STUDY REPORT

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BRANZ TEST AND EVALUATION METHOD EM3-V2 FOR BRACING RATING OF WALLS TO NZS 3604

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Preface

The BRANZ P21 test method is used to obtain the bracing ratings of wall systems for low rise buildings to meet the wind and seismic demand stipulated in the timber framed building standard, NZS 3604. The BRANZ EM3 test and evaluation method is intended to replace the BRANZ P21 test and evaluation method. This report describes the experimental and theoretical work which is the basis of the proposed BRANZ EM3 test and evaluation method as (presented in the attachment to this report).

The P21 wall bracing test and evaluation procedure was first published by BRANZ in 1979 and was revised in both 1982 and 1987. The BRANZ Technical Recommendation R10 revised the P21evaluation method to bring it into line with the 1990 revision of NZS 3604 as the previous version of NZS 3604 (1984) was based on working stress design concepts whereas the 1990 version was in limit state format. Thurston and King (1992) discussed fundamental deficiencies in the methodology used in both the P21 and R10 procedures. A proposed revised method of test and evaluation of wall racking test results is discussed by Thurston and Park (2003).

The racking resistance of long walls with openings was investigated in BRANZ Study Report 54. Field measurements of the seismic performance of timber piles was reported in BRANZ Study Report 58. The equivalent ductility of residential timber buildings is investigated in BRANZ Study Report 73. BRANZ Study Report 78 proposed a revised wall racking test and evaluation method but this was never adopted. A comparison of NZS 3604 predicted house strength and the measurements from a full sized house cyclic racking test is described in BRANZ Study Report 119.

Acknowledgments

This work was funded by the Building Research Levy. Plasterboard used in relining the house was donated by Winstone Wallboards Ltd, fibre cement board was donated by James Hardie Ltd and plywood was donated by Carter Holt Harvey Ltd.

Note

This report is intended for standards committees, structural engineers, architects, designers and others researching earthquake and wind resistance of low rise buildings.

BRANZ TEST AND EVALUATION METHOD EM3 FOR BRACING RATING OF WALLS TO NZS 3604

BRANZ Study Report SR 131 (2004) **S J Thurston**

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KEYWORDS

EM 3, racking, bracing, evaluation, NZ 3604, earthquake wind loading

ABSTRACT

This report presents the basis for changing the current BRANZ test and evaluation procedure used to establish bracing ratings. This is known as the BRANZ P21 test method and is used to obtain the bracing ratings of timber framed, plank or panel, wall systems for houses, and other low-rise structures, to meet the wind and seismic demand stipulated in the timber framed building standard, NZS 3604:1999. The demand wind and seismic loads in NZS 3604 were based on the loadings specified in the New Zealand loadings standard, NZS 4203:1992. This report presents a revised wind and earthquake test and evaluation method (called EM3) which is based on engineering analysis to ensure the as-built house strength constructed with EM3 assessed walls will achieve the NZS 4203:1992 intent in a reliable but economical manner.

One of the major differences in the two test methods is the proposed doubling of the strength of the test wall 'supplementary' end uplift restraint. The justification for this is described in the report.

Results of racking tests of similar walls to both the P21 and EM3 test procedures are presented and compared in this report to help identify the economic effect on New Zealand construction if EM3 is adopted for use.

CONTENTS **Page No.**

FIGURES

TABLES

APPENDICES

Attachment:

BRANZ Evaluation Method EM3 – V1.

1. INTRODUCTION

1.1 Purpose of the study

This report presents the basis for changing the current BRANZ test and evaluation procedure known as the BRANZ P21 test method. This is known as the BRANZ P21 test method and is used to obtain the bracing ratings of timber framed, plank or panel wall systems for houses, and other low-rise structures, to meet the wind and seismic demand stipulated in the timber framed building standard, NZS 3604 (SNZ, 1999). The demand wind and seismic loads in NZS 3604 were based on the loadings specified in the New Zealand loadings standard, NZS 4203 (SNZ, 1992). This report presents a revised wind and earthquake test and evaluation method (called EM3), which is based on engineering analysis, to ensure the as-built house strength constructed with EM3 assessed walls will achieve the NZS 4203:1992 intent in a reliable but economical manner.

1.2 Background to the P21 and EM3 test methods

The BRANZ P21 test was based on research at Forest Research Institute (Collins, 1975) and subsequent unpublished testing at BRANZ. The P21 wall bracing test and evaluation procedure was first published by BRANZ in 1979 (Cooney and Collin 1979) and was revised in both 1982 and 1987. This used a working stress type approach where the bracing strength depended on the force resisted when it was cyclically loaded to a serviceability limit deflection. (Further loading ensured that there was adequate reserve strength and ductility for the ultimate loading case.) The BRANZ Technical Recommendation TR10 (King and Lim 1991) revised the P21 evaluation method to bring it into line with the 1990 revision of the New Zealand standard NZS 3604 'Timber Framed Buildings' (SNZ,1990) and the bracing strength was generally dependent on the specimen ductility and maximum forces resisted.

This report describes the experimental and theoretical work which is the basis of the proposed BRANZ EM3 test and evaluation method as detailed in EM3-V1 (Thurston, 2004). EM3 is intended to replace the BRANZ P21 test and evaluation method (Cooney and Collins, 1979). This review was prompted by the following deficiencies in the P21 method.

1. The wall ductility is (mathematically) incorrectly evaluated in the P21 evaluation. (Thurston and King 1992).

2. The strength of most walls tested using the current P21 test is limited by the P21 'supplementary' end restraint and is not a function of the characteristics of the wall fasteners and sheathing.

3. The weak 'supplementary' P21 end restraint acts like a ductile fuse to protect the rest of the wall in a racking test and this behaviour is unlikely to be replicated in the field. This feature in itself invalidates assessment of wall ductility in houses based on P21 tests.

4. Unlike overseas standards, P21 tests give little additional bracing by the addition of a second sheathing.

5. Compared with the design standards overseas, the bracing rating used in New Zealand for walls with brittle linings (such as standard plasterboard) is proportionally significantly higher compared to the rating of more robust sheathings (such as plywood or MDF). The limitations of plasterboard lined walls was highlighted in the 1994 Northridge earthquake (Norton et al 1994) where the sudden collapse of a building was attributed to the separation of the plasterboard wall lining subsequent to pull-through of fastener heads. It is recognised that there are differences in the construction details between New Zealand and USA.

However, the failure was replicated in tests at BRANZ (Deam, 1997). The current P21 test method does not identify this sudden failure at large wall deflections.

1.3 Proposed changes to EM3-V1

This report, and the testing described in the report, has been written based on EM3-V1 (Thurston, 2004). However, during preparation of this document the author considered three changes should be made to EM3-V1. These changes are described in Appendix C.

1.4 Terminology used in this report

The following shorthand notation is used throughout this report. The terminology is common to those working in the area but is defined below to describe its specific meaning in this report.

The terminology nail or screw fastener 'working' refers to the fretting and deformation of wall sheathing directly adjacent to the fastener head caused by the shear action of the fastener in a wall racking test.

The term 'breakout' is used to describe the action of fixings pulling out of the bottom edge of the sheet as illustrated in Figure 2.

The shear strength of the fastener connection between wall sheathing and timber wall framing is referred to as the 'fastener strength'.

The relative movement between wall sheathing and timber wall framing due to the shear force transfer from the sheathing to the frame at a fastener is referred to as 'fastener slip'.

The rocking motion of a racked bracing wall, (pivoting about the compression end and lifting off the base at the tension end) as illustrated in Figure 4(b) is referred to as 'rocking'. The deflection induced at the top plate due to the 'rocking' action is referred to as the 'rocking deflection', Δ, (see Figure 6). The maximum horizontal force needed to be applied at the top plate of a bracing wall to cause 'rocking deflections', greater than 15 mm is referred to as the wall 'rocking strength'.

'Rocking' action of walls in houses is partially resisted by the adjacent construction. This is simulated in a racking test on an isolated wall by use of an artificial end restraint which is referred to as a 'rocking restraint' or 'supplementary' uplift restraint. The 'rocking restraint' used in a P21 test is referred to as a 'P21 uplift restraint' or a 'three-nail end restraint' as it uses three nails as described later in this report and as illustrated in Figure 4. The 'rocking restraint' used in an EM3 test is referred to as a 'EM3 uplift restraint' or a 'six-nail end restraint' as it uses six nails as described in detail in EM3-V1 (Thurston, 2004).

Specifically constructed 'rocking restraints' which are replicated in actual building practice are referred to as 'special uplift restraints'. An example is steel straps connecting studs to floor joists or joist blocking. Another example, (with walls constructed on concrete floors) is end steel straps fixed to the studs and wrapped around the bottom plate to resist the tendency for the two to separate. These are used in conjunction with a nearby bottom plate anchor. In other situations the wall exterior sheathing may extend past the bottom plate and be fixed directly to boundary joists. Other proprietary hold-down devices are sometimes used to prevent the wall from lifting in building practice.

The mechanism for 'rocking' is due to a combination of the vertical uplift of the bottom plate relative to the foundation beam (referred to as 'bottom plate uplift') and vertical uplift of the studs relative to the bottom plate (referred to as 'stud-uplift').

If ‘rocking’ of a bracing wall is prevented and the bracing strength is purely a function of the ‘fastener strengths’ then the wall frame deforms in a trapezoidal shape as shown in Figure 7 and the wall strength is referred to as the wall ‘trapezoidal strength’. The wall top plate deflection purely due to ‘fastener slip’ is referred to as the ‘trapezoidal deflection’ which is also shown in Figure 7.

It was noted by Thurston and Park (2003) that building racking strength is enhanced by ‘systems effects’ which they defined as the holistic response of the whole building system due to load sharing and composite action of both the structural and non-structural elements. The term ‘systems effects’ has been adopted herein.

A ‘backbone’ curve is the curve joining cyclic peaks of the specimen hysteresis loops (see Section 1.5).

1.5 EM3 hysteresis loops

A typical EM3 test involves racking a bracing wall, in a predetermined series of incrementally increasing cyclic deflections. Typical hysteresis loops (load versus deflection plot) from such a test is shown in Figure 1. The force resisted at 25, 30, 35 and 45 mm deflection (both from the first cycle and third cycle to these deflections) is extracted from the test data as shown and is used in the procedure to determine the evaluated wind and earthquake bracing rating.

1.6 Bracing of New Zealand houses for wind and earthquake forces

Earthquake and wind demand loads for timber framed building designs in New Zealand are specified in NZS 3604 (SNZ, 1999). Various lining and cladding manufacturers publish bracing strengths for their wall systems based on the BRANZ P21 (Cooney and Collins, 1979) racking test and the TR10 (King and Lim, 1991) evaluation method. The P21 tests are performed on a short length of wall with ‘supplementary’ uplift restraints to simulate continuity of actual construction. For both major building axes a designer of a particular house determines the predicted resistance of each bracing wall from manufacturers’ published wall system bracing values, sums the resistances of all walls and ensures that this exceeds the NZS 3604 stipulated demand loads. This report presents a revised bracing test and evaluation procedure, called EM3, for the determination of the element resistances.

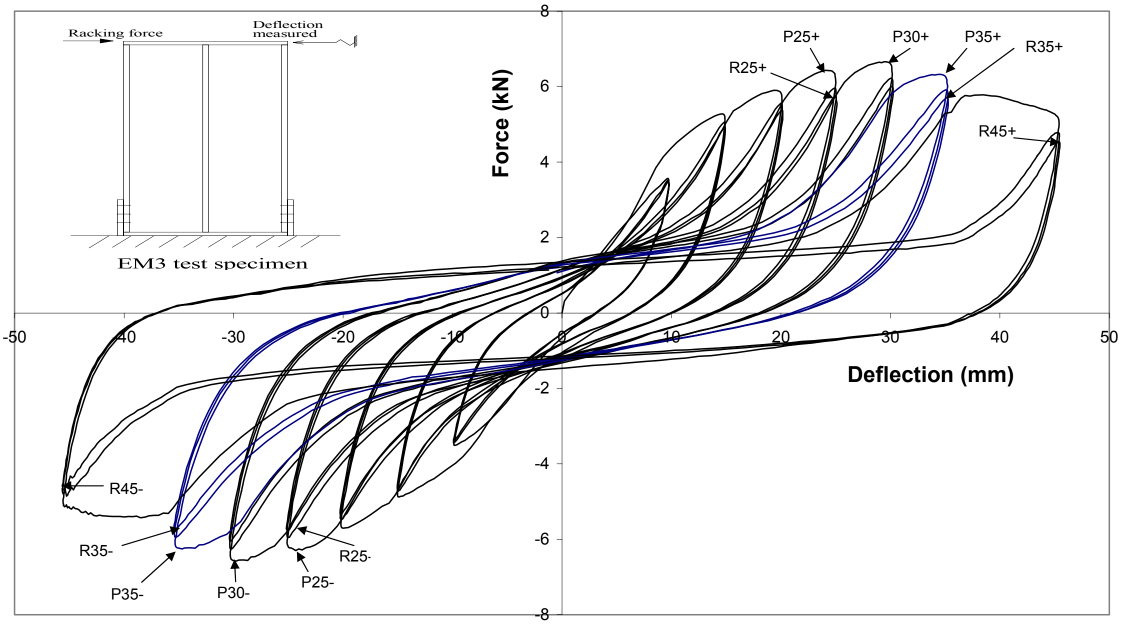


Figure 1. Hysteresis loops from a typical EM3 test.

There was a significant increase in earthquake demand bracing ratings from NZS 3604:1990 (SNZ, 1990) to NZS 3604:1999 (SNZ, 1999). This created concern in the industry as there has been little field evidence that the average NZS 3604:1990 new house will be inadequately braced in a large earthquake. However, New Zealand has not experienced a large earthquake in an urban area since the 1931 Napier earthquake, with the possible exception of the 1987 Edgecumbe earthquake. Thus, although the enhanced demand was technically justified, it is not desirable to be unduly conservative in the evaluation of wall bracing ratings.

Timber framed houses generally exhibit good racking resistance in large earthquake and wind events. However, complacency is unwise if consideration is given to the $20 billion damage to wood-frame construction resulting from the 1994 Northridge earthquake (Fischer et al 2000). The 1995 Kobe (Hanshin-Awaji) earthquake destroyed 250,000 residential buildings (Maki et al 2000) and killed 6,430 people.

In bracing calculations for houses submitted for approval to territorial authorities in New Zealand, plasterboard wall systems are most commonly used to meet the bracing demand. In the USA and England this is mainly achieved with plywood and orientated strand board (OSB) sheathed walls.

1.7 Use of statistics in derivation of design strengths

Most standards for the derivation of design parameters take into account the statistical spread of test results. For example they may use the lower five percentile probability limit. However, both the P21 and EM3 methods derive the design strengths based on the average of three tests. Using complex statistical methods to assess strength variability was rejected as there are only three replicate tests conducted to derive a bracing resistance. Using a greater number of replicate tests than three was rejected due to the cost of testing.

The methodologies do not take into account differences between site and laboratory construction. Instead, average test strengths, without any capacity reduction factors, are used. This is justified by good historic performance of houses in major events, the rarity of such events, the low life risk in the event of failure and the expected economics of the average annual cost of providing additional house bracing against the average annual cost of damage from wind and earthquake attack.

An attempt has been made to look at the effect of larger than the design events in the EM3 method but this was not made in the P21 method.

Reducing the vulnerability of particular building types is outside the scope of the EM3 and P21 methods. For example particularly torsionally susceptible buildings such as those with lower storey garage openings along an entire side are not allowed for in the methodologies. Such construction fared badly in the Northridge earthquake (Norton et al 1994). NZS 3604 (SNZ, 1999) does cater for such situations.

2. LITERATURE SURVEY

2.1 Paper by Thurston and King (1992)

This paper discussed fundamental deficiencies in the P21 evaluation method (King and Lim 1991). In was noted that the evaluation overestimates the wall ductility by 100% and does not take into account the pinched nature of the hysteresis loops. It was recommended that the evaluation be modified based on computer simulation using test hysteresis loop shapes.

2.2 Report by Thurston (1993)

Thurston (1993) presented a literature survey of wall racking and house racking research and the factors influencing wall racking strength – particularly the effect of wall openings.

Thurston performed ten cyclic racking tests with five different 6.6 m long wall configurations incorporating wall returns, typical door and window openings and various combinations of sheathings for timber framed construction on timber foundations. Generally, only standard (to NZS 3604:1990) nailing between the bottom plate and the foundation beam provided wall uplift restraint. No 'supplementary' or 'special' uplift restraints were used except that in a few instances light steel straps were used on studs either side of doorway openings. The walls had no ceilings attached and additional gravity load was only used in one instance. As ceilings with taped and filled joints at intersections with walls limit slippage at the top of the wall and as gravity loads reduce wall 'rocking' action, these two simplifications are expected to result in conservative results.

The measured wall strengths were compared with the summation of the isolated wall strengths of panels between wall openings as determined by P21 tests. It was concluded that for walls with large window (but no door) openings, that the long wall load versus deflection hysteresis loops could be fairly accurately (but conservatively) predicted by assuming that the component panels between openings were fully restrained against 'rocking' action. Thus, bracing tests using only the P21 'supplementary' uplift restraint were unduly conservative in these instances. The conclusion also held for a wall with a door opening but where hold-down straps were used on the studs on either side of the door. Without these straps the measured long-wall strengths were only 70% of the combined individual panel test results using the P21 'supplementary' uplift restraint. The existence of ceilings, trusses and roofs was expected to significantly raise this percentage.

Whereas P21 tests do not give a large increase in bracing rating in walls with sheathing on both sides versus a wall with sheathing only on one side, the long walls tests with a window (but not door openings) gave total strengths consistent with the simple addition of strengths from both sheathings.

2.3 Report by Deam (1997)

A computer program called PhylMas was developed for seismic simulation of walls with pinched hysteresis loops. Good agreement was obtained with shake table test results. A bracing rating procedure was recommended (but never adopted) based on displacement spectra generated by PhylMas which indicated large reductions in bracing ratings determined from the P21 evaluation method.

2.4 Report by Herbert and King (1998)

Herbert and King (1998) presented a literature survey of the bracing test and evaluation methods used overseas. The New Zealand system was the only one to use cyclic loading and the only one to use an external partial 'supplementary' uplift restraint. Other countries adopted a full 'supplementary' uplift restraint (USA and Japan) or none (England and Eurocodes). The Australian standards were silent on the test method by which lateral bracing was determined. However, Technical Record TR 440 (Walker and Reardon, 1978) used no 'supplementary' uplift restraint of any sort.

Herbert and King also presented a literature review of house performance under earthquake and wind racking. They quoted two authors attributing house damage in wind and earthquakes to lack of house bracing but concluded that there was generally little damage attributed to inadequate wall bracing in post-event reconnaissance reports. However, they warned that there had been few earthquakes which had severely shaken New Zealand style house construction and that the modern open plan houses with large window openings were more vulnerable.

Herbert and King presented racking test results on a variety of walls including openings and wall returns. The purpose was to quantify the actual uplift restraint and the maximum reliable displacement. They concluded that actual uplift restraint was 12 kN and the maximum reliable displacement varied considerably depending upon the sheathing tested.

2.5 Paper by Fisher et al (2000)

Fischer et al (2000) summarised 15 full-size house racking and shake table tests. This extensive literature survey is expected to cover all reported work in this study area. Many of the tests reported were on houses with construction non-typical of New Zealand construction. The reports often did not state whether the plasterboard joints were taped and filled, what wall tie down systems were used (if any) and whether wall uplift occurred. Comments below are only for those buildings which can be considered moderately typical of New Zealand construction. Values of damping measured in the various tests were 6%, 17% and 7.6%. Values of natural frequency measured were 9 Hz; 5.8 Hz without sheathing and 8.8 Hz with sheathing; 5.6 Hz and 6.5 Hz.

Most of these tests were carried out on prescriptively designed houses and all the researchers found that there was adequate strength to meet code-required lateral forces. One report noted the measured house lateral strength was 1.8 times the design wind load. Only one of these tests measured house racking strength and compared this with the sum of the predicted individual house wall strengths. In this instance the house predicted strength was based purely on ‘trapezoidal strength’ modelling ignoring wall ‘rocking’ and was slightly less than the measured strength.

Most tests indicated that ceiling diaphragms can be considered to be rigid. Where noted, all researchers found that wall diagonal braces were far less effective than sheathed wall action.
Fischer et al (2000) tested a two storey house with ‘special uplift restraints’ at some wall ends with OSB (orientated strand board) bracing walls on a shake table. They were able to get moderate agreement between test results and simulation using Ruaumoko (Carr, 2000) with Stewart (Stewart, 1987) hysteresis elements. This was similar to the approach used by Thurston and Park (2003) as described in Section 2.6. The wall hysteresis loops were determined using the Cashew software (Foltz and Filiatrault, 2000) rather than by test. Measured wall uplift peaked at 20 mm and wall horizontal slip on the floor peaked at 5 mm.

2.6 Paper by Thurston and Park (2003)

This paper developed the theoretical basis for the new EM3 evaluation procedure. It was purely a computer study where buildings were analysed by inelastic time history seismic analysis using the Ruaumoko 2D software (Carr, 2000) and the Stewart hysteresis element (Stewart, 1987). It was assumed that the houses were constructed with only one type of bracing wall which was modelled within the analysis software from experimental hysteresis loops.

The analyses used a suite of earthquakes which had elastic spectra corresponding to the design elastic spectra of NZS 4203 (SNZ, 1992). Computer models of single and two storey buildings, with wall elements having pinched hysteresis loop shapes defined to cover the usual range of sheathed timber framed wall behaviour, were analysed under excitation from these earthquakes. For each modelled structure, a series of computer runs was performed to compute the maximum deflection, Δmax, for a range of the seismic weights, W. The third cycle peak strength ,R, at deflection Δmax is determined from the bracing test.

Thurston and Park showed that NZS 3604 (SNZ,1999) calculates the seismic demand base shear force, V, in Zone A from V = 0.241W. EM3 calculates the bracing wall seismic design strength from F1 x F2 x R where F1 is a function of wall deflection, F2 a function of building 'systems effects' (put = 1.0 for this consideration) and R is the strength characteristic of the hysteresis loops noted in the paragraph above. Putting the NZS 3604 demand force equal to the EM3 evaluated wall resistance gives: 0.241W = F1 x R or F1 = 0.241W/R. Hence, for each seismic weight, W, and computed deflection, Δmax, from the computer runs in the paragraph above, the value of R was determined as computer output which enabled F1 to be determined. Thurston and Park plotted F1 versus Δmax for different basic hysteresis loops for single storey buildings and this relationship averaged between plywood and plasterboard walls is adopted in the EM3-V1 method (Thurston, 2004). One change made to the Thurston and Park recommendations is that the overload cycling is taken to 45 mm rather than the recommended 55 mm because BRANZ testing noted that end strap breaking and other brittle mechanisms sometimes occurred between 50-55 mm which distorted the results.

Houses are in fact an amalgam of different strength/stiffness walls and to ensure compatibility between different walls the EM3 method specifies a small deflection range over which the walls are evaluated. Thurston and Park proposed that the value of R be determined in the deflection range 25 mm ≤ Δmax ≤ 35 mm and the EM3 method, (EM3-V1, Thurston, 2004), uses this deflection range for both wind and earthquake loading.

To account for the strength enhancement due to the presence of non-structural elements and also 'systems effects' they introduced a second factor, F2, which varies between 1.0 and 1.2 depending on the 'toughness' of the wall. The toughness is defined in equations in EM3-V1 (Thurston, 2004) as the ability of the wall to deflect to 45 mm with only small strength loss.

Thurston and Park also analysed many two storey buildings with a range of floor/roof weights and inter-storey stiffness distributions. They found good agreement with the F1 relationship proposed for single storey buildings provided NZS 3604:1999 reduced the demand seismic shear forces by 10% for two storey buildings and the seismic inertia load distribution method is as per equation 4.8.2 of NZS 4203 (SNZ, 1992) but with the 0.92 factor replaced by 1.0 and no additional load directed at the top.

In sensitivity analyses, Thurston and Park found that their results were not unduly sensitive to changes in the assumptions made.

2.7 Report by Thurston (2003)

An existing experimental house on the BRANZ site was relined with plasterboard and cyclically racked to failure. The plank exterior (with a single row of nails per plank) was expected to add little to the bracing resistance. The report compares the actual house strength with the strength determined using the NZS 3604:1999 design provisions (i.e., summing wall strengths derived from P21 tests). Free vibration tests to measure the house natural frequency and damping were also reported.

The averaged cyclic strength of the lined house was 50% greater than that predicted based on summing all the component walls and assuming all the walls were restrained against uplift. Although it was recognised that this is but one example of a typical New Zealand house, it indicated that simple summing of all component bracing walls gives a conservative estimate of total-house strength for single-storey structures. It also indicated that non-designated bracing walls should be used when computing house bracing strength.

Measured wall uplifts were very small, despite the walls having only single bottom-plate nails at 600 mm centres and fixings of internal walls being only into the sheet flooring. This implied that stud straps to resist uplift may not be necessary – except perhaps at doorway openings (Thurston, 1994).

The testing also indicated that 'cantilever' diaphragm action may be inadequate to transfer face loads from near the ends of a building to internal walls. Taking this into account, Appendix D of Thurston (2003) made recommendations for changes to provisions of NZS 3604:1999 on the distribution of bracing elements. However, at the house ultimate load no cracking occurred within the plasterboard ceiling and cracking along the wall/ceiling junction was light.

The average percent of critical damping, λ, determined from the house free vibration tests was 8.2%. The house natural frequency was measured as 20.8 Hz. This is a higher frequency than found by others (Fisher et al, 2000) and is attributed to the lightness of house construction.

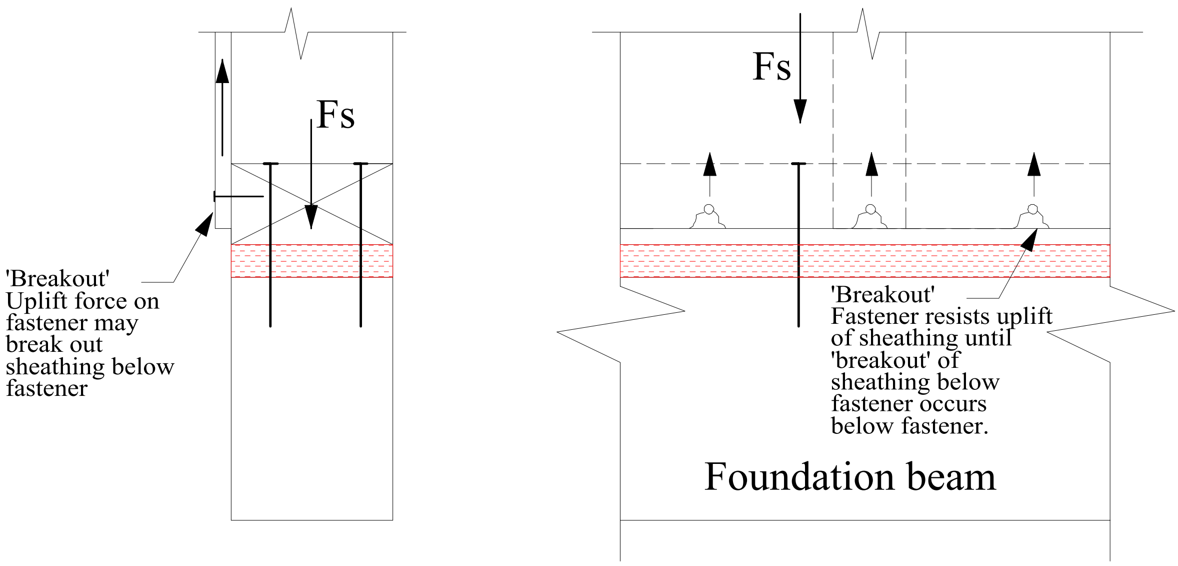


Figure 2. Transfer of sheathing uplift force to bottom plate via bottom plate nail.

2.8 Design codes used overseas

Herbert and King (1998) reviewed wall racking tests used overseas. The racking resistance of walls is determined by summing the strength of panels between door and window openings. New Zealand appears to be the only country which attempts to simulate in-service continuity effects when testing (isolated) bracing panels, by providing partial end stud 'supplementary' uplift restraint. Other countries adopt either a full end stud hold-down (Japan and USA) by means of tie-rods or have zero 'supplementary' uplift restraints (Australia and the U.K.) Using no restraint can give unduly conservative results. Full 'supplementary' uplift restraint is excellent for comparing the performance of sheathings under pure shear load. However, it does not give an upper limit for when 'rocking' will occur in actual construction. More importantly, it does not simulate the potential of fixings in the bottom plate to pull-out perpendicular to the edge of the sheathing under 'rocking' action as shown in Figure 2. The difficulty of the partial 'supplementary' uplift restraint is in the selection of the level of restraint to simulate actual construction. Too low a level (such as the current P21 uplift restraint) and all sheathings get the same bracing ratings. This rating then becomes a function purely of the 'supplementary' uplift restraint and not the sheathing fixing strength. This advantages weak sheathing systems.

3. PERFORMANCE OF BRACING WALLS IN ACTUAL CONSTRUCTION

3.1 Deflection compatibility

Houses in New Zealand are generally constructed with timber framed walls each with a variety of wall lengths, sheathing and fastening systems. The result is many different bracing systems, each of which achieves peak bracing resistance at different deflections. This incompatibility precludes the simple addition of peak strengths to obtain total lateral resistance. For instance, plasterboard (without fibreglass in the core) wall bracing systems generally reach peak resistance at 10–15 mm deflection and then drop in strength while plywood systems continue providing dependable and increasing resistance up to approximately 60 mm deflection. The proposed EM3 method addresses this problem by assessing the bracing resistances in a small deflection band (25-35 mm) to ensure at least moderated compatibility.

3.2 'Systems effects' and damping

Tests reported herein have noted that houses under racking load are stiffer and stronger than the sum of the individual wall panels due to the holistic response of the complete system. This is due to load sharing and composite action, of both the structural and non-structural elements, and the lateral restraint due to wall 'rocking' action being small, owing to the transfer of house weights to the ends of the bracing elements. In addition, the taped and filled joints between plasterboard sheet lining at both wall ends and ceiling are expected to significantly increase wall racking strength, as illustrated in Figure 3, not only due to the increased uplift restraint at wall ends but by changing the deformation mechanism from the sheet rotating about its centroid, as shown in Figure 4(c), to close to pure translation along the bottom plate. The sheet to bottom plate connection strength is enhanced by the zone under the windows.

The EM3 takes some account of this enhanced stiffness and strength (hereafter called 'systems effects'), by increasing the theoretical strength by a factor which varies between 1.0 and 1.2. This is justified by noting the good historic performance of houses under large wind and earthquake events.

When non-structural walls and other items contributing to the 'systems effects' fail, they are still likely to contribute to the damping. However, the analyses used as a basis for EM3 (Thurston and Park, 2003) only assumed 5% damping ratio (as also assumed in development of the NZS 4203 spectra (SNZ,1999)). This damping ratio is at the low end of reported damping measured in house tests (Fisher et al, 2000), indicating that the assumption is conservative.

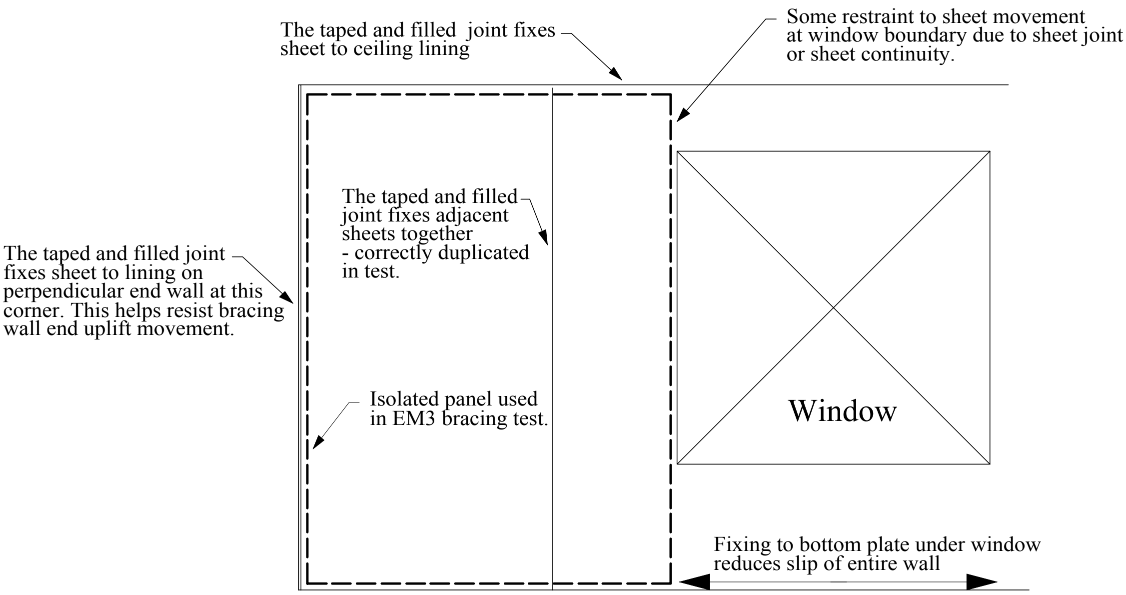


Figure 3. Restraint of wall lining in real buildings.

3.3 House bracing wall design philosophy

A bracing test evaluation method needs to take into account the return period of the design event and the likely life risk and property damage of the design event.

The design loads in NZS 4203 (SNZ, 1999) were based on the philosophy that there is only a 10% probability that NZ houses will experience an earthquake exceeding the design earthquake in any 50 year period.

Historically, timber framed bracing walls have performed well in large earthquakes.

A suitable design philosophy is considered to be that bracing walls only sustain minor cracks and are readily repairable after a design ultimate event, and do not collapse in an extreme event.

3.4 Bracing provided by 'nominal walls'

Where lateral loading is specifically considered, most house bracing standards in use around the world only consider the bracing due to specific wall bracing systems. An exception is AS 1684.2 (SA, 1999) which also assigns bracing ratings for 'nominal walls' which are defined to be wall framing lined with plywood, plasterboard, fibre cement board or the like when the frames are fixed to both the floor and roof or ceiling framing. In AS1684.2 the design bracing values used are 0.45 kN/m for walls sheathed on one side only (9 BU/m) and 0.75 BU/m (15 BU/m) for walls sheathed on both sides. It is recommended that this nominal bracing rating concept be adopted in New Zealand. However, the author considers that the values used above may be increased provided the stipulation is also made that the fixings around the perimeter of the bracing wall must be at a maximum of 300 mm centres. For instance a standard plasterboard system with fasteners at 150 mm centres has a published bracing rating of 50 BU/m. For fixings centres at 300 mm this becomes 25 BU/m >> 9 BU/m.

4. RACKING BEHAVIOUR OF ISOLATED TEST WALLS

4.1 Purpose of ‘supplementary’ uplift restraints

Most New Zealand house bracing walls are only fixed to the floor with bottom plate nailing (timber floors) or bolting (concrete floors).

Some walls founded on timber floors also have steel straps connecting the end studs to floor joists or joist blocking. Some walls founded on concrete floors have end steel straps fixed to the studs and wrapped around the bottom plate to resist the tendency for the two to separate. In other situations the wall exterior sheathing may extend past the bottom plate and be fixed directly to boundary joists. Other hold-down devices are sometimes used to stop the wall from lifting. For reference in this report, all the devices in this paragraph are defined as ‘special uplift restraints’.

If bracing panels are isolated from the surrounding structure and laboratory tested under horizontal racking loads, without any ‘supplementary’ end restraints to simulate continuity of actual construction, bracing walls without ‘special uplift restraints’ would fail at a low racking load. This is due to ‘rocking’ of the test wall about the bottom compression corner as shown in Figure 4(b). The associated uplift at the other (tension) end is due to either uplift of the bottom plate as the nails pull out of the timber floor or the studs lift from the bottom plate as the end nails withdraw. Only the latter movement is significant with construction on concrete floors if the bolt fixing the bottom plate to the floor does not fail and a large washer is used beneath the bolt head.

When bracing panels are built into a house, the wall sheathing, framing continuity and gravity effects provide resistance to uplifting, thereby reducing ‘rocking’ effects and increasing the house racking stiffness.

Gerlich (1987) proposed the ‘three-nail’ end restraint used in the current P21 tests as a replacement for the nail-pullout restraint previously used in the P21 method. It was designed to replicate the minimum nail fixing used at wall corner junctions. Gerlich ignored the additional uplift restraint due to the usual taped and filled joints in corners of plasterboard lined houses. However, no attempt was made to replicate the (weaker) uplift restraint typical at doorway openings. Gerlich stated that he had conservatively ignored the additional restraint provided by ceiling framing members, upper storey joists etc. Gerlich reported on tests where he attributed the apparently ductile wall performance to being the ductile movement due to yielding of the ‘three-nail’ end restraint rather than the wall shear deformation.

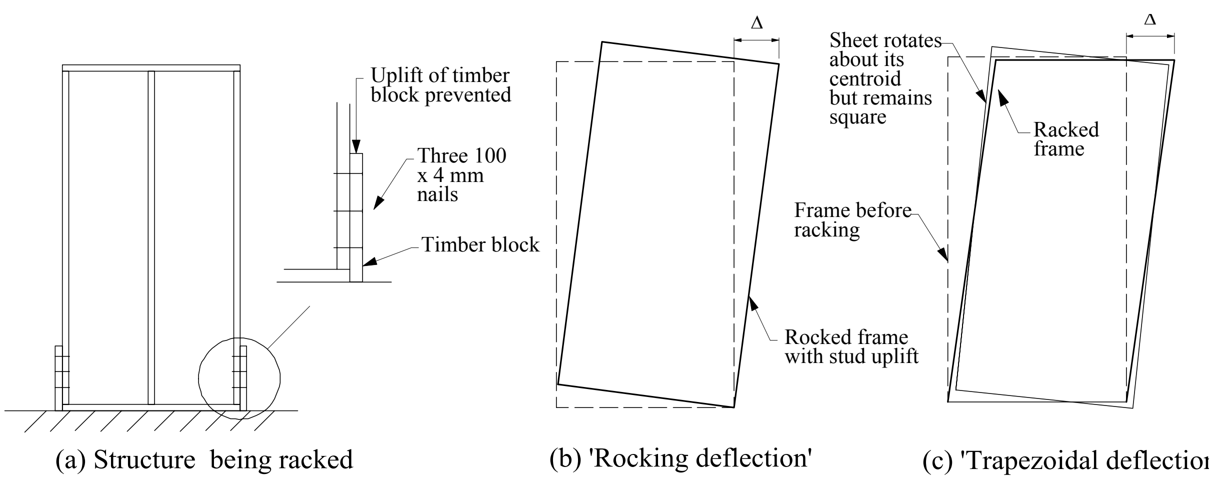


Figure 4. Components of racking wall deflection and sketch of the BRANZ P21 uplift restraints.

4.2 Appropriate magnitude of 'supplementary' uplift restraint?

The question needs to be asked, "should the New Zealand bracing test use a full 'supplementary' uplift restraint (like USA and Japan), a partial restraint like the current 'P21 uplift restraint' or none (like England, Eurocodes and TR 440 Australia)".

4.2.1 Zero 'supplementary' uplift restraint

If no 'supplementary' uplift restraints are used in a bracing test then significantly lower wall bracing ratings are expected unless strong 'special uplift restraints' are incorporated as part of the wall construction. This will lead to the necessity of using these restraints in building construction and consequently give rise to significant increased building cost. The author considers that there is no justification for this change in construction based on the good reports of performance of New Zealand and comparable overseas houses in large wind and earthquake events.

4.2.2 Full 'supplementary' uplift restraint

Precluding panel 'rocking' action by provision of end restraints which prevent any end stud-uplift has some attraction as the test will then be a measure of the 'fastener strengths' and the racking strength by definition will be the 'trapezoidal strength'. This can be predicted reasonably accurately from load versus 'fastener slip' tests (Thurston, 1993, 2000, Patton-Mallory and McCutcheon, 1987). The design wall racking strength derived from the test can be reduced to account for 'rocking' if required. The 'rocking strength' is a function of actual building construction with the critical parameters being the presence of 'special uplift restraints', wall axial load and construction adjacent to the panel boundaries (e.g. at doorways, corners, window edges).

Modifying the P21 test to use full 'supplementary' uplift restraints was rejected because:

1. Under 'rocking' action the fixings may break out of the edge of the bottom of the sheet as shown in Figure 2. This is not simulated in a racking test with full 'supplementary' uplift restraint but is a real phenomena in a house subjected to racking (Thurston, 2000). Having a test method with full supplementary' end restraint would encourage systems with small fastener edge distances which is undesirable. It would not penalise systems with low 'breakout' strength.

2. Under tests with full 'supplementary' uplift restraint there is no advantage given to systems which use 'special uplift restraints' or have large 'breakout' strength.

3. The bracing ratings of walls constructed using multi-layers of sheathings on each side and tested with full 'supplementary' uplift restraints can be very high. However, 'rocking' action in actual construction would prevent this high rating from being achieved.

Therefore a partial restraint system was chosen with the important point being the choice of an appropriate magnitude of 'supplementary' uplift restraint.

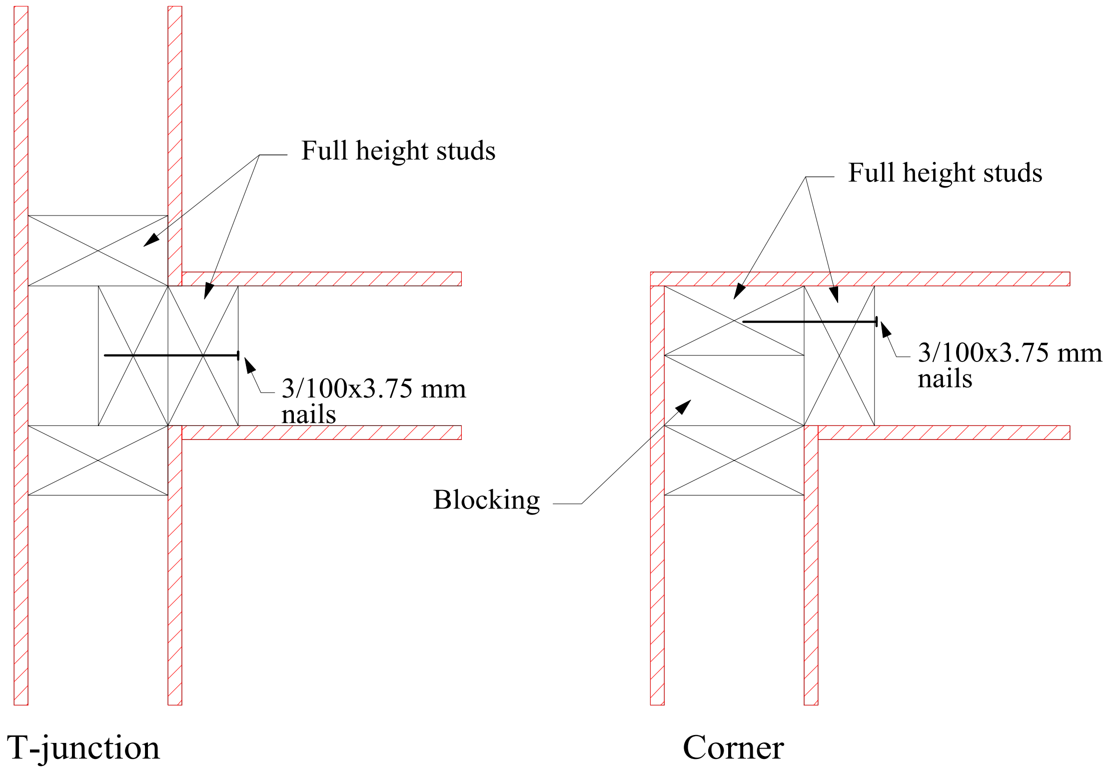


Figure 5. Typical wall junction showing continuity construction affecting uplift restraint.

4.3 Magnitude of 'supplementary' uplift restraint

In the P21 test method the 'supplementary' uplift restraint is effectively the strength of three nails in shear as shown in Figure 4(a) which results in a relatively low 'rocking strength'. Consequently, the strengths of many P21 bracing tests are governed purely by 'rocking' and not by the cladding 'fastener strength'.

The 'P21 uplift restraint' was designed (Gerlich, 1987) to model the minimum restraint provided by the nailing specified in NZS 3604:1984 from the wall end stud to the return wall framing as shown in Figure 5 in walls without taped and filled joints at corner junctions. Such conditions generally only occur in garages and attics. Most New Zealand house wall junctions are lined and taped and filled at the corners. In these instances the racking induced uplift force is likely to be transmitted from the racked wall to the return wall via the taped and filled lining joint. Thurston (1993) measured this strength as 8 kN/m along the joint, which is sufficiently high that the generalisation can be made that the uplift loads in the racked wall panels can be transmitted to the return wall linings provided the return wall can carry such uplift forces.

Herbert and King (1998) tested walls with openings and return walls to determine the effective uplift restraint. They concluded that the construction restraint at timber framed walls with fully taped and filled joints at a corner could be duplicated by a 12 kN connection restraint. Thurston (2003) measured the strength of the '6-nail' proposed 'EM3 uplift restraint' at 6 mm uplift as 9.50 kN with 35 mm wide studs and 8.96 kN with 45 mm wide studs and which are slightly less than the Herbert and King recommendation. However, they concluded that the effective uplift restraint at a standard door lintel was only 3 kN – i.e., half that used in the current P21 test procedure.

Thurston (1993) tested long walls with various openings and concluded (see Section 2.2) that the bracing of long walls with window openings could conservatively be determined from the sum of the component panels between window openings when tested with full uplift restraint provided:

Hold-down straps were used at door openings.
The walls were lined with plasterboard with taped and filled joints.

Thurston (2003) racked a full sized house and concluded (see Section 2.7) that the house racking strength could conservatively be determined from the sum of all wall zones between openings when tested with full uplift restraint provided the walls were lined with plasterboard and had taped and filled joints. It should be noted that the house did not contain doorway openings in the direction Thurston tested.

Full scale testing (see Section 2.5) of a plasterboard lined house which did include some ‘special uplift restraints’ found that the measured house strength exceeded the predicted strength assuming all walls had full uplift restraint.

It is therefore concluded that use of the ‘EM3 uplift restraint’ is conservative for tests simulating wall racking strength for houses with plasterboard lining with taped and filled joints except near door openings without hold-down straps. The ‘EM3’ method in EM3-V1 (Thurston, 2004) downgrades the rating by a factor of 0.8 for unlined walls and also for top storey and single storey houses at walls terminating in door openings not using a hold-down strap. This is discussed further in Section 11.4.

The ‘supplementary’ end restraint in the proposed EM3 test method uses six gun-nails rather than three hand-driven nails of the P21 test method. Gun-nails were adopted as firing these nails causes less vibration damage to the wall sheathing fixings than hammering hand driven nails. The time saving in using gun-nails was of secondary concern.

4.4 Geometry of wall deformation in a partial end restraint test

4.4.1 Wall deflection due to ‘rocking’ action

A sketch of the component of wall deflection due to ‘rocking action’ is shown in Figure 6. The wall lifts a distance, $\delta 1$, at the tension end and sinks a distance, $\delta 2$, at the compression end. The deflection $\delta 1$ is usually due to lifting of the bottom plate from the foundation beam and/or lifting of the stud from the bottom plate. The deflection $\delta 2$ is usually due to indentation of the stud (under compression load) into the bottom plate and is usually smaller in magnitude than $\delta 1$.

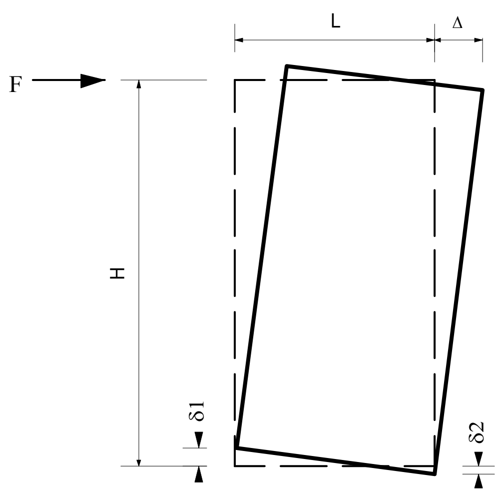


Figure 6. Wall ‘rocking deflection’.

From the geometry in Figure 6 the horizontal wall deflection, Δ, due to 'rocking' can be expressed:

$\Delta = (\delta 1\text{-}\delta 2)xH/L$ ………….. (1)

(The sign convention in this equation is that deflections $\delta 1$ and $\delta 2$ are positive for uplift movement.)

4.4.2 Wall deflection due to 'fastener slip'

A sketch of the component of wall deflection due to 'fastener slip' (i.e., no 'rocking' action) is shown in Figure 7 (Thurston and Flack, 1980). (For clarity the deflections have not been plotted to scale.) The angles, α, β and γ are measured in radians. Under this action the wall sheathing (shown shaded) rotates but does not change shape (ie it retains a rectangular shape). The frame is assumed to act as pin ended members with the top and bottom plate remaining horizontal (as there is no 'rocking' action).

The horizontal deflection of the wall can be found from the measured slip deflections between the sheathing and the timber frame referred to as 'fastener slip'. In the derivation below all 'fastener slip' deflections are taken as absolute positive and use the following nomenclature:

S_{top} = horizontal slip between sheathing and top plate
S_{bottom} = horizontal slip between sheathing and bottom plate
S_{side1} = vertical slip between sheathing and stud on side 1
S_{side2} = vertical slip between sheathing and stud on side 2

The rotation of the sheet relative to the studs, β, can be found from geometric considerations:

$\text{Tan}\,\beta = \{(S_{top} + S_{bottom})/H\}$
Thus, $\beta \approx (S_{top} + S_{bottom})/H$ because the angle change is small.

Similarly:

$\alpha \approx (S_{side1} + S_{side2})/L$

The short side of the sheet rotates an angle α to the horizontal. Therefore (as the sheet does not change shape) the long side of the sheet also rotates an angle α to the vertical. As shown in Figure 7, the frame rotates an angle β further than this. i.e., from geometric considerations: $\gamma = \alpha + \beta$

The horizontal deflection of the top plate, Δ, is given by:

$\Delta = H\text{Tan}\gamma = H\text{Tan}(\alpha + \beta) \approx H(\alpha + \beta)$ …………. (2)

Thus, substituting for α and β derived above into eqn (2) gives:

$\Delta \approx (S_{top} + S_{bottom}) + (S_{side1} + S_{side2})H/L$ ……. (3)

Note, that if the 'fastener slip' along each side is constant and given by: $e_n = S_{top} = S_{bottom} = S_{side1} = S_{side2}$ and if H/L is put = 'a' (the wall aspect ratio), then eqn (3) beomes identical to the deflection of a shear wall due to 'fastener slip' given in Eqn. 5.29 of NZS 3603 (SNZ, 1993) for a single sheathing panel.

Bracing wall horizontal deflections measured in the tests described later in this report are compared with a predicted deflection of the top plate, Δ_p, given by:

$\Delta_p = \Delta_{slip} + \Delta_r$ …….. (4)

Where:

Δ_{slip} = $(S_{top} + S_{bottom}) + (S_{side1} + S_{side2})H/L + \Delta_{base}$ (see Eqn. (2))

Δ_{base} = slip of base of wall relative to the foundation beam

Δ_r = $(\delta1-\delta2)xH/L$ (see Eqn. (1))

Rocking deflections, δ, measured in this report were taken at mid-thickness of the end studs on the opposite face of the studs to the sheathing on single sheathed walls. It was noted in many tests where the stud separated from the bottom plate that significant bottom plate rotation occurred as shown in Figure 12. Thus the perpendicular to the sheet edge movement of the fasteners was less than the measured stud-uplift movement which explained why the fasteners had not broken out of the sheet edge until the measured stud movement far exceeded the fastener edge distance.

4.4.3 Wall deflection due to other deformations

Eqn (4) does not include shear and flexural deformation of the sheathing and framing, movement due to any horizontal separation of the studs from the plates and slip of the foundation beam relative to the foundation. These movements are expected to be small. Patton-Mallory and McCutcheon (1987) stated that they usually account for only 5%-10% of the total deflection.

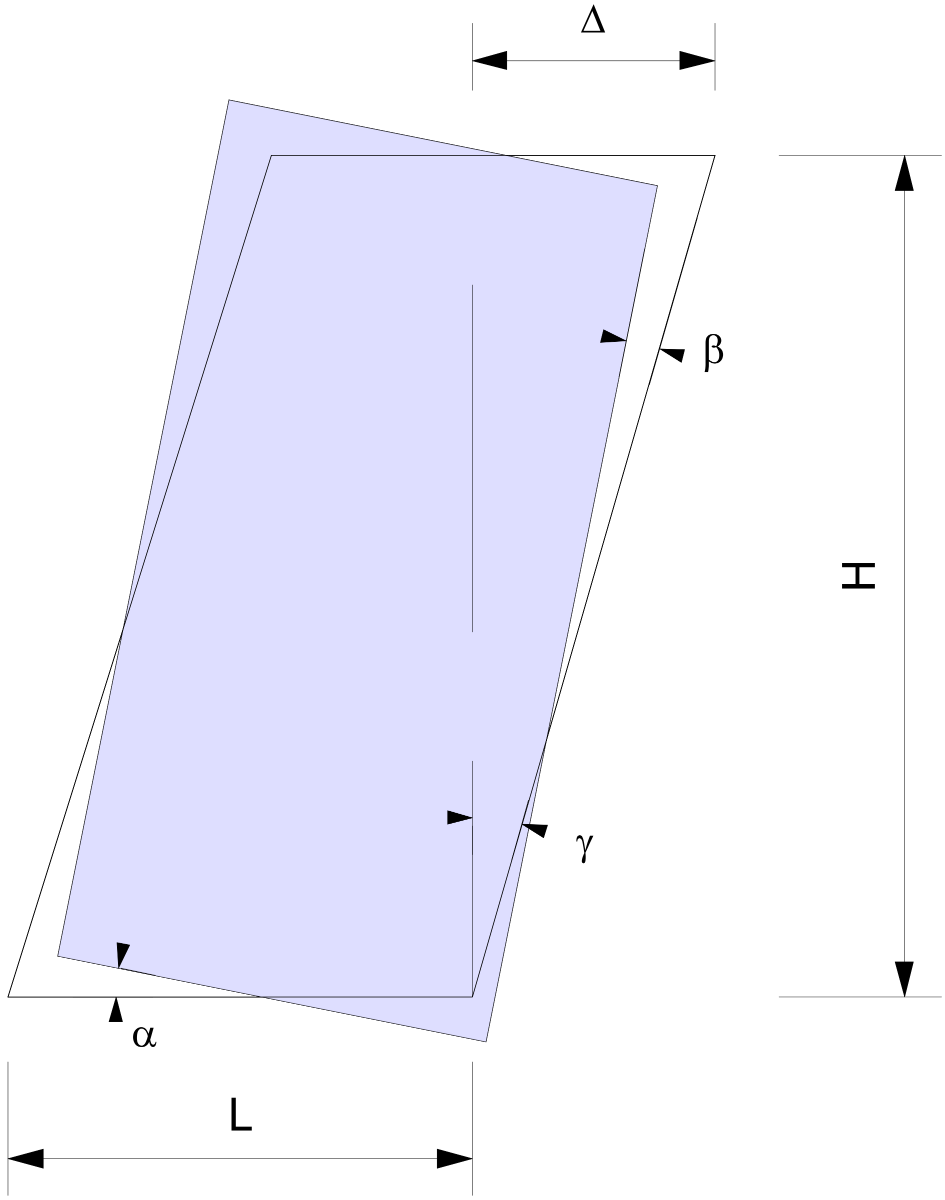


Figure 7. Deflection of wall due to 'fastener slip'.

4.4.4 Predicting wall deflection from load versus 'fastener slip' data

If the relationship between horizontal shear load at the top plate and 'fastener slip' is known then the McCutcheon (1985) method or else the CASHEW computer program developed by Foltz and Filiatrault (2000) may be used to predict the wall racking deflection for zero panel 'rocking'. Patton-Mallory and McCutcheon (1987) found excellent agreement between their method and test results. Appendix A of Thurston (2003) took 'rocking' into account and also obtained excellent agreement with both methods and his test results. However, this project is not concerned with predicting test results, but rather determining the proportion of measured deflection attributable to 'fastener slip' and the proportion due to whole panel 'rocking' and for this end Eqn (4) is more useful.

4.5 Maximum wall bracing force in P21 or EM3 tests.

4.5.1 Mechanisms resisting panel 'rocking'

The strength of a bracing panel is the lesser of the 'rocking strength' and the panel strength from other mechanisms (usually governed by 'fastener strength' but could be by sheet rupture, frame fracture etc.) 'Rocking' action can result in sheet 'breakout' as shown in Figure 2.

If the limit of a bracing panel strength is the 'supplementary' uplift restraint, this will give an apparently ductile behaviour, limit the maximum racking force that can be applied and act like an 'isolator' to protect other wall failure modes.

The tendency for a test panel to 'rock' about the compression corner as shown in Figure 4(b) is resisted by the following mechanisms (see Figure 11):

1. Supplementary uplift restraints (Fs). (See Figure 4(a)).
2. Any 'special uplift restraints' (Fer).
3. Bottom plate fixing to the foundation beam (Fn) (usually nails in a simulated timber floor or bolts in a simulated concrete floor). The fasteners must transfer this vertical uplift force from the sheathing into the bottom plate and the sheathing 'breakout' strength often proves to be inadequate to fully utilize the bottom plate anchorage strength. 'Breakout' is illustrated in Figure 2.
4. Stud to bottom plate nailing. See Figure 13.
5. Self weight of the panel.

4.5.2 Limitations of racking force due to rocking reported in other tests

Thurston (1993) predicted the 'backbone' curves for fibre cement sheathed walls of various lengths, having full 'supplementary' uplift end restraint, by applying the McCutcheon (1985) theory using measured load versus 'fastener slip' data. The deflections due to 'rocking deflection' were then added to give the predicted 'backbone' curve in a P21 test. For this strong sheathing, the results clearly showed that walls of 2.4 m long or shorter would be governed by 'the rocking strength' whereas walls 3.0 m or longer would be governed by the 'trapezoidal strength'.

4.5.3 Measurement of the strength of the P21 and EM3 uplift restraints

In Appendix B.2 of Thurston (2003), he measured the strength of the 'P21 end restraint' and the 'EM3 end restraint' through 35 mm and 45 mm thick timber studs (kiln dried machine stress graded F5 Radiata pine) using an average from six replica. The joints were cyclically tested at 0.2 mm/second. The averaged backbone curves to the hysteresis loops are given in Figure 8 for a 'four-nail' joint and indicate similar strengths for both power-driven nails and hand-driven nails with little variation due to thickness of stud. Data is taken from this plot in calculations given below. The value used for simulation of P21 restraint (4.85 kN) is taken at a 'fastener slip' of 6 mm and is proportioned for three hand-driven nails using the mean test results for 35 and 45 mm studs. The value used for simulation of EM3 restraint (9.23 kN) is taken at a slip of 6 mm and is proportioned for six power-driven nails using the mean test results for 35 and 45 mm studs.

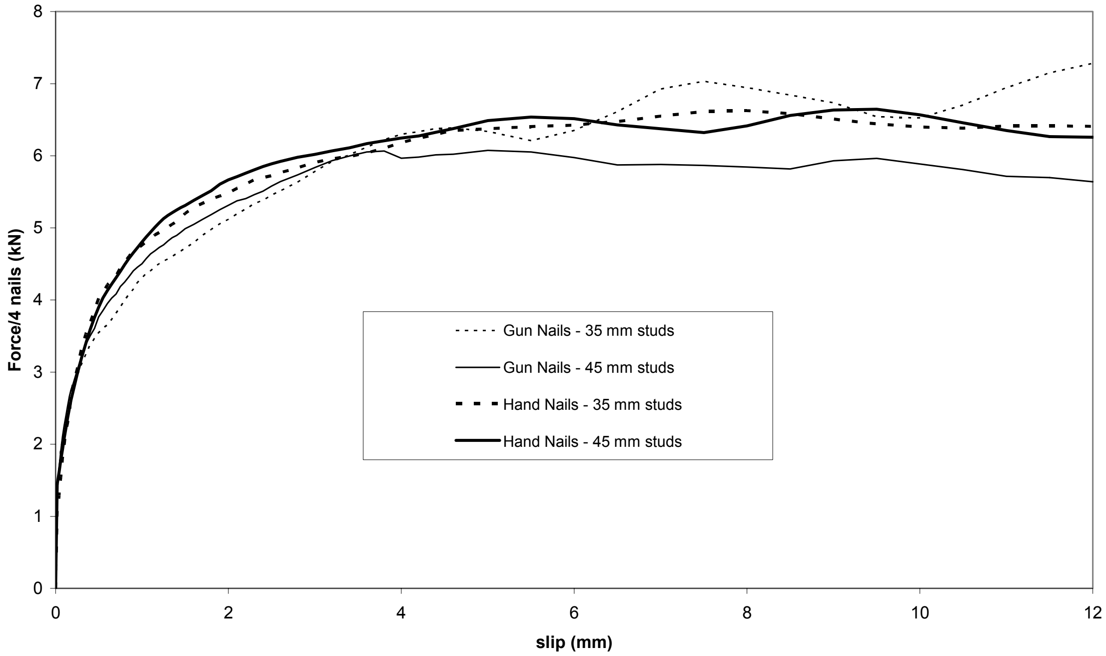


Figure 8. Backbone curves to hysteresis loops for a 'four-nail' end restraint joint.

Herbert and King (1998) separately measured the strength of a P21 uplift restraint as 6.5 kN which is greater than the 4.85 kN derived above. They used a different load rate which was not stated (load rate can significantly influence measured strengths), used the maximum load achieved rather than peak load in the 0-6 mm range and their tests were monotonic rather than cyclic. For these reasons these results are not adopted.

4.5.4 Tensile strength of bottom plate to timber foundation nailed connection

Table 8.19 of NZS 3604:1999 specifies options of hand-driven and power-driven nails for fixing bracing walls to timber floor framing. Unless specifically specified as part of the bracing system the EM3 test specifies the hand-driven option be used. This is to give consistency between testing Laboratories and a lower bound solution. The requirement is that that the bottom plate shall be fixed down by two hand driven 100 x 3.75 mm nails at 600 mm centres commencing 50 mm inside the end studs.

Section A.2 gives the measured pullout strength of nailed bottom plate to timber framing connections performed as part of this project and as reported by others. It was concluded that for the calculations used below the pullout strength of the two bottom plate nails should be taken as 3.08 kN. However, this force can only be activated if the fixing between sheet and bottom plate or stud and bottom plate is strong enough to transfer the force between sheet and bottom plate. 'Breakout' of the fixing from the sheet edge (as shown in Figure 2) is common.

4.5.5 Tensile strength of bottom plate to concrete foundation connection

NZS 3604:1999 does not specify the required anchorage of bracing walls to slabs as this is considered to be an essential part of the bracing system and is expected to be reflected in the test results. Section 7.5.12.4 of NZS 3604 (SNZ, 1999) requires the bottom plate connection on all external walls (excluding bracing walls) to have a lower 5 percentile pullout design strength of 8 kN. The average strength will be significantly greater than the design strength (Shelton, 2003). M12 bolts or R10 dowels may be used as stipulated in Section 6.11.9 of NZS 3604 (SNZ, 1999) and are expected to have an average pullout strength greater than 8 kN.

4.5.6 Tensile strength of bottom plate to timber stud nailed connection

Table 8.19 of NZS 3604 specifies four options for fixing wall framing studs to plates. For consistency between testing Laboratories, the EM3 test specifies only a single option. The requirement is that unless specifically specified as part of the bracing system that the plates shall be nailed to the studs with two 90 x 3.15 mm of a specified brand of power-driven nails.

Section A.1 of this report gives the measured pullout strength of nailed bottom plate to stud joints in tests performed as part of this project and as reported by others. It was concluded that for calculations used below, the pullout strength of the two nail connection between stud and plate should be taken as 1.53 kN. The fixings between sheet and bottom plate also help to transfer the uplift force between sheet and bottom plate until 'breakout' of the fixing from the sheet edge (as shown in Figure 2) occurs.

4.5.7 Shear strength of bottom plate to timber foundation nailed connection

The measured shear strength of the P21 end restraint per nail (see Section 4.5.3) is expected to be similar to the shear strength of the nailed bottom plate connection. A 1.2 m long wall has six nails between bottom plate and foundation. The P21 restraint strength measured by Thurston (2003) was 4.85 kN for three hand driven nails which proportions to 9.7 kN for six nails which is 8.08 kN/m (162 BU/m). This can be considered the maximum wall shear before the bottom plate slip becomes large.

A 2.4 m long wall has ten nails between bottom plate and foundation. Proportioning as above gives a maximum shear force of 16.1 kN or 6.74 kN/m (135 BU/m).

Use of 'special end restraints' may increase this value. In actual construction fixing of the bottom plate beneath windows and non-structural walls is likely to ensure that bottom plate shear connection failure does not occur provided fixings are into the floor framing.

4.5.8 Shear strength of bottom plate to concrete foundation bolted connection

The shear strength of walls using M12 bolts at 1200 mm centres can be estimated from the 9.74 kN design shear strength of an M12 bolt parallel to the grain given in Table 4.10 of NZS 3603 (SNZ,1993). The average strength will be significantly higher and is clearly unlikely to limit maximum wall racking ratings.

4.5.9 Strength of hold-down straps on bracing walls with timber foundations under cyclic loading

Typical details of steel hold-down straps on timber foundations as published in manufacturers' technical information are shown in Figure 9. The "6 kN" and "12 kN" labels in the figure does not necessarily refer to the connection strength. If the bracing wall is not directly over joists then blocking must be used as shown in the figure.

Under cyclic loading the hold-down strap tends to buckle. From Table 10, the straps have increased the wall '1.2 m long rocking' strength by 6.07-3.47 = 2.60 kN/m for walls with a 'P21 end restraint' and 8.16-5.59 = 2.57 kN/m for walls with an 'EM3 end restraint'. The average value (2.59 kN/m) will be used which proportions to 3.10 kN for a 1.2 m long wall. Taking into account the wall has an aspect ratio of 2.0, the uplift strength of the strap is effectively 2 x 3.10 = 6.20 kN.

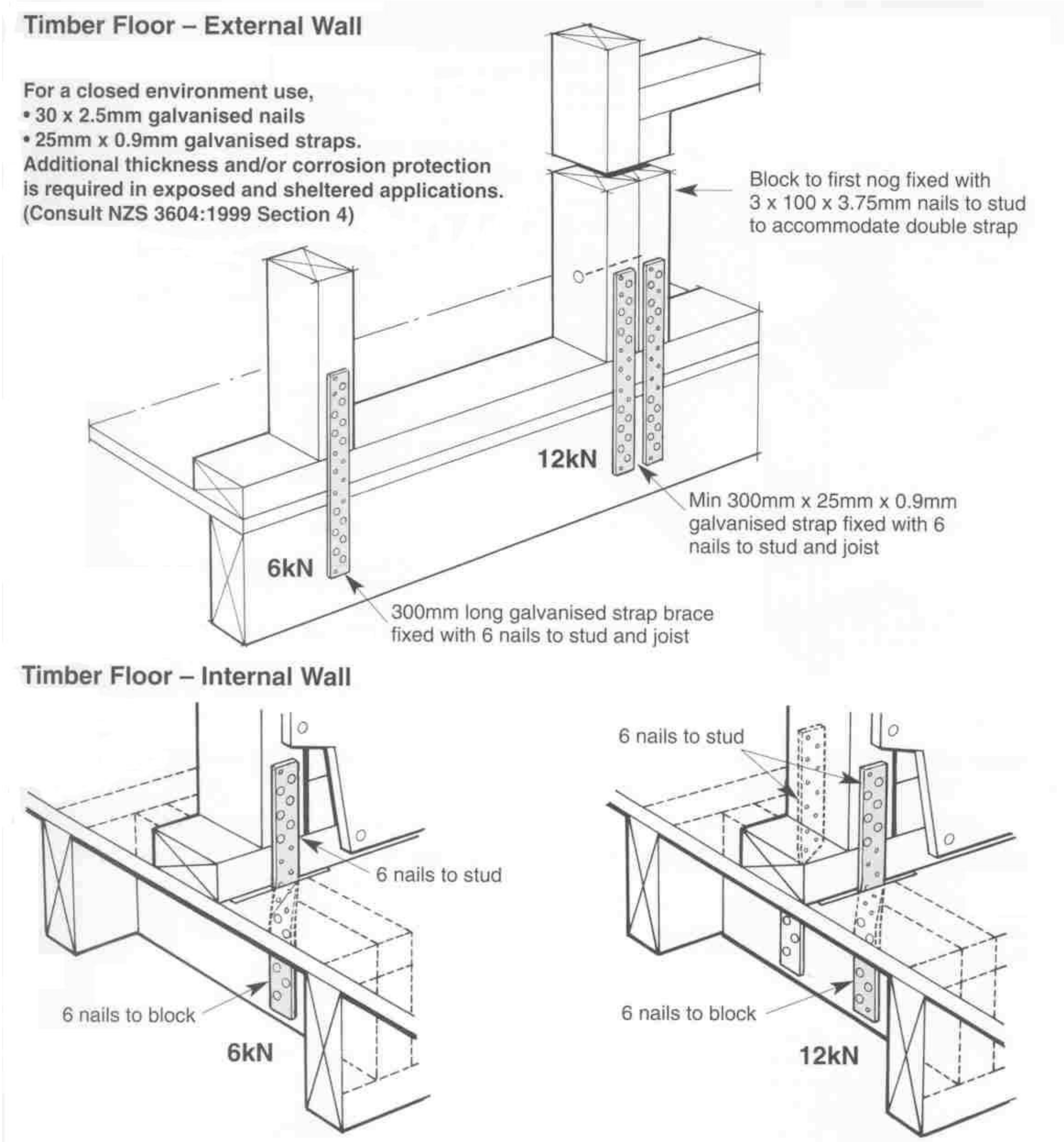


Figure 9. 6 kN hold-down straps for bracing walls on a timber foundation (printed with permission from Winstone Wallboards).

4.5.10 Strength of hold-down straps on bracing walls with concrete foundations under cyclic loading

Typical details of steel hold-down straps on concrete foundations as published in manufacturers' technical information are shown in
Figure 10. The "6 kN" and "12 kN" labels in the figure does not necessarily refer to the connection strength. However, a comparison of Tests 50 and 51 in Table 15 shows that the detail in
Figure 10 increased the maximum racking force resisted by 7.52-5.11 = 2.41 kN or the strap strength was 2 x 2.41 = 4.82 kN. It was noted that this was not a good detail as the bottom plate split at the top and bottom nails into the plate. The same comparison cannot be made with the 2.4 m long walls as the stud did not separate from the bottom plate in the test using a strap (Test 44) and thus the full strength of the strap was not determined.

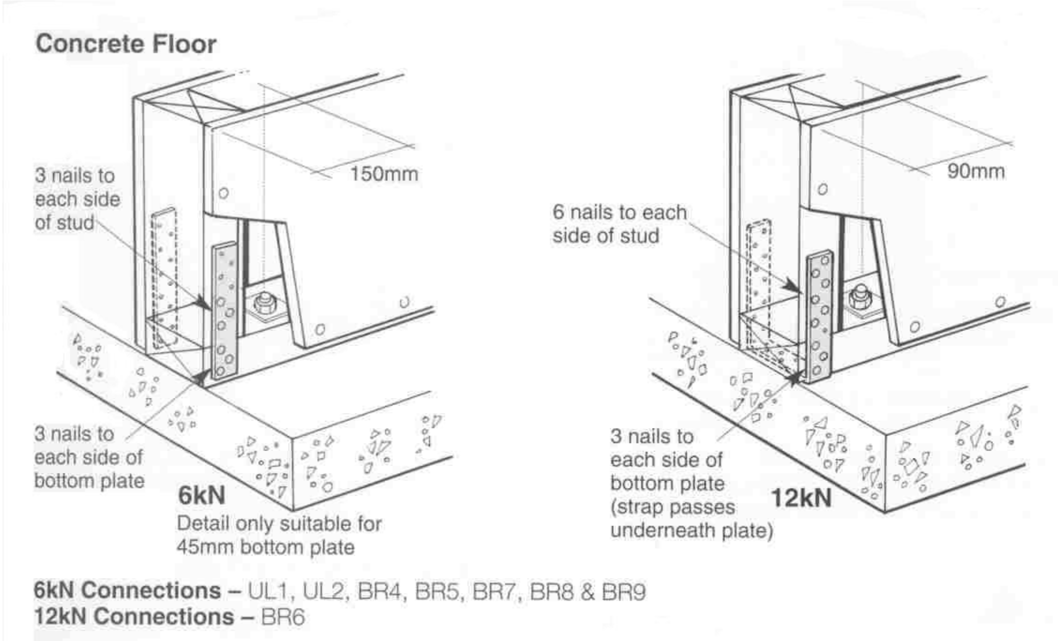


Figure 10. 6 kN hold-down straps for bracing walls on a concrete foundation (printed with permission from Winstone Wallboards).

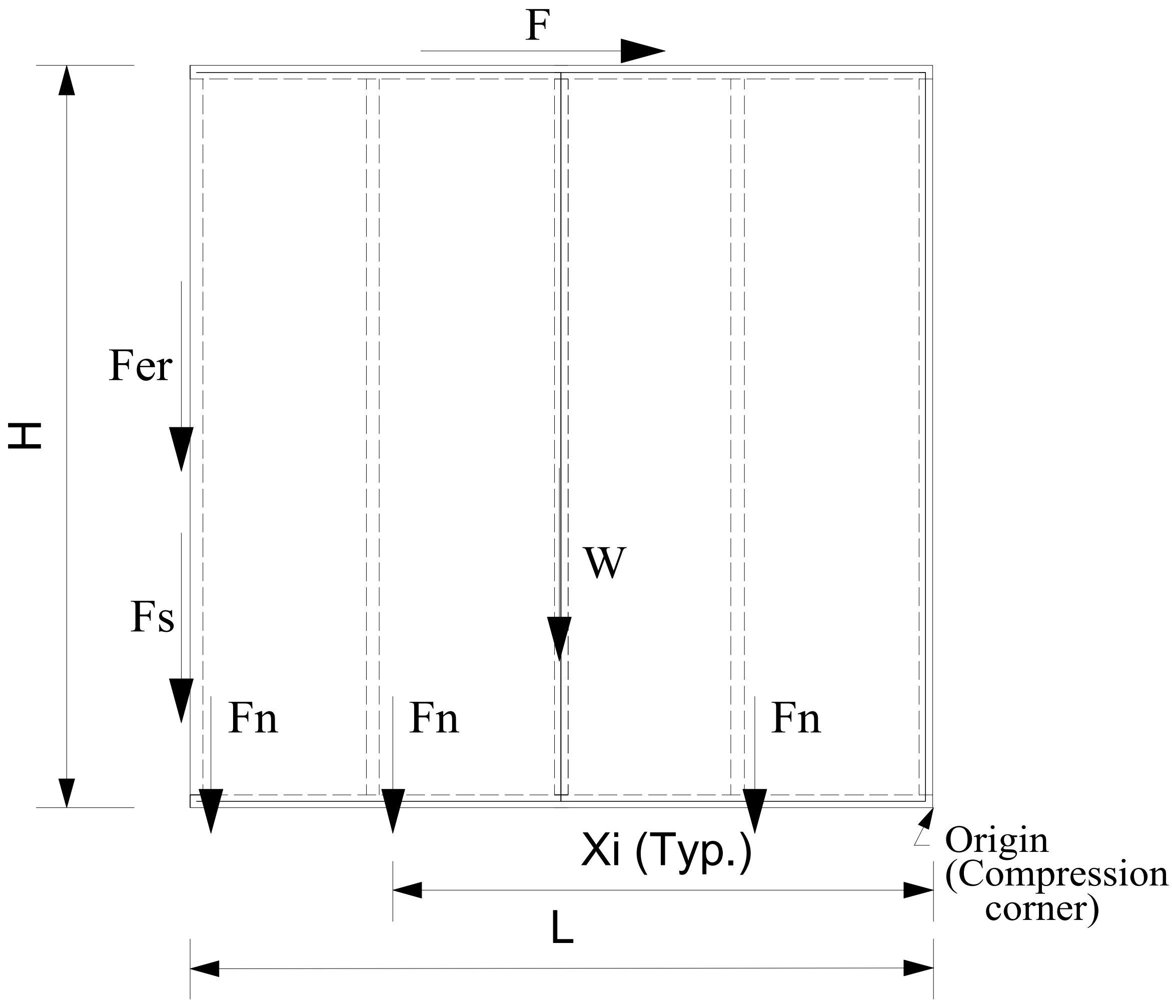


Figure 11. Forces to resist 'rocking' motion due to 'bottom plate uplift'.

4.5.11 Calculation of racking force to induce 'rocking' due to 'bottom plate uplift'

In the tests described in this report, 'rocking' action occurred either due to 'bottom plate uplift' from nails pulling out of the timber foundation beam or else through 'stud-uplift' off the bottom plate. This latter action is covered in Section 4.5.12.

The racking force, F, as shown in Figure 11, tends to 'rock' the panel about the bottom compression corner marked as the origin in Figure 11. This is resisted by the panel self weight, W, the 'supplementary' uplift restraint, Fs, 'special end restraints' (if any) of strength Fer and the sum of the nail fixings between bottom plate and foundation beam. Each of these nails is taken as having a strength Fn and acting at a variable distance from the compression corner – taken as distance Xi where Xi = 600, 1200, etc for all fastener locations.

Taking moments about the origin in Figure 11 results in the following equation:

$$F \times H = (Fs + Fer) \times L + W \times L/2 + \Sigma(Fn \times Xi) \quad \ldots\ldots.. (5)$$

From Equation (5) the maximum wall bracing force per unit wall length of a 2.4 m high wall is presented in Table 1 in units of BU/m. For this calculation W was calculated assuming the weight of the sheathing was 10 kg/m^2 and the density of the framing was 450 kg/m^3. The other values used are as derived in Sections 4.5.3 - 4.5.9. Section 4.5.3 gave Fs = 4.85 kN for the P21 'three-nail' end restraint and 9.23 kN for the EM3 'six-nail' end restraint. Where specified, a hold-down strap was assumed to be used between end studs and foundation with strength Fer = 6.2 kN (see Section 4.5.9).

The force Fn is the double nail strength between bottom plate and foundation and was taken as 3.08 kN at 600 mm centres. Values of Fn = 1.5 and 0 kN were also used to examine weak sheathings where it was assumed that the 'breakout' strength of bottom fixings were inadequate to mobilise plate uplift. By comparing the bracing ratings in Table 1 for the three assumptions it can be seen that the ability of the wall sheathing to transmit the uplift force from the sheathing to the bottom plate increases the bracing rating of the wall significantly. This is the 'breakout' strength which is a function of sheathing strength and fastener edge distance.

Assuming no 'breakout', the increase in maximum bracing rating between a single and double sheathed wall before 'rocking' dominates, is small. This can be seen by comparison of the single sheathing and double sheathing values in Table 1.

Also shown in Table 1 is the maximum bracing rating of walls when 'rocking' dominates based on the equations in Section D.3 of Thurston (1993). These are based on the P21 test results from a large range of racking tests performed at BRANZ. These 'rocking strengths', based on wall bracing tests, are all slightly higher than the theoretical values under the column headings: Fn = 3.08 kN.

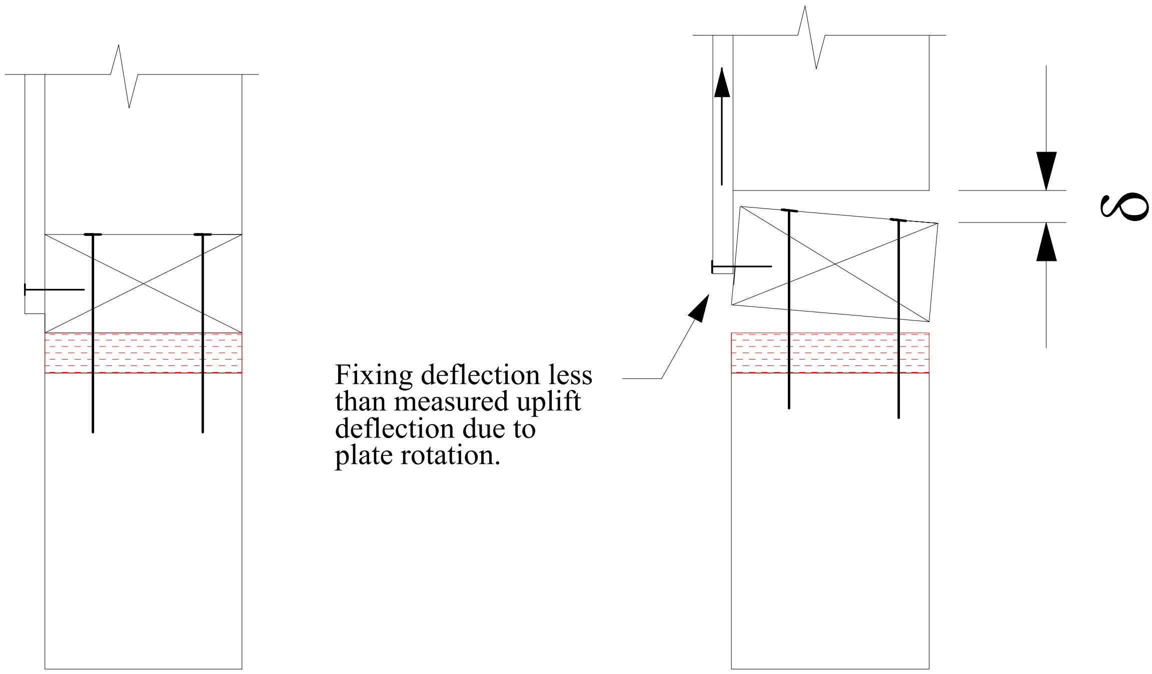
Table 1. Theoretical bracing rating of walls based on 'bottom plate uplift' (BU/m length).

		Single Sheathing			Double Sheathing			
	Straps?	Fn = 3.08 kN	Fn = 1.54 kN	Fn = 0 kN	Fn = 3.08 kN	Fn = 1.54 kN	Fn = 0 kN	*
P21 end restraint								
0.9 m long wall	Yes	131	112	94	132	113	95	148
0.9 m long wall	No	79	61	42	80	61	43	-
1.2 m long wall	Yes	131	113	94	133	114	95	149
1.2 m long wall	No	80	61	42	81	62	43	99
1.8 m long wall	No	94	68	43	96	70	45	107
2.4 m long wall	No	108	76	44	110	78	46	116
3.0 m long wall	No	121	83	45	124	86	48	141
EM3 end restraint								
0.9 m long wall	Yes	167	149	130	168	150	131	
0.9 m long wall	No	116	97	78	116	98	79	
1.2 m long wall	Yes	168	149	130	169	150	132	
1.2 m long wall	No	116	98	79	117	99	80	
1.8 m long wall	No	130	105	80	132	107	81	
2.4 m long wall	No	144	112	81	146	115	83	
3.0 m long wall	No	158	120	81	161	123	84	

Legend to Table 1.

* Maximum bracing rating based on the equations in Section D.3 of Thurston (1993).

In Figure B.5 of Thurston (2003) he showed that the 'P21 end restraint' only accounted for approximately 50% of the 'rocking' restraint of 1.2 m long test walls.



(a) Before loading (b) After loading

Figure 12. Effect of rotation of bottom plate.

4.5.12 Calculation of racking force to induce 'rocking' due to 'stud-uplift'

'Rocking' action of walls on timber foundations is either due to 'bottom plate uplift' or else 'stud-uplift' off the bottom plate. For construction on concrete foundations, the bottom plate is assumed to be rigidly fixed to the foundation by bolts placed at 1200 mm centres commencing 100 mm inside the wall end studs (as specified in EM3-V1 (Thurston, 2004)). The 'rocking' mechanism is due to a 'stud-uplift' from the bottom plate as shown in Figure 13. For this mechanism to occur there must be a similar movement between bottom plate fasteners perpendicular to the sheet edge, say δ', as the stud lifts by say, δ. However, bottom plate twisting (see Figure 12) and other effects usually result in δ' being less than δ. The tendency for 'breakout' to occur due to 'stud-uplift' is illustrated in Figure 13. For instance, if 'studs uplift' δ = 20 mm and δ' = 12 mm, then the fasteners will have broken out of the sheet edge (see Figure 2) and the ability of the affected fasteners along the bottom plate to transfer shear load will have been lost.

The same equations apply as in Section 4.5.11 except that the Fn now equals Fj being the strength of the stud to bottom plate joint (taken as 1.53 kN from Section 4.5.6) for all studs. A fixing strength between fasteners and sheathing perpendicular to the sheathing edge, Fp, of 3.0, 1.5 and 0 kN/m has been assumed. Equation (5) is now re-expressed as:

$$F \times H = (Fs + Fer) \times L + W \times L/2 + \Sigma(Fj \times Xj) + Fp \times L^2/2 \text{ …….. (6)}$$

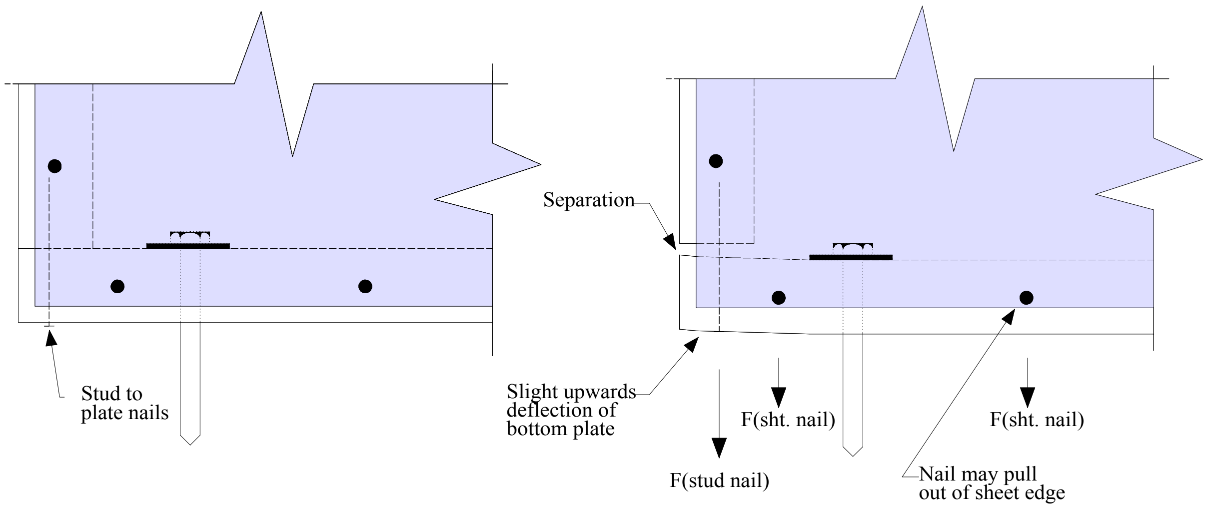
Where specified as being used, the end strap strength is taken as 6.2 kN. However, Section 4.5.10 gave different strengths for stud/plate steel strap connections for construction on concrete foundation. i.e., for construction as shown in
Figure 10 the strength = 4.82 kN.

To simulate a wall with a double sheathing the value of Fp has been doubled as well as considering the extra weight.

Table 2 gives the maximum bracing rating of walls. Assuming Fn = 3.08 kN, then Table 2 indicates that 'stud-uplift' will likely occur before 'bottom plate uplift' unless the 'breakout' strength is high or end straps are used to connect studs to bottom plate in a simulated concrete foundation construction. However, racking tests need to be done on both foundation types to see if walls tested on a concrete foundation have lower strength relative to walls on a timber foundation due to lower wall ductility and rupture of fasteners perpendicular to the sheet edge.

Table 2. Theoretical bracing rating of walls based on 'stud-uplift' (BU/m length).

		Single Sheathing			Double Sheathing		
	Straps?	Fp = 3 kN/m	Fp = 1.5 kN/m	Fp = 0 kN	Fp = 3 kN/m	Fp = 1.5 kN/m	Fp = 0 kN
P21 end restraint							
0.9 m long wall	Yes	123	118	112	135	124	113
0.9 m long wall	No	72	66	60	84	73	61
1.2 m long wall	Yes	128	120	113	144	129	114
1.2 m long wall	No	76	68	61	92	77	62
1.8 m long wall	No	91	80	68	115	93	70
2.4 m long wall	No	106	91	76	138	108	78
3.0 m long wall	No	120	102	83	161	123	86
EM3 end restraint							
0.9 m long wall	Yes	160	154	149	172	161	149
0.9 m long wall	No	108	103	97	120	109	98
1.2 m long wall	Yes	164	157	149	180	165	150
1.2 m long wall	No	112	105	97	129	114	99
1.8 m long wall	No	127	116	105	152	129	107
2.4 m long wall	No	142	127	112	174	144	114
3.0 m long wall	No	157	138	119	197	160	122



(a) Before racking (b) After racking

Figure 13. 'Rocking' action for wall on concrete foundations.

5. WIND UPLIFT

5.1 EM3 approach to wind uplift forces

Wall ‘rocking’ is largely resisted by house gravity load and tension connections to the foundations. Wind uplift reduces the racking load at which a house bracing wall will ‘rock’. This is recognised in the EM3 test wind evaluation which ignores the strength enhancement due to ‘systems effects’ whereas an enhancement factor F2 is used for earthquake resistance which varies between 1.0 and 1.2.

The factor F3 = 0.8 except = 1.0 for walls with plasterboard lining and taped and filled joints which satisfy one of the following:

(a) Neither end of the wall terminates at either a door opening or free end.
(b) The ends of the wall at door openings and free ends use steel end straps.
(c) The wall is directly below a suspended floor above.

These criteria in the EM3 method are expected to reduce the need for end straps in building construction to being only at door openings where they are most effective (Thurston, 1993 and Herbert and King, 1998). Most houses are lined with plasterboard with taped and filled joints and most walls will satisfy one of the criteria above. There are expected to be few buildings where total wind resistance reduces under EM3.

5.2 Magnitude of wind uplift forces

Shelton (2003) calculated the net uplift forces at the base of continuous walls during design level wind storms as 7.7 kN/m in the Very High wind zone and 5.4 kN/m in the High wind zone. Shelton calculated that the net uplift force could reach 16.6 kN at the base of walls terminating at wide windows but considered 12 kN was a more realistic maximum figure. As illustrated in Figure 14, for wind parallel to either building major axis, the maximum wind uplift forces are likely to occur in face loaded walls (i.e. perpendicular to the wind direction) whereas the maximum wind bracing demand is on walls parallel to the wind direction. Thus, wind uplift forces are expected to have little effect on house bracing resistance except perhaps at building corners. Uplift at building corners is usually not critical in construction using linings with taped and filled corner joints as vertical forces can be transmitted across the joint. A more critical situation may be where the wind is at 45° to the major building axis when walls will be subjected to both wind uplift and bracing demand. However, the magnitude of both is expected to be reduced by a factor of approximately $1/(\sqrt{2})$.

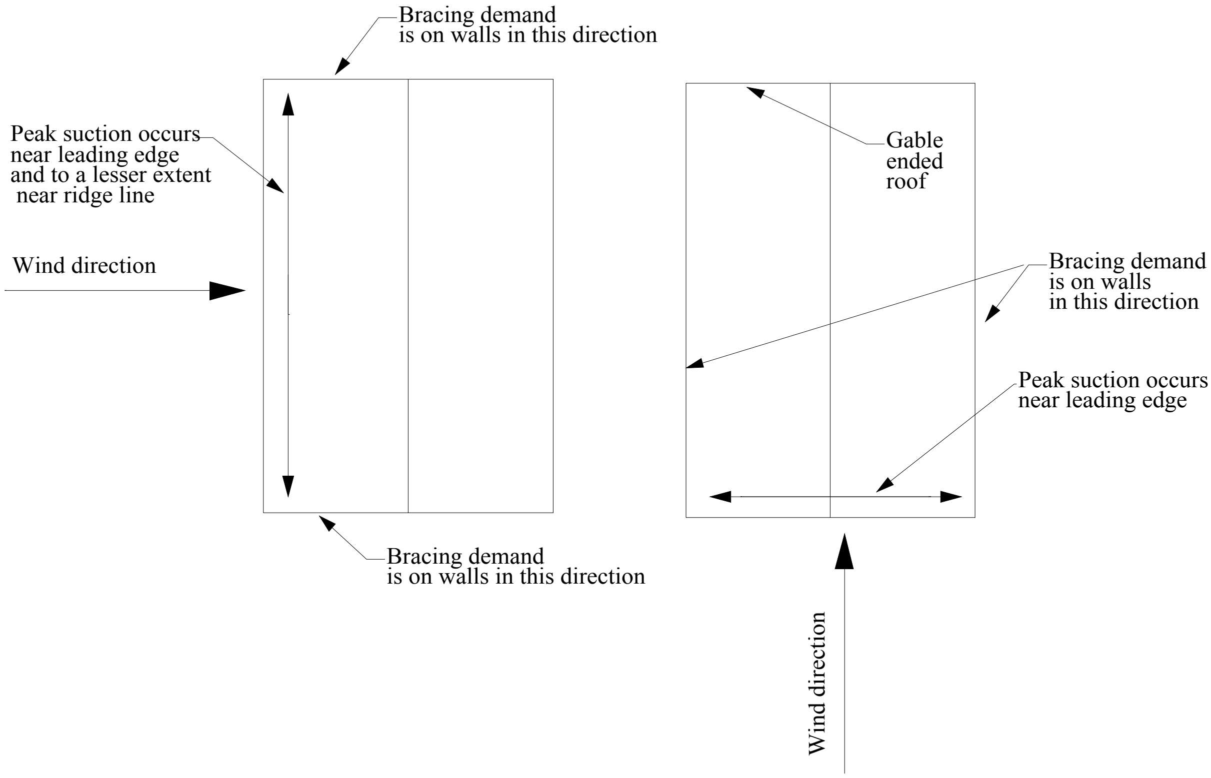


Figure 14. Combined roof uplift forces and wall racking forces.

5.3 Criteria for critical wind uplift conditions

Wind uplift forces will cause a wall to rock at lower racking loads which needs to be considered in the EM3 method. Although the author knows of no cases of wind-induced wall damage in modern New Zealand houses it is considered prudent not to increase the wind bracing resistance as evaluated from the current P21 method by more than 10% for walls having a high bracing rating (say, 100 BU/m). For walls having a very high bracing rating (say, 150 BU/m) no increase in bracing rating is recommended. Walls having a low bracing rating (say, 70 BU/m) will not be subjected to high 'rocking' demand and so can sustain a significant increase in assessed bracing resistance before 'rocking' action dominates. The next section investigates whether this is achieved under the proposed EM3 method.

5.4 Comparison of P21 and EM3 wind bracing ratings

The wind bracing ratings of walls as derived from EM3 and P21 methods are compared in this section assuming each is tested to the EM3 test regime but with different 'supplementary' end restraints and evaluations. It is assumed that the average of the 'push' and 'pull' strengths of a hypothetical wall construction = P using the P21 end restraint and evaluation. The same wall but with an EM3 end restraint and evaluation is assumed to have an average strength of E.

5.4.1 P21 test

A full P21 test assigns a design bracing strength of 0.9P. This 10% reduction on actual strength was due to predicted strength decrease due to repetitive loading at levels below the design level. However, the EM3 regime uses repetitive loading and thus it is considered that the design load using a P21 evaluation can be considered to be 1.0P.

5.4.2 EM3 test

Walls considered are those which most potentially affected by wind uplift – i.e., those for which F3 = 0.8. The wall is therefore assigned a design bracing strength of 0.8E.

5.4.3 Calculated ratio of EM3:P21 wind bracing rating

The calculation will be done for a bracing panel of width, L (m), which is governed by 'rocking'.

In Section 4.5.3 it was shown that the P21 end restraint was effectively an end force of 4.85 kN. Thus, substituting P for F and 4.85 for Fs, Equation (5) can be re-expressed:

$P \times H = (4.85 + Fer) \times L + W \times L/2 + \Sigma(Fn \times Xi)$ …….. (7)

In Section 4.5.3 it was shown that the EM3 end restraint was effectively an end force of 9.23 kN. Thus, substituting E for F and 9.23 for Fs, Equation (5) can be re-expressed:

$E \times H = (9.23 + Fer) \times L + W \times L/2 + \Sigma(Fn \times Xi)$ …….. (8)

From Equation (8) - Equation (7) and by putting H = 2.4 m and using 1 kN = 20 BUs:

$E-P=(9.23-4.85) \times L/H = 1.825 \times L$ kN = $36.5 \times L$ Bus

Or $E = P + 36.5 \times L$ (in units of BUs) …………………. (9)

From Sections 5.4.1 and 5.4.2, the ratio of EM3 to P21 wind bracing rating $W_{EM3}:W_{P21}$ is given by 0.80E/1.0P

Substituting for E from Equation (9) gives:

$W_{EM3}:W_{P21} = 0.80(P+ 36.5xL)/(1.0P) = 0.8+29.2L/P$ ……. (10)

Consider a panel with a P21 bracing rating = K BU/m. $\therefore$ P = KL.

Hence $W_{EM3}:W_{P21} = 0.8+29.2L/P = 0.8+29.2/K$ ……… (11)

Substituting for various values of K in Equation (11) gives the relationships presented in Table 3.

Table 3. Predicted ratio of EM3 to P21 wind bracing rating ($W_{EM3}:W_{P21}$) for various wall bracing ratings.

Wall bracing rating K (BU/m)	$W_{EM3}:W_{P21}$
70	1.22
100	1.09
150	0.995

The values in Table 3 meet the criteria listed as desirable in Section 5.3. It will be noted that these are independent of wall length.

6. EM3-V1 SERVICEABILITY LIMITS.

Serviceability limits have not been used in EM3: V1. they can be simply added using the procedure discussed in this section, even though they are unlikely to govern.

6.1 Wind

From NZS 4203:1992; the ratio of serviceability wind to ultimate wind force = 0.75^2 = 0.563. BRANZ believes that at the serviceability load that the wall deflection should be less than a predefined limit (say 6, 8 or 10 mm) where this limit is the deflection at which damage commences to plasterboard walls – usually taken as 8 mm or H/300.

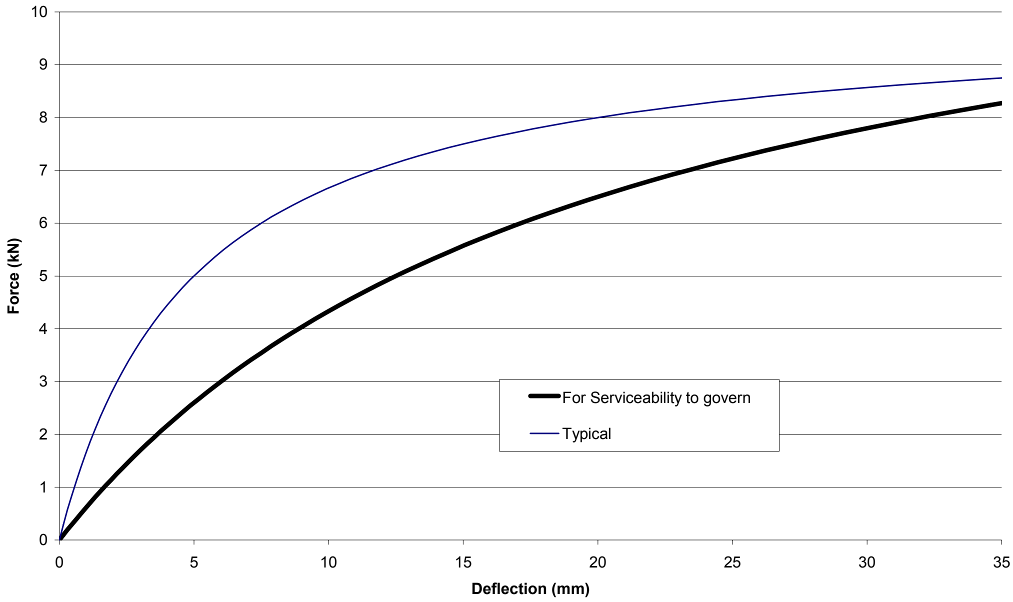


Figure 15. Racking test 'backbone' curves showing the shape when serviceability commences to govern wind bracing rating.

Hence the wind bracing rating, W, is the lesser of: Pu (Maximum resisted load at 25, 30 or 35 mm) and $R_S/0.563$ where R_S is the average load resisted after the cycles to the serviceability deflection limit of 6, 8 or 10 mm.

The serviceability criteria only governs when the loading slope is near linear as shown in Figure 15. Further, at serviceability loads the house has a great deal of non-structural stiffening and is less likely to exceed the serviceability deflection. Thus, serviceability loading is probably a non-issue – but this can be added to EM3 if desired.

6.2 Earthquake

From NZS 4203:1992; the earthquake demand force at serviceability limit and assuming $\mu = 1.25$ is given by:

F = Ch(T1,1.25) x Sp x R x Z x Ls

However, NZS 3604:1999 demand tables are based on

F' = Ch(T1,3) x Sp x R x Z x Lu

Hence, F/F' = Ch(T1,1.25) / Ch(T1,3) x Ls / Lu = 0.8/0.35/6 = 0.381 (for a building of period T1 = 0.4 seconds).

Hence, as the wall is designed from NZS 3604:1999 tables, the earthquake bracing rating, E, is the lesser of the seismic rating from EM3: V1 and Rs/0.381 where Rs is the residual load after 3 cycles to the serviceability deflection of, say, 8 mm. The latter criterion is unlikely govern.

7. ECONOMIC ANALYSIS OF EFFECT OF CHANGING FROM P21 TO EM3

The tests reported later and summarised in Table 16 show that the earthquake bracing resistance rating from the EM3 method is less than that derived from the P21 method, particularly for plasterboard and some other cladding systems which can act in a brittle manner. On the other hand generally the wall wind bracing rating has increased for all except the standard plasterboard wall system which reached peak resistance at low deflection, likely to be incompatible with the stronger bracing systems. The increased cost of meeting bracing demand in New Zealand houses is examined below first by a detailed analysis and then by an approximate method. The approximate method is expected to give an upper bound solution.

7.1 Detailed analysis

The ratio of earthquake to wind demand forces was obtained from NZS 3604 (SNZ, 1999) for all seismic zones and wind zones. This assumed a typical house geometry being a rectangular house with 0.6 m roof overhang, footprint dimensions 15 m x 8 m, stud height 2.7 m high for the lower storey and 2.4 m for the upper or single storey.

The economic analysis described below assumed that house bracing walls consisted of two option for the percentage of wall lengths as shown in Table 4. These were combined with the assumed type of bracing sheathing percentages to give four analysis assumptions listed in Table 5.

Table 4. Assumed bracing wall lengths used in the economic analysis.

Length Set		Description
A	20%	1.2 m long walls without end straps
	10%	1.2 m long walls with end straps
	70%	2.4 m long walls without end straps
B	50%	1.2 m long walls without end straps
	20%	1.2 m long walls with end straps
	30%	2.4 m long walls without end straps

Table 5. Assumptions on house wall bracing sheet material

Material	Length Set A (See Table 4)			Length Set B
	Assumption 1	Assumption 2	Assumption 3	Assumption 4
FC	3%	15%	3%	3%
PLYA	2%	5%	2%	2%
BRL	9%	10%	80%	9%
PLB	86%	70%	15%	86%

From the bracing ratings shown in Table 16 of this report, the above assumptions were calculated to have average bracing ratings per metre length of house wall given in Table 6 except that the 'systems effect' values for PLB was taken as 1.2 rather than 1.0 and was used to modify the values in Table 16.

Table 6. Average bracing resistance for assumptions shown.

Assumption	Average bracing rating (BU/m) of house walls derived using the following assumptions			
	EM3		P21	
	Wind	Earthquake	Wind	Earthquake
1	60.5	25.7	56.0	43.2
2	70.4	33.5	60.6	47.9
3	88.8	43.3	68.3	52.8
4	63.6	28.5	56.5	44.0

If the percentage of PLY and FC in Table Table 5 remain constant, but the percentage of BRL and PLB is varied, the EM3 earthquake bracing rating in Table 6 can be shown to be:

EM3 earthquake bracing rating/m = A + B x P
Where P = percent of Braceline and
A = 23.5 BU's/m for Assumption 1 and 3, 31.0 BUs/m for Assumption 2 and 25.3 BUs/m for Assumption 4
B = 0.248 BU's/m for Assumptions 1, 2 and 3 and 0.360 for Assumption 4.

The following process was then followed to calculate the economic effect of changing from P21 to EM3:

1. 3000 houses from the BRANZ database, selected by random throughout New Zealand, were analysed.
2. For each house analysed the ratio of earthquake to wind demand forces were obtained using NZS 3604 tables as described above. This is called $\text{Ratio}_{\text{EQ to wind}}$.
3. A nominal value was assumed for wind demand forces of 2000 BU if the house was in a high wind zone. This nominal value was factored by: 1.29, 1, 0.71 and 0.53 for wind zones Very High, High, Medium and Low respectively to account for the ratio of wind design speeds for these zones. This is called DemandWind. The earthquake demand force (DemandEQ) is therefore given by:

   DemandEQ = DemandWind x $\text{Ratio}_{\text{EQ to wind}}$.
4. The length of bracing walls in the house was calculated for both a P21 based resistance design and a EM3 based resistance design as described below:

   Assuming the house was designed using the P21 test procedure:

   The length of bracing walls to resist wind load was determined from:

   $L_{P21(wind)}$ = DemandWind / (P21_{Wind} rating)

   i.e. $L_{P21(wind)}$ = DemandWind /56.0 for Assumption 1.

   The length of bracing walls to resist earthquake load was determined from

   $L_{1(EQ)}$ = DemandEQ/ ($\text{P21}_{\text{Earthquake}}$ rating)

   i.e. $L_{P21(EQ)}$ = DemandEQ /43.2 for Assumption 1.

   L_{P21} was set equal to the maximum of $L_{P21(EQ)}$ and $L_{P21(wind)}$

   Assuming the house was designed using the EM3 test procedure:

The length of bracing walls to resist wind load was determined from:

$L_{EM3(wind)}$ = DemandWind / ($EM3_{Wind}$ rating)

i.e. $L_{EM3(wind)}$ = DemandWind /60.5 for Assumption 1.

The length of bracing walls to resist earthquake load was determined from

$L_{EM3(EQ)}$ = DemandEQ/ ($EM3_{Earthquake}$ rating)

i.e. $L_{EM3(EQ)}$ = DemandEQ /25.7 for Assumption 1.

L_{EM3} was set equal to the maximum of $L_{EM3(EQ)}$ and $L_{EM3(wind)}$

5. The percentage of walls which must be braced with BRL is then found from solving for P in the equation:

   DemandEQ = (A + B x P) x L_{P21} where A, B and P are defined above.

   The EM3 has been estimated in Table 7 by assuming there is a cost per extra metre of bracing wall requirements = \$7.2/m - (i.e. the cost to replace one sheet of nominal wall with one sheet of BRL/1.2).

   Thus the extra cost = \$7.20 x (P - 9)/100 x LP21 for Assumption 1. (The 9% is the percent of BRL currently being used from Table 5).

   The cost was added over all 3000 buildings.

The results of the economic analysis are given in Table 7. The results show the average cost is sensitive to the assumptions made.

Table 7. Results of economic analysis

Assumption	Cost per house
No.	**(\$)**
1	\$145
2	\$99
3	–
4	\$80

This costs given in Table 7 makes the following assumptions – all of which will tend to increase the cost per house shown in the table compared to what the true cost will be.

1. Many houses are currently found to have more than adequate wall bracing to meet demand forces using only PLB sheet bracing. However, it was assumed that all houses are currently only just adequate to meet the demand forces.
2. It is proposed that the next revision of NZS 3604 take account of nominal walls to meet bracing demand. It is assumed that this is not adopted.
3. The greater end restraint in the EM3 test means that for houses where walls are lined on one side and clad on the other with a strong bracing system such as FC or PLY a significant increase in bracing rating will result. This is conservatively ignored.
4. The savings in the use of end straps likely with EM3 has been ignored.

7.2 Approximate analysis giving upper limit of average cost per house

An expected average upper limit cost is determined below which very conservatively assumes that all house bracing walls are purely designed for earthquake. If wind effects are also considered the additional cost of EM3 will reduce.

A conservative average value for house seismic bracing wall demand is assumed to be 1600 BU's.

Note that in Table 6 the mix of walls in Assumption 1 (Mix 1) have an average earthquake P21 resistance of 43.2 BU/m whereas the mix of walls in Assumption 3 (Mix 3) have an average earthquake EM3 resistance of 43.3 BU/m – i.e., effectively the same value.

Therefore a building with Mix 1 which has been satisfactorily designed for earthquake as determined from the P21 test will also be satisfactory with Mix 3 in the EM3 test.

Hence, an upper bound of the cost of the change from the P21 test to the EM3 test = \$7.2/m x (80%-9%) x 1600/43.2 = \$189. As expected, this is greater than the more accurate analysis above, where the value derived was \$145.

8. TEST PROGRAMME

8.1 Purpose

The purpose of the test programme was to:

1. Determine the maximum shear load which could be applied to P21 and EM3 tests before 'rocking' occurred for 1.2 m long walls on timber and concrete foundations (with and without steel straps on the end studs) and 2.4 m long walls without straps.
2. To compare the ratio of bracing resistances obtained from P21 and EM3 tests for both wind and earthquake loads to ensure the change from P21 to EM3 is intuitively reasonable.
3. To help develop procedures for conservatively estimating EM3 ratings from P21 test results (and perhaps indicative tests) to avoid the need for manufacture's to fully retest their established bracing systems.

8.2 Construction details

Frame details for the 1.2 and 2.4 m long walls simulating timber floor construction are shown in Figure 16 and Figure 17 respectively. The framing joints were nailed as specified in NZS 3604 (SNZ, 1999). Two 90 x 3.15 mm gun-nails were driven though the plates into the studs. The framing consisted of machine stress graded F5 90 x 45 mm kiln dried radiata pine with a mid-height nog except for walls with a single lining of SPB or BRL (see Table 8) when the studs were 90 x 35 mm and no nogs were used.

Where a steel end strap was used on construction simulating a timber foundation the detail was as shown in Figure 9 for fixing on the side of the joist. The strap was always used on the side representing the wall exterior.

Construction simulating concrete foundations was similar to that shown in Figure 16 and Figure 17 except that no nails were used to fix the bottom plate to the foundation. Instead, coach screws with 50 x 50 x 3 mm steel washers were used at every second marked nail locations in these figures starting from the first nail. This effectively placed the coach screws

at 1.2 m centres. Where a steel end strap was used the detail was the ‘6 kN’ connection as shown in
Figure 10 for 1.2 m long walls. As it was found that this detail resulted in splitting of the bottom plate with consequent strength loss, the detail shown in Figure 18 was used for 2.4 m long walls. This detail used a mild steel strap on each side of the stud. Each plate was bent to enable a leg at least 30 mm long to fit under the bottom plate. Six nails are used per plate as shown. An alternative construction where the strap was continuous under the bottom plate is also acceptable and is considered to be equivalent.

Details of the wall sheathings used are given in Appendix B with generic information given in Table 8. All sheet material used was of overall dimensions 2.4 m high and 1.2 m wide except for Test 44 which simulated concrete foundations and used FC cladding of length 2.47 m to allow the sheet to overlap the bottom plate by 60 mm.

Table 8. Details of sheathing materials used in wall construction.

Label	Material Type	Sheet thickness (mm)	Board weight (kg/m^2)
SPB	Standard plasterboard	9.75	7.44
BRL	Plasterboard with glass fibre reinforced core	9.83	8.91
FC	Fibre cement	7.39	11.20
PLYA	Plywood	7.45	4.11
PLYB	Plywood	12.10	6.38

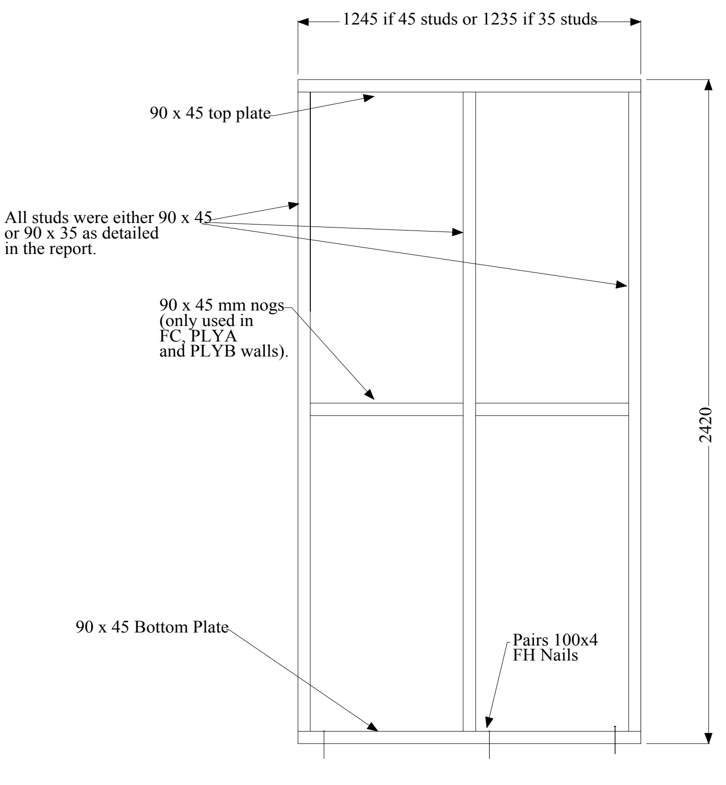


Figure 16. Frame construction for 1.2 m wide walls.

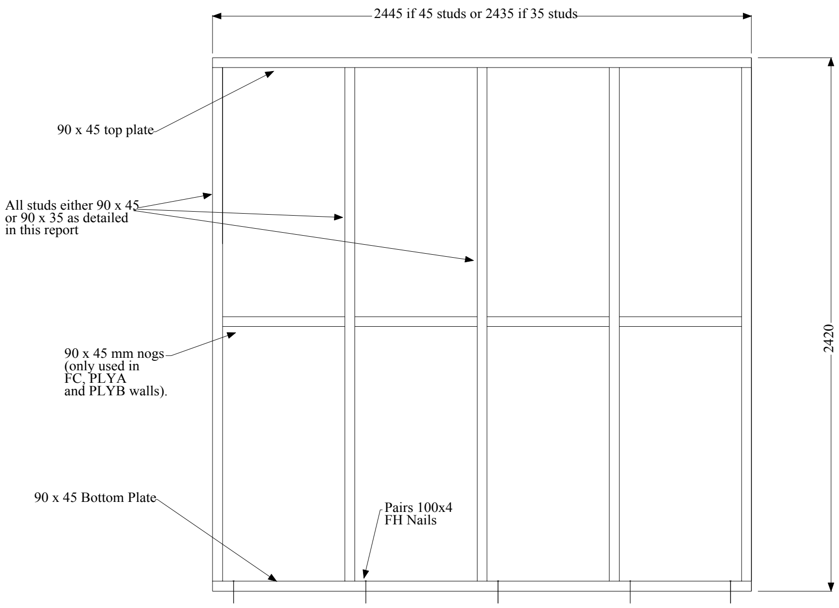


Figure 17. Frame construction for 2.4 m long walls.

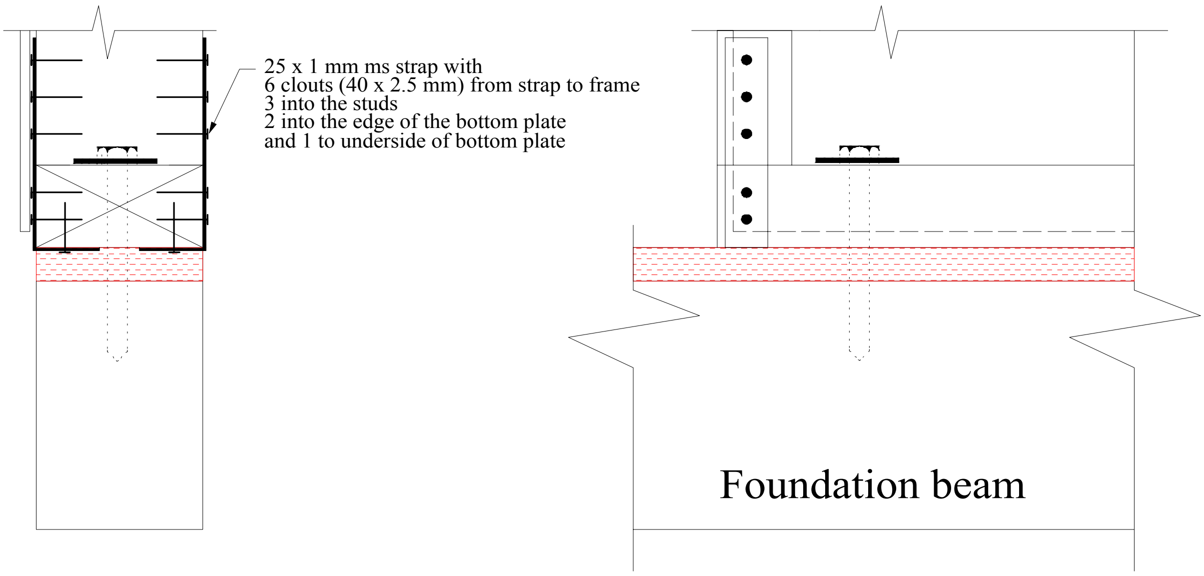
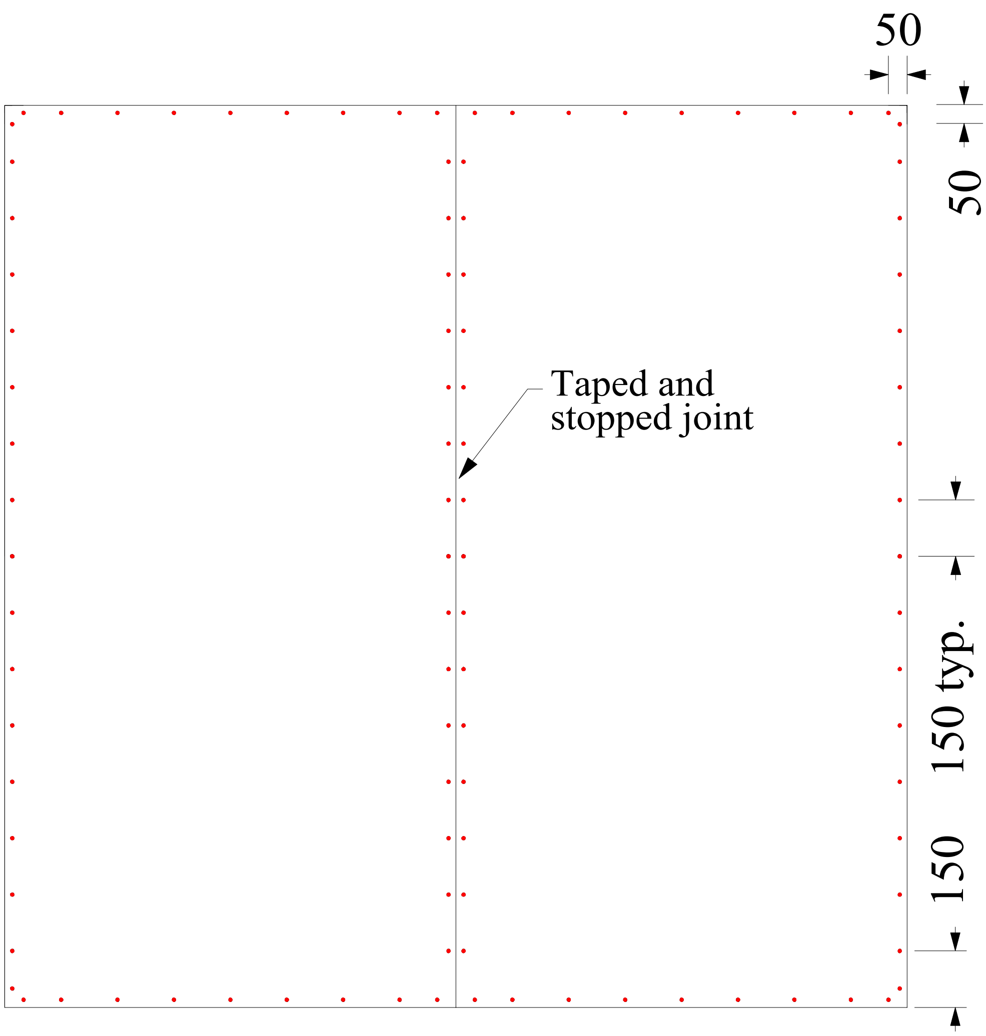


Figure 18. End strap detail used on some 2.4 m long walls constructed on simulated concrete foundations.

The fixings used with each sheathing material are given in Table 9. The proprietary names for these fixings are given in Appendix B.

Table 9. **Fastener types used.**

Label	Lining Type	Generic description
GGS screw	SPB	25 mm long drywall plasterboard steel screw
GBN nail	BRL	30 mm long, 2.8 mm shank diameter gold passivated nail, with 16 mm diameter 1.0 mm thick attached washer
GBC clout	BRL	30 mm long, 2.8 mm shank diameter gold passivated nail with 7 mm diameter head. (Used only in the framing at sheet joints.)
PWA clout	PLYA	30 mm long x 2.5 mm shank diameter, 7.5 mm flat head diameter galvanized clout.
PWB clout	PLYB	50 mm long x 2.8 mm shank diameter, 7.5 mm flat head diameter galvanized clout.
FCSS nail	FC	40 mm long x 2.8 mm shank diameter, 7 mm flat head diameter stainless steel ring shank nail.



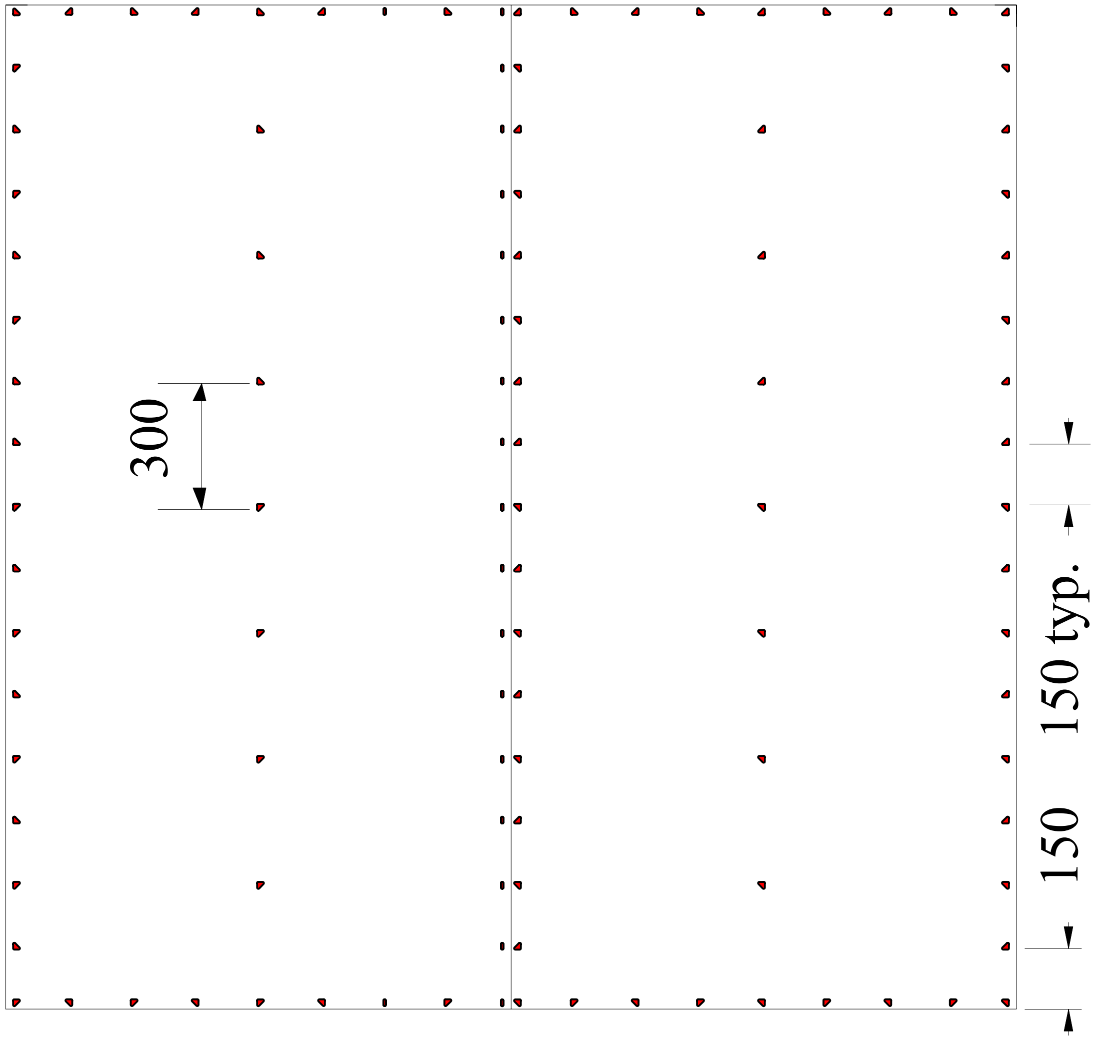
(a) SPB sheets

All fixings are 25 mm GGS drywall screws. Refer to Table 9.
Screws around sheet perimeter are located 12 mm from the sheet edge. Screws start at 50 mm from the sheet corners in both horizontal and vertical direction. The next screw in is 150 mm from the sheet corner. Screws around the sheet perimeter continue at 150 mm spacing.
No fixings in the body of the sheet.

(b) BRL sheets

Fixings around the wall perimeter are BRL nails. Refer to Table 9.
Fixings along the middle joint between sheets are GBC clouts.
Fixings are located 12 mm from the sheet edge and start at 50 mm from the sheet corners in both horizontal and vertical direction. The next fixing is 150 mm from the sheet corner. Fixings around the sheet perimeter continue at 150 mm spacing. No fixings in the body of the sheet.

Figure 19. Method of fixing SPB and BRL sheets.



All fixings are PWA clouts for PLYA sheathing and PWB clouts for PLYB sheathing. Refer to

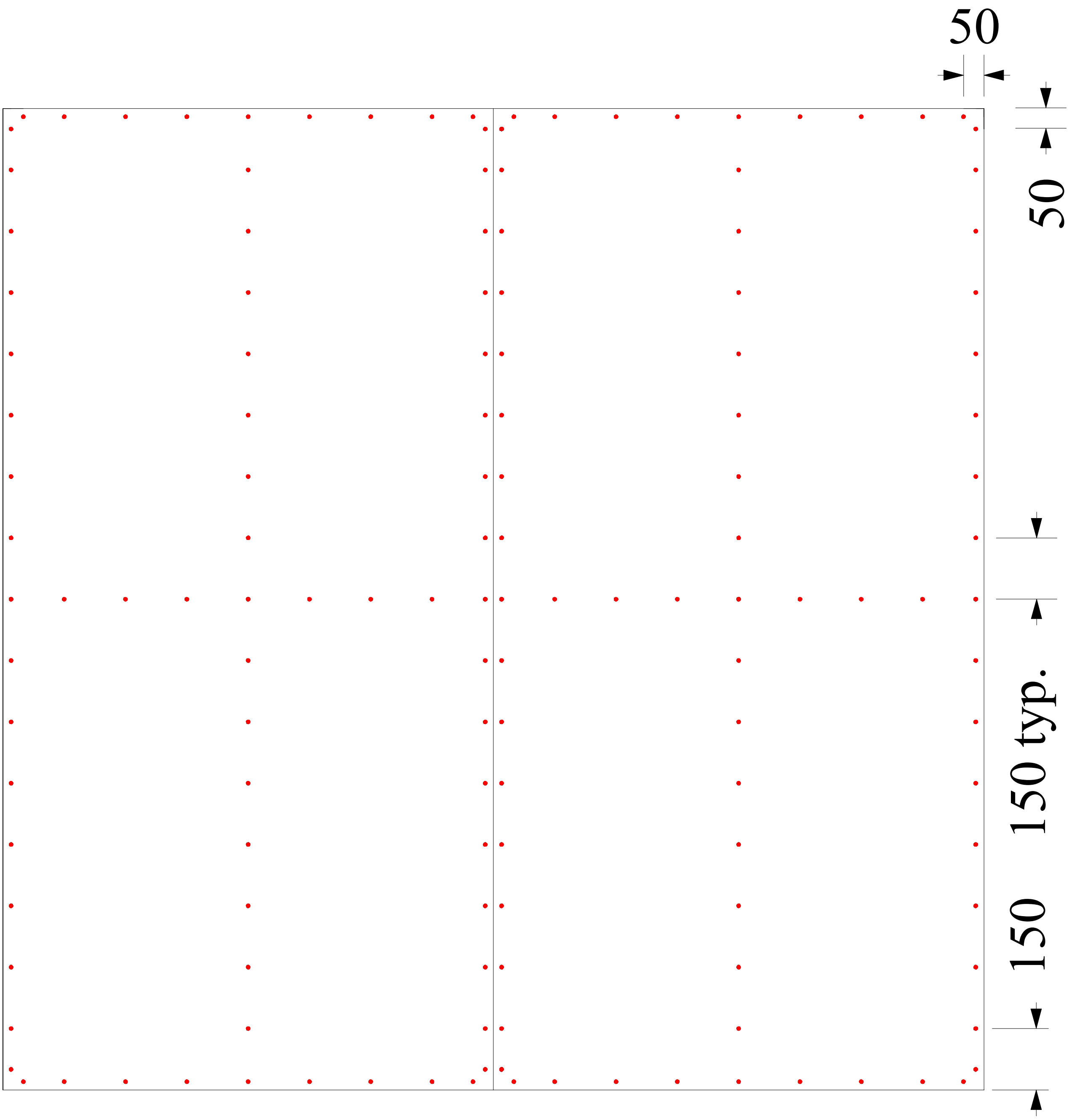
Table 9.

Fixings around the sheet perimeter are located 7.5 mm from the sheet edge of PLYA and 7.5 mm from back face of the shiplap for PLYB cladding. There is a single clout at each sheet corner. The next clout in is 150 mm from the sheet corner. Clouts around the sheet perimeter continue at 150 mm spacing.

Clouts on studs at middle of sheets are at 300 mm centres starting 300 mm from the sheet edge.

No clouts are used to mid-height nogs.

Figure 20. **Method of fixing PLYA and PLYB sheets.**



All fixings are FCSS nails. Refer to Table 9.

Nails around sheet perimeter are located 12 mm from the sheet edge. Nails start at 50 mm from the sheet corners in both horizontal and vertical direction. The next nail in is 150 mm from the sheet corner. Nails around the sheet perimeter continue at 150 mm spacing.

Nails on studs at middle of sheets are at 150 mm centres starting 150 mm from the sheet edge.

There are three nails along sheet mid-height nogs at 150 mm centres as shown.

Figure 21. Method of fixing FC sheets

8.3 Supplementary end restraints

A ‘supplementary’ end restraint of either the P21 system (shown in Figure 4(a)) or the EM3 system shown in EM3-V1 (Thurston, 2004) was used in each wall test. The restraint used is identified with each specific test.

8.4 Test arrangement and equipment

The racking test specimens were installed in a rigid steel loading frame. Horizontal load was applied to the mid-length of the specimen top plate with a 100 kN closed loop electro-hydraulic ram and measured with two load cells in series. The first was a 100 kN load cell and the other was either a 10 kN load cell in the 1.2 m long wall tests or a 25 kN load cell in the 2.4 m long wall tests.

Nylon rollers were used to prevent out-of-plane movement of the top plate.

Linear potentiometers were used to measure the horizontal displacement of the top plate, vertical uplift of the studs at either end of the specimen, and horizontal displacement of the bottom plate.

For single sheathed walls the slip of the sheathing relative to the framing was measured at mid-height of each frame side.

The test load and displacement measurements were recorded using an IBM compatible PC running a software program to record the data. The load cell was calibrated to BS 1610 Grade 1 accuracy and the linear potentiometers were calibrated to an accuracy of 0.2 mm.

8.5 Test programme

There were 26 racking tests performed on 1.2 m long walls as shown in Table 12, and 11 tests performed on 2.4 m long walls as shown in Table 17. These tables relate the sheathing, end restraint, use of end straps and reference construction drawing to the Test Specimen Number.

8.6 Test procedure

The loading sequence consisted of three displacement controlled cycles at each level of the following top plate displacements ±(8,15,20,25,30,35 and 45 mm deflection).

8.7 Date and location of tests

The tests were carried out in during September and October 2003 at BRANZ Ltd, Judgeford, New Zealand.

8.8 Typical test results

Data from all tests were processed by automated spreadsheet analysis. Typical output plots from these spreadsheets are shown within this subsection. Comparisons of results between tests are given in Sections 9 and 0 for the 1.2 and 2.4 m long walls respectively.

A typical set of applied load versus top plate deflection hysteresis loops is given in Figure 22.
A typical time history set of measured ‘fastener slip’ is shown in Figure 23 at the top and bottom plate and stud closest to the ram (ram stud) and the stud at the far end of the wall (gauge stud). The ‘fastener slip’ at the bottom plate (shown in thick bold in the plot) was greater than the other measured ‘fastener slips’ which were all of similar magnitude in this

example. The measurement, denoted as ‘Base’ with a dotted line, is the slip between bottom plate and the foundation beam.
In all spreadsheets, the measured wall top plate deflection was compared with the predicted ‘rocking deflection’ and that due to ‘trapezoidal deflection’. The predicted deflections were computed as per Eqn (4) of Section 4.4.2. A typical scan history plot is given in Figure 24. These comparisons are summarised in tables later in this report.

In all spreadsheets, the ‘backbone’ curve (defined as the envelope to the hysteresis loops) was compared with the prediction from Δ_p of Eqn (4). A typical plot is given in Figure 25.

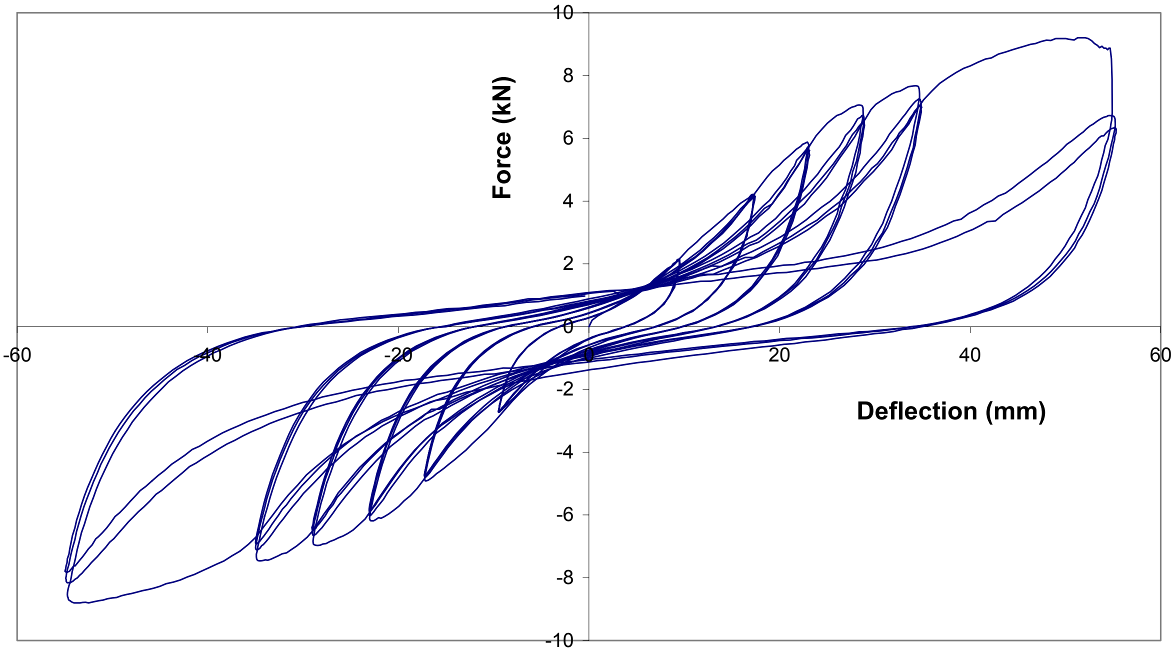


Figure 22. Typical load versus deflection hysteresis loops.

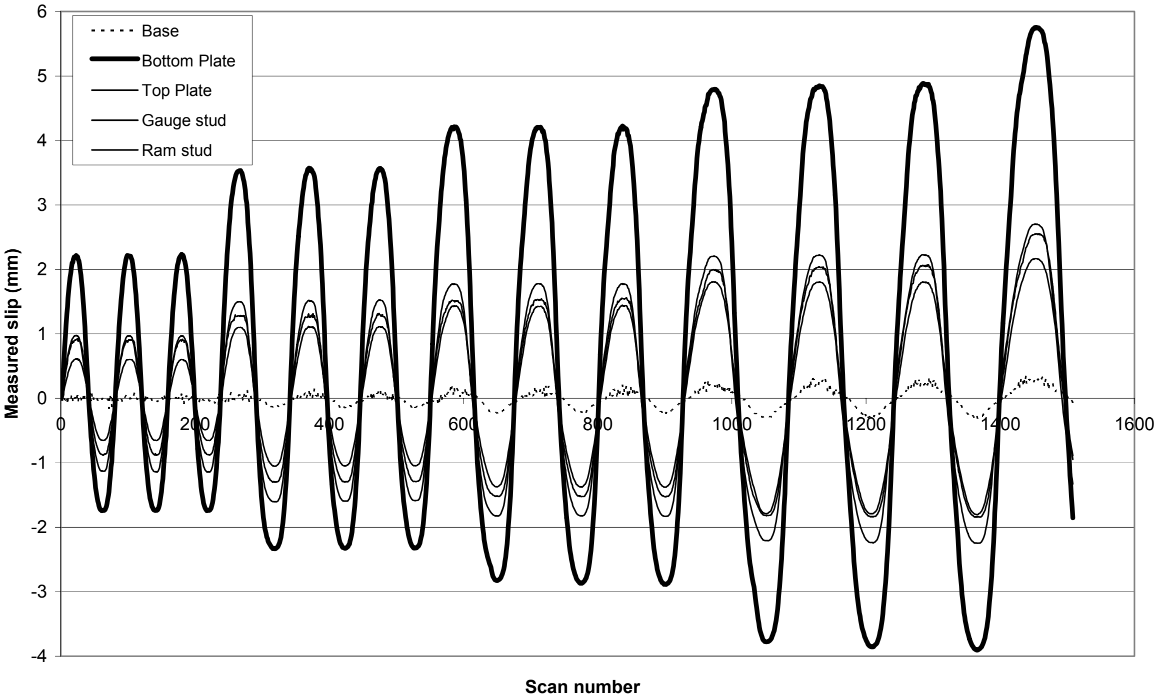


Figure 23. Typical plots of 'fastener slip'.

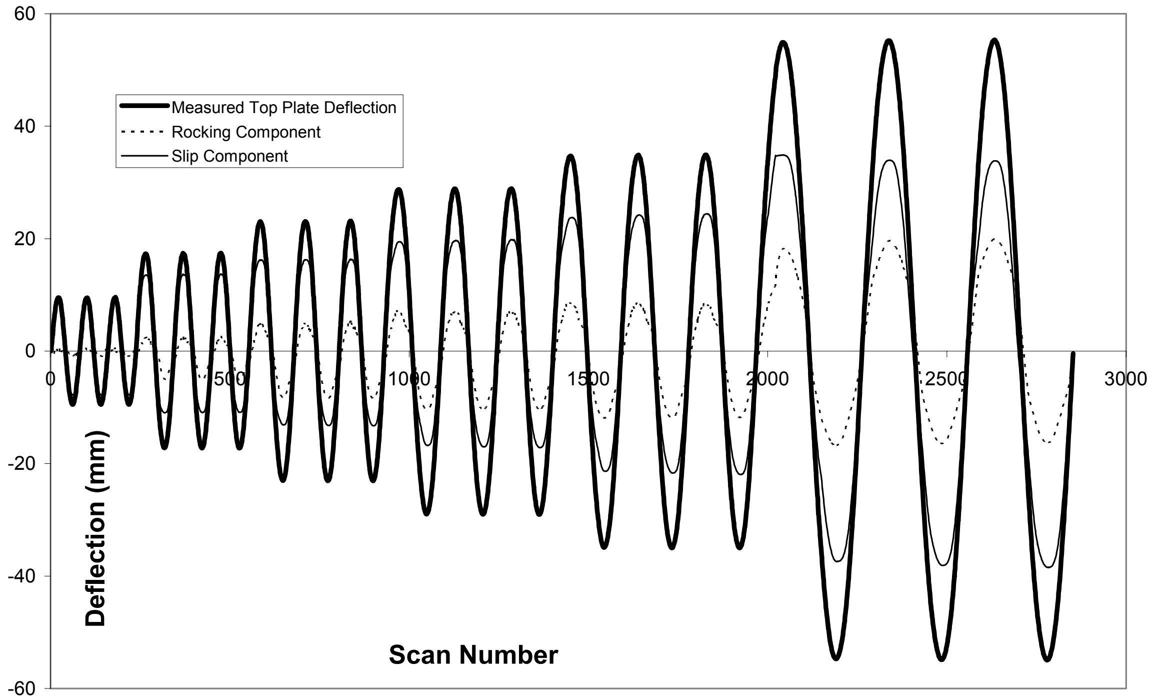


Figure 24. Typical plots of components of wall deflection.

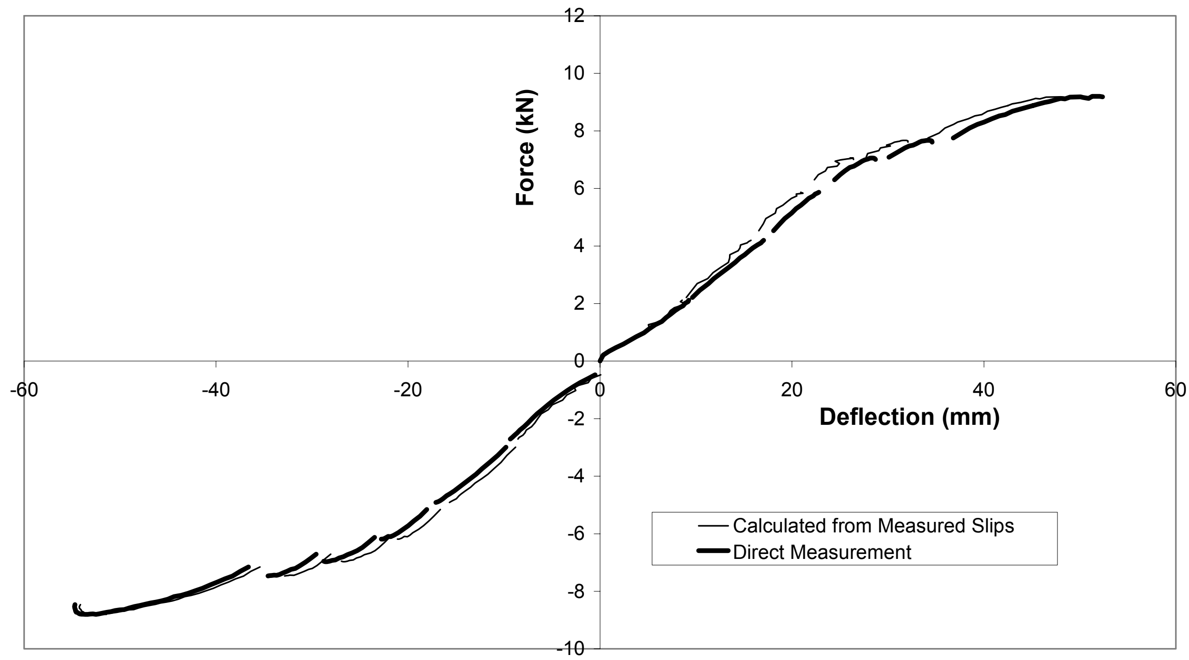


Figure 25. Typical plot showing comparison of measured backbone curve and prediction from measured 'fastener slip' data.

9. TEST RESULTS FOR 1.2 M LONG WALLS

9.1 Specimen Construction

Construction details for the 1.2 m long walls are summarised in Table 12. There were six types of wall sheathing configurations used, the first five being for single sided walls and the sixth being for walls sheathed on two sides. Table 12 identifies the following:

(1) Wall type. Walls of the same type use the same framing, sheathing and sheathing fixings.

(2) Test Specimen Number.

(3) Supplementary end restraint ('three-nail' or 'six-nail').

(4) Method used to attach the bottom plate to the foundation beam. This was either pairs of 100 x 4 mm flathead nails at 600 mm centres (for construction simulating timber foundations) or coach screws at 1200 mm centres.

(5) Use of 6 kN end straps. If used they were either Type T as shown in Figure 9 for timber foundations, Type C1 as shown in Table 10 or Type C2 as shown in Figure 18 for concrete foundations.

(6) Wall sheathing used.

(7) Figure number/s which show the sheathing fixing details.

9.2 Test observations

Observations made during the testing process are summarised in Table 13. Generalised observations and comments for each wall type are given below.

9.2.1 Walls on a simulated timber foundation

Wall Type 1. This had a plasterboard wall lining with no fibreglass reinforcing in the core. In the 'three-nail' end restraint (P21 type) tests without end straps (Test 5) 'rocking' dominated. As the 'breakout' strength of the fasteners was low and there was insufficient strength to transfer the uplift force from the sheet to the bottom plate to pull out the nails from the foundation beam, the studs instead lifted from the bottom plate and 'breakout' of the lining fixings occurred. 'Rocking' action was small in the other three tests (Test 6, 7 and 8) of this wall type and instead the sheet acted in almost pure shear with uniform 'working' of the nails around the sheet perimeter in two tests and mainly along the top plate in the other. These latter tests more accurately portrayed the relatively brittle nature of the lining fixings rather than giving the apparently ductile performance of the first specimen (i.e., Test 5) which was in reality just the 'rocking' behaviour of the wall 'supplementary' uplift restraints.

Wall Type 2. This had a plasterboard wall lining with fibreglass reinforcing in the core. In the P21 type tests of the wall without end strap (Test 9) 'rocking' dominated. Minor 'breakout' occurred near the bottom plate corner nails. In the other tests (Test 10 and 11) with either a 'six-nail' end restraint and 'three-nail' end restraint plus end strap a moderate amount of 'rocking' occurred (both stud-uplift from the bottom plate and bottom plate uplift) and 'working' occurred around the entire sheet perimeter particularly top and bottom plate. However, with Test 12 ('six-nail' end restraint plus end strap) 'rocking' action was small and instead the sheet acted in almost pure shear with uniform 'working' of the nails around the sheet perimeter.

Wall Type 3 and 4. These were plywood sheathed test specimens. In the P21 type tests of the wall without end strap and with a 'three-nail' end restraint (Tests 1 and 13) 'rocking' dominated. No damage to the test specimen was observed. In the tests with either a 'six-nail' end restraint and 'three-nail' end restraint plus end strap, (Test 2, 3, 14 and 15), a moderate amount of 'rocking' occurred (both stud-uplift from the bottom plate and bottom plate uplift) but despite 'fastener slip' movements measured up to 3 mm there was little evidence of sheathing damage. However, with Tests 4 and 16 ('six-nail' end restraint plus end strap) 'rocking' action was small and instead the sheet acted in almost pure shear with uniform 'working' of the nails around the sheet perimeter. Generally, 'rocking' was greater and 'fastener slip' less in the Wall Type 3 tests relative to the Wall Type 4 tests which indicated that the Wall Type 3 'fastener strength' was greater.

Wall Type 5 and 6. These were fibre cement sheathed test specimens with Type 6 also including a plasterboard sheet on the other face. These walls were all dominated by 'rocking' (both stud-uplift from the bottom plate and bottom plate uplift from the foundation beam) with 'fastener slip' and fastener 'working' being small. Uplift was more dominant in Type 6 walls which is not surprising as the total sheathing fixing strength was greater in these walls. Some 'fastener slip' was detected in the walls with 'six-nail' end restraint plus end straps and these showed signs of nail 'working' near the bottom corners and in the large deflection cycles the end sheet fixings 'broke out' of the sheet edges.

9.2.2 Walls on a simulated concrete foundation

The two walls (Tests 50 and 51) on simulated concrete foundations both had a similar visual performance. All studs lifted from the bottom plate which introduced tension between the bottom of the sheet and the bottom plate. Eventually, fixing 'breakout' occurred commencing in the bottom corners but at larger deflections, eventually spreading along the whole length of the bottom plate.

9.3 Comparison of predicted and measured wall deflection

A comparison of the measured wall top plate deflection with the predicted deflections (‘rocking deflection’, Δ_r, and the ‘trapezoidal deflection’ due to ‘fastener slip’, Δ_{slip}) is shown in Table 14. Based on Eqn (4) (Section 4.4.2) the total predicted top plate deflection, $\Delta_p = \Delta_r + \Delta_{slip}$. The percentages in each pair of columns of Table 14 labelled ‘rocking’ and ‘slip’ should therefore add to 100%. At higher wall deflections the summation was close to this 100% value but at lower wall deflections they generally added to significantly less than 100% with the difference being attributed to wall shear and flexural deformation. Also shown for interest is the ‘fastener slip’ at the bottom plate in the column labelled ‘Base’.

The percentage of wall deflection due to ‘rocking’ measured for each wall type, is plotted in Figure 26 and Figure 27 for construction without and with end straps respectively.

Figure 26 shows that the deflection of 1.2 m long walls without end straps are generally governed by ‘rocking’ for walls having either the ‘three-nail’ or the ‘six-nail’ end restraint. The exception is Wall Type 1 (which used a small head fastener and a lining of plasterboard without fibreglass reinforcing) for which most of the top plate deflection with the ‘six-nail’ end restraint is associated with ‘fastener slip’ rather than ‘rocking’. The ‘rocking’ deflection of Wall Type 2 (plasterboard with fibreglass reinforcing) is significantly less with the ‘six-nail’ end restraint than the ‘three-nail’ end restraint indicating that the wall strength with the ‘three-nail’ end restraint is governed by ‘rocking’ whereas with the ‘six-nail’ end restraint it is governed by the ‘fastener strength’.

A comparison of Figure 26 and Figure 27 shows that ‘rocking’ is less dominant in 1.2 m long walls with end straps although there is still a large component of ‘rocking deflection’ in the strong plywood sheathed, fibre-cement sheathed and walls sheathed on both sides (i.e. Wall Types 3-6).

It is informative to estimate the magnitude of ‘fastener slip’ at, say, 36 mm wall deflection, if the ‘rocking deflection’ is, say, 50% of the total deflection and thus the ‘trapezoidal deflection’ is also 50%. From Eqn. (3) of Section 4.4.2, the average ‘fastener slip’ is 50%x36/6 = 3 mm. At this ‘fastener slip’ plasterboard fixings are usually past their peak shear strength whereas plywood fixings are not.

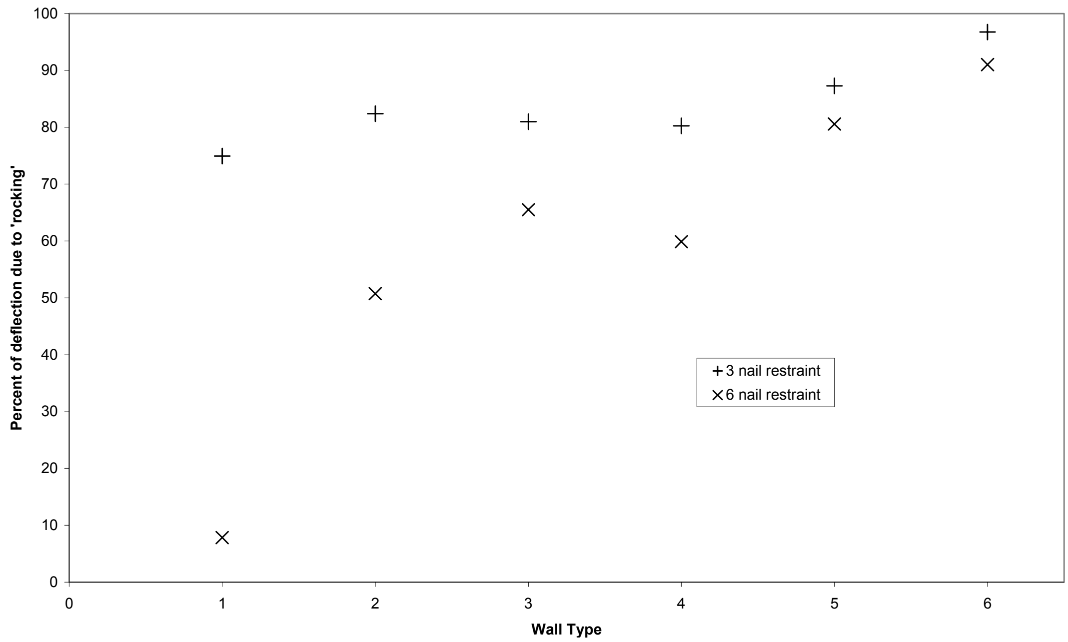


Figure 26. ‘Rocking deflection’ as a percent of total deflection at 25 mm wall deflection in 1.2 m long walls without end straps.

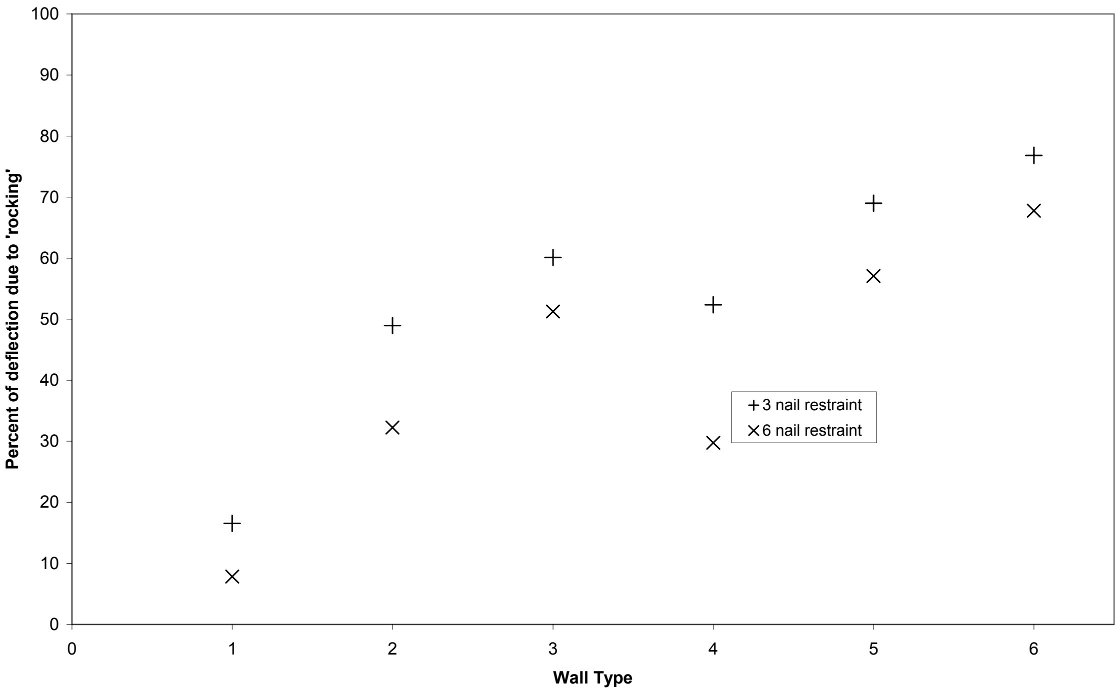


Figure 27. 'Rocking deflection' as a percent of total deflection at 25 mm wall deflection in 1.2 m long walls with end straps.

9.4 Maximum strengths of 1.2 m long walls

Table 15 summarises the average peak loads resisted in the tests for each wall type (given as the average of the wall peak push and peak pull loads). For the walls with 'six-nail' end restraints the information required for EM3 evaluations is given – namely the first cycle and residual (i.e., 3^{rd} cycle) peak loads at wall top plate deflections of 25 mm, 30 mm and 35 mm as well as the peak resisted load. For the walls with 'three-nail' end restraints the information required for P21 evaluations is given – namely the peak and residual (i.e. 3^{rd} cycle) peak resisted loads.

Figure 28 and Figure 29 plot the maximum wall strength per unit wall length achieved in the bracing tests. The strength shown is the average for the push and pull loading directions. For walls without end straps the strength was determined by the wall 'rocking strength' for all except Wall Type 1 which failed at a lower load. Where the 'rocking' mechanism is the studs uplifting from the bottom plate then the 'rocking strength' is influenced by the fastener 'breakout' strength as discussed in Section 4.5.12 and thus the 'rocking strength' is influenced by both sheathing type and wall uplift restraint system. For walls with end straps, only Wall Type 3 and 4 were governed by the 'rocking strength' for the 'six nail' end restraint and Wall Types 3, 4, 5 and 6 for the 'three nail' end restraint. The average strength of these specified1.2 m long walls (assumed to be the 'rocking strength') is given in Table 10.

The values in Table 10 are compared with the theoretical values from Section 4.5.11 in Table 11. The measured ratios are all moderately close to, but always less than 1.0, indicating that the actual strength was less than the theoretical strength for bottom plate uplift. Agreement can be made exact between measured and the theoretical strength for stud-uplift by selecting the best fit value for Fp between 3 and 1.5 kN/m in Table 2.

Table 10. 'Rocking strength' for 1.2 m long walls.

End Restraint	Walls without end straps	Walls with end straps
'three nail'	3.47 (kN/m)	6.07 (kN/m)
'six nail'	5.59 (kN/m)	8.16 (kN/m)

Table 11.Ratio of measured to predicted 'rocking strength' due to bottom plate uplift for 1.2 m long walls.

End Restraint	Walls without end straps	Walls with end straps
'three nail'	0.87	0.93
'six nail'	0.96	0.97

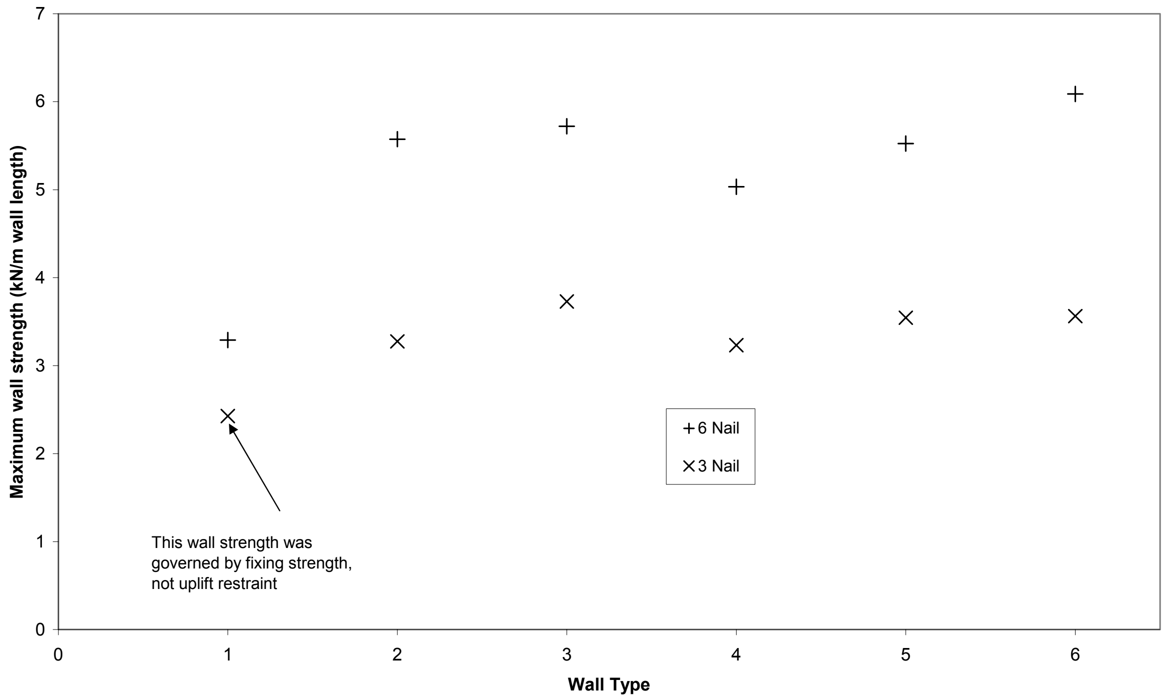


Figure 28. Influence of end restraint on wall strength for 1.2 m long walls without end straps.

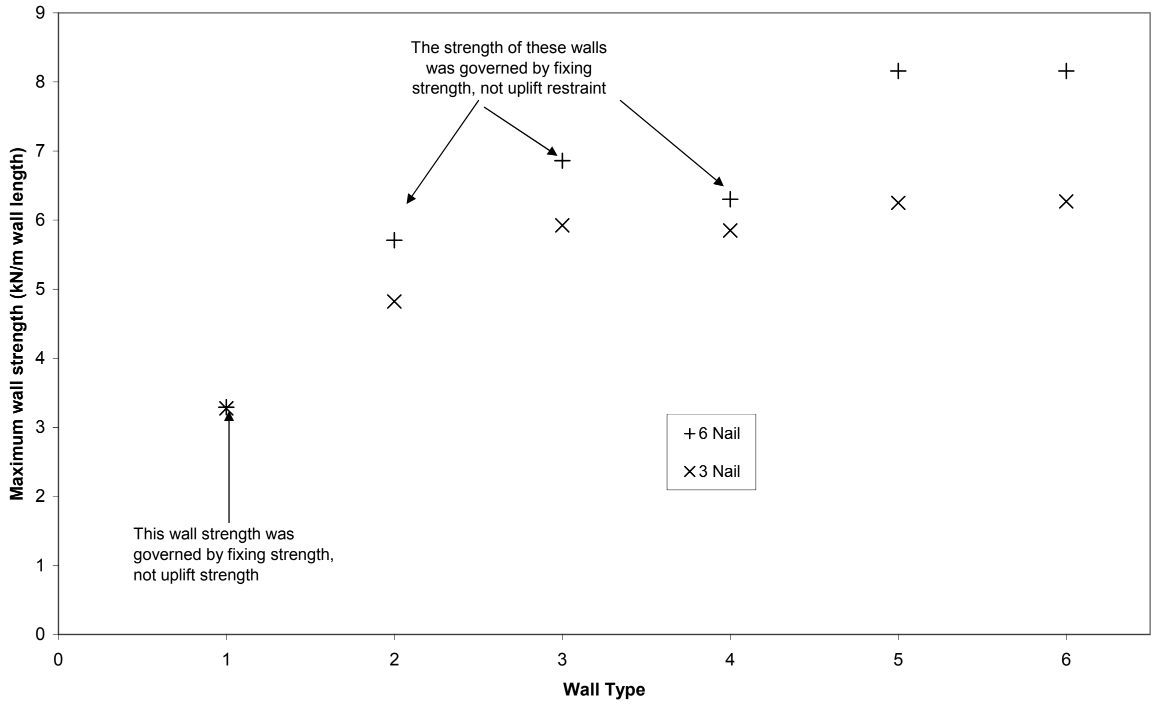


Figure 29. Influence of end restraint on wall strength for 1.2 m long walls with end straps.

9.5 Bracing ratings of 1.2 m long walls

The bracing values evaluated from the test results as per EM3-V1 (Thurston, 2004) and the P21 procedure described by King and Lim (1991) are given in Table 16. With the exception of the EM3 tests on Wall Type 1, all walls showed an increase in wind bracing rating when going from construction without to with end straps. As the EM3 test on Wall Type 1 was not limited by 'rocking' action, the addition of end straps did not enhance the strength.

The ratio of the EM3 to P21 evaluated bracing values are also given in Table 16 and these are plotted in Figure 30 and Figure 31 for 1.2 m long walls with and without end straps respectively.

With the exception of Wall Type 1 with straps the EM3 test has increased the wind bracing ratings. The wind bracing rating for Wall Type 1 with straps has dropped because the P21 test evaluation was based on the maximum wall load whereas the EM3 test evaluation was based on the maximum wall load between 25-35 mm deflection. Wall Type 1 with straps reached peak load at low deflections and the strength was dropping with increased deflection well before the 25 mm deflection level was reached. For compatibility with other bracing systems the author believes the EM3 criteria to be justified.

As it was observed that the EM3 to P21 ratio increased with increasing wall strength, the data has been replotted in Figure 32 against this variable. The trend for this increase is clear and is explained by the stronger walls being limited by 'rocking' action in the P21 type tests and thus able to benefit from the 'six-nail' end restraint in the EM3 tests.

In walls without end straps (with the exception of Wall Type 1) the change in earthquake bracing ratings is small. The rating for Wall Type 1 has dropped because it was governed by rocking in the P21 tests but showed brittle failure with no strength enhancement in the EM3 tests. However, walls with straps showed significant reduction in evaluated earthquake rating.

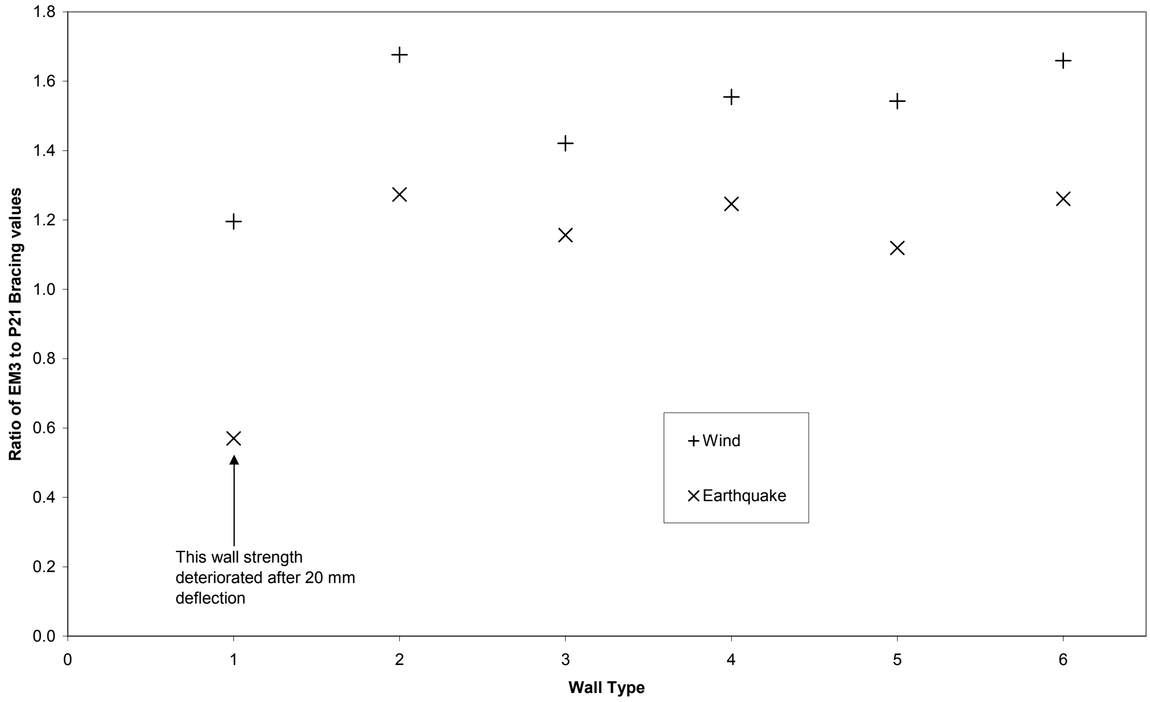


Figure 30. Ratio of bracing values from EM3 to P21 procedures for 1.2 m long walls without end straps.

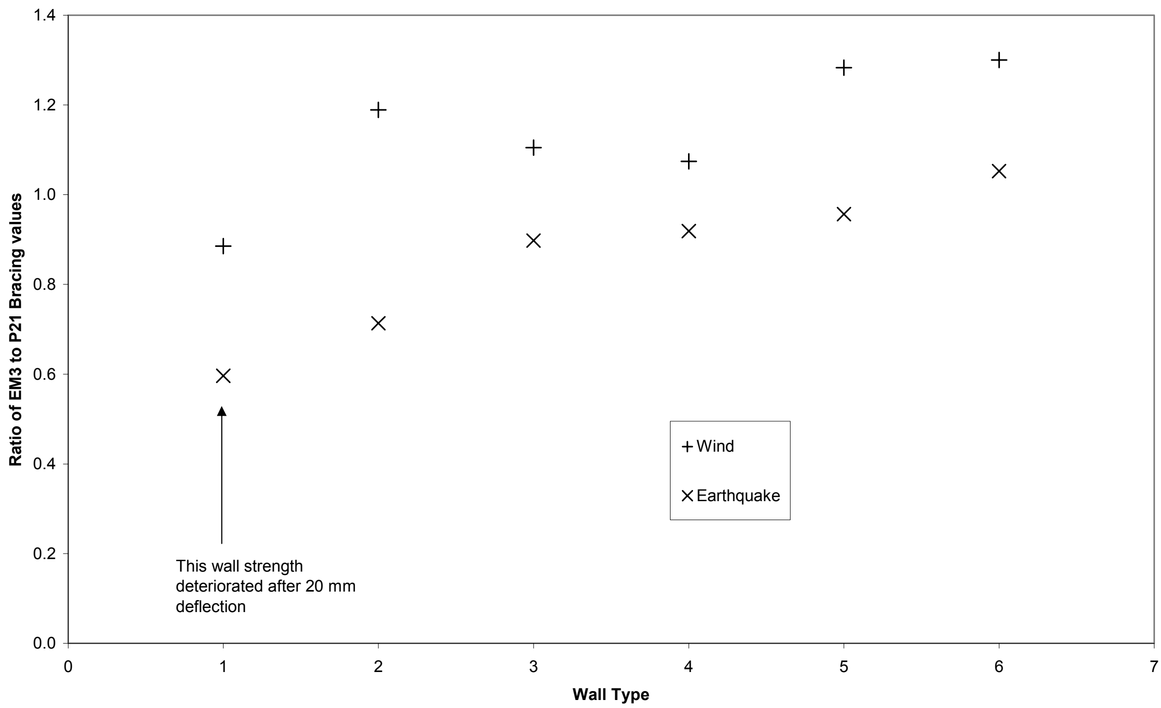


Figure 31. Ratio of bracing values from EM3 to P21 procedures for 1.2 m long walls with end straps.

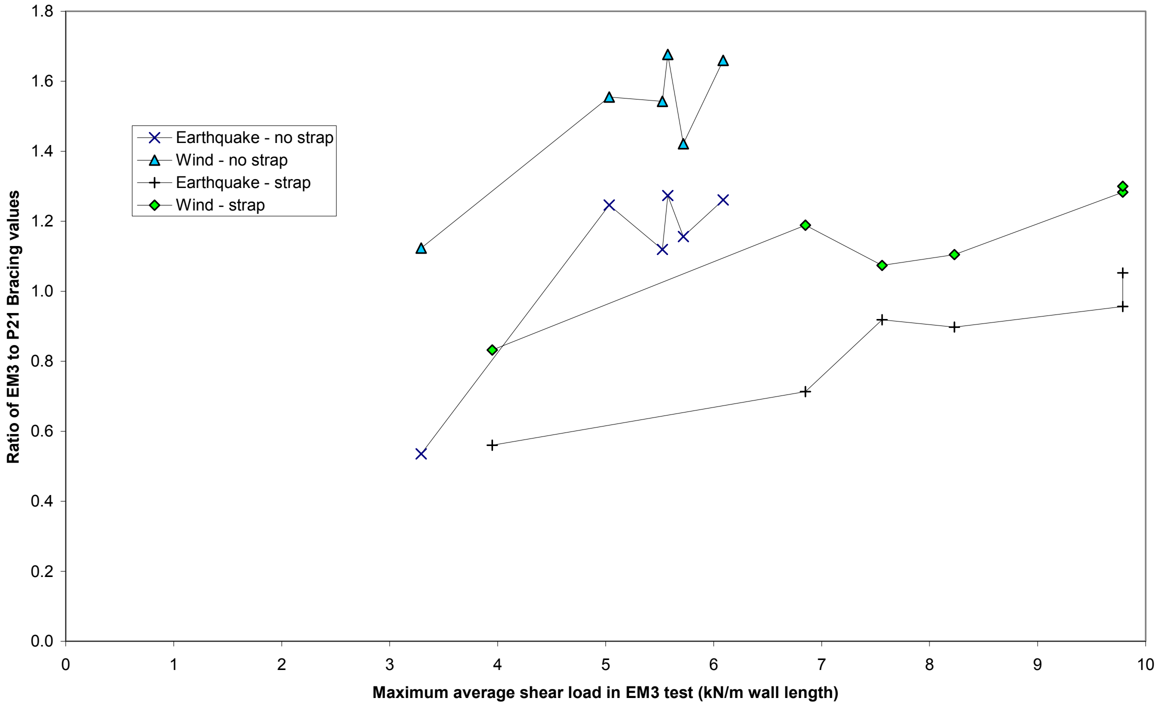


Figure 32. Ratio of bracing values from EM3 to P21 procedures for 1.2 m long walls as a function of wall maximum strength in the EM3 test.

9.6 Test results of walls on simulated concrete foundations

A comparison of results in Table 15 and Table 16 for corresponding walls with a 'six-nail' uplift restraint (i.e., Test 19 with 50 and 20 with 51) shows that the peak strengths of the bolted bottom plates on simulated concrete foundations was significantly less than for the walls on simulated timber foundations – particularly for the walls with no end straps. This is attributed to the rocking mechanism of stud lifting which pulled fixings out of the bottom of the sheet, creating a more brittle failure mechanism than would occur with bottom plate uplift. Historically BRANZ considered that the bracing rating for walls on concrete foundations could be determined from tests on timber foundations. However, these results show that bracing tests need to be performed on walls with both timber and concrete foundations.

When an end strap was included on a concrete foundation (see Figure 10) a brittle mechanism still occurred, but the bracing strength did exceed that for the wall on timber foundation without end straps (compare Test 19 with 51).

The detail used for fixing the hold-down strap to the bottom plate in Test 51 (Figure 10) proved to be poor because splitting occurred at the top and bottom nail into the bottom plate. Hence, in the tests on 2.4 m long walls a revised detail was used.

9.7 Conclusions from 1.2 m long wall racking tests

Walls without end straps. The behaviour was largely governed by 'rocking' action with the maximum resisted load being the 'rocking strength' for all walls except Wall Type 1 (plasterboard without fibreglass reinforcing) which was governed by the 'fastener strength'. However, this lining is the most common in New Zealand houses and most houses rely heavily on this as a bracing wall in bracing calculations.

Walls with end straps using a 'three-nail' uplift restraint. The strength of Wall Type 2 (plasterboard with fibreglass reinforcing) walls was governed by the 'trapezoidal strength' which was slightly less than the 'rocking strength'. As expected this wall had little strength enhancement when using the 'six-nail' uplift restraint. The remaining walls were governed by the 'rocking strength'.

Walls with end straps using a 'six-nail' uplift restraint. Only Wall Types 5 and 6 were governed by the 'rocking strength'. These both had a cladding of fibre cement (with Wall Type 6 also having a lining of plasterboard).

It was concluded that if the average actual wall uplift restraint in houses is greater than the 'three-nail' uplift restraint, then plasterboard systems are being advantaged by the current P21 test relative to other stronger sheathings.

With the exception of Wall Type 1 with straps the EM3 test has increased the wind bracing ratings. The wind rating for Wall Type 1 with straps has dropped because it reached high load at low deflection and at deflections compatible with the peak strengths of other bracing systems it had lost much of its strength.

In walls without end straps (with the exception of Wall Type 1) the change in earthquake bracing ratings from P21 to EM3 is small. The rating for Wall Type1 dropped because it was governed by rocking in the P21 tests but showed a less ductile failure with no strength enhancement in the EM3 tests. However, walls with straps showed significant reduction in evaluated earthquake rating.

The bracing strength of the tested bolted walls on simulated concrete foundations was significantly less than for the walls on simulated timber foundations – particularly for the walls with no end straps. This was attributed to the rocking mechanism of stud lifting followed by 'breakout' (fixings pulled out of the bottom of the sheet) which gave a weak 'rocking' mechanism whereas the bottom plate uplift mechanism gave a ductile deformation mechanism which protected the more system from failure from the more brittle stud-uplift mechanism. Bracing tests need to be performed for walls on both concrete and timber simulated foundations as either may be weaker.

The detail used for fixing the end strap to the bottom plate of walls on concrete foundations (Figure 10) proved to be poor because splitting occurred at the top and bottom nail into the bottom plate.

Table 12. Summary of construction details for 1.2 m long walls.

Wall Type	Test Number	End Restraint	Bottom Plate Fixing	End Straps	Sheathing Side 1	Sheathing Side 2	Figure Reference Number
1	5	3-nail	Nails	None	SPB	None	Figure 15
1	6	3-nail	Nails	Type T	SPB	None	
1	7	6-nail	Nails	None	SPB	None	
1	7	6-nail	Nails	Type T	SPB	None	
2	9	3-nail	Nails	None	BRL	None	Figure 15
2	10	3-nail	Nails	Type T	BRL	None	
2	11	6-nail	Nails	None	BRL	None	
2	12	6-nail	Nails	Type T	BRL	None	
3	1	3-nail	Nails	None	PLYA	None	Figure 16
3	2	3-nail	Nails	Type T	PLYA	None	
3	3	6-nail	Nails	None	PLYA	None	
3	4	6-nail	Nails	Type T	PLYA	None	
4	13	3-nail	Nails	None	PLYB	None	Figure 16
4	14	3-nail	Nails	Type T	PLYB	None	
4	15	6-nail	Nails	None	PLYB	None	
4	16	6-nail	Nails	Type T	PLYB	None	
5	17	3-nail	Nails	None	FC	None	Figure 17
5	18	3-nail	Nails	Type T	FC	None	
5	19	6-nail	Nails	None	FC	None	
5	20	6-nail	Nails	Type T	FC	None	
5	50	6-nail	Bolts	None	FC	None	
5	51	6-nail	Bolts	Type C1	FC	None	
6	21	3-nail	Nails	None	FC	SPB	Figure 15 and Figure 17
6	22	3-nail	Nails	Type T	FC	SPB	
6	23	6-nail	Nails	None	FC	SPB	
6	24	6-nail	Nails	Type T	FC	SPB	

Table 13. Observations made during testing of the 1.2 m long walls.

Wall Type	Test Specimen Number	Observations
1	5	Nails broke out at the bottom of the sheet from 15 mm wall deflection. Little damage occurred elsewhere around the sheet perimeter. End studs lifted from the bottom plate but the bottom plate did not lift from the foundation beam.
1	6	Most of the wall movement was due to 'fastener slip' at the top plate. The sheet eventually pulled partially away from the top plate.
1	7	Negligible stud or bottom plate uplift. Uniform nail 'working' around entire sheet perimeter.
1	8	Negligible stud or bottom plate uplift. Uniform nail 'working' around entire sheet perimeter.
2	9	Mainly 'rocking' action mainly due to bottom plate uplift but some stud-uplift occurred at the ram end. No damage was observed at nail fixings except along the bottom plate (particularly at bottom corner fixings) which showed signs of 'working'.
2	10	'Working' occurred at all the perimeter nails - particularly along the top and bottom plate and the bottom corners. The bottom plate uplifted and to a lesser extent one end stud.
2	11	Initially the fixings in the bottom corners were 'working'. There was some uplift of the bottom plate and to a lesser extent uplift of the ram stud from the bottom plate. As the test progressed 'working' extended around the entire perimeter of the wall. At the end of the test some of the nails had broken near the shank/wall interface.
2	12	Uplift was small. 'Working occurred almost uniformly around the entire sheet perimeter. Many nails had broken near the shank/wall interface.
3	1	The deformation mechanism was dominated by 'rocking' action due to bottom plate uplift. No damage due to nail 'working' could be seen.
3	2	Bottom plate lifted a bit.
3	3	Bottom plate lifted at one end and stud-uplift occurred at the other. Small 'fastener slip'
3	4	Some lifting of bottom plate. Three nails pulled out of bottom of sheet which would have weakened the wall - probably due to small edge distance. Significant 'working' of top plate sheet nails.
4	13	The deformation mechanism was dominated by 'rocking' action mainly due to bottom plate uplift but stud-uplift was significant at each end . No damage due to nail 'working' could be seen.
4	14	Both the bottom plate and stud-uplifted at each end. There was little apparent damage to sheet.
4	15	The bottom plate uplifted one end and the stud at the other end. No apparent damage to sheet.
4	16	Significant 'working' occurred at all nail heads – particularly one end stud. A small amount of mainly stud-uplift occurred at each end.
5	17	Bottom plate uplift was significant with a little stud-uplift. No 'fastener slip' or damage was observed.
5	18	Mainly stud-uplift occurred with no significant 'fastener slip' or damage.
5	19	Only got bottom plate uplift and a little stud-uplift. Minor 'fastener slip'. One bottom corner nail 'broke out'.
5	20	Got some small 'fastener slip' but again mainly stud and plate uplift. Bottom corner nails 'broke out'.
5	50	Most of the movement was due to was stud-uplift. This was all 3 studs but more at the ends. Nails pulled through bottom edge, first at ends but eventually along all bottom plate.
5	51	This was very similar to Test 50 despite the presence of the end strap. The bottom plate split at the top and bottom strap nail and some of the nails partially pulled out in the latter stages of the test.
6	21	Significant bottom plate uplift occurred but no nail 'working' was observed.
6	22	Significant bottom plate uplift occurred but no nail 'working' was observed.
6	23	Significant bottom plate uplift occurred but no nail 'working' was observed.
6	24	Getting mainly bottom plate uplift. One bottom corner nail 'broke out' at one end of the FC sheet and at both end corners of the PLB sheet.

Table 14. ‘Rocking deflections’ and ‘trapezoidal deflections’ as a percent of total deflections determined from 1.2 m long wall tests.
(Percentage due to ‘fastener slip’ at the bottom plate is also shown as ‘Base’)

Wall Type	Test Specimen Number	8 mm total wall deflection			25 mm total wall deflection			35 mm total wall deflection		
		‘Rocking’ %	Slip %	Base %	‘Rocking’ %	Slip %	Base %	‘Rocking’ %	Slip %	Base %
1	5	66.8	32.3	5.9	75.0	28.9	7.2	77.3	24.9	8.3
1	6	27.3	61.3	4.8	16.6	84.5	5.5	5.7	90.7	2.6
1	7	19.3	73.6	8.4	7.8	98.0	9.3	3.1	97.7	7.7
1	8	19.3	73.6	8.4	7.8	98.0	9.3	3.1	97.7	7.7
2	9	74.8	19.3	3.3	82.4	17.4	3.4	87.9	13.7	3.5
2	10	45.9	48.1	9.4	48.9	47.5	9.5	41.7	60.1	12.1
2	11	48.8	39.1	5.3	50.7	46.7	5.7	48.3	53.4	6.6
2	12	37.4	43.8	6.6	32.3	63.6	9.1	23.1	79.2	12.1
3	1	58.6	18.1	4.1	81.0	11.8	2.4	91.3	7.1	1.3
3	2	50.5	24.8	5.1	60.1	26.2	5.5	67.7	27.1	4.8
3	3	48.6	30.9	5.1	65.5	24.2	3.5	66.7	25.3	3.1
3	4	65.0	23.7	7.3	51.3	35.5	6.9	56.0	37.0	7.4
4	13	70.0	26.4	4.1	80.2	19.6	3.0	87.6	14.8	2.1
4	14	50.7	29.2	4.8	52.3	36.8	5.1	55.8	39.0	4.8
4	15	52.5	39.7	5.7	59.9	36.8	4.1	63.3	34.4	3.4
4	16	7.4	91.7	6.9	29.8	66.5	6.3	30.9	67.8	6.6
5	17	66.0	4.6	1.8	87.3	2.3	0.9	93.3	1.4	0.5
5	18	65.7	23.3	3.7	69.0	23.7	4.1	74.2	22.0	4.5
5	19	66.4	13.5	2.7	80.6	11.6	2.3	87.9	8.5	1.7
5	20	46.0	13.8	4.3	57.1	11.1	5.0	57.9	11.1	4.9
5	50	49.8	40.6	3.9	78.5	21.5	2.6	85.7	16.7	1.9
5	51	58.8	7.6	4.4	66.3	9.4	4.1	75.1	8.9	5.6
6	21	89.4			96.7			98.7		
6	22	89.5			91.0			91.7		
6	23	76.0			76.8			81.4		
6	24	74.8			67.8			74.3		

Table 15. Summary of peak loads resisted (kN).

Wall	6 nail end restraint								3 nail end restraint	
Type	Peak loads at deflections			Residual loads at deflections				Peak	P	R
	25 mm	30 mm	35 mm	25 mm	30 mm	35 mm	45 mm			
1.2 m long walls on simulated timber foundations (no strap)										
1	3.54	3.15	2.72	2.83	2.53	1.97	0.58	3.95	2.91	2.47
2	6.22	6.49	6.65	5.49	5.73	5.68	5.15	6.69	3.93	3.38
3	5.87	6.53	6.88	5.29	5.89	6.10	5.90	6.87	4.48	3.84
4	5.26	5.76	6.03	4.89	5.21	5.44	5.64	6.04	3.88	3.48
5	6.58	6.46	6.28	6.03	5.99	5.74	5.23	6.63	4.26	3.57
6	6.25	7.10	6.81	5.82	6.26	6.03	5.20	7.31	4.28	3.55
1.2 m long walls on simulated timber foundations (strap)										
1	6.92	7.82	8.21	6.41	7.36	7.63	7.80	8.23	3.93	2.41
2	6.55	6.86	6.59	5.99	5.75	5.04	4.33	6.85	5.79	5.01
3	6.97	7.82	8.26	6.41	7.36	7.57	7.85	8.23	7.11	6.42
4	6.03	7.05	7.54	5.70	6.48	6.77	7.79	7.56	7.02	6.36
5	8.48	9.26	9.77	7.89	8.45	8.51	7.44	9.79	7.50	6.71
6	8.57	9.33	9.77	7.95	8.49	8.91	8.73	9.79	7.53	6.75
2.4 m long walls on simulated timber foundations (no strap)										
1	6.32	5.68	4.64	5.00	4.14	3.44	2.27	6.32	6.31	4.86
2	10.35	10.04	8.82	8.60	7.69	6.54	2.37	10.63	8.18	5.90
3	11.09	11.53	11.83	10.15	10.39	10.56	9.44	11.83	8.52	7.63
5	13.56	14.11	13.85	12.08	12.50	12.34	10.01	14.35	9.79	8.22
1.2 m long walls on simulated concrete foundations										
50*	4.92	4.98	5.11	4.64	4.49	4.63	4.21	5.24		
51*	7.43	7.52	7.16	6.63	6.58	6.32	4.72	7.68		
2.4 m long walls on simulated concrete foundations										
41*	13.21	13.70	10.85	11.68	10.35	9.80	7.67	13.35		
43*	17.84	18.65	13.29	15.84	15.13	6.22	5.81	18.65		
44*	15.31	16.04	16.14	13.59	14.18	13.96	8.63	16.31		

Legend to Table 15

*. Numbers shown are the test numbers. See Table 12 and Table **17** for construction details of the 1.2 m wide and 2.4 m wide walls respectively.

Table 16. Summary of EM3 and P21 bracing values derived from all bracing tests.

Wall Type	EM3		P21		Ratios EM3/P21		Maximum load	
	Wind BU/m	EQ BU/m	Wind BU/m	EQ BU/m	Wind	EQ	EM3 (kN)	P21 (kN)
1.2 m long walls with no end straps								
1	58	24	49	41	1.20	0.58	4.0	2.9
2	110	72	65	56	1.68	1.27	6.7	3.9
3	106	74	75	64	1.42	1.16	6.9	4.5
4	100	72	65	58	1.55	1.25	6.0	3.9
5	110	67	71	60	1.54	1.12	6.6	4.3
6	118	75	71	59	1.66	1.26	7.3	4.3
1.2 m long walls with end straps								
1	55	22	66	40	0.83	0.56	4.0	3.9
2	114	60	96	83	1.19	0.71	6.9	5.8
3	131	96	118	107	1.10	0.90	8.2	7.1
4	126	90	117	98	1.07	0.92	7.6	7.0
5	161	107	125	112	1.28	0.96	9.8	7.5
6	163	118	125	112	1.30	1.05	9.8	7.5
2.4 m long walls with no end straps								
1	52	20	53	40	0.994	0.488	6.320	6.305
2	86	36	68	49	1.265	0.740	10.630	8.180
3	99	68	71	64	1.388	1.068	11.830	8.520
5	118	67	82	69	1.442	0.978	14.345	9.790
1.2 m long walls on simulated concrete foundations								
50*	85	60					5.24	
51*	125	70					7.68	
2.4 m long walls on simulated concrete foundations								
41*	111	55					13.35	
43*	155	72					18.65	
44*	134	77					16.31	

Legend to Table 16

*. Numbers shown are test numbers. See Table 12 and Table **17** for details of the 1.2 m wide and 2.4 m wide walls respectively.

Table 17. Summary of construction details and EM3/P21 ratios for 2.4 m long walls.

Wall Type	Test Specimen No.	End Restraint	Bottom Plate Fixing	End Straps	Sheathing	Figure Reference Number
1	31	3-nail	Nails	None	SPB	Figure 15
1	32	6-nail	Nails	None	SPB	
2	34	3-nail	Nails	None	BRL	
2	36	6-nail	Nails	None	BRL	Figure 15
3	37	3-nail	Nails	None	PLYA	
3	38	6-nail	Nails	None	PLYA	Figure 16
5	40	3-nail	Nails	None	FC	
5	41	6-nail	Bolts	None	FC	
5	42	6-nail	Nails	None	FC	Figure 17
5	43	6-nail	Bolts	Type C2	FC	
5*	44	6-nail	Bolts	None	FC	

Legend to Table 17

5*. The sheet overlapped the bottom of the bottom plate by 60 mm.

Table 18. Observations made during testing of the 2.4 m long walls.

Wall Type	Test Specimen Number	Observations
1	31	The ram end stud-uplifted but not the bottom plate. No stud-uplift at the other end but a small amount of bottom plate uplift. A large amount of screw 'working' along the bottom plate and to a lesser extent the top plate. At the larger deflections significant 'working' also occurred along the studs.
1	32	As per 31.
2	34	Generally dominated with 'rocking' action mainly due to stud-uplift although some bottom plate uplift at one end. The stud-uplift broke the nails out of the lining on some of the bottom plate and then 'fastener slip' along the bottom plate became large.
2	36	Started off with stud-uplift one end and plate uplift the other. After 24 mm dominated by stud-uplift both ends and fixings had broken out at the base and then 'fastener slip' along the bottom plate became large. Some 'working' on ram stud occurred. Then the sheet pulled off bottom plate and the sheet just became a passenger.
3	37	The bottom plate was lifting and also twisting. Later end nails pulled out of the bottom corners ends of the plywood. However, the wall behavior was generally dominated by 'rocking' action with some working along the bottom plate but little elsewhere.
3	38	Only a little bottom plate uplift occurred at low deflections. From 15 mm deflection the end studs lifted and then the nails pulled out of bottom of sheathing and 'fastener slip' a the bottom plate became large. Later the sheathing lifted off some of the bottom plate nails.
5	40	This was dominated by 'rocking' action with very little 'fastener slip'. On the ram end 50% of the movement was due to stud-uplift and 50% was due to plate uplift until end nail 'breakout' when the stud-uplift dominated. On the other end the bottom plate uplifted with no nail 'breakout'.
5	41	This had a bolted bottom plate. On both ends the studs uplifted with no bottom plate uplifting. From 25 mm imposed deflection some of the nails along the bottom plate broke out of the sheathing and 'fastener slip' along the bottom became large. 'Fastener slip' was not observed elsewhere.
5	42	Initially the bottom plate uplifted but at deflections greater than 15 mm the studs also uplifted at each end. No 'fastener slip' was observed.
5	43	This had a bolted bottom plate. The uplift seemed very small as the straps never broke and the bolts held. Some curvature of bottom plate was observed. The nails broke out of the bottom of the sheet at an early stage - over almost all the wall length. Some strong 'working' on end studs. After the nails pulled out of the bottom of the sheet the sheet pulled off the bottom plate and the only load transfer was stud flexing.
5	44	This had a bolted bottom plate and the sheet extended 60 mm below the bottom plate. The bolts held the bottom plate firmly in position throughout the test. The studs lifted. Initially this just worked the end couple of nails and 'fastener slip' along the bottom plate was significant . General 'fastener slip' around the sheet perimeter could also be detected. At 30 and 36 mm imposed deflection the nails pulled from the sheet in the end few nails. At 45 mm the sheet lifted from the bottom plate and it lost most of its strength.

Table 19. 'Rocking deflections' and 'trapezoidal deflections' as a percent of total deflections determined from 2.4 m long wall tests.
(Percentage due to 'fastener slip' at the bottom plate is also shown as 'Base')

Wall Type	Test Specimen Number	8 mm wall deflection			25 mm wall deflection			35 mm wall deflection		
		'Rocking'	Slip	Base	'Rocking'	Slip	Base	'Rocking'	Slip	Base
1	31	34.2	41.6	19.4	27.3	62.5	34.1	15.3	62.5	32.9
1	32	24.3	51.8	21.2	21.1	71.3	37.7	12.9	73.4	40.0
2	34	46.1	28.1	7.7	65.6	24.3	13.1	67.4	29.5	21.2
2	36	53.8	48.1	9.4	48.9	47.5	9.5	41.7	60.1	12.1
3	37	55.0	18.1	4.1	81.0	11.8	2.4	91.3	7.1	1.3
3	38	44.6	24.8	5.1	60.1	26.2	5.5	67.7	27.1	4.8
5	40	65.1	4.6	1.8	87.3	2.3	0.9	93.3	1.4	0.5
5	41	36.8	23.3	3.7	69.0	23.7	4.1	74.2	22.0	4.5
5	42	54.5	13.5	2.7	80.6	11.6	2.3	87.9	8.5	1.7
5	43	74.8	13.8	4.3	57.1	11.1	5.0	57.9	11.1	4.9
5	44	46.6	40.6	3.9	78.5	21.5	2.6	85.7	16.7	1.9

10. TEST RESULTS FOR 2.4 M LONG WALLS

10.1 Specimen Construction

Construction details for the 2.4 m long walls are summarised in Table 17. There were four types of wall sheathing configurations used, all being single sided walls. Section 9 provides a description of the column headings used in Table 17.

10.2 Test Observations

Observations made during the testing process are summarised in Table 18. Generalised observations and comments for each wall type are given below.

10.2.1 Walls on a simulated timber foundation

Wall Type 1. This had a plasterboard wall lining with no fibreglass reinforcing in the core. In both tests (31 and 32) 'rocking' action was very small and the dominant mechanism was screw 'working' along the bottom plate and to a lesser but significant extent elsewhere around the wall perimeter.

Wall Type 2. This had a plasterboard wall lining with fibreglass reinforcing in the core. In the P21 type tests with a 'three-nail' end restraint (Test 34) 'rocking' dominated by lifting of the studs. This caused 'breakout' near the bottom plate corner nails and 'fastener slip' along the bottom plate then became large. In the test (Test 36) with a 'six-nail' end restraint a moderate amount of 'rocking' occurred from stud-uplift and 'working' occurred along one stud and the bottom plate and 'breakout' occurred near the bottom plate corners until the 45 mm cycles when the sheet detached from the bottom plate.

Wall Type 3. These were plywood sheathed test specimens. In the P21 tests with a 'three-nail' end restraint (Tests 37) 'rocking' dominated with some damage being observed at nail fixings along the bottom plate. In the test with a 'six-nail' end restraint (Test 38) a moderate amount of 'rocking' occurred due to stud-uplift. However, 'fastener slip' and 'breakout' along the bottom plate dominated at large wall deflections.

Wall Type 5. These were fibre cement sheathed test specimens (Test 40 and 42). These walls were all dominated by uplift (both stud from the bottom plate and bottom plate from the foundation beam) with 'fastener slip' being small.

10.2.2 Walls on a simulated concrete foundation

Tests 41 and 43 were the same except the sheathing overlapped the bottom plate by 60 mm in Test 43 construction. Test 43 was performed to investigate whether nail 'breakout' would be less significant if fixing edge distance was increased. In both tests all studs lifted from the bottom plate which tended to lift the sheet from the bottom and the fasteners heads in the bottom corners either pulled through the sheet face or 'broke out' from the bottom of the sheet. This eventually spread along all the bottom plate.

10.3 Comparison of predicted and measured wall deflection

A comparison of the measured wall top plate deflection with the predicted deflections ('rocking deflection', Δ_r, and the 'trapezoidal deflection' due to 'fastener slip', Δ_{slip}) is shown in Table 19. Based on Eqn (4) (Section 4.4.2) the total predicted top plate deflection, $\Delta_p = \Delta_r + \Delta_{slip}$. Also shown for interest is the 'fastener slip' at the bottom plate under the column labelled 'Base'. Some of this data is plotted in Figure 33.

Figure 33 shows that the deflection of strong 2.4 m long walls (without end straps) is generally governed by 'rocking' for walls having either the 'three-nail' or the 'six-nail' end restraint. The exception is Wall Type 1 (plasterboard lined without fibreglass reinforcing) which had low 'rocking deflections'. The 'rocking deflection' of Wall Type 2 (plasterboard with fibreglass reinforcing) is significantly less with the 'six-nail' end restraint than the 'three-nail' end restraint indicating that the wall strength with the 'three-nail' end restraint is governed by 'rocking' whereas with the 'six-nail' end restraint it is governed by the 'fastener strength'.

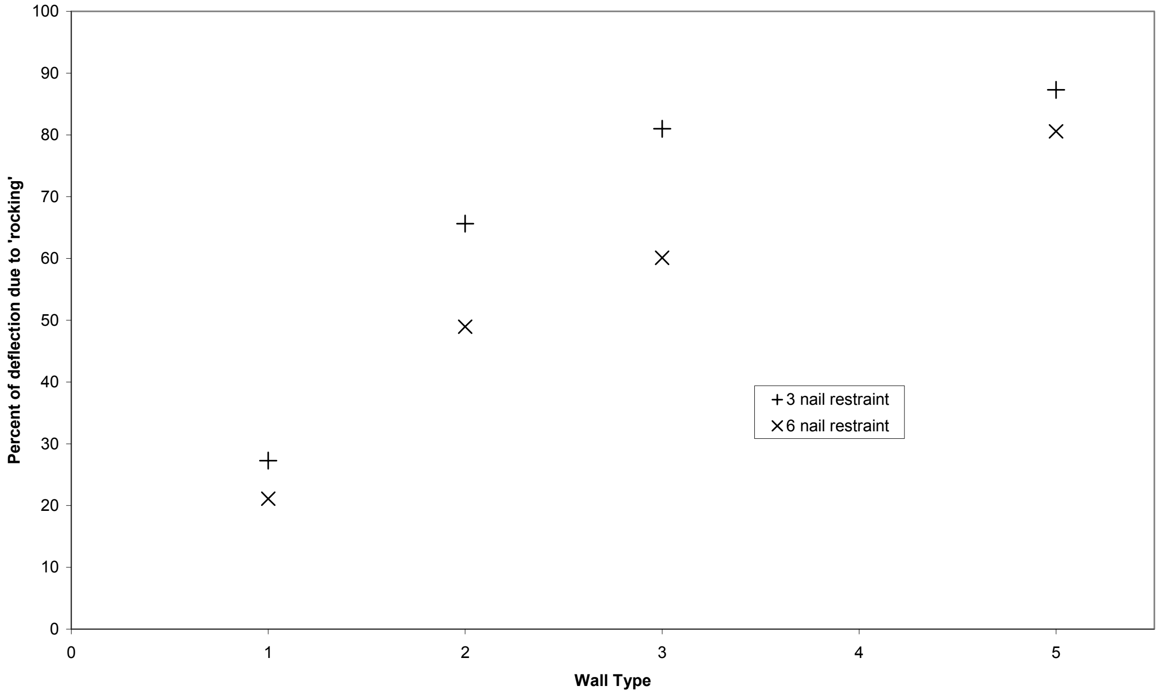


Figure 33. 'Rocking deflection' as a percent of measured top plate deflection at 25 mm wall deflection in 2.4 m long walls (no end straps).

10.4 Maximum strengths of 2.4 m long walls

Table 15 summarises the average peak loads resisted in the tests for each wall type (given as the average of the wall peak push and peak pull loads) and Figure 34 plots the maximum wall strength per unit wall length achieved in the bracing tests. For walls without end straps the strength was determined by the wall 'rocking strength' for all except Wall Type 1 which failed at a lower load. Where the 'rocking' mechanism is the studs uplifting from the bottom plate then the 'rocking strength' is influenced by the fastener 'breakout' strength as discussed in Section 4.5.8 and thus the 'rocking strength' is influenced by both sheathing type and wall hold-down system. The averaged 'rocking strength' of these 2.4 m long walls is given in Table 20.

The values in Table 20 are compared with the theoretical values from Section 4.5.11 in Table 21. The measured ratios are significantly less than 1.0, indicating that the actual strength was less than the theoretical strength for bottom plate uplift. The measured 'rocking strength' was actually very close to the theoretical strength for stud-uplift in Table 2 for zero fixing strength between fasteners and sheathing perpendicular to the sheathing edge (i.e., Fp = 0 kN) which corresponds with the observation that most walls failed by stud-uplift. It is likely that when the contribution from the second and third stud in bottom plate to stud force was large then 'breakout' had occurred indeed making Fp = 0.

Table 20. 'Rocking strength' for 2.4 m long walls.

End Restraint	Walls without end straps
'three nail'	3.81 (kN/m)
'six nail'	5.45 (kN/m)

Table 21. Ratio of measured to predicted 'rocking strength' due to bottom plate uplift for 2.4 m long walls.

End Restraint	Walls without end straps
'three nail'	0.71
'six nail'	0.76

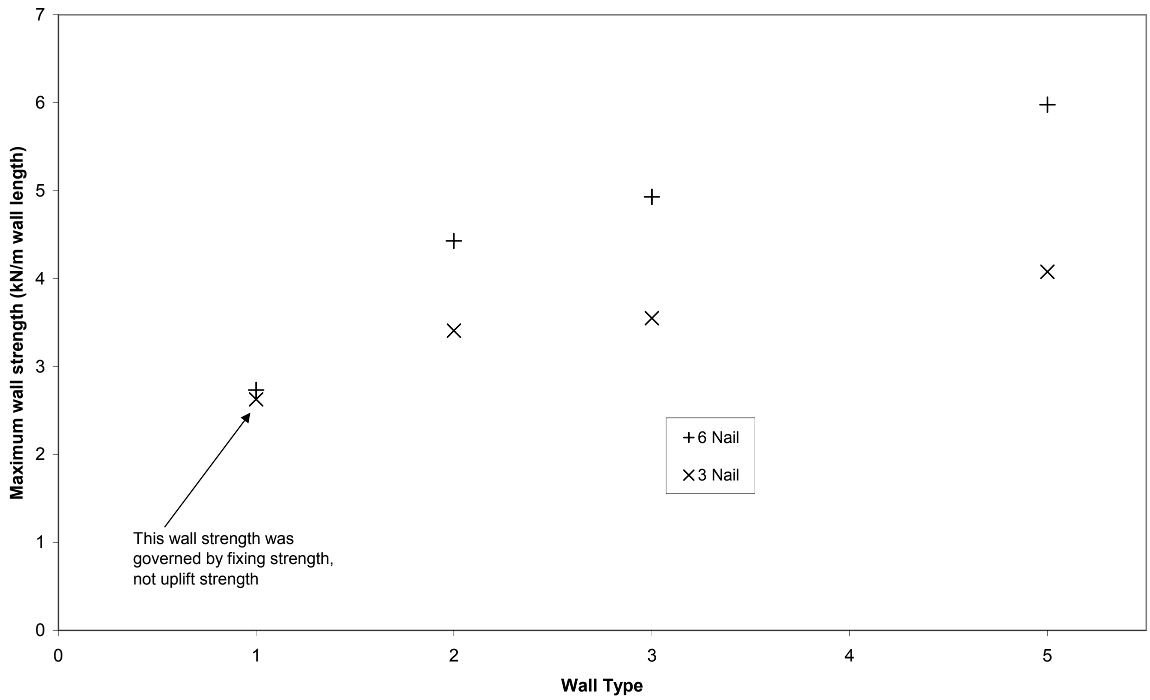


Figure 34. Influence of end restraint on wall strength for 2.4 m long walls (no end straps).

10.5 Bracing ratings of 2.4 m long walls

The bracing values evaluated from the test results as per EM3-V1 (Thurston, 2004) and the P21 procedure (King and Lim, 1991) are given in Table 16. The ratio of the EM3 to P21 evaluated bracing values are also given in Table 16 and these are plotted in Figure 35.

The EM3 test procedure has increased the wind bracing ratings for all tests except Wall Type 1 which showed no change.

As it was observed that the EM3 to P21 ratio increased with increasing wall strength, the data has been replotted in Figure 36 against this variable. The trend for this increase is clear and is explained by the stronger walls being limited by 'rocking' action in the P21 type tests and thus able to benefit from the 'six-nail' end restraint in the EM3 tests.

The change in earthquake bracing ratings is significant particularly for the weaker walls which do not gain as much advantage from the increased end restraint of the EM3 method.

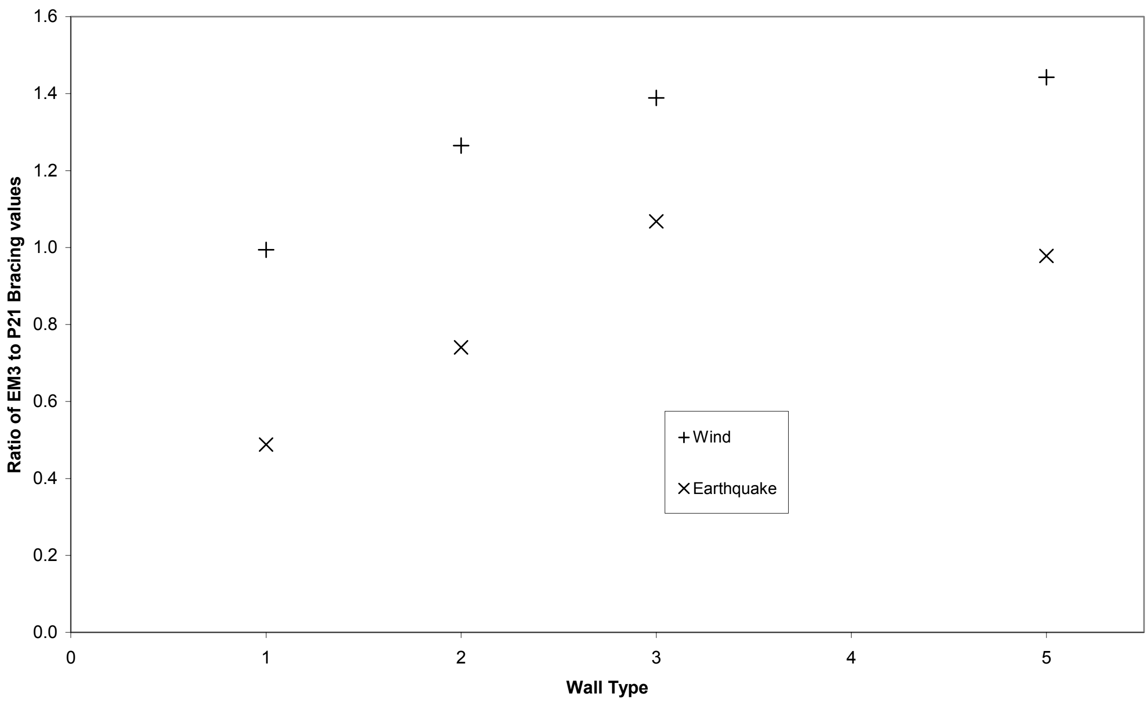


Figure 35. Ratio of bracing values from EM3 to P21 procedures for 2.4 m long walls.

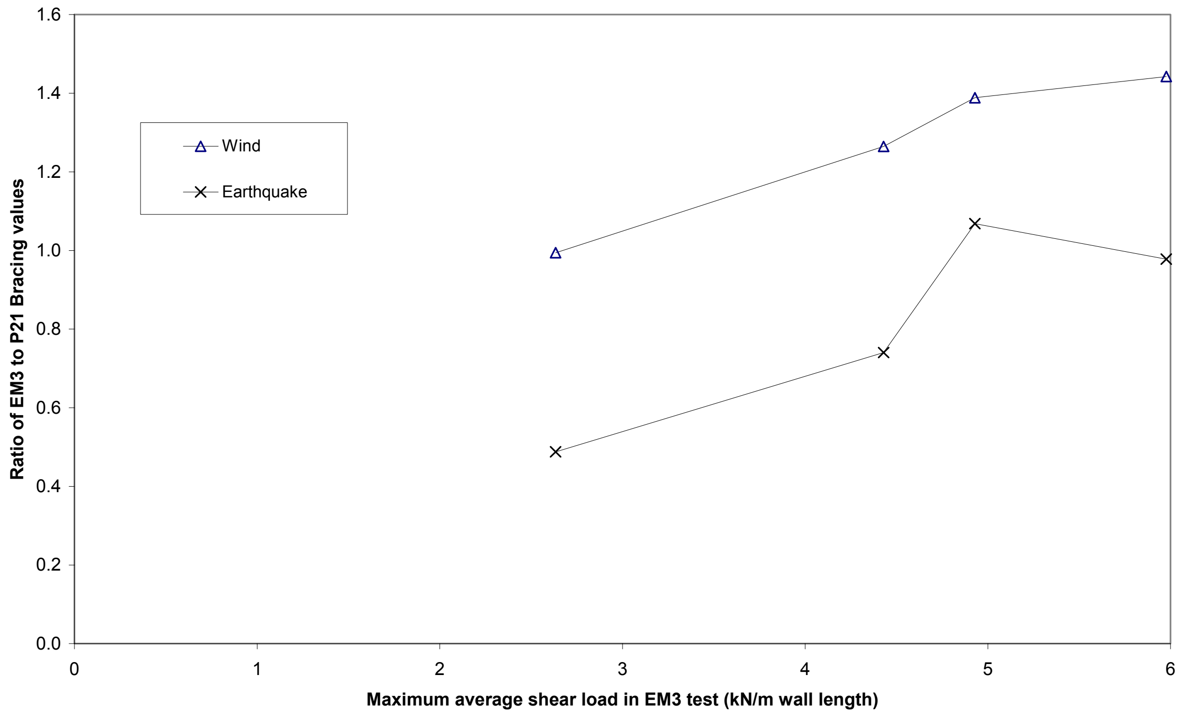


Figure 36. Ratio of bracing values from EM3 to P21 procedures for 2.4 m long walls as a function of wall strength.

10.6 Test results of walls on simulated concrete foundations

A comparison of results in Table 15 and Table 16 for corresponding walls with a 'six-nail' uplift restraint (i.e., Test 41 with 42 – Refer to Table 17) shows that although the peak strengths of the bolted walls on simulated concrete foundations were similar to the walls on simulated timber foundations the earthquake bracing strengths were significantly lower. This is attributed to the 'rocking' mechanism of stud lifting which pulled fixings out of the bottom of the sheet resulting in a brittle weak 'rocking' mechanism whereas the bottom plate uplift mechanism enabled gave a more ductile deformation mechanism. These results show that bracing tests need to be performed for walls on both timber and concrete foundations.

If an end strap was wrapped around the end studs of the wall on a concrete foundation (see Figure 18) a brittle mechanism still occurred but the bracing strength did exceed that for the wall on timber foundation (c.f. Tests 42 and 43).

The detail used for fixing the hold-down strap to the bottom plate in Test 51 (Figure 10) proved to be poor because splitting occurred at the top and bottom nail into the bottom plate. Hence in the tests on 2.4 m long walls a revised detail was used which did not split (Figure 18).

Increasing the edge distance of the fixings at the bottom of the sheet (c.f. Tests 41 and 44) increased the peak load resisted by 21%.

10.7 Conclusions from 2.4 m long wall racking tests

The conclusions are similar to those found for the 1.2 m long walls in Section 9.7 except that the failure mechanism of 'rocking' by end stud lifting and subsequent 'breakout' of the fixings from the bottom plate was more predominant.

11. MAGNITUDE OF FACTORS USED IN THE EM3 METHOD

11.1 EM3 Method - summary of principles

The bracing ratings are the average of three tests. This reflects the knowledge that extreme events are rare, houses are stronger under racking than the sum of the ratings of individual bracing walls and life risk under overload conditions is small.

All bracing systems are evaluated at deflections in the range 25-35 mm to ensure a reasonable compatibility of mixed systems in house construction.

The derivation of seismic resistance is compatible with the seismic load demand spectra in NZS 4203 (SNZ, 1992) which is used as a basis of load tables in NZS 3604 (SNZ, 1999).

The earthquake bracing rating = F1 x F2 x F3 x R where R is the average push/pull residual load after three load cycles to 25, 30 or 35 mm. R may not be greater that 1.05 x the smallest of the push or pull load to penalise non-symmetrical systems.

The wind load bracing rating = F3 x P where P is the average push/pull peak load from load cycles to 25, 30 or 35 mm. P may not be greater that 1.05 x the smallest of the push or pull load to penalise non-symmetrical systems.

11.2 F1 factor

The F1 values used are a function of deflection of the bracing wall at which the test wall peak loads are measured. F1 values were selected based on inelastic time history computer analysis of actual test data (Thurston and Park, 2003) using earthquakes giving similar seismic response spectra to those used in NZS 4203 (SNZ, 1992).

11.3 F2 factor

The factor F2 varies between 1.0 and 1.2 and is used in recognition of building ‘systems effects’ as discussed in this report. The exact value of F2 depends on the ability of the wall to deflect to 45 mm with little strength reduction and is a reflection of the ductility of the wall. In a test on a single storey house, Thurston (2003) found that a value of 1.5 was applicable to the house tested assuming all walls (even nominal walls) were accounted for.

11.4 F3 factor

This factor is generally 1.0 but can be 0.8 in situations such as walls without plasterboard lining on one side or walls terminating at doorways without end straps.

Appendix C proposes that the value of F3 given as 0.8 above be reduced to 0.7.

12. MODIFYING EXISTING P21 RESULTS TO EM3

It will be necessary for manufacturers to retest to EM3 even if they have already tested to the P21 test. Predicting the results of EM3 tests based on P21 tests is difficult as the P21 tests were often just a test of the wall panel ‘rocking strength’ rather than a function of ‘fastener strength’. However, the following options are recommended:

Option 1. Perform a single indicative test to EM3. An experienced testing laboratory may then assess the EM3 rating based on this test and the previously acquired P21 test results. An upper limit of bracing strength which may be assessed is 0.95 x the indicative EM3 test evaluation.

Option 2. This is only available to fibre cement or wood based sheathings (eg plywood or MDF). The EM3 wind rating may be taken as the P21 rating for wind. For walls without end straps the EM3 earthquake rating may be taken as 0.9 x the P21 rating for earthquake. For walls with end straps the EM3 earthquake rating may be taken as 0.8 x the P21 rating for earthquake.

Option 3. This may be applied to walls not covered by Option 2. The EM3 wind rating may be taken as 0.8 x the P21 rating for wind. The EM3 earthquake rating may be taken as 0.5 x the P21 rating.

Option 4. An experienced testing laboratory may assess the EM3 rating based on the hysteresis loops from a P21 test for walls which did used an end strap but will not do so in the proposed system.

13. MAJOR CHANGES BETWEEN P21 AND EM3

The following are the key changes that are being proposed for introduction within EM3 over the approach which has been previously applied within the P21method.

1. The loading protocol uses three cycle sets that are increased in amplitude until the specimen is unable to reliably sustain lateral load. The resulting assessment is much less influenced by the selection of the target displacement chosen for the ultimate limit state excursions. Similarly knowledge of the behaviour of the specimen beyond its peak load carrying deformation is required. Systems that degrade slowly will achieve a higher rating than their stiff/brittle counterparts through the application of F2, the Robustness factor.
2. The connection point between the specimen and the loading device is recommended to be central to the length of the wall to minimise any tilt effects that can occur as the wall rocks under load.

3. The connection between the specimen and the base of the test frame is now an integral part of the scope of application for which the bracing rating can be applied. The increasing emphasis on deformation compatibility together with the acknowledgement that walls with rigid hold-down devices typically experience a greater resistance but at a much lower level of deformation. Thus walls affixed to a concrete slab will need to be tested with a simulated bolted base fixity (which can be expected to be stronger and stiffer) while those affixed to a timber floor will need to be tested with simulated nail connections.
4. The basis of assessing the specimen ductility has changed. The K4 factor in the P21 test based on a crude assessment of ductility has been replaced with an F1 factor. F1 values have been derived from time-history analysis of the design level earthquake (as prescribed within the NZ Loading Standard NZS 4203 (SNZ, 1993)) and the pinched hysteretic model appropriate for degrading systems. It may be considered as a 'quasi-ductility' coefficient which translates the bracing rating demand published in section 5 of NZS 3604:1999 (based on a ductility factor of 3 and fundamental response period of 0.4 seconds) to the mass which can be sustained by the wall without the wall exceeding its maximum reliable displacement as assessed during the test.
5. The error in assessing the 'equivalent yield' displacement (being displacement at half peak load so that a totally brittle system was incorrectly assessed as having a ductility of 2) has also been addressed. Fully brittle systems will now be correctly assigned a ductility of 1.
6. The robustness of bracing walls together with an allowance for redundancy are incorporated within the F2 factor. This is based on the ability of the test wall to carry load at overload deflections. This is combined with a 'systems effect' allowance of 20% resulting in an F2 factor that ranges from 1 to 1.2. This enhancement is justified on the basis that provided the wall can sustain large deformation without collapse, the contribution of secondary and non-structural components, hitherto ignored, can also be assumed to provide some resistance.
7. For EM3 wind bracing rating, the 'systems effect' allowance of 20% has been ignored to take into account the potential for wind uplift forces. However, for strong wall sheathings or for walls with sheathings on both sides the greater end restraint of the EM3 procedure generally results in an increase in wind rating compared to the P21 method.
8. Deformation compatibility between bracing walls within an elevation of a building has resulted in the introduction of a more clearly defined deformation band being between 25 and 35 mm.
9. The F3 factor has been introduced for those construction situations where the greater end 'uplift restraint' used in the EM3 method cannot be justified – (ie at doorways and non-lined walls).

14. RECOMMENDED CHANGES TO NZS 3604:1999

Over the last few years BRANZ has made recommendations in various reports for modification of the bracing requirements of NZS 3604. BRANZ would like the NZS 3604 committee to consider the following changes.

- Call up the EM3 test method to supersede the P21 test method.
- Thurston (2003) recommended that a nominal bracing rating be allowed for walls without a proven bracing rating but sheathed with plasterboard, plywood, fibre cement etc between floor and ceiling and fixed at a minimum of 300 mm centres around the sheathing perimeter to timber framing. (Refer to Section 3.4.)

- NZS 3604 requires exterior walls to have a minimum bracing rating of 0.5 kN/m length which is low and does not relate to the house bracing demand. Thurston (2003) instead proposed a minimum bracing rating in walls as a function of whether the wall was an exterior or interior wall, and the average bracing demand per unit width of the building.
- Beattie (2004) recommended a reduction in bracing capacity of walls supported within the span of joists.

15. SPECIFIC DESIGN OF NON-NZS 3604 BUILDINGS

Non-NZS 3604 type buildings do not necessarily have the bracing wall 'continuity' nor the total-building 'systems effects' nor the damping inherent in NZS 3604 buildings.

All non-3604 buildings need to be designed to ensure that the predominant displacement mode is due to slip between sheathings and framing and that other brittle failure mechanisms are suppressed. Special end uplift restraints should be designed to resist panel overturning.

Panel over-strength needs to be considered and other elements designed for the associated greater force using a capacity design procedure. The end uplift restraints, chords and sheathing need to be designed for this over-strength. It is recommended that panel over-strength be considered to be 1.5 x the panel wind ultimate limit state bracing rating.

Designers must ensure that bracing walls sheathing are fixed to framing on all edges, the top and bottom of the sheathing are fixed to a diaphragm at the (roof/ceiling/floor) and the bottom plate is held down and prevented from sliding.

The EM3 test will ensure the above walls, so designed, have an effective ductility of at least 3.0 at the assessed bracing deflection.

16. CONCLUSIONS

An EM3 test and evaluation procedure has been recommended to replace the existing P21 method for determining bracing ratings of walls that may be used for the design of lightweight structures to NZS 3604. Building performance under design level winds and earthquakes is expected to be more accurately and reliably represented by the summation of walls tested and evaluated by the EM3 method.

The P21 test had many fundamental defects which are discussed in this report. One major defect was that most walls tested failed due to 'rocking' action which was more a test of the supplementary end restraints used in the test than the wall construction itself. The EM3 test doubles the supplementary end restraints and justification for this is provided in the report.

Under test method EM3, as against P21, it was found that plasterboard walls without fibreglass reinforcing and with 6 mm head fasteners did not increase in strength with the increased supplementary end restraints and reached peak loads at low deflections incompatible with other bracing wall systems. These walls had a relatively brittle failure mechanism. Consequently both their wind and earthquake bracing rating will decrease. However, for the other bracing walls tested the following conclusions were made:

- The wind bracing rating will increase (see Table 16 and Figures 26, 27 and 31).
- The seismic rating will increase for 1.2 m long walls without end straps but generally decrease for 1.2 m walls with end straps and 2.4 m long walls.

An analysis (based on a BRANZ data base of actual building construction) indicated that the change from the P21 test to the EM3 test would add a maximum $145 expense to the average house if the 'systems effect' factor of F2 = 1.2 is used. If less conservative assumptions were made this value reduces significantly.

The changes in EM3 are expected to reduce the number of end straps used in buildings except at door openings and wall free ends in critical situations. End straps are difficult to install on site and tests have shown they are generally only of benefit at door openings and wall free ends.

The EM3 method will allow advantage to be gained from walls sheathed on both sides. This was not possible with the P21 method as the walls 'rocked' at such a low load.

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APPENDIX A. SMALL SAMPLE TESTING

A.1 Tensile strength of nailed bottom plate to stud joint

Rocking action of timber framed walls is usually associated with either lifting of the stud from the bottom plate or lifting of the bottom plate from the foundation beam as discussed in Section 4.5.11. Tests measuring the strength of the stud/bottom plate joint are discussed in this section and the results are compared with measurements reported elsewhere.

The monotonic tests were performed in the BRANZ Structures Dartec test machine. The setup is shown in Figure 37. The timber was short lengths of kiln dried machine stress graded F5 radiata pine with the bottom plate being 90 x 45 mm and the stud 90 x 35 mm. The stud was held in hydraulic grips in the machine top jaws and bottom plate bolted to the machine bottom platen. The load rate was based on the EM3 test method, assuming that the bracing test results are performed at a deflection rate of 4 mm/second. It was further assumed that 50% of the wall movement was due to wall 'rocking' by lifting of the stud and that the test wall had an aspect ratio of 2.0. This resulted in a plate to stud separation rate of 50%x4/2 = 1 mm/second which was used in tests described below.

Ignoring the skewed nail options, Table 8.19 of NZS 3604 specifies two methods for connecting wall studs to plates; namely two 100 x 3.75 mm long flat head nails or three 90 x 3.15 mm power-driven nails. The NZS 3604 standards committee (Shelton, 2000) designed the joint to meet the shear demand of this joint under face loading. Based on tests (Shelton, 2000b[1]) BRANZ has issued opinions[i] for some nail manufacturers that only two of their proprietary nails may be used at these joints in NZS 3604 buildings provided the wall height does not exceed 2.7 m. Thus, the bracing wall testing described in this report used two such nails to connect the studs to the frames. The testing described in this Appendix measures the tensile strength of the following stud to plate connections:

(1) Two bright 100 x 3.75 mm long flat heat nails

(2) Two 90 x 3.15 mm coated and galvanized Paslode JDN gun-nails (i.e., power-driven nails)

(3) Two 90 x 3.15 mm coated but not galvanized Paslode JDN power-driven nails

[i] This information is quoted with the consent of clients, Paslode New Zealand formerly ITW Construction Products.

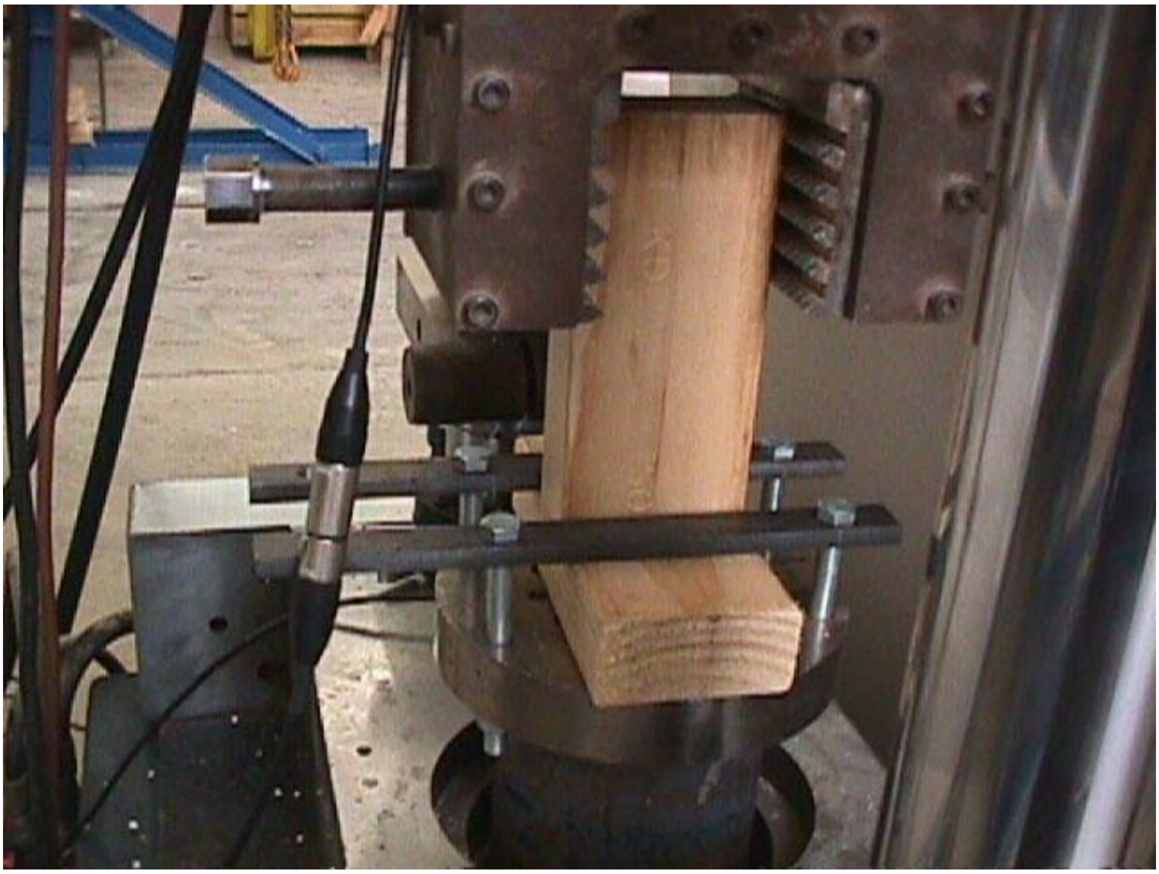

Figure 37. Test setup used to measure stud/plate tension strength.

Sample plots for load versus pull-out deflection of the galvanised power-driven nails are shown in Figure 38. Note that after peak load has been reached the load resisted by power-driven nails drop rapidly until reaching a plateau. The drop is attributed to the failure of the nail/timber glue bond. The ‘residual load’ is expected to be due to the nail/timber friction.

Table 22 lists statistical data on the peak load, the resisted load at 4 mm deflection and, for the power-driven nail tests only, the ‘residual load’. It can be seen that the power-driven nails had significantly stronger pull-out strength. A large variability can be expected due to variation of end grain conditions.

The connection strength between bracing wall stud and bottom plate is due to a combination of:

(1) the stud/bottom plate nailing

(2) the sheet to bottom plate fixing

(3) end straps between stud and bottom plate if applicable

Item (2) above is a significant portion of the connection strength and from fastener slip tests at BRANZ it is known that these peak at fastener slips of approximately 3 mm which means peak racking strengths will occur at stud/bottom plate vertical movements somewhat greater than this, say 4-6 mm. However, the peak load in the stud-bottom plate tension tests occurred at a pullout deflection between 0.5-1.3 mm which would only activate low uplift resistance due to the sheet to bottom plate fixing. It is therefore expected that the peak ‘rocking strengths’ of Figure 28 occurs when stud/plate joints opens more than 4 mm. Table 22 presents the ‘two-nail’ joint strength for opening of 4 mm and the average value of 1.53 kN for power-driven nails which are not galvanised is used in theoretical analysis in Section 4.5.12.

Using a load rate of 2 kN/minute, Herbert and King (1998) gave peak pullout strength of two 90 x 3.15 mm Paslode gun-nails driven through a 45 mm thick bottom plate into the end grain of a 90 x 35 mm timber stud as 4.1 kN. This is similar to the value in Table 22 (4.26 kN) but as noted in the paragraph above it is the strength at significant joint movement, say 4 mm, which is of most significance.

Table 22. Withdrawal strength of two-nail plate to stud connection.

	Nail Type		
	Power	**Power**	**Hand**
Galvnanised?	Yes	No	No
Number of samples	6	8	7
First Peak			
Mean strength/joint	4.26	3.05	1.61
SD joint strength	0.56	0.63	0.34
Post Peak			
Mean strength/joint	2.98	1.81	
SD joint strength	0.64	0.62	
4 mm deflection strength			
Mean strength/joint	2.61	1.53	1.16
Standard deviation joint strength	0.29	0.71	0.20

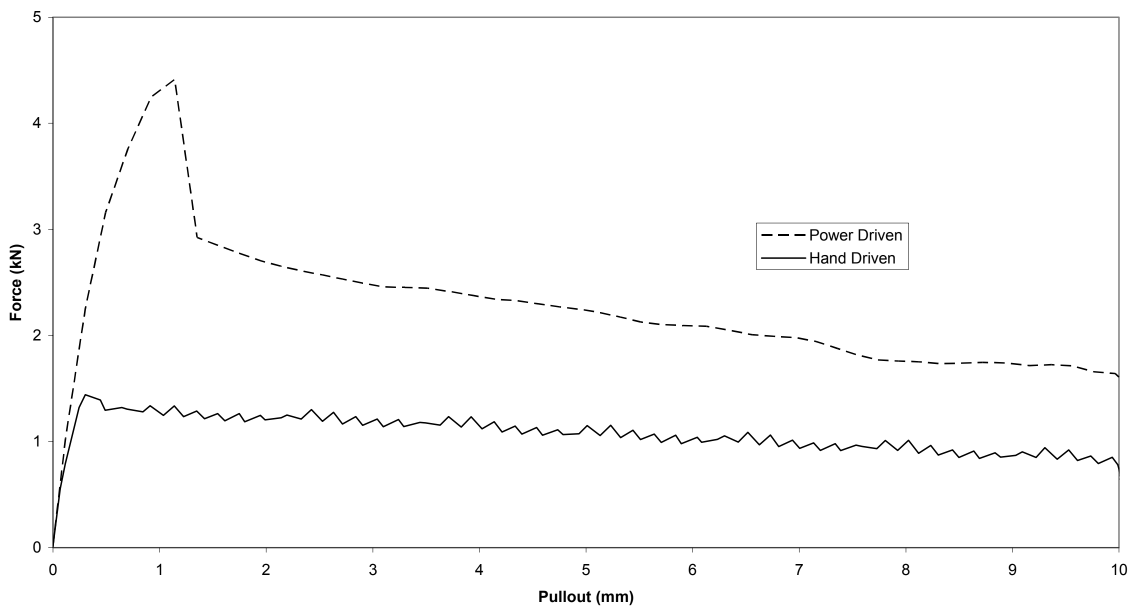

Figure 38. Typical relationship between tension load and two-nail stud/plate joint deflection.

A.2 Tensile strength of bottom plate to timber foundation beam nailed connection

'Rocking' action of timber framed walls is usually associated with either lifting of the stud from the bottom plate or lifting of the bottom plate from the foundation beam as discussed in Section 4.5.10. Tests measuring the strength of the bottom plate to foundation beam joint is discussed later in this section and the results are compared with measurements reported elsewhere.

The monotonic tests were performed in the BRANZ Structures Dartec test machine using a load rate of 1 mm/second for the reasons discussed in Section A.1. The setup is shown in Figure 39. A strip of particleboard was nailed to a length of 90 x 45 mm kiln dried radiata pine to form a simulated foundation beam. A length of 90 x 45 mm kiln dried radiata pine bottom plate was nailed to the foundation beam using either a bright 100 x 4 mm flat head nail or a Paslode 90 x 3.15 mm Power-driven coated nail (non- galvanised).

Table 8.19 of NZS 3604 specifies two options for connecting external wall bottom plates and internal bracing walls to floor framing, namely two 100 x 3.75 mm long flat heat bright nails or three 90 x 3.15 mm power-driven nails at 600 mm centres. For internal walls the options are one 100 x 3.75 mm long flat heat nails or one 90 x 3.15 mm power-driven nails at 600 mm centres. In the bracing tests of this study report the fixing used was two 100 x 3.75 mm long flat heat nails at 600 mm centres.

Figure 39. Test setup used to measure bottom plate to timber foundation beam tension strength.

Sample plots are shown in Figure 40. Note that with power-driven nails after peak load has been reached, the load drops rapidly until reaching a sloping plateau. The drop is attributed to the failure of the nail/timber glue bond. The residual load is expected to be due to the nail/timber friction.

Table 23 lists statistical data on the peak load, the resisted load at 4 mm deflection and, for the power-driven nail tests only, the ‘residual load’. It can be seen that the power-driven nails had significantly stronger pull-out strength but only slightly greater strength at 4 mm pullout deflection.

This bottom plate/foundation beam strength for each pair of hand-driven 100 x 3.75 bright flat head nails used in the theoretical analysis in Section 4.5.11 was derived from the average value from Table 23 of 2 x 1.54 kN = 3.08 kN for two bright nails. Despite being at a slower rate of 2 kN/minute Herbert and King (1998) measured a larger average first peak pullout strength of two 100 x 4 mm hand driven bright nails as 3.8 kN. On the other hand, Thurston (1993) measured the average strength as 2x1.32 = 2.64 kN using an even slower load rate of 1 kN/minute which appears to be compatible with the values measured in this test series (note: greater load rates generally results in greater strengths).

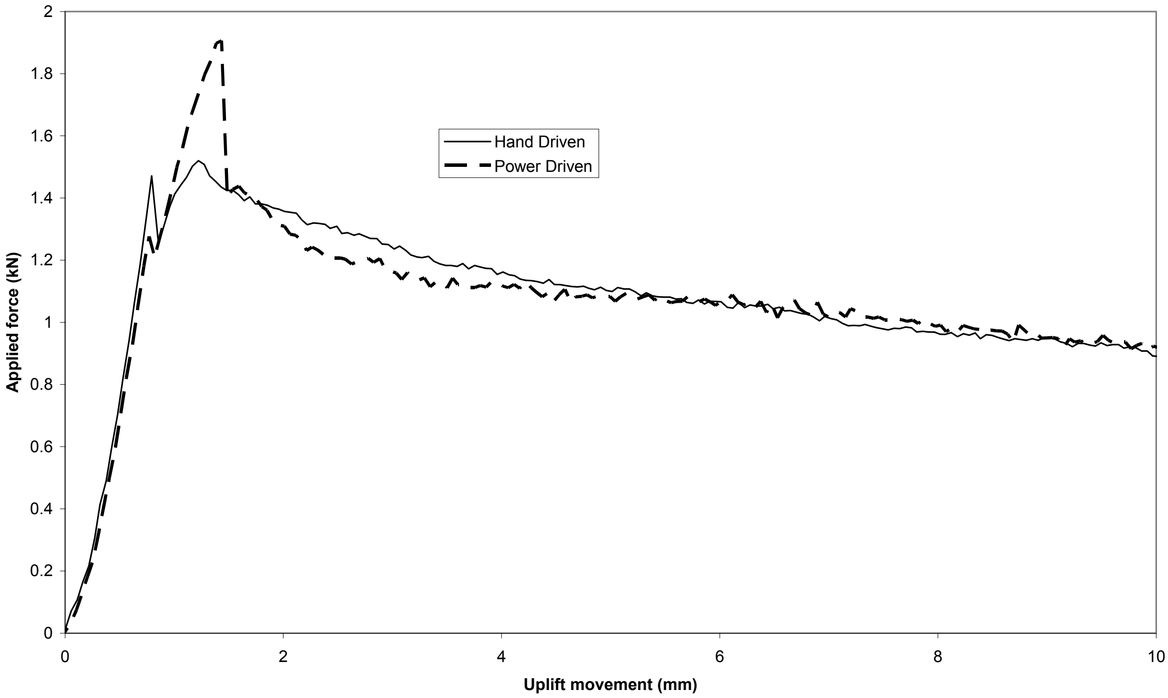

Figure 40. Typical relationship between tension load and single nail plate to foundation beam joint deflection.

Table 23. Withdrawal strength of single nail on plate to stud connection.

	Power	**Hand**
Number of samples	6	8
First Peak		
Mean strength/joint	2.06	1.54
SD joint strength	0.19	0.23
Post Peak		
Mean strength/joint	1.57	
SD joint strength	0.22	
4 mm deflection strength		
Mean strength/joint	1.47	1.25
Standard deviation joint strength	0.21	0.19

APPENDIX B. PROPRIETARY PRODUCTS USED

Three proprietary sheathings were used in the experimental programme described in this report and referred to as Type SPB or BRL, FC,PLYA, PLYB These products are defined below:

Table 24. 2.4 x 1.2 m Sheathings used in the test.

Label	Description
SPB	10 mm Standard Gib® manufactured by Winstone Wallboards Ltd. This was an off-white paper faced gypsum plaster based board.
BRL	10 mm Gib Braceline® with fibreglass reinforcement in the core manufactured by Winstone Wallboards Ltd. This was an blue paper faced gypsum plaster based board.
PLYA	7.0 mm Grade D-D untreated plywood manufactured by Carter Holt Harvey Ltd.
PLYB	12.0 mm ShadowClad Groove plywood manufactured by Carter Holt Harvey Ltd.
FC	7.5 mm Monotek fibre cement sheet manufactured by James Hardie.

The following sheet fixing types were used in this report. The generic descriptions are given in Figure 9 and correspond to the proprietary names given in Table 25.

Table 25. Fixings used in the testing reported herein.

Name used in this report	Sheathing material	Proprietary name
GGS screw	SPB	GIB® Grabber drywall screws
GBN nail	BRL	GIB Braceline® nails
GBC clout	BRL	GIB® clout
PWA clout	PLYA	Plywood 30 mm clout
PWA clout	PLYB	Plywood 50 mm clout
FCSS nail	FC	Hardiflex® nail

Note: Results obtained in this study relate only to the samples tested, and not to any other item of the same or similar description. BRANZ does not necessarily test all brands or types available within the class of items tested and exclusion of any brand or type is not to be taken as any reflection on it.

This work was carried out for specific research purposes, and BRANZ may not have assessed all aspects of the products named which would be relevant in any specific use. For this reason, BRANZ disclaims all liability for any loss or other deficit, following use of the named products, which is claimed to be reliance on the results published here.

Further, the listing of any trade or brand names above does not represent endorsement of any named product or imply that it is better or worse than any other available product of its type. Laboratory tests may not be exactly representative of the performance of the item in general use.

APPENDIX C. PROPOSED MODIFICATIONS TO EM3 -V1

This report, and the testing described in the report, has been written based on EM3-V1 (Thurston, 2004). However, during preparation of this document the author considered the following two changes should be made to EM3-V1.

Change 1. Instead of 'six-gun-nails' in the end restraint, it is proposed to use eight. This is because:

- most of the tests were still governed by rocking.
- the degree of uplift restraint, namely 9.23 kN (see Section 4.5.3) is less than the 12 kN value recommended based on tests by Herbert and King (1998) and the full restraint recommended based on tests by Thurston (1993). The full scale house testing by Thurston (2003) indicated that an 'eight-nail' end restraint would still be conservative.
- it would raise the seismic rating of strong sheathing systems, in particular walls sheathed on both sides while not reducing the rating of the weaker sheathing systems. This would increase average house bracing rating and hence reduce the amount of bracing walls required and improve the economics of the change from the P21 to EM3 system. The full scale house testing by Thurston (2003) indicated that this was justified.

Change 2. If Change 1 is made then it is recommended that where factor F3 = 0.8 it is reduced to 0.7. For lined houses with taped and filled joints this situation only occurs in single storey or upper storey walls at door openings or wall free ends. The new value of F3 will counteract the potentially greater net uplift force from Change 1. It is also the value recommended by Thurston (1993) based on his tests of long walls. This change will encourage the use of end straps at doorway openings where tests have shown they are most effective, but give less advantage to their use elsewhere. End straps are difficult to use on site and are often omitted.