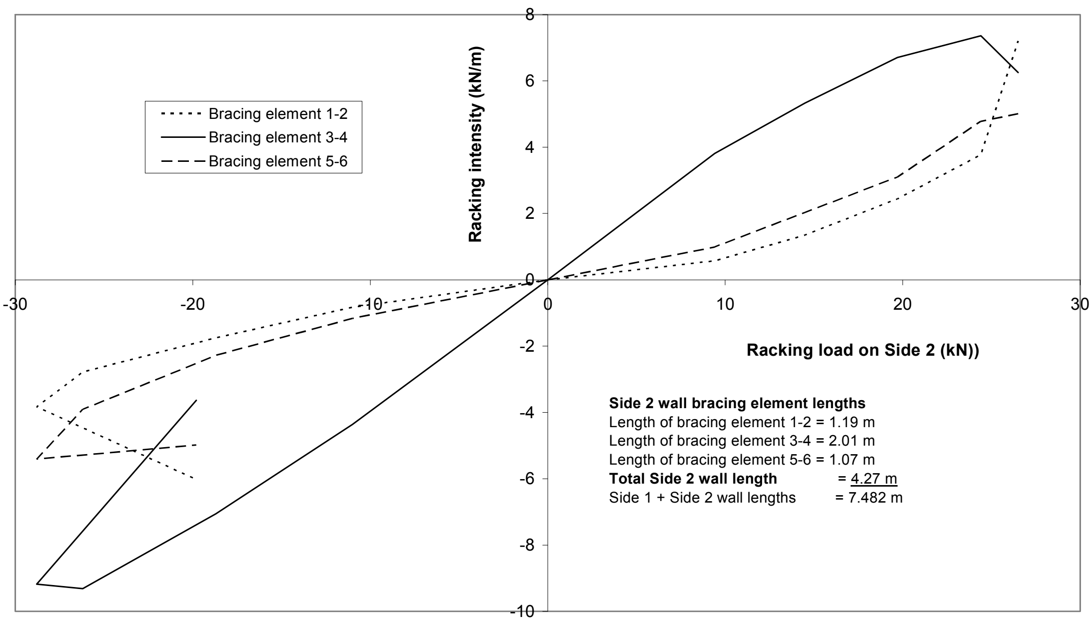


Figure 24. Racking loads in wall bracing elements of Side 2 for Stage I Test 2.

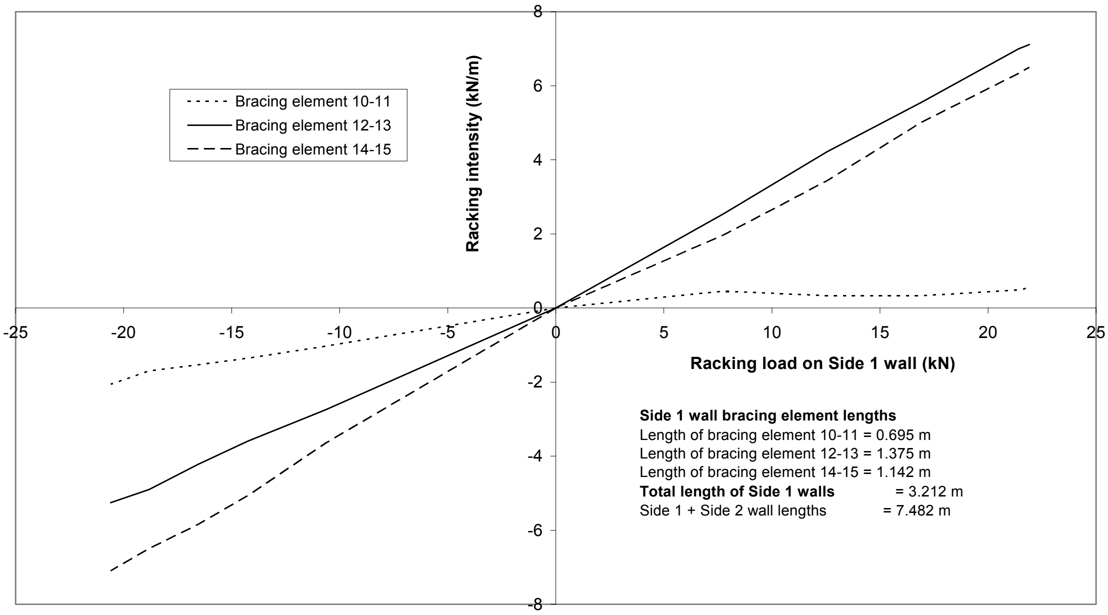


Figure 25. Racking loads in wall bracing elements of Side 1 for Stage I Test 2.

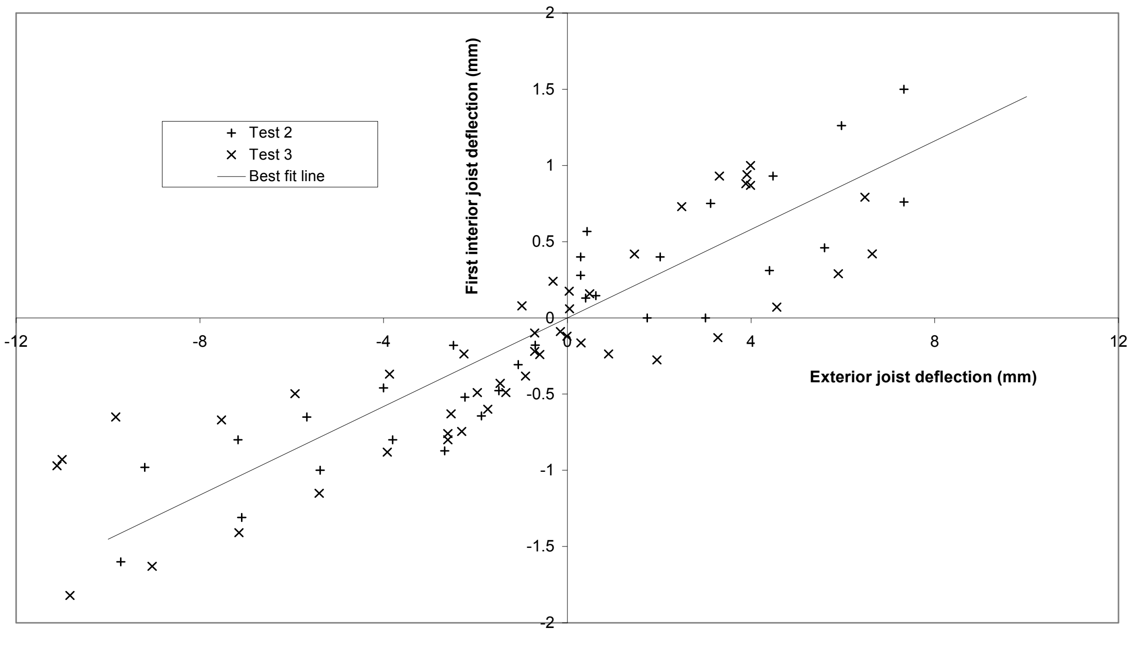


Figure 26. Deflection of first interior joist versus deflection of exterior joist for Stage I Test 2 and Test 3.

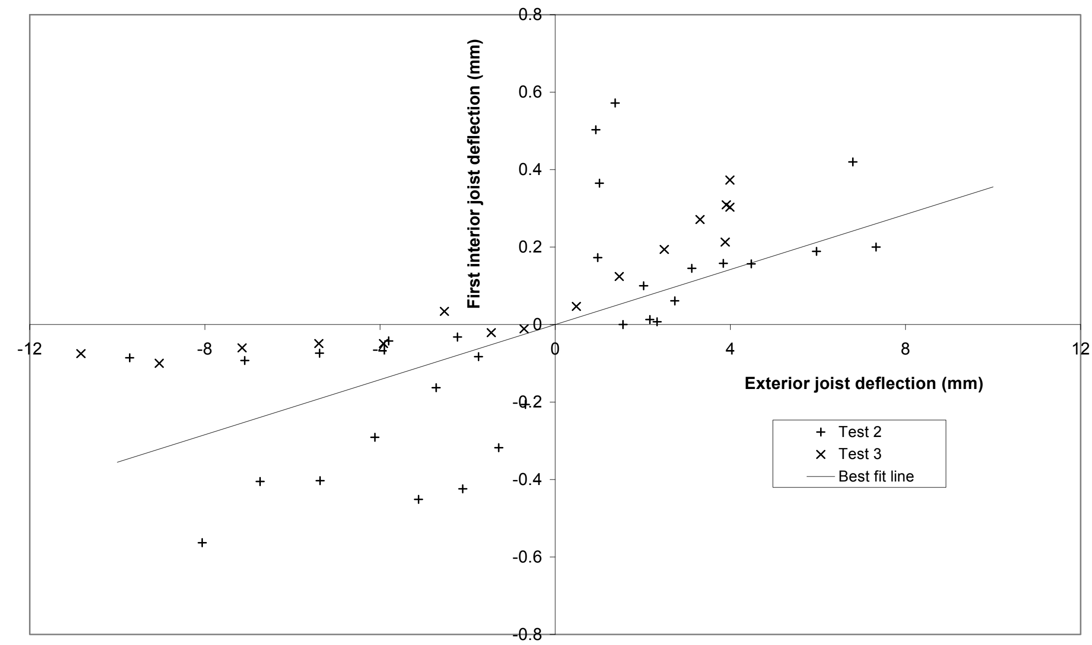


Figure 27. Deflection of second interior joist versus deflection of exterior joist for Stage I Test 2 and Test 3.

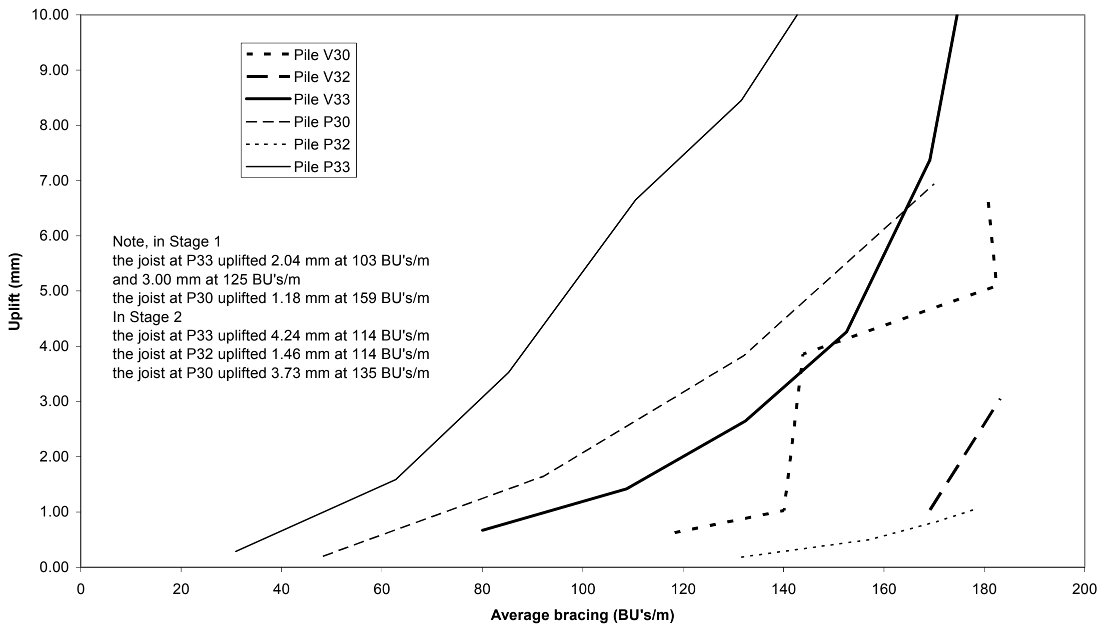


Figure 28. Joist uplift from bearer versus bracing load for Stage I Test 3.

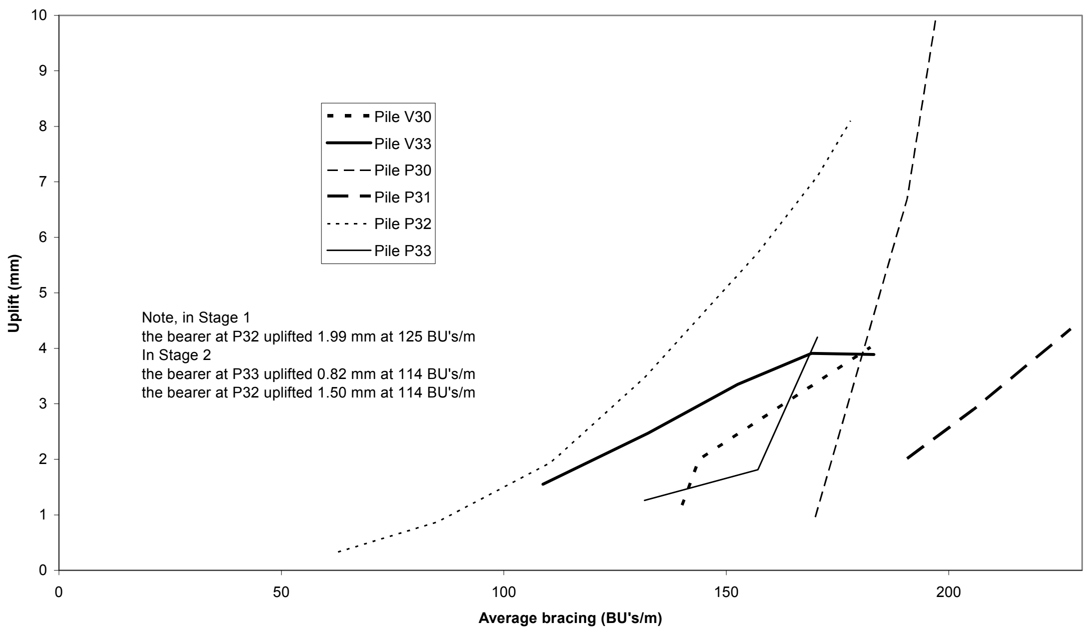


Figure 29. Bearer uplift from pile versus bracing load for Stage I Test 3.

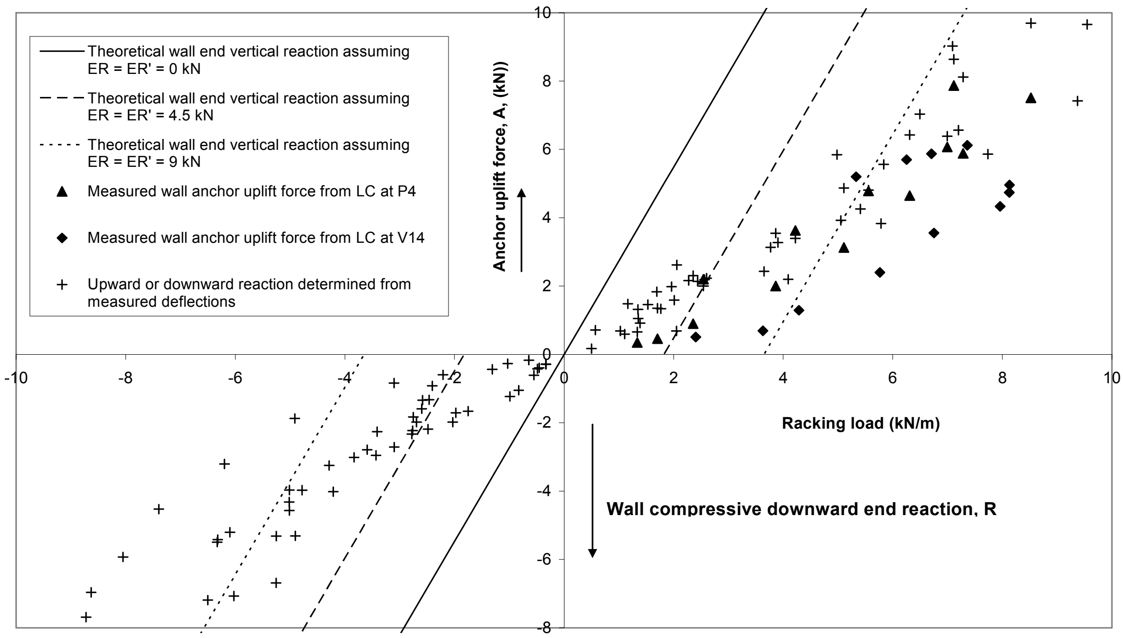


Figure 30. Comparison of theoretical and measured uplift forces on bracing walls for Stage I.

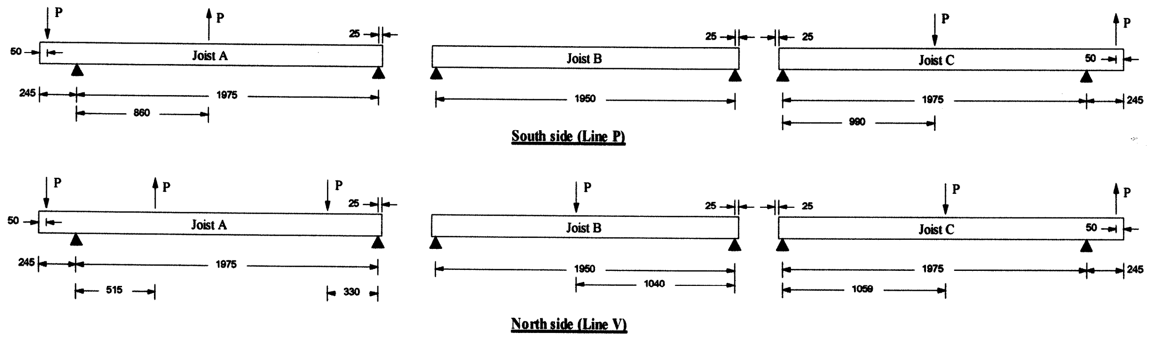


Figure 31. Models for calculation of joist deflection for Stage I.

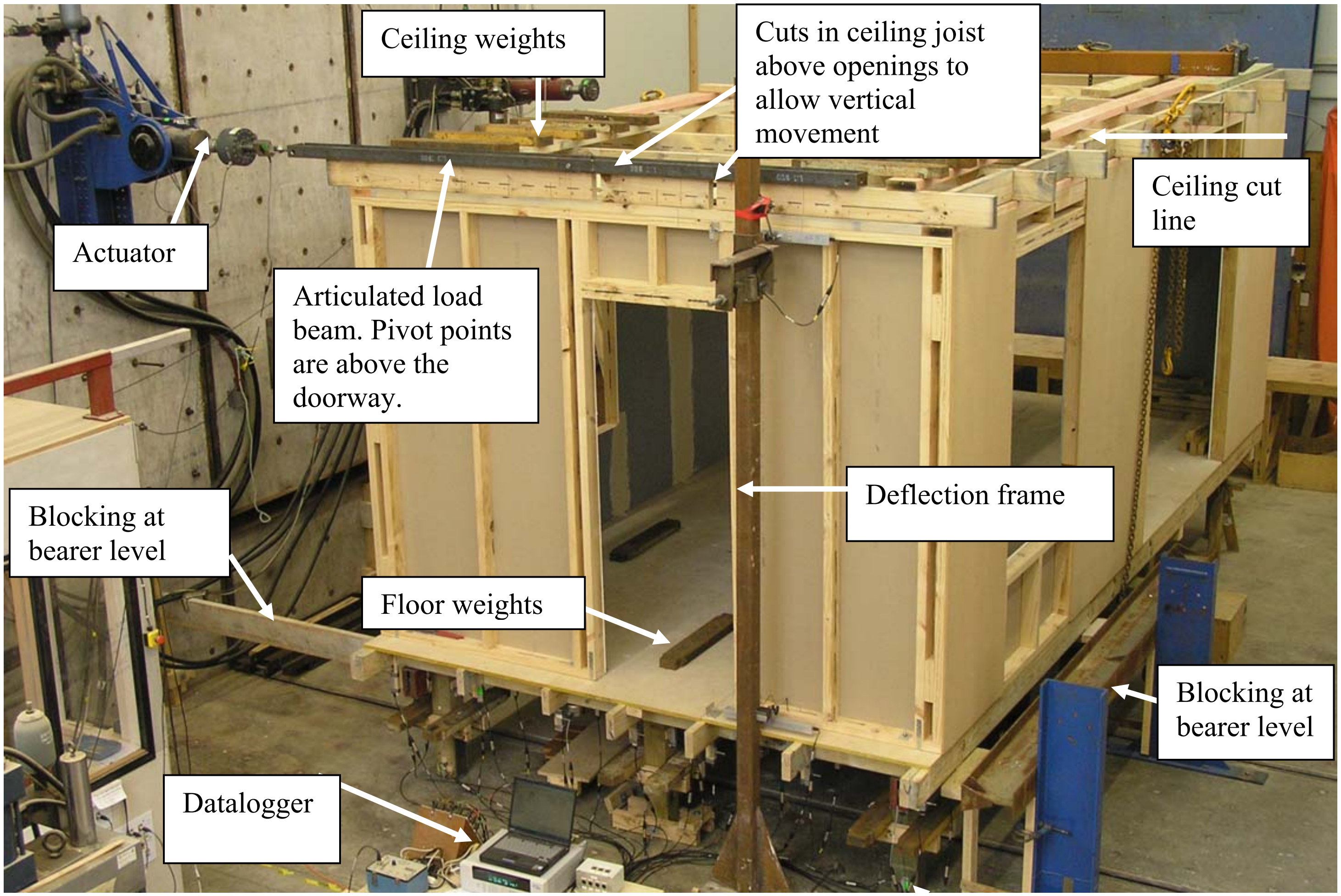


Figure 32. Set-up for Stage II testing. Racking test of End 1.

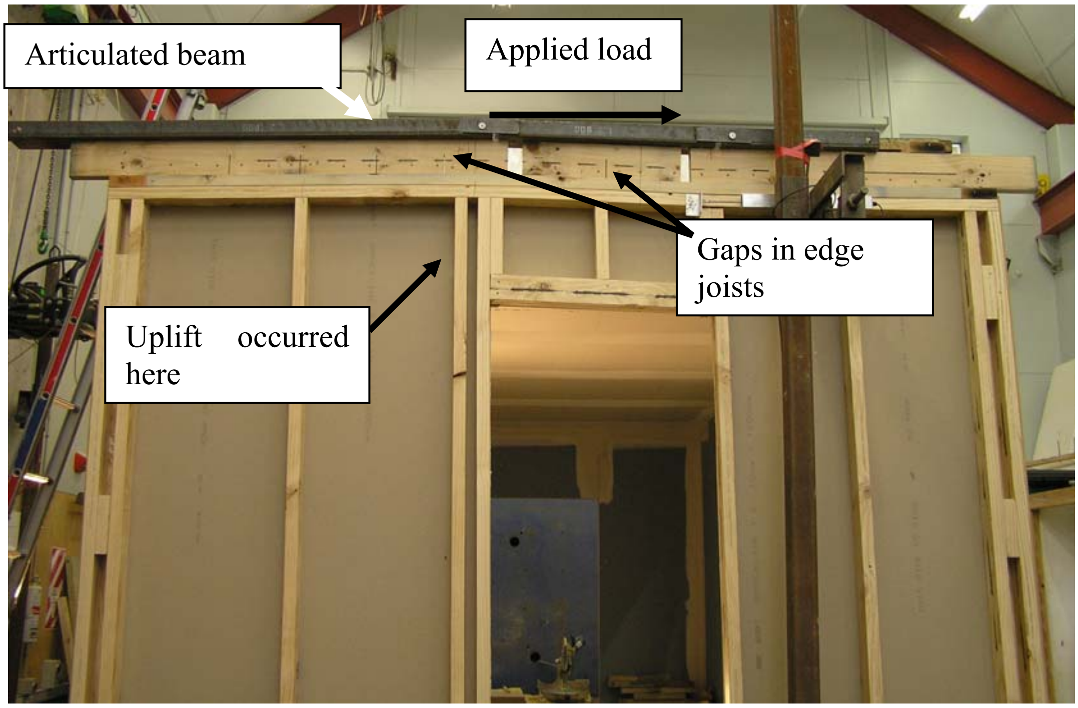


Figure 33. Deformation of wall due to wall uplift at End 1 during Stage II.

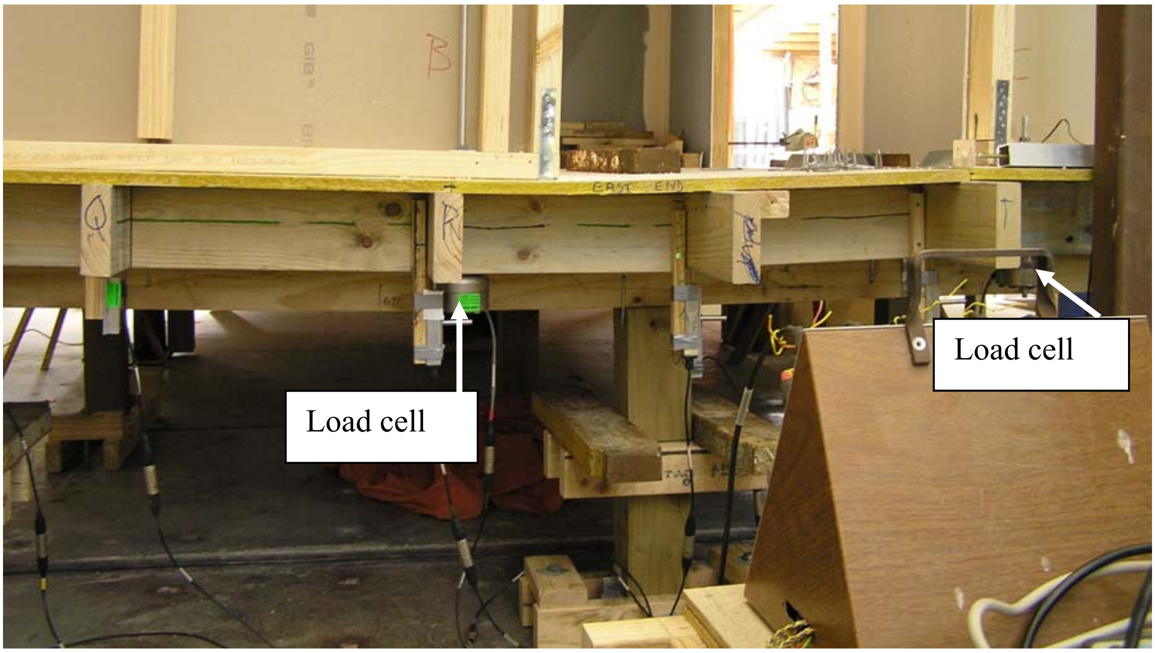


Figure 34. Deformation of floor and blocking at End 1 during Stage II.

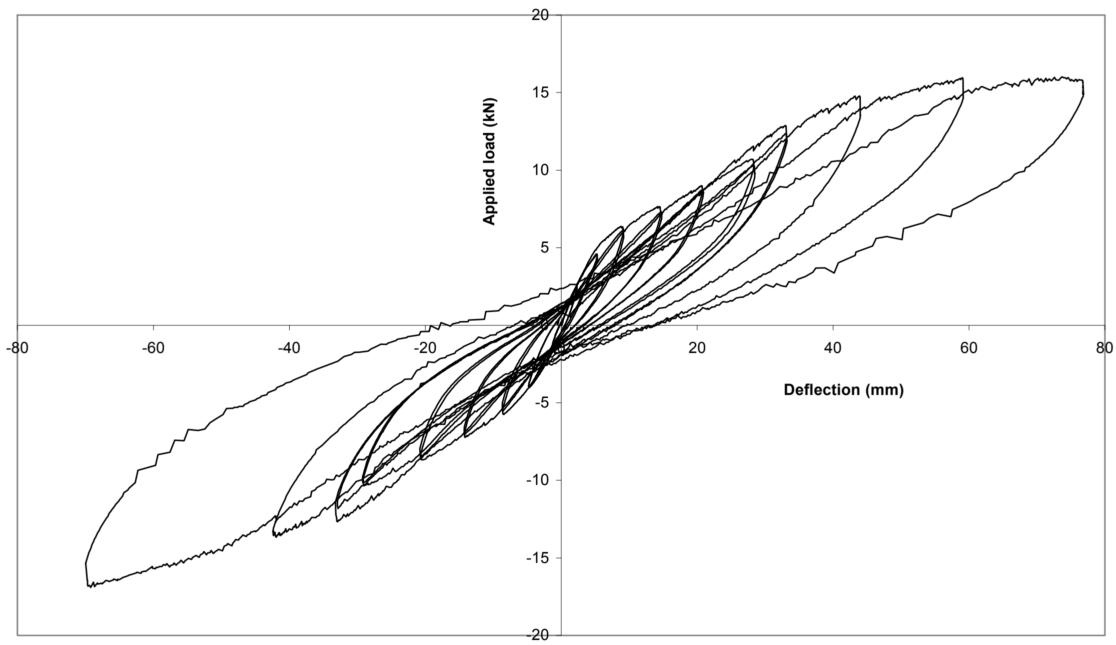


Figure 35. Racking force versus End 1 deflection for Stage II.

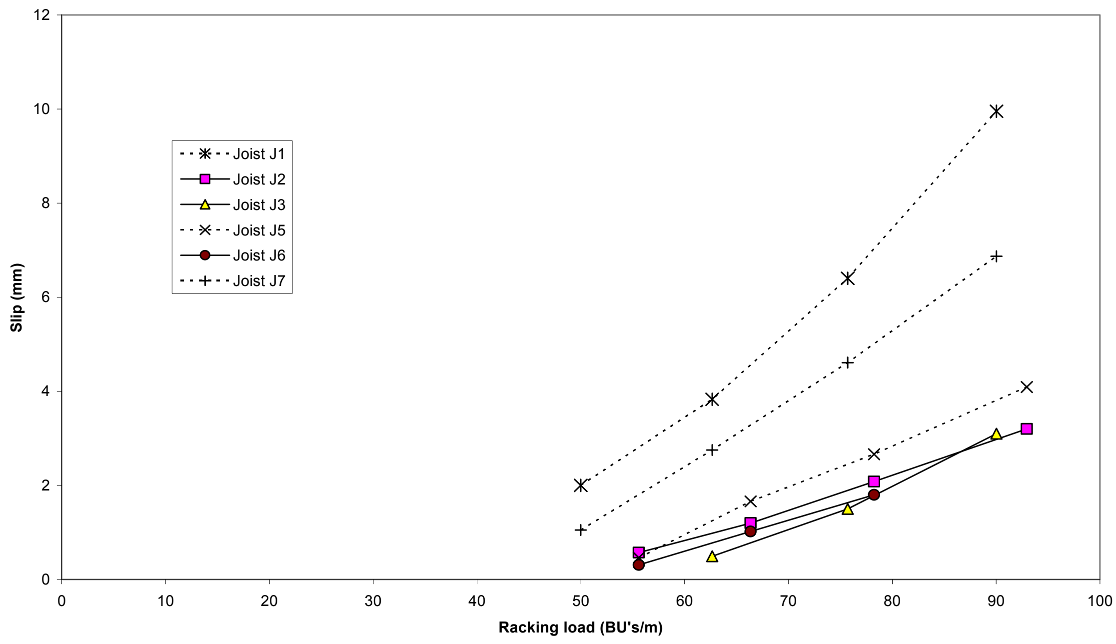


Figure 36. Joist uplift from bearer versus racking load at End 1 for Stage II.

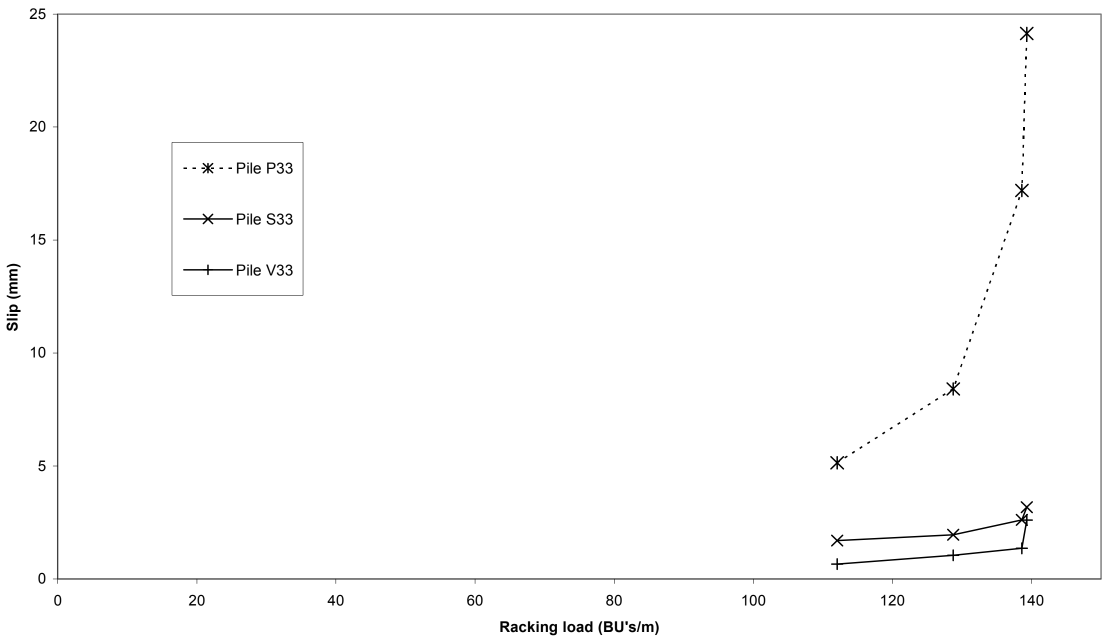


Figure 37. Bearer uplift from the pile versus racking load at End 1 for Stage II.

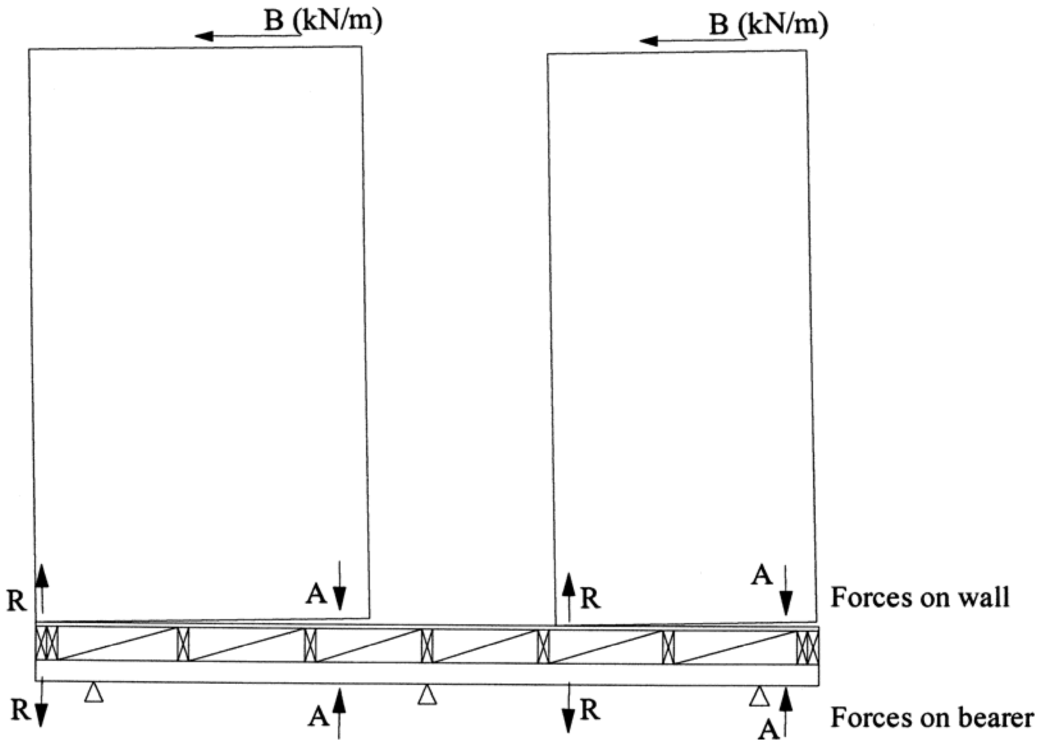


Figure 38. Forces transferred from bracing wall to bearer at End 1 for Stage II.

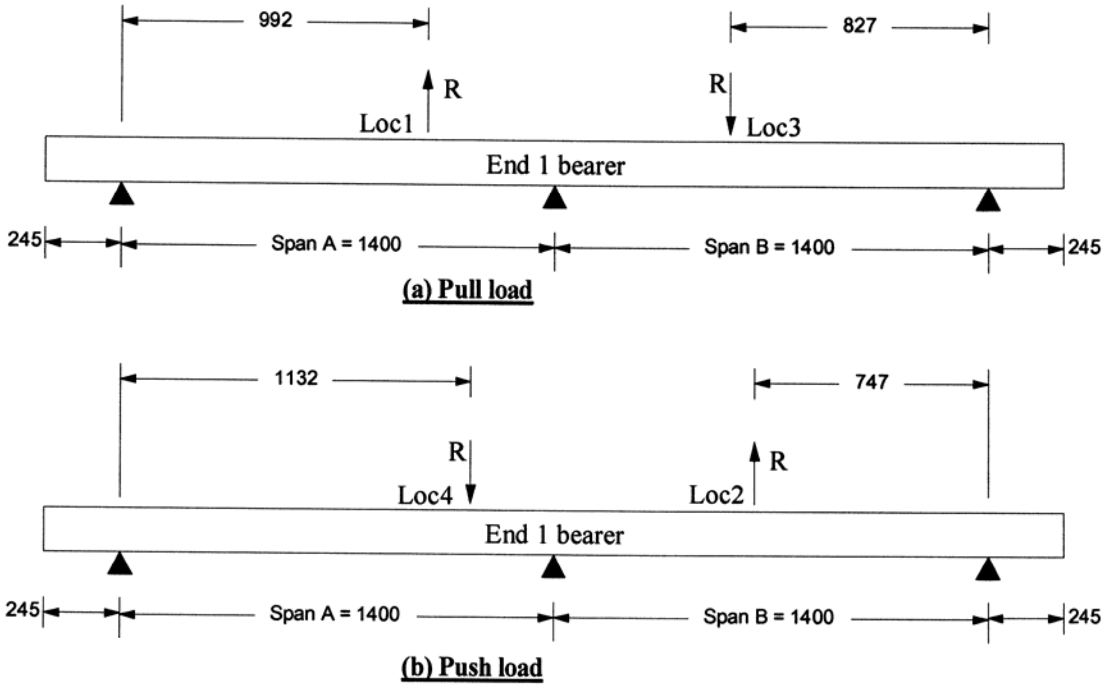


Figure 39. Computer model of loads on bearer at End 1 for Stage II.

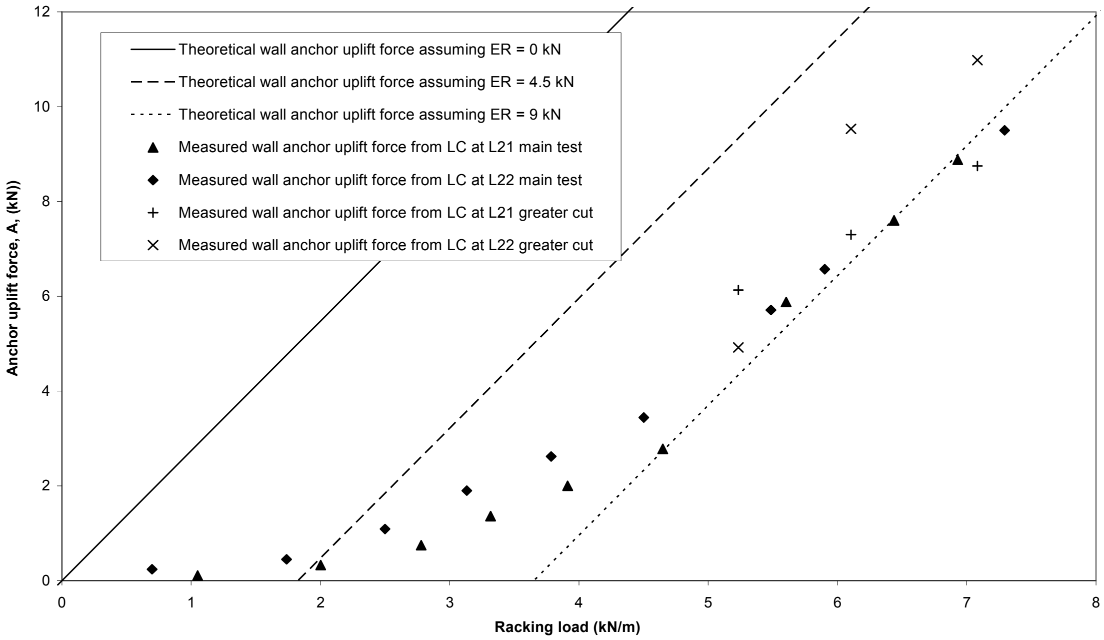


Figure 40. Comparison of theoretical and measured uplift forces on a bracing wall at End 1 for Stage II.

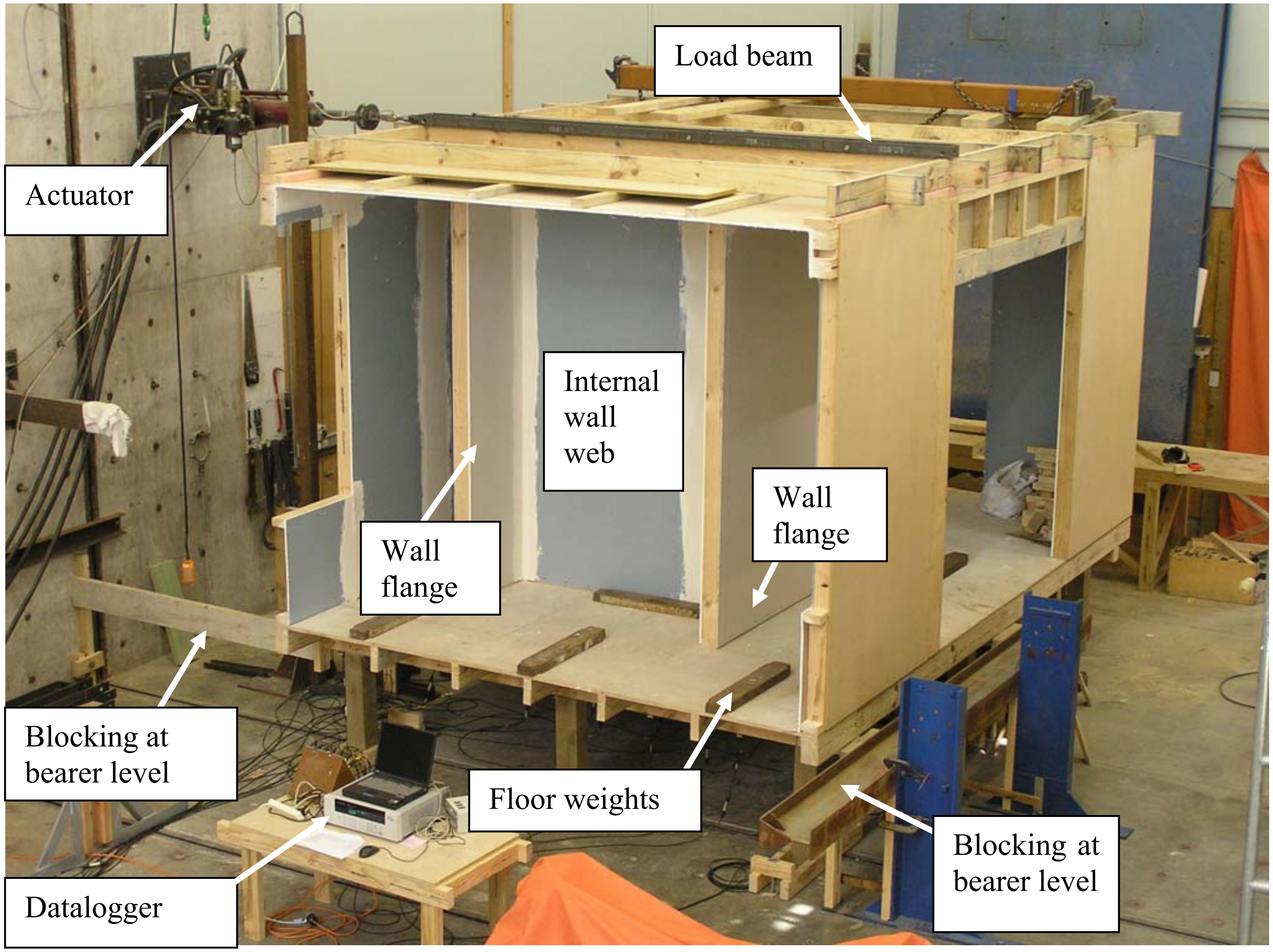


Figure 41. Set-up for Stage III testing. Racking test of internal wall.

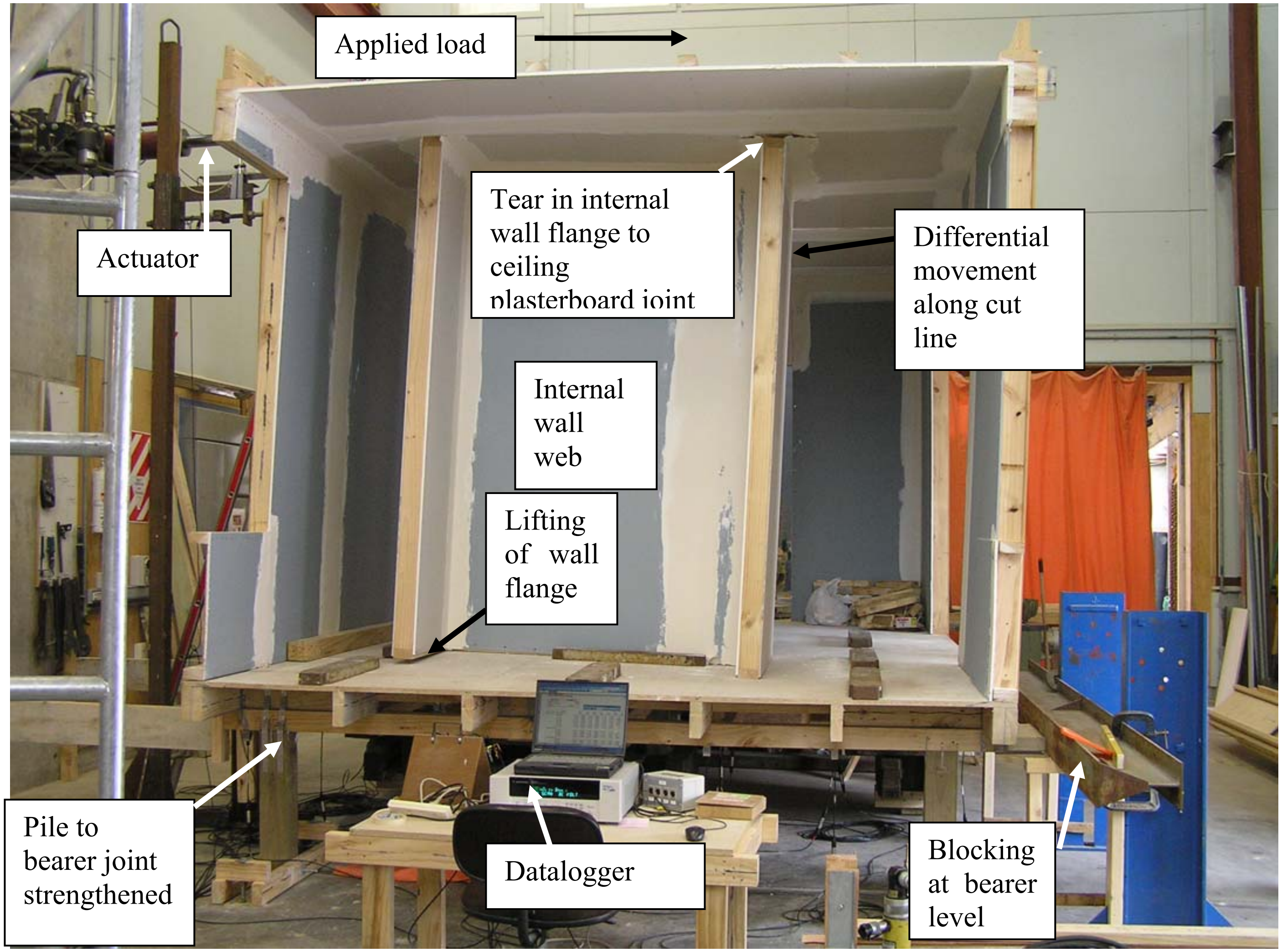


Figure 42. Photograph taken at large imposed deflections. Internal wall Stage III.

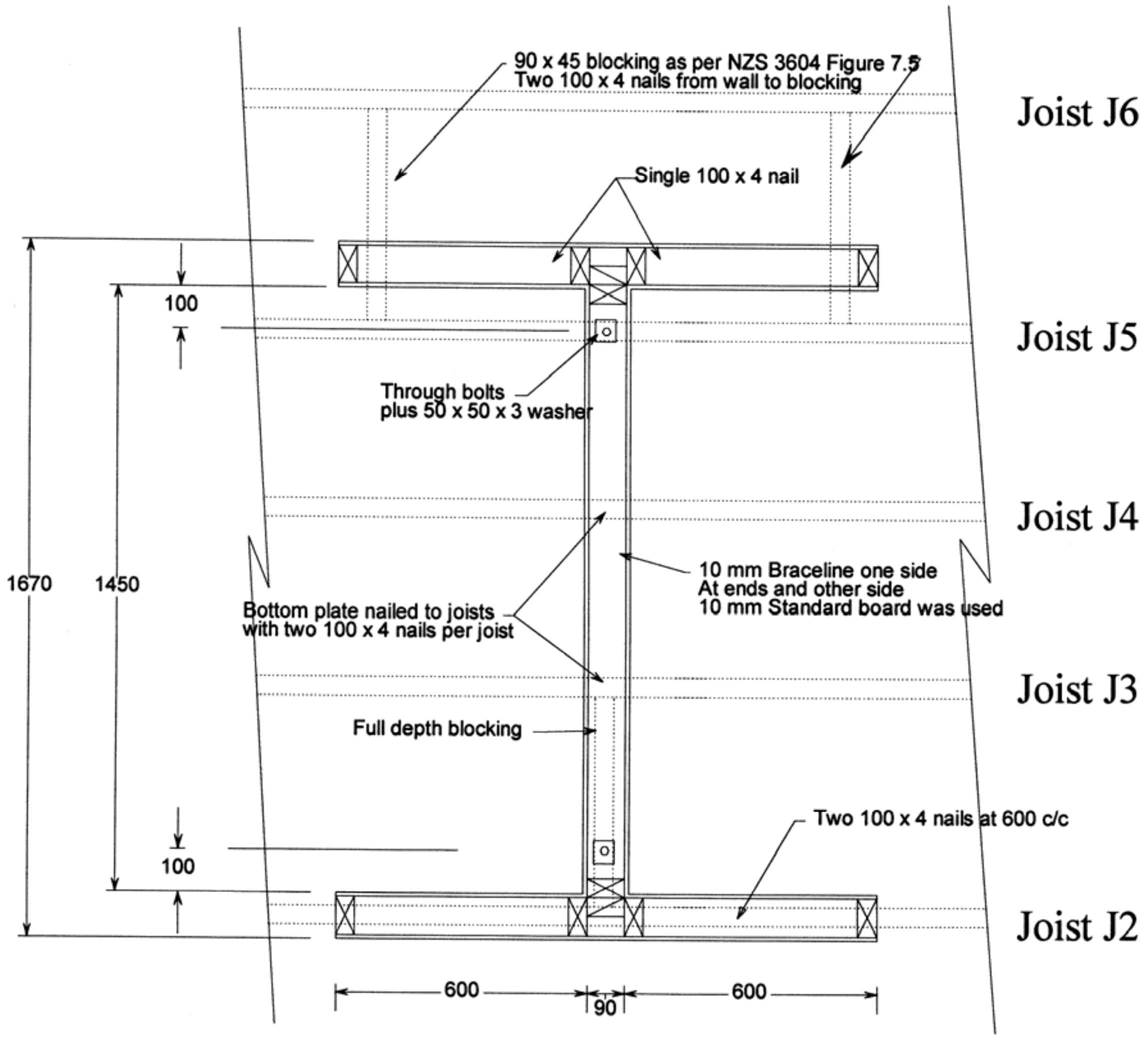


Figure 43. Floor plan showing internal wall.

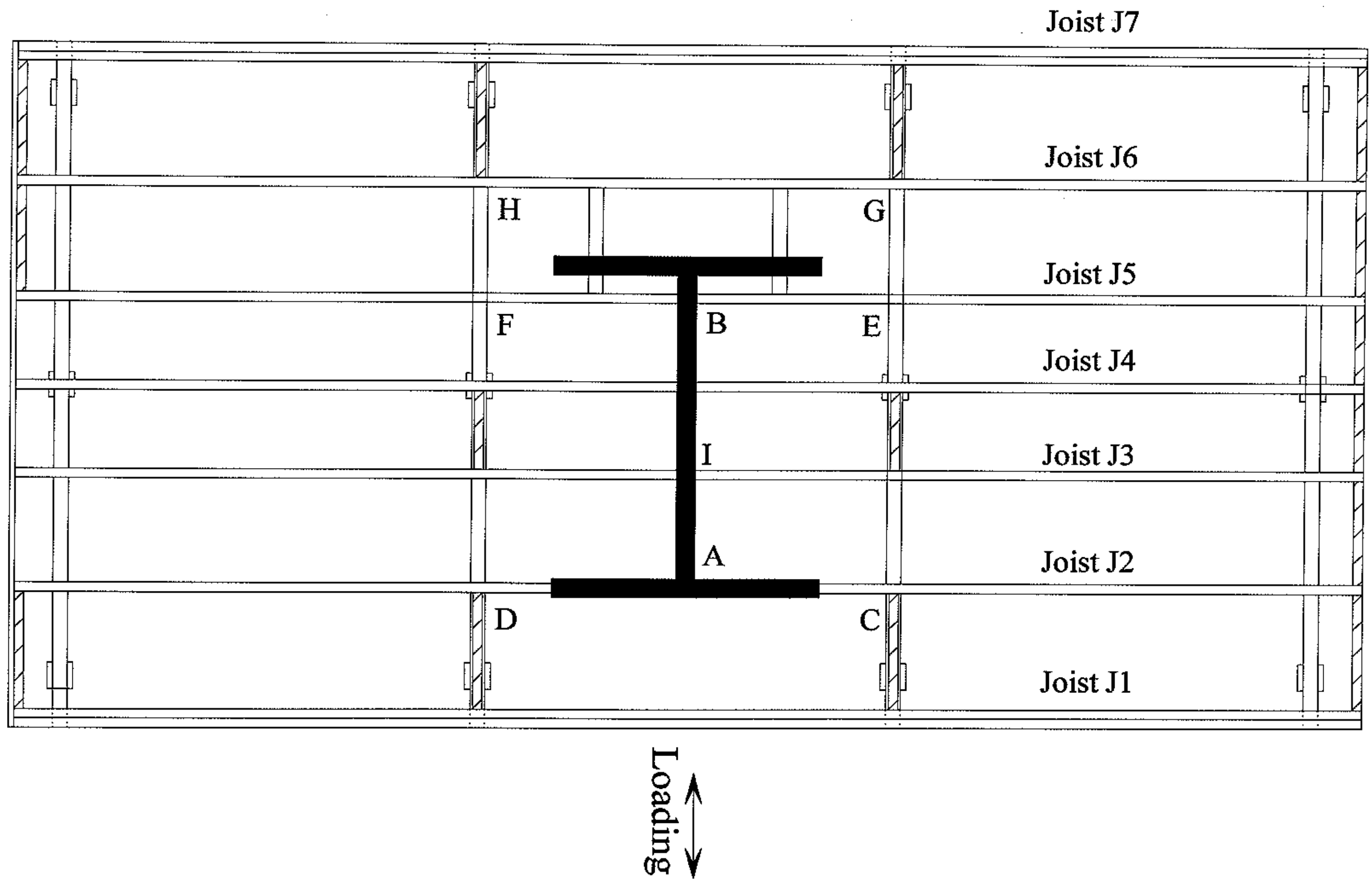


Figure 44. Test room floor plan showing framing members and nomenclature.

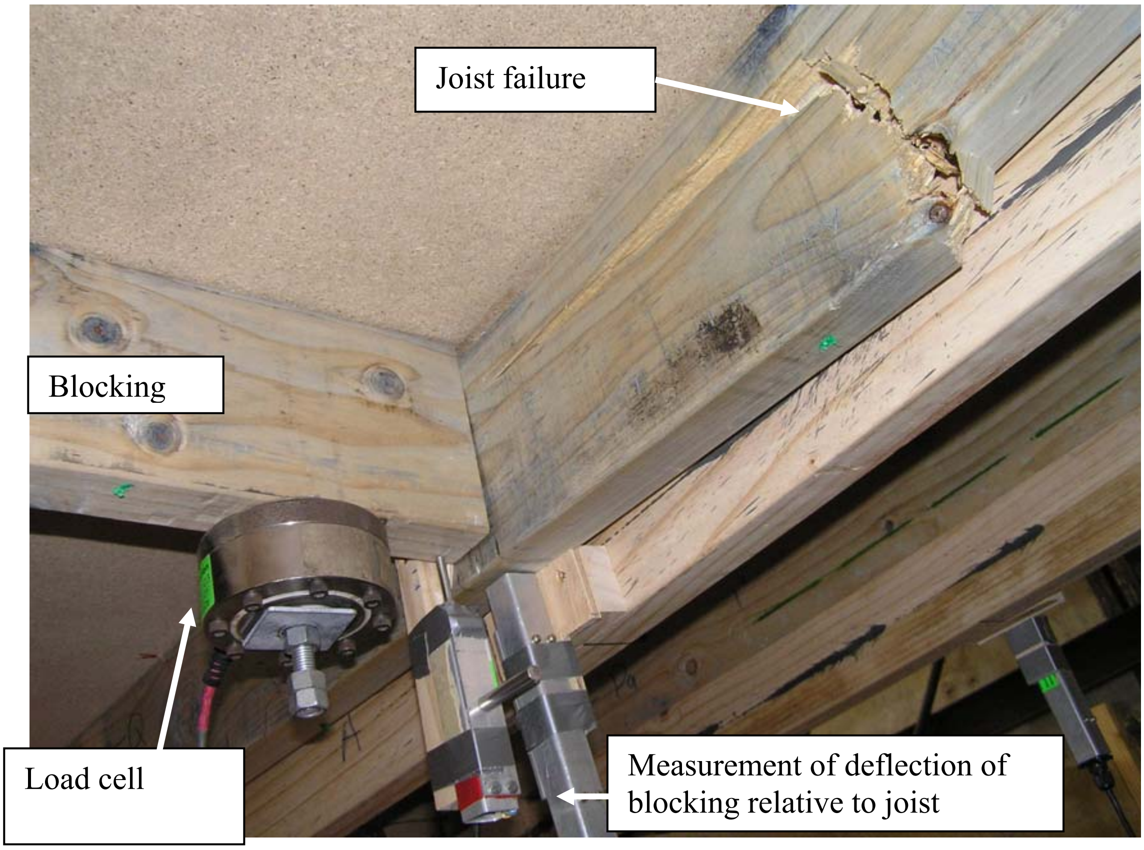


Figure 45. Flexural failure of Joist J2.

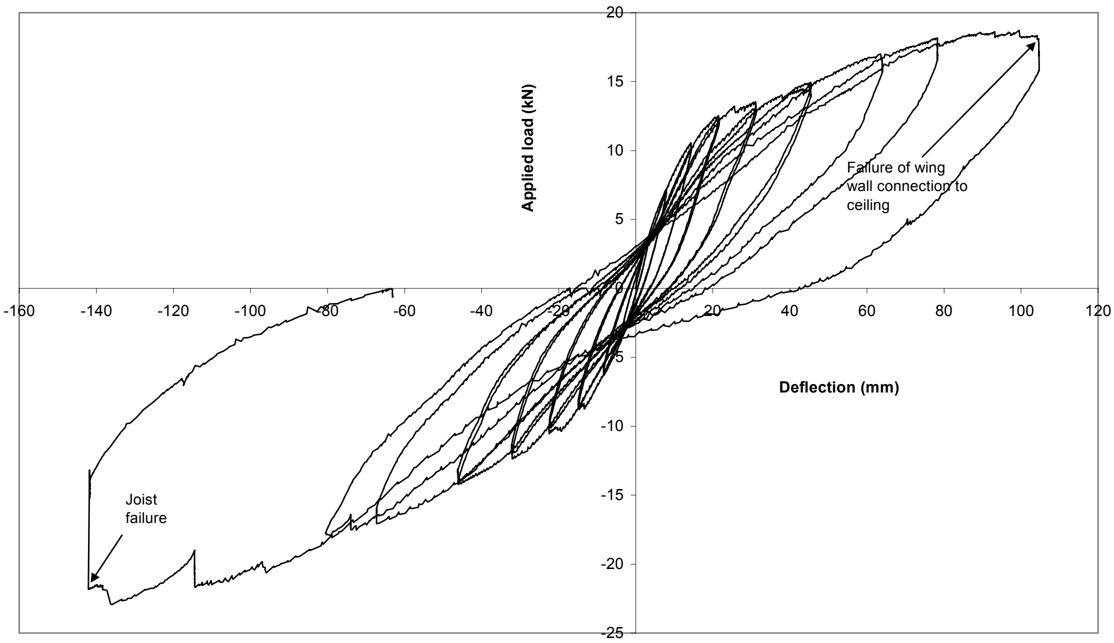


Figure 46. Racking force versus internal wall deflection for Stage III.

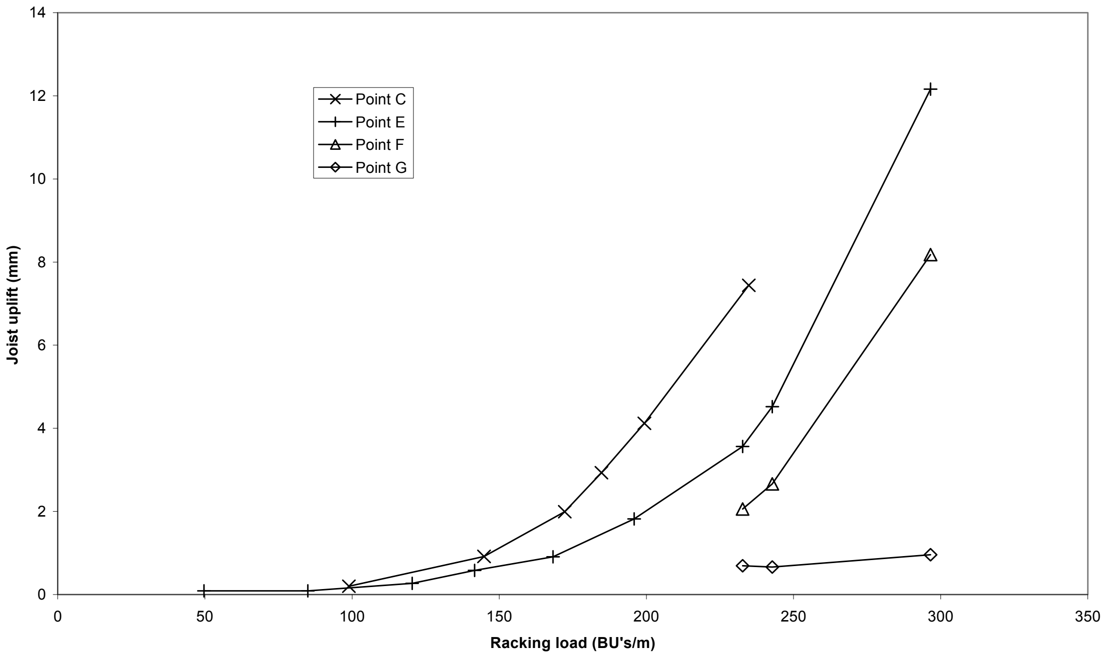


Figure 47. Joist-to-bearer vertical movement during Stage III at locations shown. (See Figure 43 for locations.)

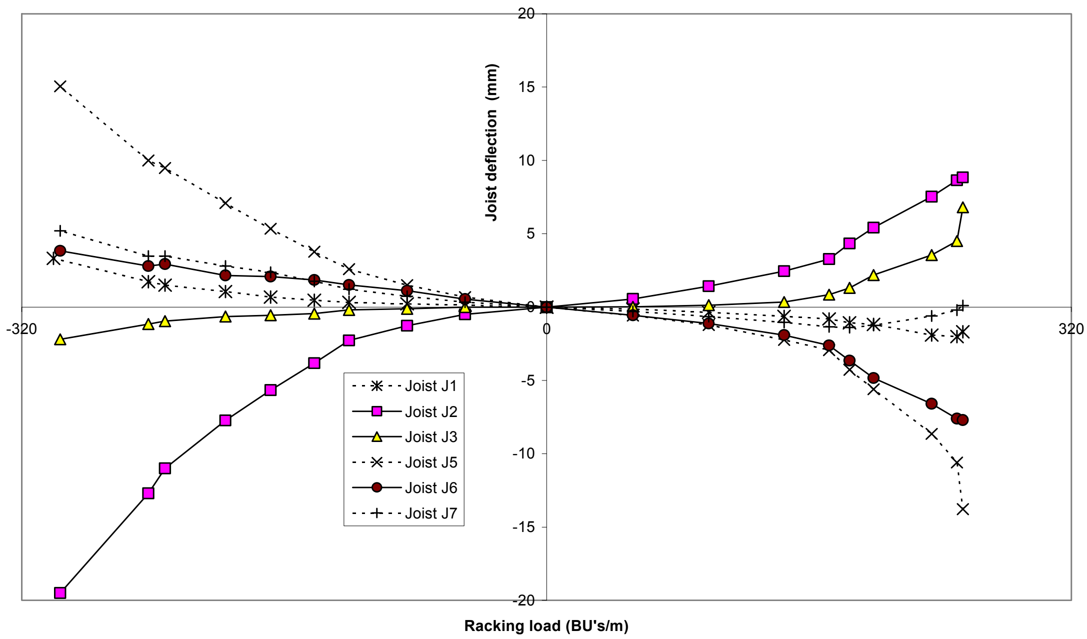


Figure 48. Joist vertical mid-span deflections versus racking load for the internal wall during Stage III.

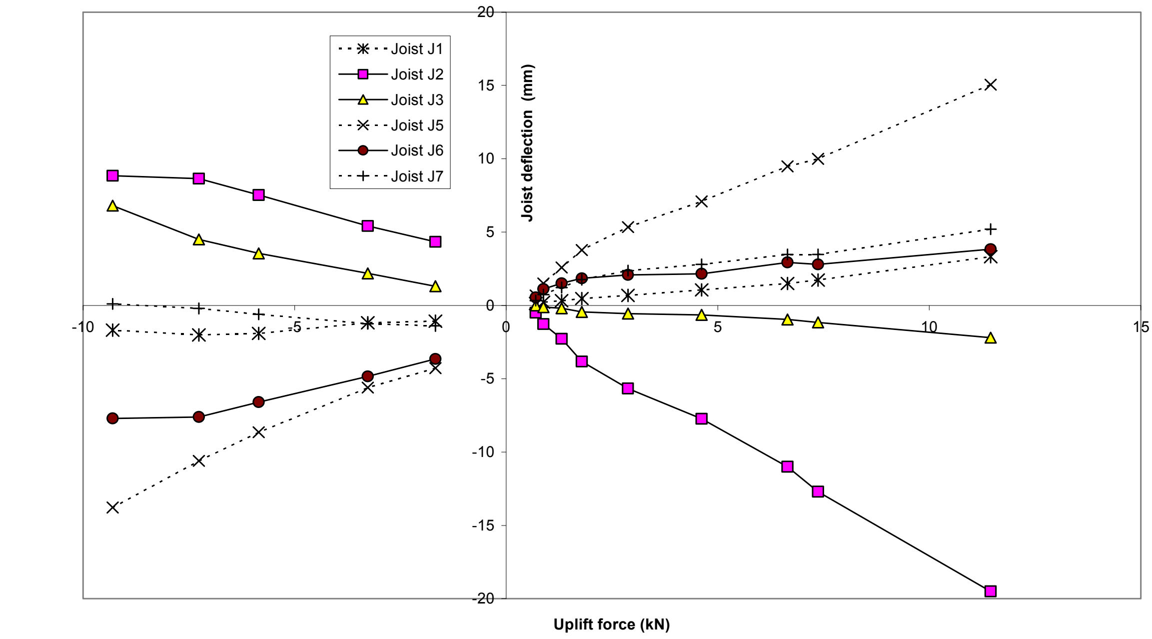


Figure 49. Joist vertical mid-span deflection versus measured uplift force for the internal wall during Stage III.

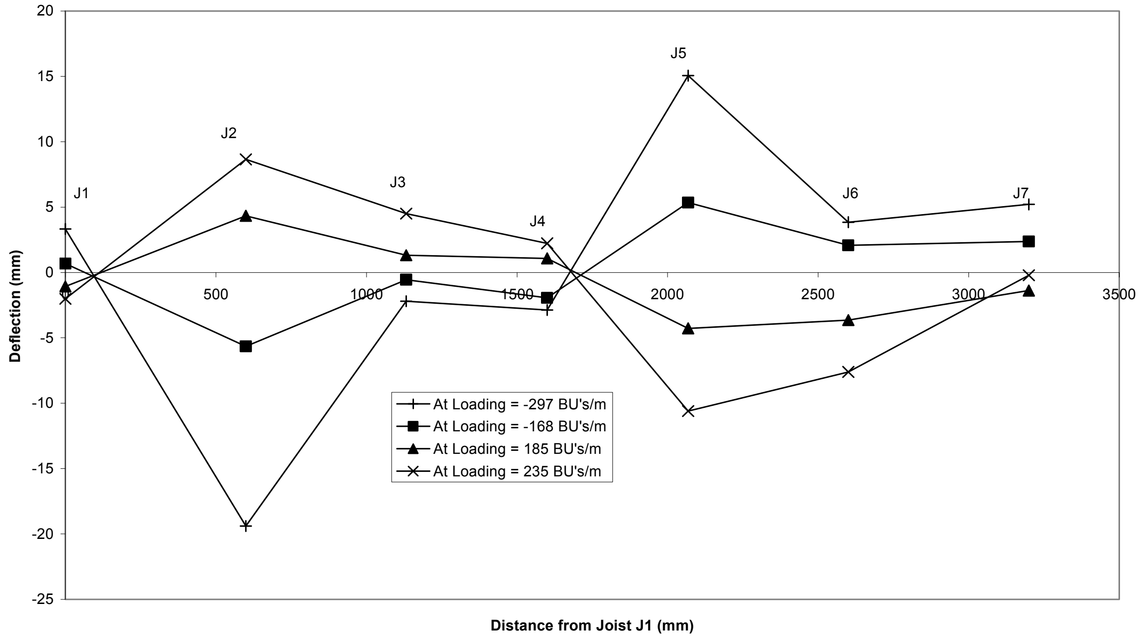


Figure 50. Floor deflection profiles at joist mid-span for the internal wall during Stage III.

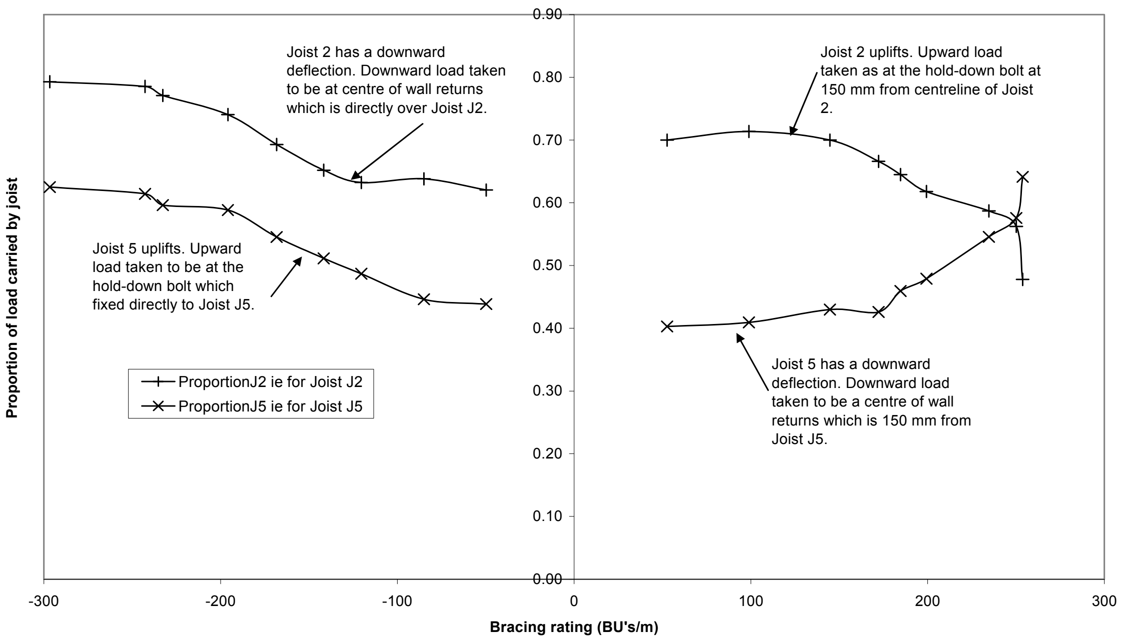


Figure 51. Proportion of racking load vertical load carried by Joist J2 and J5 for the internal wall during Stage III.

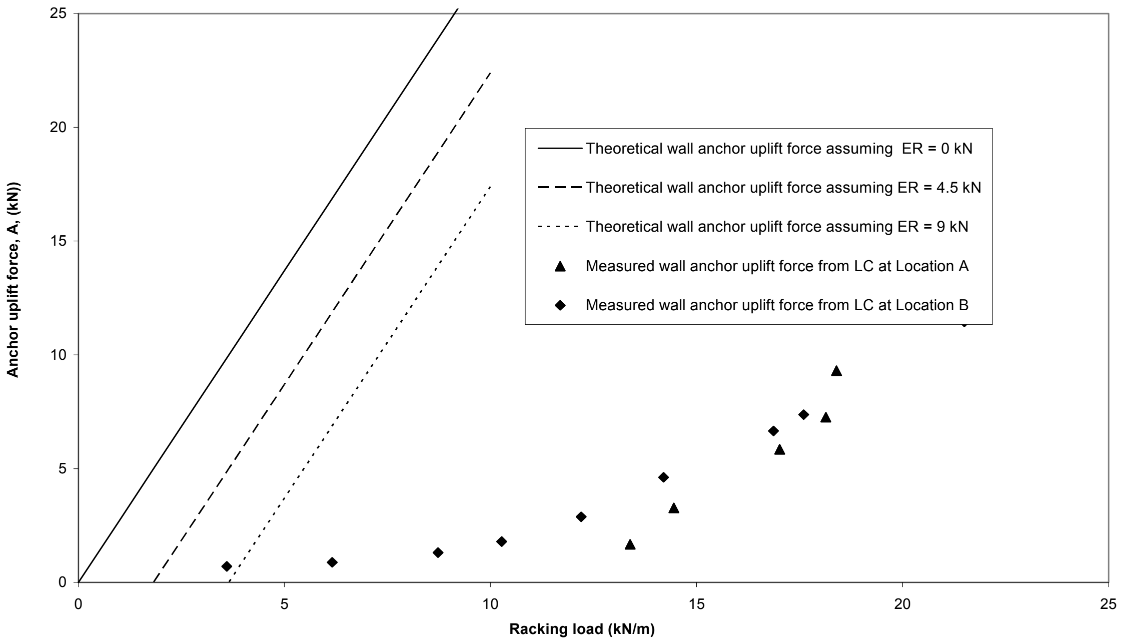


Figure 52. Comparison of theoretical and measured uplift forces on a bracing wall for Stage III.

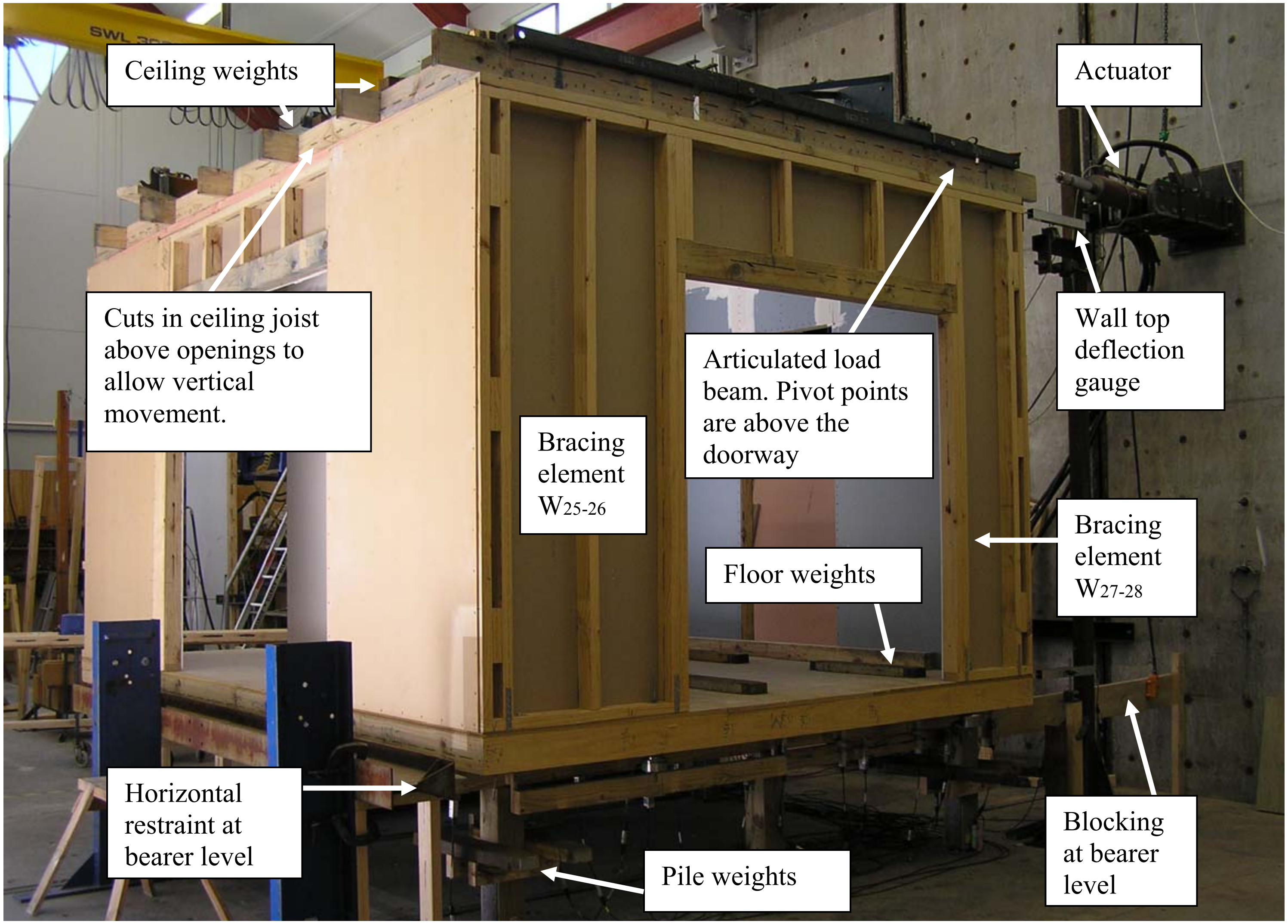


Figure 53. Set-up for Stage IV testing. Racking test of End 2. Stage IV testing.

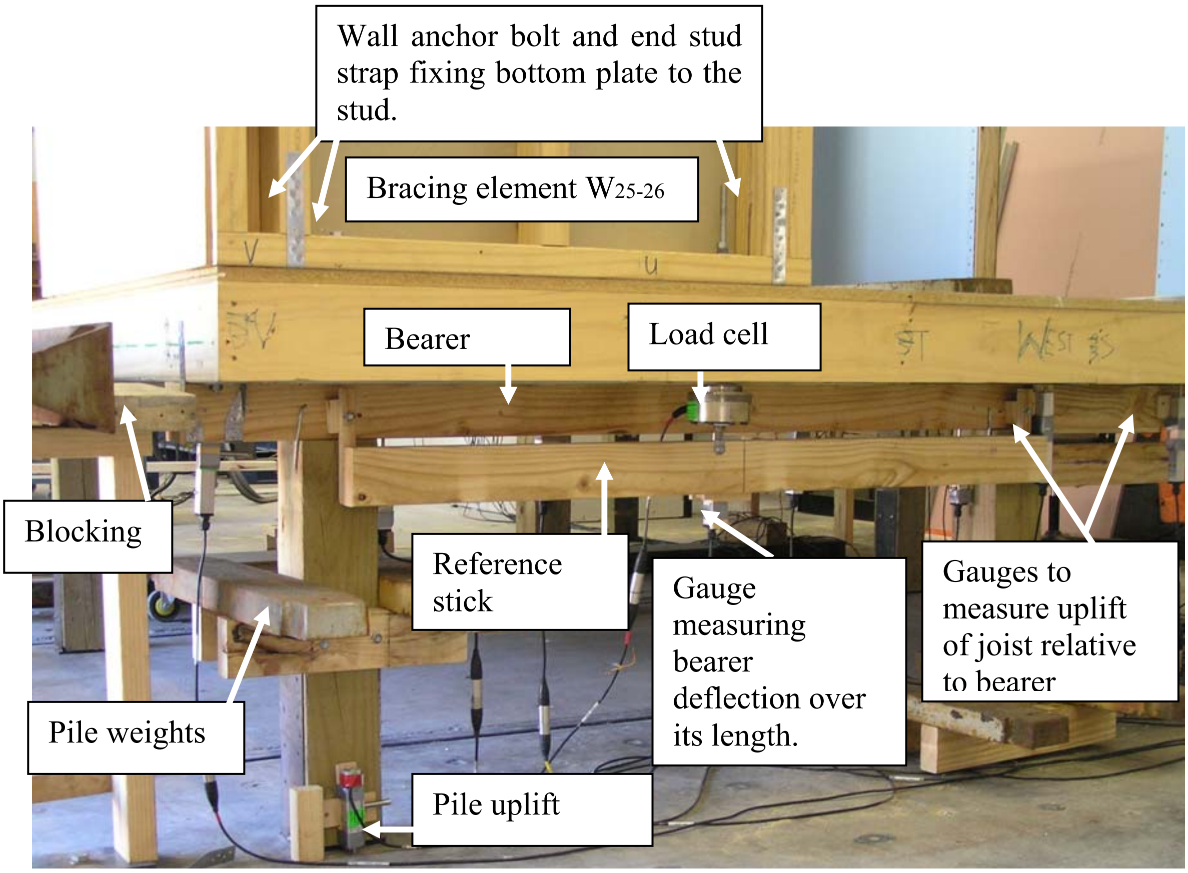


Figure 54. Gauges and details at bracing element W25-26.

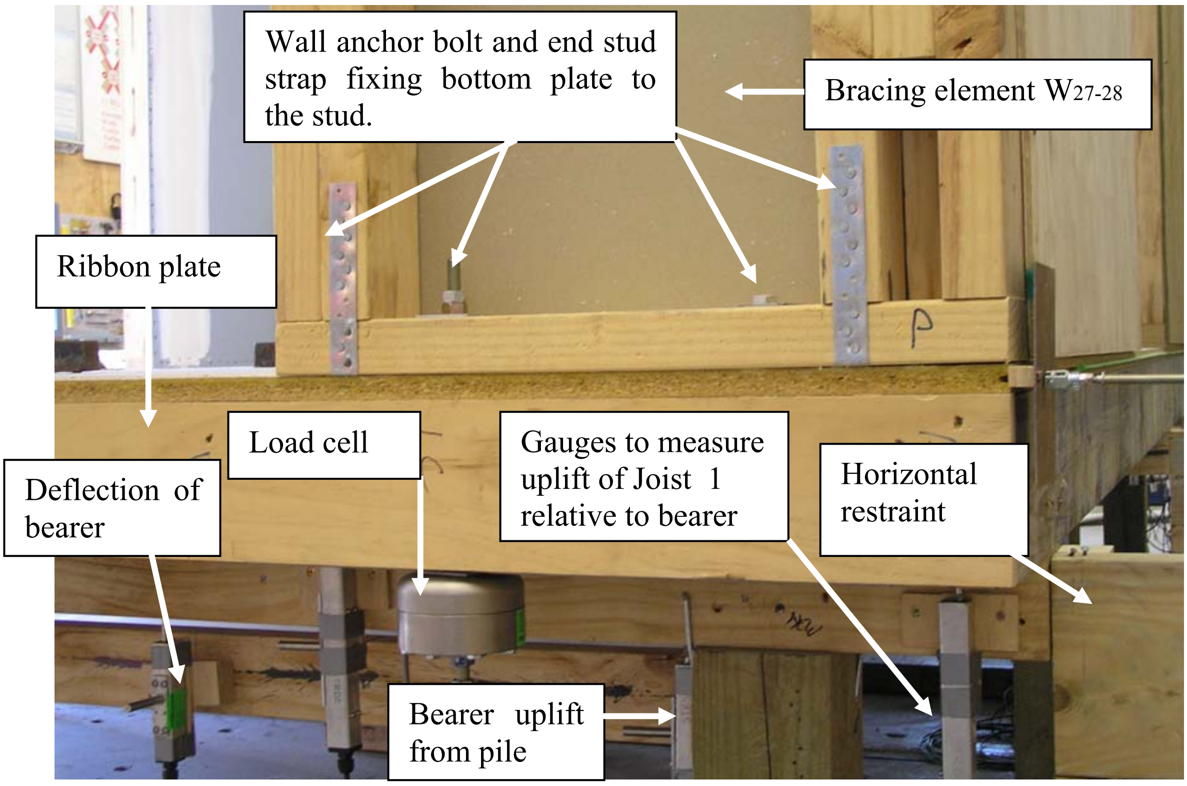


Figure 55. Gauges and details at bracing element W27-28.

Figure 56. Plasterboard cracking at End 2 in Stage IV.

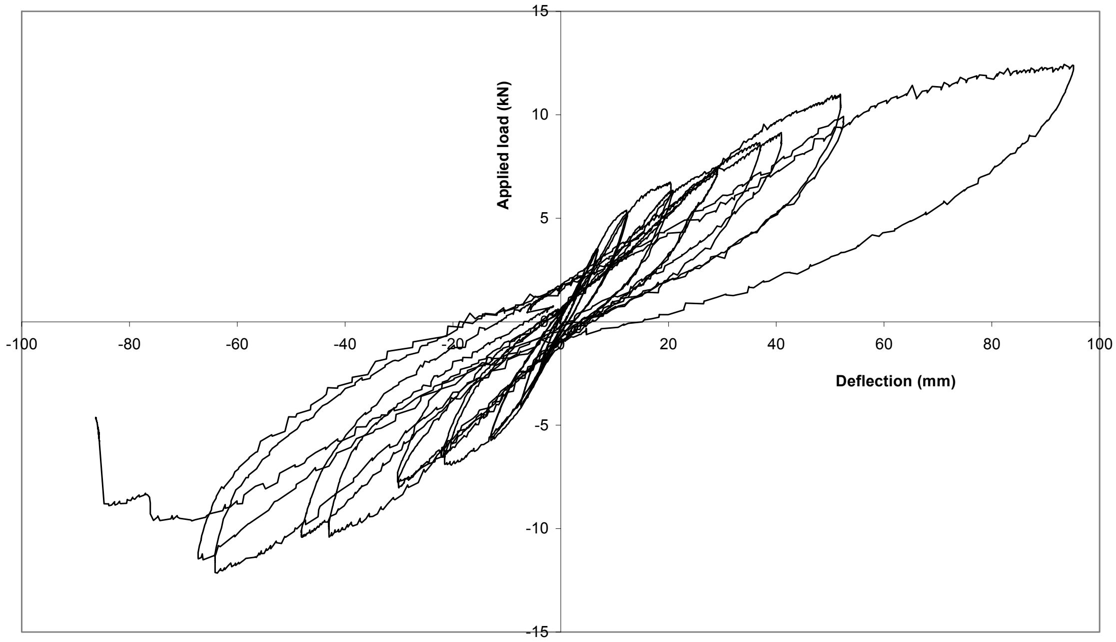


Figure 57. Racking force versus End 2 deflection for Stage IV.

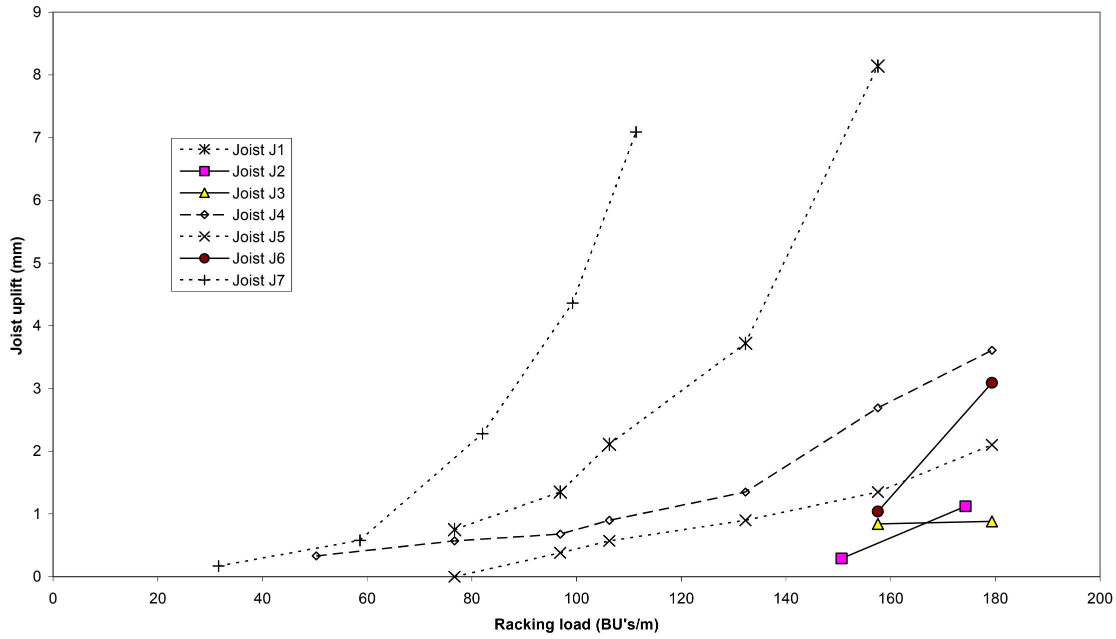


Figure 58. Joist uplift from the bearer below. End 2 Stage IV.

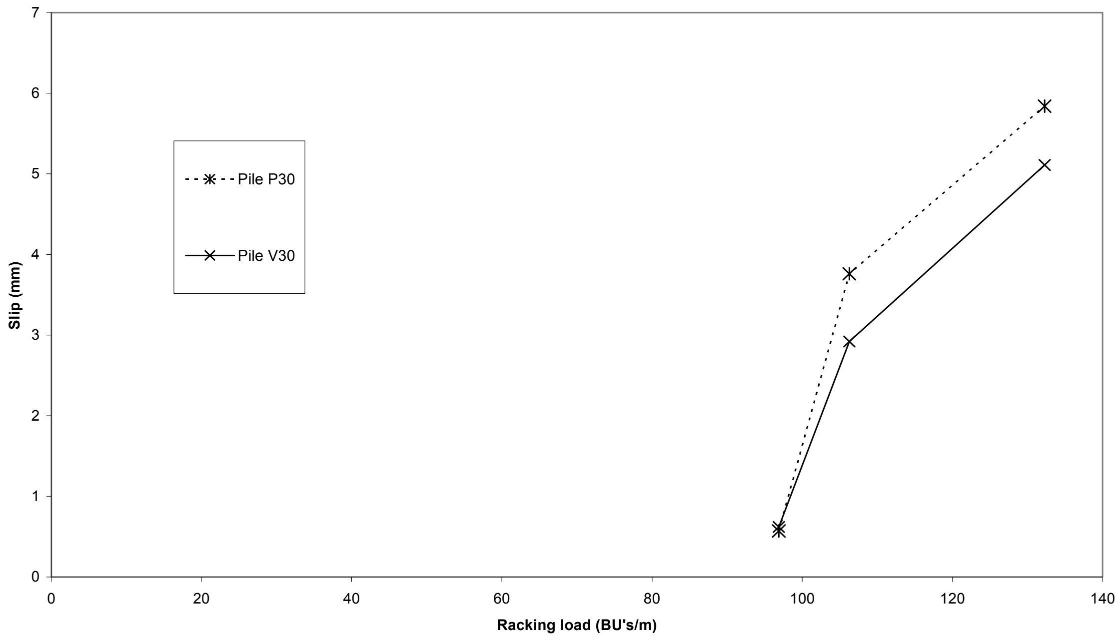


Figure 59. Plot of bearer uplift from the pile below. End 2 Stage IV.

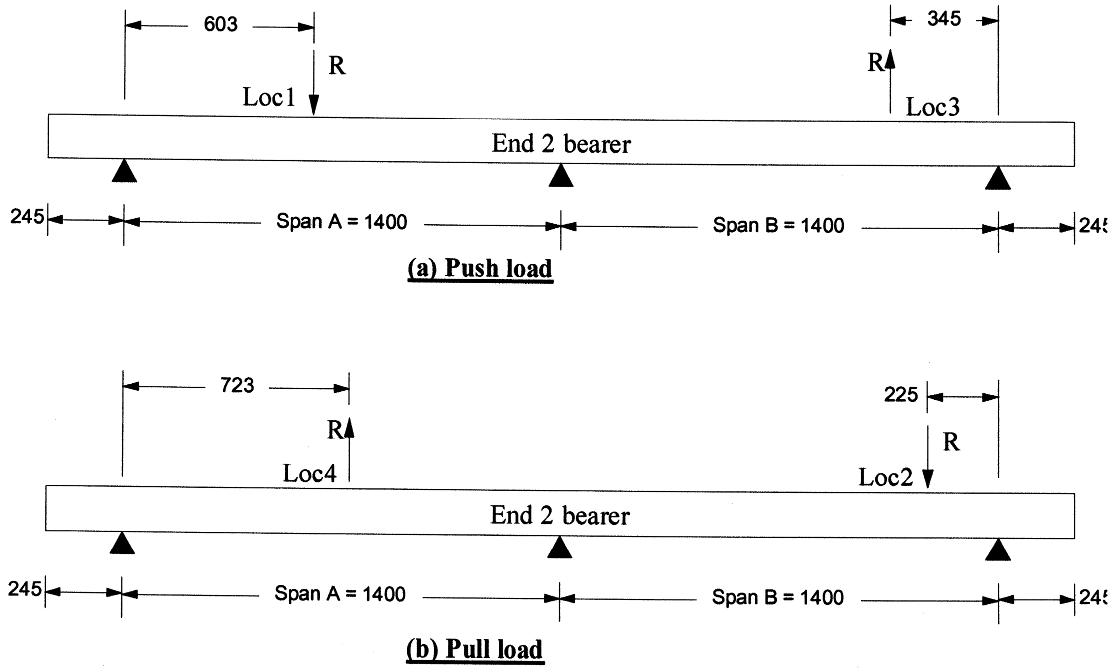


Figure 60. Computer model of loads on bearer.

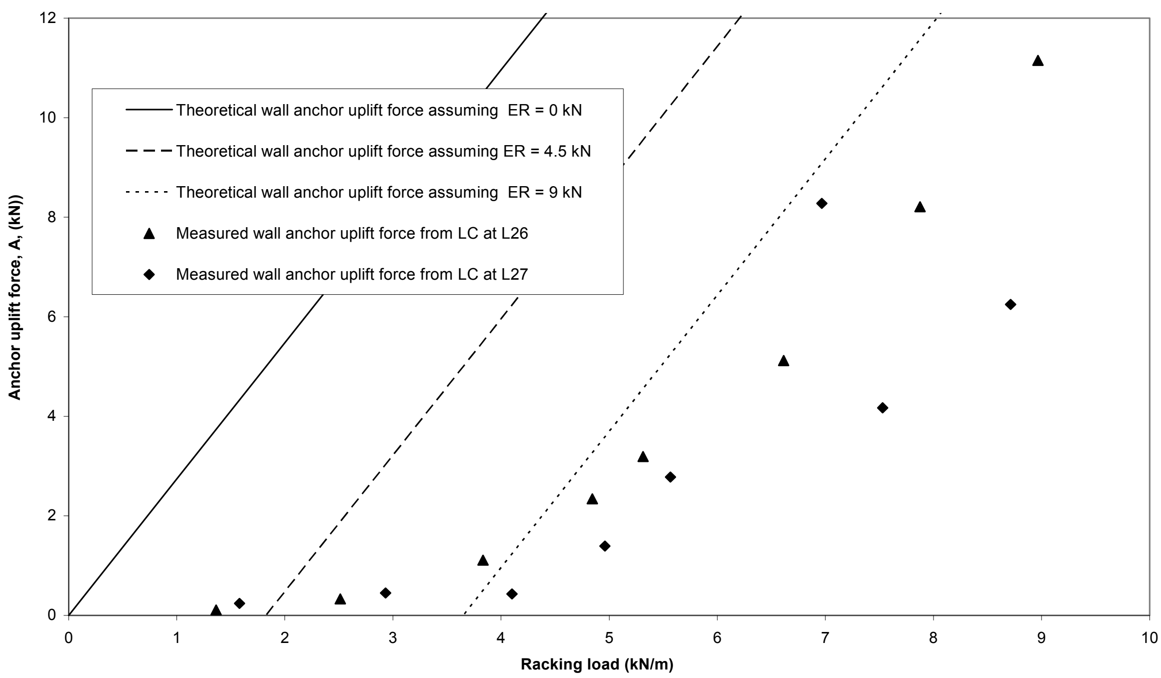


Figure 61. Comparison of theoretical and measured uplift forces on a bracing wall at End 2 for Stage IV.