STUDY REPORT

No. 119 (2003)

Full-Sized House Cyclic Racking Test

S. J. Thurston

The work reported here was jointly funded by Building Research Levy and the Foundation for Research, Science and Technology from the Public Good Science Fund.

ISSN: 0113-3675

Preface

This is the first BRANZ report on seismic testing of a full-sized timber framed New Zealand house. 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. House wall bracing ratings are usually derived in New Zealand from the BRANZ P21 test (BRANZ Technical Recommendation No 10). A proposed revised method of evaluation of the P21 test results are discussed by Thurston and Park (2002) based on expected whole house performance.

Acknowledgments

This work was jointly funded by the Building Research Levy and the Foundation for Research, Science and Technology from the Public Good Science Fund. Plasterboard used in relining the house was donated by Winstone Wallboards.

Note

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

FULL-SIZED HOUSE CYCLIC RACKING TEST

BRANZ Study Report SR119 (2003)
S J Thurston

REFERENCE

Thurston 2002. Full-Sized House Cyclic Racking Test. BRANZ Study Report SR119 (2003), Judgeford, Wellington.

ABSTRACT

Earthquake and wind loads for timber framed house designs in New Zealand are specified in NZS 3604 (1999). Various lining and cladding manufactures publish bracing strengths for their wall systems based on the BRANZ P21 racking test. The P21 tests are carried out on a short length of wall with contrived end restraints to simulate continuity of actual construction.

To verify that this design approach is realistic, an existing house was relined and cyclically racked to failure. This paper compares the actual house strength with the strength determined using the NZS 3604:1999 design provisions. Free vibration tests to measure the house natural frequency and damping are also reported.

CONTENTS

Page No.

Appendices

.

1. INTRODUCTION

Houses in New Zealand are generally constructed with light timber framed (LTF) walls each with a variety of lengths, sheathing and fastening systems. The result is many different bracing systems, each of which achieves peak bracing resistance at different deflections. Thurston and Park (2002) discuss this incompatibility problem and also the basis for changing the current test and evaluation procedure used to establish bracing ratings. The current test (King and Lim 1991) is known as the BRANZ P21 test method (Cooney and Collins 1979) and is used to obtain the bracing resistance ratings of timber framed wall systems to meet the wind and seismic demand stipulated in the timber framed buildings standard, NZS 3604 (SNZ 1999).

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 woodframe 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. 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. Most bracing resistance of New Zealand homes is provided by plasterboard wall systems whereas, overseas, similar buildings use plywood and orientated strand board (OSB) lined walls in the belief that plasterboard is too brittle.

Fischer et al (2000) summarises 15 full size house racking and shake table tests. Most of these tests were carried out on prescriptively designed houses and the researchers found that there was adequate strength to meet code-required lateral forces. Only one of these tests measured house racking strength and compared this with the sum of the predicted individual house wall strengths, which is the approach of this report.

Without contrived end restraints to simulate continuity of actual construction, bracing walls without end straps tested as separate elements would fail at a low load due to rocking of the wall at the bottom compression end. In the testing reported herein and by Thurston (1994) it was observed that the behaviour of house walls is more consistent with the fastener slip model shown in Figure 1. The restraint used in the current P21 test method is effectively three nails in shear, as shown in Figure 1. Thurston and Park advocate increasing the restraint from a “three-nail” to a “six-nail” restraint in recognition of the inability of fixed wall panels in houses to rotate in practice.

The full scale racking test described in this report was a pilot investigation into the degree of conservatism implicit in the P21 test procedure which satisfies the bracing demands prescribed in NZS 3604.

2. DESCRIPTION OF TEST HOUSE

The test house is a simple single-storey, plasterboard lined, fibre-cement weatherboard clad, standard Fletcher Homes house, typical of those at the low cost end of the market available around 1990. It was bought unmodified from their catalogue. A plan of the house is shown in Figure 2 and photographs are given in Figure 3.

The house was initially bought by BRANZ with the intention of being used only for fire testing purposes and was simply placed on timber blocks nailed to a concrete foundation. By the time the structural testing project had started, most of the plasterboard in the house was fire-damaged, although the framing had not charred. All the house ceiling and wall plasterboard was stripped and relined except for the bathroom, toilet, wardrobes and one bedroom, all of which were undamaged by fire. The new lining was placed vertically with PLB gypsum plasterboard screwed at either 150 mm or 300 mm nominal centres and BRL bracing grade gypsum plasterboard fixed as a nailed bracing wall. Full details of all linings and fasteners used are given in Appendix C.

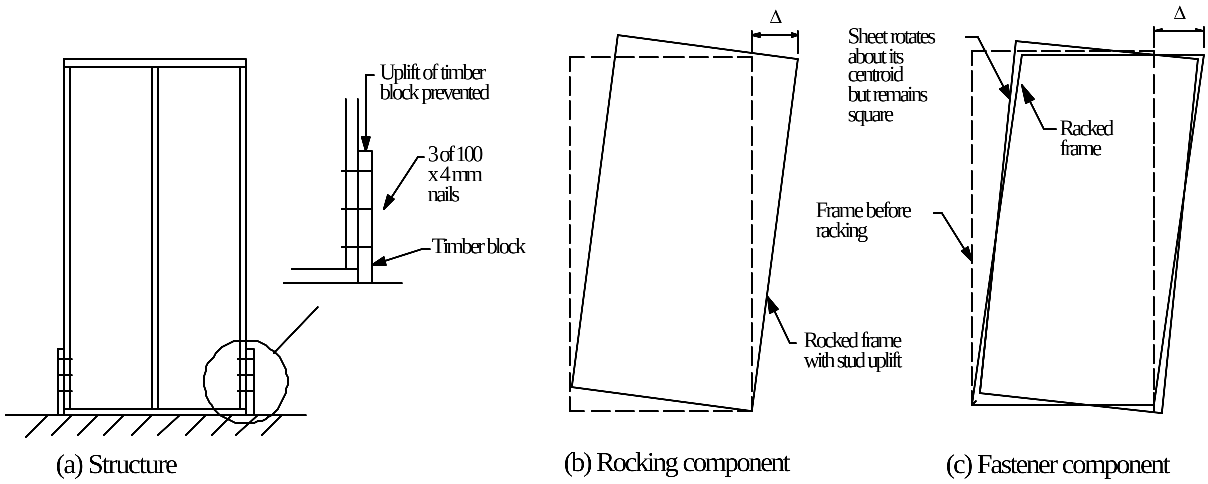


Figure 1. BRANZ P21 end restraints and components of deflection

The 20 mm particleboard floor was gun-nailed to 140 x 45 mm radiata pine joists at 450 mm centres which, in turn, were supported on 140 x 45 mm radiata pine bearers. The bearers were at 1800 mm centres and were supported on timber blocking set on a concrete pad. As the house was in a temporary location, the blocking foundation had little lateral strength and separate restraints were installed to resist horizontal house movement at floor level. (See Figure 4.)

Wall frames (2.4 m high) were prefabricated using gun-nails from kiln-dried radiata pine of 90 x 45 mm for exterior walls and 65 x 45 mm for interior frames. Horizontal nogging was at 800 mm centres. Studs were at 600 mm centres. No supplementary stud hold-down straps or devices were used throughout the entire building. The bottom plates were gun nailed to the flooring using only a single 90 mm long x 3.28 mm diameter gun nail at 600 mm centres, whereas NZS 3404 requires pairs of nails. At exterior walls and at interior Wall4 (Figure 2) these gun-nails penetrated into floor joists. The remaining interior walls were not located above floor joists and hence the bottom plate nails were only fastened to the particleboard floor. The exterior wall top and bottom plates were generally only continuous for 3 m when they butted together without any direct connections.

The walls and ceiling were lined with 10 mm plasterboard, as noted above. All plasterboard joints were fully plastered and reinforced with paper tape. No skirting was used at the base of the walls and no ceiling coves at the top of the walls. Timber ceiling battens of 68 x 32 mm at 400 mm centres ran perpendicular to trusses which were spaced at 1 m centres and spanned the 6 m width of the house. Ceilings were screwed to these battens at 200 mm centres using standard gypsum plasterboard fixing details. At exterior walls the ceilings were fixed to double top plates. At interior walls the ceilings were not directly fixed to the wall plates. These two details are illustrated in Detail 5 of Appendix C.

The corrugated galvanised steel roof (without sarking) sloped 20° to a centre ridge and had gable ends.

The exterior of the house was clad with fibre cement planks of cross section 300 mm x 6 mm. These were placed in the overlapping weatherboard style and were fixed to the adjacent stud near the bottom of the planks with a single 90 mm x 4 mm jolt head nail at 600 mm centres. The top of the plank was slotted under the plank above but was not fastened to it. Thus, the lateral resistance provided to the house by the exterior cladding is considered to be small.

This house has some weaker than usual racking resistance features. In particular, the fixing of bottom plates to the floor by only a single nail at 600 mm; lack of coving and skirting and wall hold-down straps and most internal walls were not over joists. On the other hand, the west exterior wall is expected to be particularly strong as it has no openings. Also, there were no doors in the exterior walls in the racking direction. (Thurston (1994) found that when long walls are racked they may experience uplift problems at door openings.)

3. TEST SETUP AND INSTRUMENTATION

Figure 4 shows a general view of the test arrangement. The house was cyclically racked by loading at the ceiling plane at four locations along the house length. Load was applied by means of two hydraulic jacks (see Figure 3) each fixed to a separate reaction frame. The reaction frame was bolted to an existing concrete pad. To prevent the pad from lifting, large blocks of concrete were stacked around the reaction frame. Each jack loaded a steel strut, which in turn loaded a steel beam (see Figure 2). Each end of each steel beam was connected to a timber beam (Figure 2), which spread the applied force along the adjacent house wall by use of strips of plywood located in the ceiling cavity. (See Figure 3(c).) An oil manifold connected the two systems to ensure that the same force was applied to both jacks and thus equal load was applied to the four timber beams. The ceiling diaphragm was strengthened by sheets of plywood in two places (see Figure 2) to facilitate more direct load transfer to a specific house wall. Further distribution of load was expected to take place from the ceiling acting as a diaphragm.

Measurements included force applied by each jack, in-plane deflection of walls relative to the floor (as shown in Figures 2 and 3(e)), house twist (Figure 3(f)), plasterboard crack width and wall uplift movement.

4. PREDICTING HOUSE STRENGTH

An objective of this study is to compare the measured house racking capacity in the north-south (N-S) direction with that which would be theoretically calculated by summing the transverse wall racking strength from the P21 (Cooney et al 1979) bracing procedures as stipulated in NZS 3604:1999. The purpose was not to compare the actual strength of this house with the demand loadings of NZS 3604.

The overall wall deflection in a racking test has two major components (see Figure 1); namely (1) that which is due to fastener slip between sheathing and framing, and (2) that which is due to total body rocking motion of the entire wall. There is little shear deformation of the panel itself. The latter movement is a function of the artificial end restraint. “Three-nail”, “six-nail” and total end uplift restraint have been considered in this paper. The predicted curve joining the cyclic load versus deflection plot peaks (hereafter called a “backbone curve”) of each house N-S wall was obtained as follows:

Laboratory testing was used to determine the load-versus-deflection curve for each type of fastener used to fix the plasterboard linings for the long sheet edge (both parallel to the papered edge and parallel to the cut, non-papered, edge). The wall load-versus-deflection relationship due to fastener slip was calculated using a theory developed by McCutcheon (1985) and also using the CASHEW software (Foltz et al 2000). Appendix A gives detailed information on this step. A good agreement was found between both theoretical models and full P21 test results. The relatively small deflection due to shear distortion of the sheathing was included in these calculations.

(a) The rocking component of deflection (see Figure 1) was obtained by examination of vertical movement at ends of bracing panels from historic P21 racking tests. Appendix B derives the relationships used in these calculations.

(b) The backbone curve for each wall was obtained by summing (a) to the theoretical wall deflection from shear distortion and fastener slip.

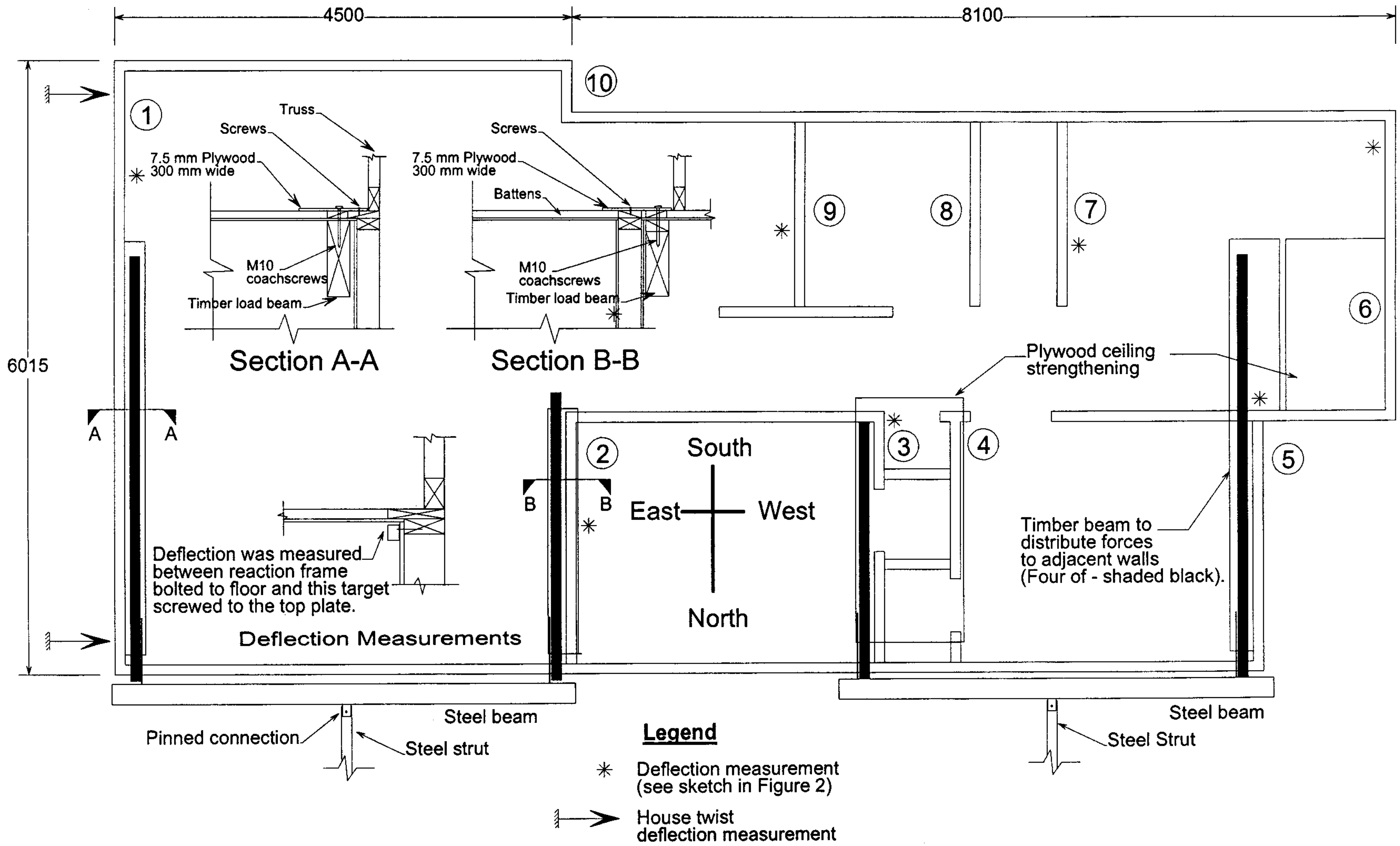


Figure 2 House plan and cross section

(a) General view of test in progress

(b) Connection to timber loading beams

(c) Plywood strip in roof space to connect timber loading beams to wall top plate

(d) Hydraulic jack, load cell and connection to reaction frame

Figure 3. Test photographs

(e) Measurement of deflection of walls

(f) House twist measurements

(g) Loading rig on west end of house

(h) General view from the east side

Figure 3. Test Photographs … continued

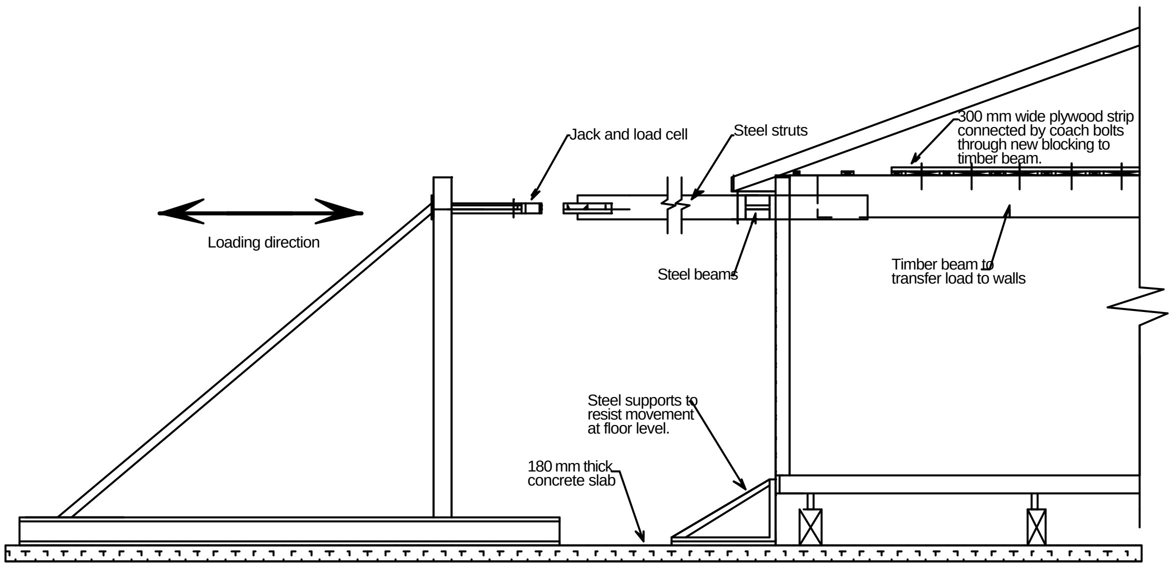


Figure 4 Test setup

The house backbone curve was obtained by summing the N-S wall backbone curves as discussed in Section 5.

5. COMPARISON OF MEASURED AND PREDICTED HOUSE STRENGTH

5.1 Phase 1 Testing

Test hysteresis loops are shown in Figure 5 for the early (near elastic) house response and in Figure 6 for the entire Phase 1 testing.

After three cycles to ±60 kN (maximum wall deflection being 7 mm) a close inspection of the house found no plasterboard cracking, even though the hysteretic response had become inelastic. The first damage was observed when the total applied load reached 71 kN when BRL bracing grade gypsum plasterboard cracking was observed commencing from the east wall window openings. Cracks from each corner of both of these windows had extended to the ceiling or floor after the three cycles to ±78 kN. As these cracks had either cut through or were on the panel side of, the panel fasteners, (see Figure 7) the bracing panel strength is expected to be severely diminished. During subsequent cycling these cracks widened and the fasteners along this plasterboard bottom plate pulled through the adjacent plasterboard edge. The east wall windows and the sliding door on the east end of the north face jammed.

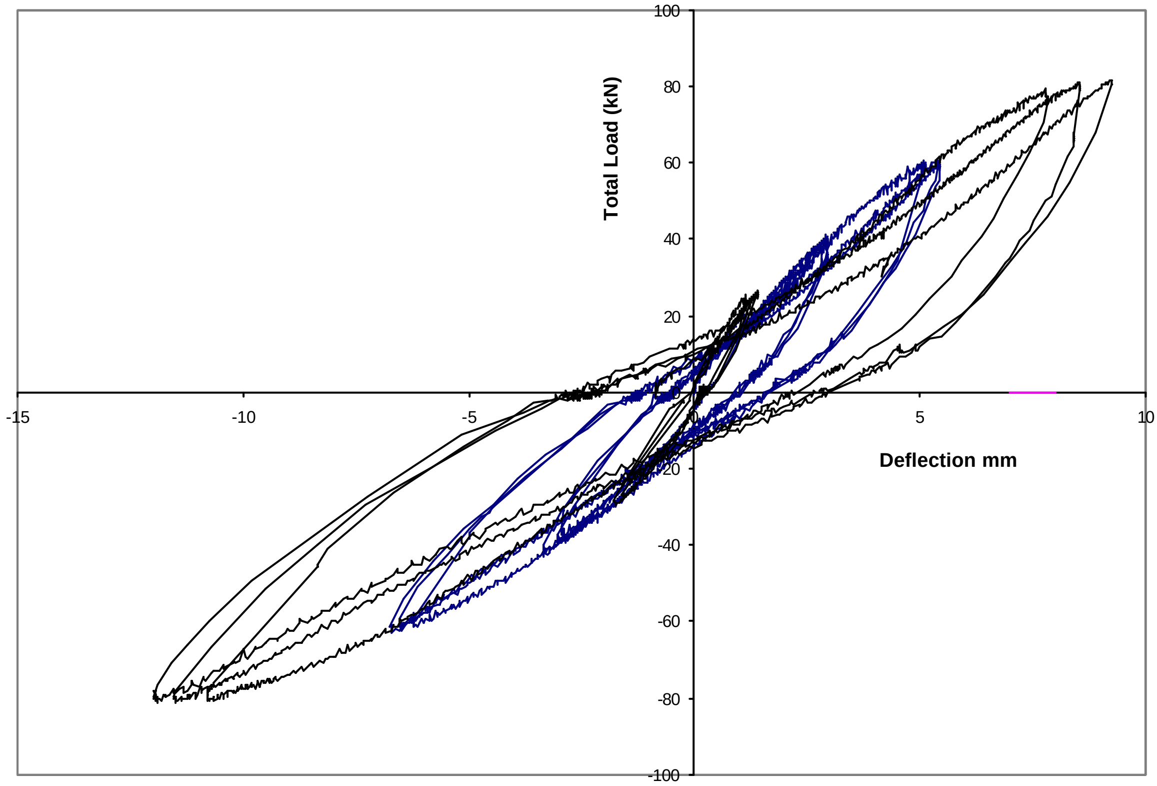


Figure 5. Racking hysteresis loops – early stages of test.

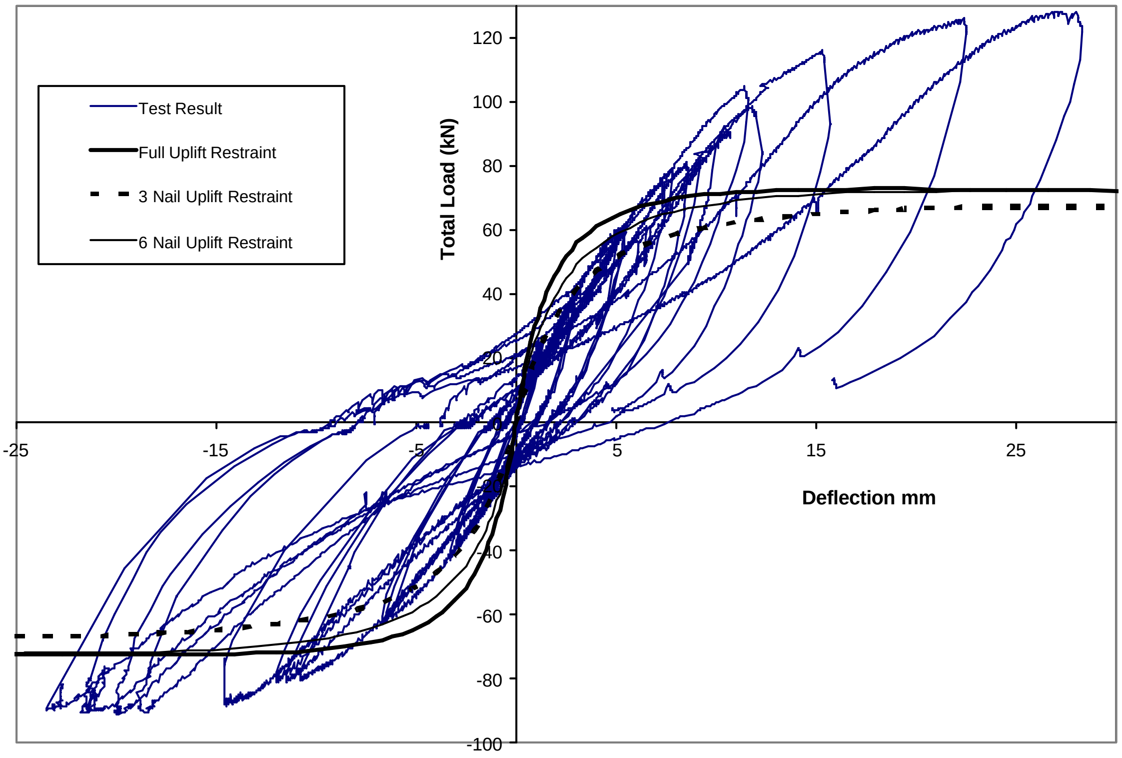


Figure 6. Forces resisted by house versus deflection of Wall3 - Phase I

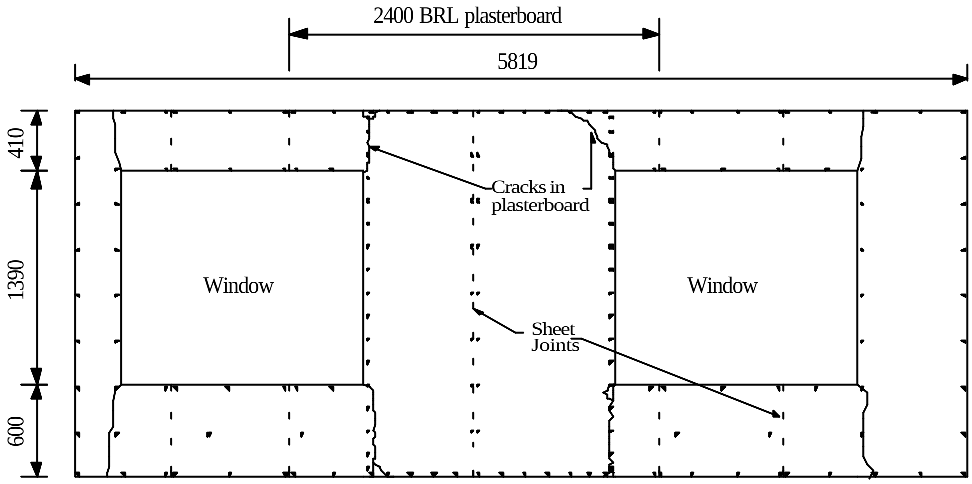


Figure 7. Plasterboard cracks formed in the East Wall (Wall1)

Slippage of the bottom plate of internal Wall2 on the particleboard floor (see Figure 2) was observed at cycling to ±78 kN and this increased at subsequent cycles eventually reaching ±15 mm. Slippage of the bottom plate of Wall7 reaching ±6 mm was also monitored.

Fine plasterboard cracks had formed from wardrobe door corners during cycling to +114, -90 kN and the vertical plasterboard junction of Wall5 with the interior wall had cracked. During the final load cycle, 2 mm uplift occurred at the now lightly cracked interior junction of Wall8. The ceiling/wall junction of Wall8 had also lightly cracked. With the exception of Wall1 and Wall8, no cracks were observed in any plastered joints. Fastener heads fixing the plasterboard to the bottom plate had worked into the plasterboard in all N-S walls, except for the west exterior wall, which only experienced 4 mm deflection and showed little signs of stress. Despite the walls having no hold-down devices, apart from minimal bottom-plate nailing, only Wall8 had uplifted.

House twist was found from east-west deflection measurements relative to the ground of the ends of the east exterior wall. (See Figures 2 and 3(f)). The maximum monitored out-of-plane movement was 2.1 mm at the north end and 0.9 mm at the south end - indicating that house twist was small.

A plot joining the absolute values of successive cyclic peak deflections of each wall is given in Figure 8. It can be seen that as Wall1 weakens it deflects relatively further than other walls at the same peaks. The large differences in house wall deflections illustrated in Figure 7 implies that the ceiling diaphragm action was far from being perfectly rigid. (Full rigidity is often assumed.)

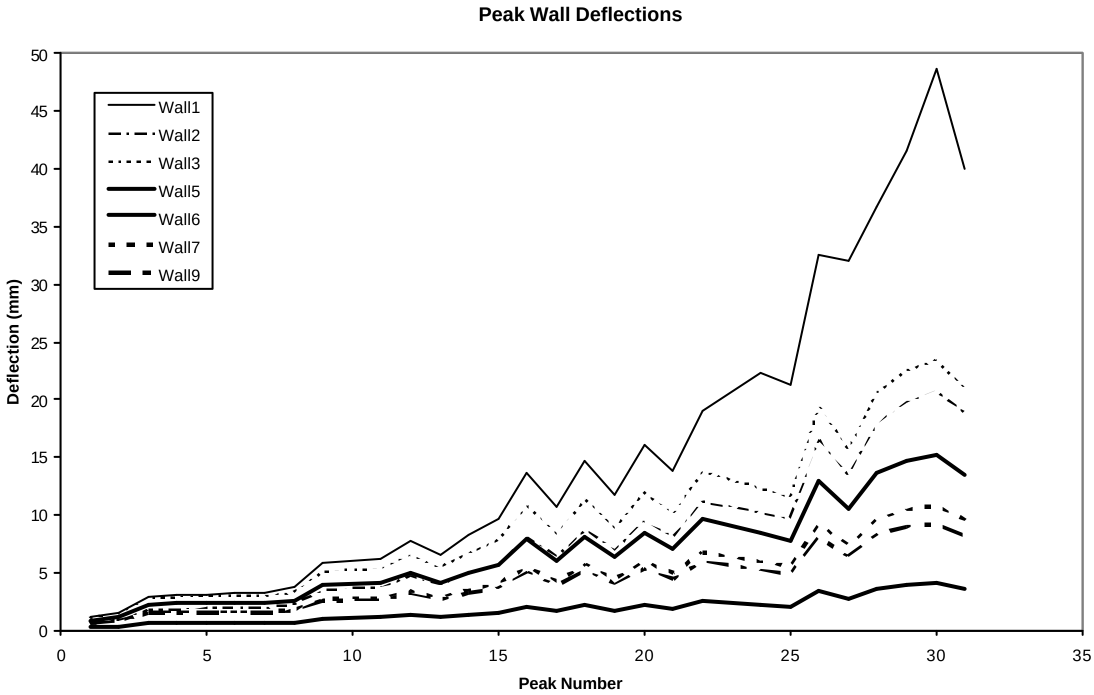


Figure 8. A comparison of the absolute values of peak deflections experienced by each wall

The prediction of total-house total load versus deflection response to monotonic lateral load was obtained by two methods using different assumptions – called Method A and B respectively.

Method A assumed all walls deflected the same amount at any given total-house applied load. The predicted house response is then just the summation of the response curves of each individual wall.

Method B was based on the response curve of Wall3. For any selected deflection of Wall3, Δ_3, the deflection measured at another wall, say Δ_i for $Wall_i$ was found from test measurements. The corresponding predicted force in $Wall_i$ (say f_i) was obtained from using the load/displacement relationships discussed in Section 4. By summation of f_i over all walls the total predicted house lateral force F corresponding to Δ_3 was obtained. This is compared to the measured total load resisted by the house versus the deflection of Wall3 (see Figure 2) in Figure 6.

Figure 6 shows good agreement between the "three-nail uplift restraint" theory and measurement for initial stiffness. However, compared to the uplift full-restraint prediction, actual house strength was more than 75% greater for push loading and 25% greater for pull (average 50%). This is attributed to the plastered lining joints preventing the sheet from moving at the top of bracing walls and at corners and various "systems effects", which is the term for the cumulative influence of moment and shear resisting "effects" which do not readily lend themselves to rational engineering analysis.

The shape of Figure 6 is similar to the P21 test hysteresis loops. Using this similarity, hysteretic models for whole house behaviour can be derived from the predicted whole house backbone curves. The whole house hysteresis model will then enable analysis of the house under seismic attack by inelastic time-history computer analysis in a similar manner as described by Thurston and Park 2002.

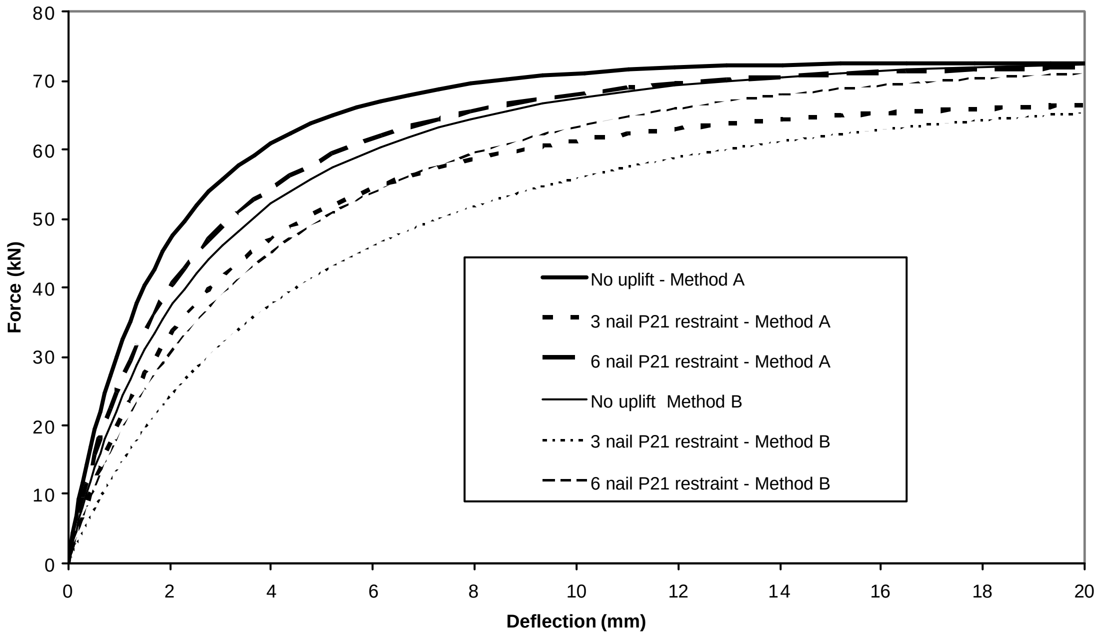


Figure 9. Comparison of backbone curves for predicted house response for two assumptions of wall relative deflections

Table 1. Prediction of house bracing rating from summation of all bracing elements

Wall No.	Wall Length (m)	Bracing kN/m	Bracing kN
1A	0.295	1.60	0.47
1B	1.630	5.75	9.37
1C	0.710	1.60	1.14
2A-East	2.400	5.75	13.80
2-West	2.400	1.38	3.30
3-East	1.670	1.38	2.30
3- West	1.670	1.38	2.30
4-East	1.670	1.38	2.30
4-West	1.575	3.20	5.04
5	1.670	1.60	2.67
6	2.770	3.20	8.86
7-West	1.775	1.60	2.84
7-East	1.775	1.38	2.44
8-West	1.775	1.38	2.44
8-East	1.775	1.38	2.44
9-West	1.775	1.38	2.44
9-East	1.775	1.60	2.84
10	0.520	1.60	0.83
Total Bracing Rating.			67.82

A comparison of the predictions of Method A and B is given in Figure 9. At small deflections the force is significantly higher with Method A but there is little effect at large deflections – i.e., on the ultimate load. This is because when Wall3 has reached 20 mm deflection all walls are still close to their ultimate load using Method B. In fact, some are past their ultimate load and on the declining slope. Hence, the predicted ultimate load of the house is largely independent of the assumed relative deflections of the house walls for any range between Method A and B.

Another way of predicting the total-house bracing rating is to sum the bracing ratings from the published P21 gypsum plasterboard bracing ratings for all (including nominal) walls shown in Table 1. The published ratings used are the wind bracing values, rather than earthquake, as the comparisons are with the backbone curves. The result in Table 1 is approximately 5% less than the ultimate load predicted by the Assumption A and Assumption B methods discussed above. The fastener types, fastener patterns, lining types and elevations drawings of all walls listed in Table 1 are given in Appendix C. The bracing ratings for the BRL and PLB system nailed at 150 mm centres were taken as 5.75 kN and 2.75 kN directly from the manufacture's bracing book. The bracing ratings for nailing at 300 mm centres was taken as half this – i.e., 2.75/2 = 1.375 kN/m. The bracing ratings for screws are known to be slightly greater than for nails and from the P21 bracing test results presented in Appendix A of this report, the PLB system bracing values determined for screws was 3.2 and 1.6 kN/m for 150 and 300 mm centre fixings respectively. Thus, the measured test house strength was more than 55% greater (averaged over the push and pull directions) than predicted if all bracing elements are included and established bracing ratings applied.

5.2 Phase 2 Testing

The cracking shown in Figure 7 is likely have severely weakened Wall1. The system used to load the house applied equal force at the four timber beam locations (i.e., including Wall1) as shown in Figure 2. If greater forces were to be applied to the house and as Wall1 had degenerated, much of the force applied at Wall1 would have been needed to be transferred by diaphragm action to other walls. However, as noted above, diaphragm action was "soft." Hence, to enable the test to proceed Wall1 was relined as discussed below.

The BRL bracing grade gypsum plasterboard for Phase I testing had been cut around the window corners, as shown in Figure 7, and as recommended by the manufacturer. To investigate whether placement of the plasterboard so that sheet joints aligned with window edges would give a more favourable crack pattern, all the plasterboard in Wall1 was replaced, but with the joints so aligned. However, when the total-house applied load reached 60 kN on the subsequent loading, the centre joint in the BRL bracing grade gypsum plasterboard wall between window openings (see Figure 7) failed. It is likely that this was due to poor workmanship in the joint repair and inadequate curing time of the joint. However, loading continued and the recorded house response is shown in Figure 10.

Windows in Wall1 and the glass in the North wall sliding doors cracked. Wall2 bottom plate slipped 20 mm on the particleboard floor and consequently failed by separating from the North wall. The plasterboard on Wall7 cracked at the corners of the door opening and uplifted 16 mm at the door opening and the wall bottom plate slipped 5 mm. (Previously no uplift had been measured here.) Wall9 uplifted 15 mm at the corridor junction and joint cracking occurred along the ceiling/wall joint and also at the wall/hall corridor vertical joint of this wall. Most walls were left on a distinct lean at test completion of Phase II testing.

5.3 Free Vibration Testing

The jack pumping system included a quick release device which rapidly dumped the oil out of the jacks. Without oil the jacks offered low resistance to house movement and thus the house was effectively left to freely vibrate. This testing commenced before Phase I with the house being displaced to 30 kN (approximately 2 mm house movement), the jacks released and the house decay vibration motion was monitored by accelerometers.

The motion of an elastic deforming element will decay due to what is referred to as Coulomb damping which Chopra (2001) considered to be the cumulative effect of friction within the structure and air resistance to motion. It is separate from hysteretic damping, which is the energy absorbed by the inelastic action of structural elements. An inelastic time history analysis of a structure effectively simulates hysteretic damping by the shape of the hysteresis loop used in the analysis. When a house deflects into its non-linear range the damping is a combination of hysteretic damping and Coulomb damping. Thus, it is important that the house not be racked to deflections which would result in a significantly non-linear house response if only Coulomb damping is desired to be measured in a free vibration test.

The inherent critical damping ratio, λ, in a freely vibrating system can be determined from the ratio, R, of the (i)th and (i+j)th peaks of either displacement or acceleration. Chopra (2001) provided equations relating the damping to R.

$$\frac{\lambda}{\sqrt{1-\lambda^2}} = \frac{1}{2\pi j}\ln R. \quad \textit{For small } \lambda \ (< 0.3), \quad \lambda = \frac{1}{2\pi j}\ln R \qquad \ldots\ldots. (1)$$

A typical free vibration decay curve from these tests for an accelerometer at the top of Wall9 is shown in Figure 11. The first full vibration cycle after the quick release was ignored because the jacks were still extruding oil and would, therefore, have added artificial damping to the motion. The average critical damping ratio, λ, determined from the remaining free vibration curves and using Eqn (1) was 8.2%. The house natural frequency was determined from the time between successive peaks in the free vibration plots. The frequency so determined was 20.8 Hz.

6. CONCLUSIONS

The averaged (push + pull) cyclic strength of a plasterboard lined house was 50% greater than that predicted based on summing all the component walls and assuming all the walls are restrained against uplift. Although it is recognised that this is but one example of a typical New Zealand house, it indicates that simple summing of all component bracing walls will give a conservative estimate of total-house strength for single-storey structures. It also indicates that nominal walls should be used when computing house bracing strength.

Only small wall uplift was measured during the testing, despite the walls having minimum bottom-plate nailing. This implies 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 makes recommendations for change to provisions of NZS3604 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.

7. REFERENCES

Chopra Anil K. Dynamics of Structures. 2001. Theory and Application to Earthquake Engineering. Second Edition. Prentice Hall. New Jersey. 844 pages.

Cooney, R.C. and Collins, M.J. 1979 (revised 1982, 1987, 1988, 1990). A wall bracing test and evaluation procedure. BRANZ Technical Paper P21. Judgeford, New Zealand.

Foltz, B. and Filiatrault, A. 2000. A computer program for cyclic analysis of wood shear walls.

Fischer, D.., Filiatrault, A., Folz, B., Uang, C-M. and Seible, F. 2000. Shake Table Tests of a Two-Storey House. Structural Systems Research Report No. SSRP 2000/15, Department of Structural Engineering, University of California, San Diego.

King, A.B. and Lim, K.Y.S. 1991. Supplement to P21: an evaluation method of P21 test results for use with NZS 3604:1990. BRANZ Recommendation No. 10. Judgeford, New Zealand.

Maki, N. and Hayashi H. 2000. Building codes and tradeoffs for earthquake risk reduction:disaster management for housing.12WCEE 2000, Auckland.

McCutcheon, W.J. 1985. Racking deformations in wood shear walls. J. Strc. Eng., Am. Soc. Civil Eng. 111(2):257-269.

Standards New Zealand 1992. General Structural Design and Design Loadings For Buildings. NZS 4203. Wellington, New Zealand.

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

Thurston S.J. 1994. Racking resistance of long sheathed timber famed walls with openings. BRANZ, Study Report 54. Judgeford, New Zealand.

Thurston S.J. and Park S.G., 2002 Seismic Design of New Zealand Houses. Paper submitted to NZNSEE.

Thurston S.J. 2003. Full sized house cyclic racking test. Paper submitted to 2003 Pacific Earthquake Conference, Christchurch.

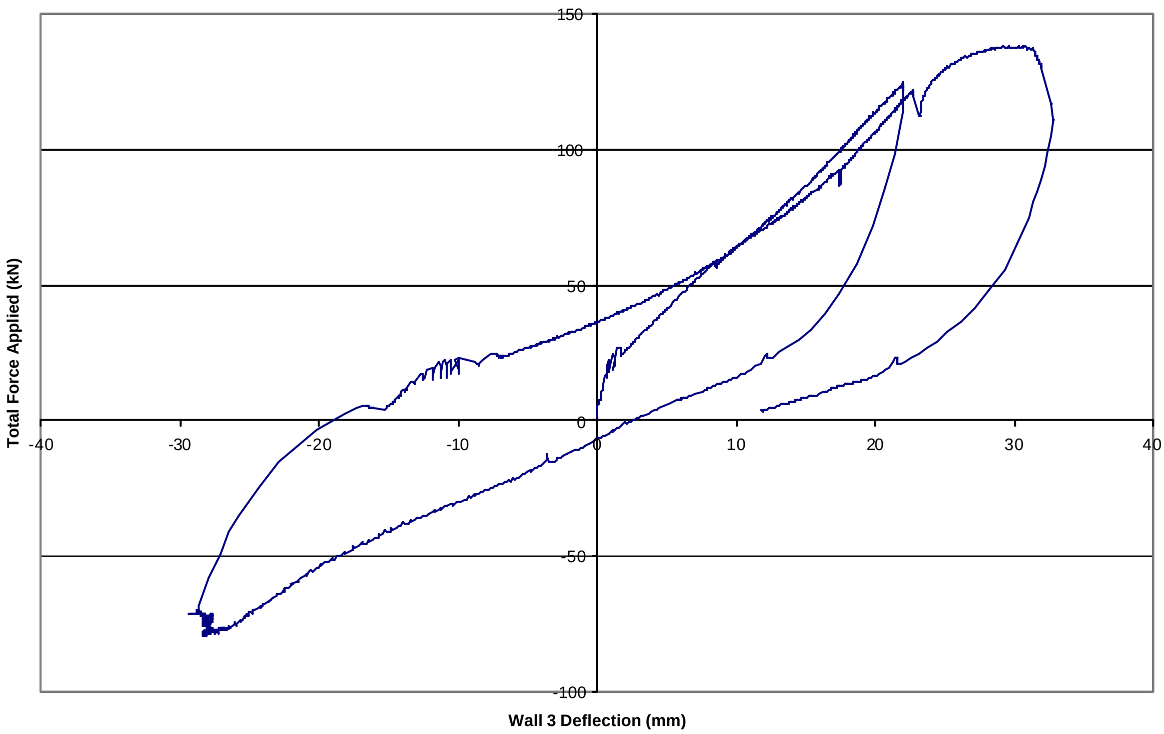


Figure 10. Forces resisted by house versus deflection of Wall3 - Phase II

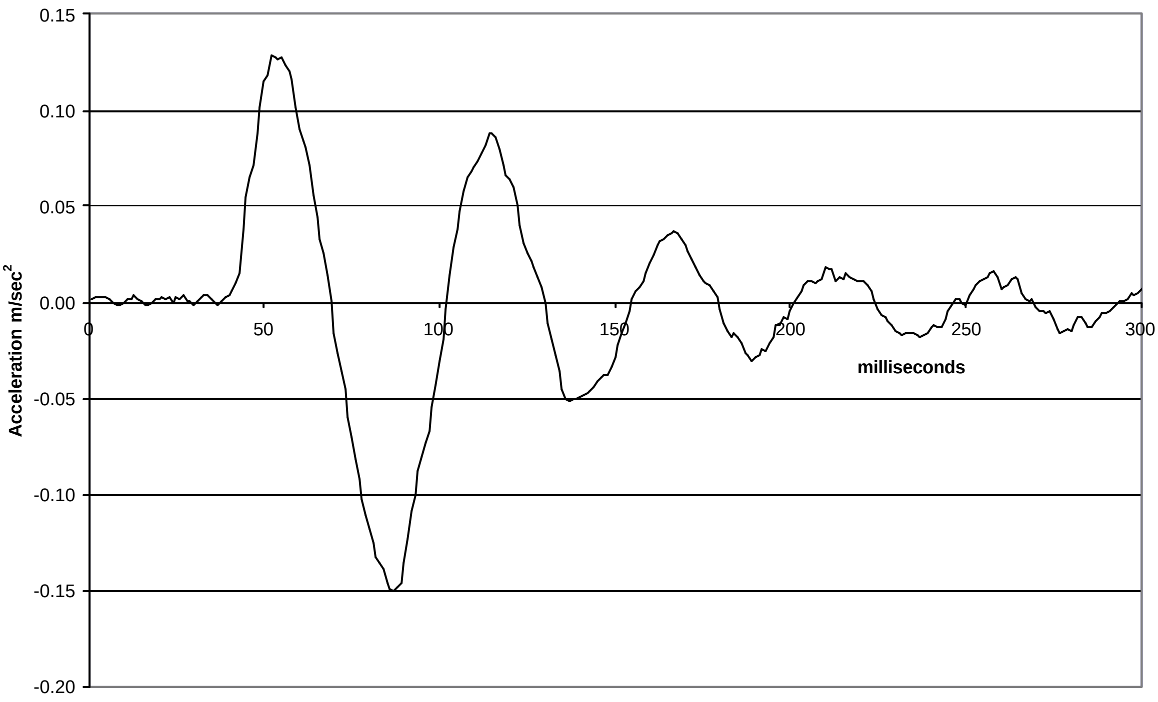


Figure 11. Accelerations measured during a free vibration test

Appendix A: Relationship Between P21 Tests and Predictions from Fastener Slip Data

A.1 Introduction

This appendix has been written as a stand-alone document and the references are listed at the back of the appendix rather than in the main text.

Many researchers have attempted to relate the load-versus-deflection behaviour of bracing panels to the material properties. Walker[1], Kuenzi[2] and Burgess[3] derived formulae assuming a linear relationship between fastener load and slip between panel and framing and that the panel was in uniform shear.

Patton-Mallory and McCutcheon[5] obtained excellent agreement between the theory derived by McCutcheon[6] and test data reported by Patton-Mallory et al.[7] for isolated walls with total uplift restraint. Thurston[8] also found the theory gave good agreement with results from racking tests.

Patton-Mallory and McCutcheon[5] found that Eqn. A.1 for nail load slip relationship and Eqn. A.2 best predicted the wall load (R) deflection (Δ_f) relationship. The formulae are only valid where there is no separation between the framing joints and where there is no sheathing buckling or rupture. The deflection, Δ_f, is the wall horizontal deflection due to fastener slip alone. The additional deflection, Δ_s, due to panel shear distortion (given by equation A.3) usually accounts for about 5%-10% of the total deflection. The constant C was not used by Patton-Mallory and McCutcheon[5], but it has been inserted in the equations below because Thurston[8] found that changing the value of C can provide a better fit to experimental data.

$$p = \frac{A\,d_n}{B + d_n{}^C} \qquad \text{....................... (A.1)}$$

where:

p = nail load (kN) at slip δ_n (mm)

A, B and C are constants

$$R = A\sum \left(\frac{K^2 \Delta_f}{B + (K\Delta_f)^C} \right) \qquad \text{................ (A.2)}$$

where $K = \sin a \sqrt{\left(\frac{x}{L}\cos a\right)^2 + \left(\frac{y}{H}\sin a\right)^2}$

(nail coordinates x,y and panel geometry H, L and α are defined in Figure A.1).

$$\Delta_s = \frac{RH}{GtL} \qquad \text{.................(A.3)}$$

where G = sheathing shear modulus

t = sheathing thickness

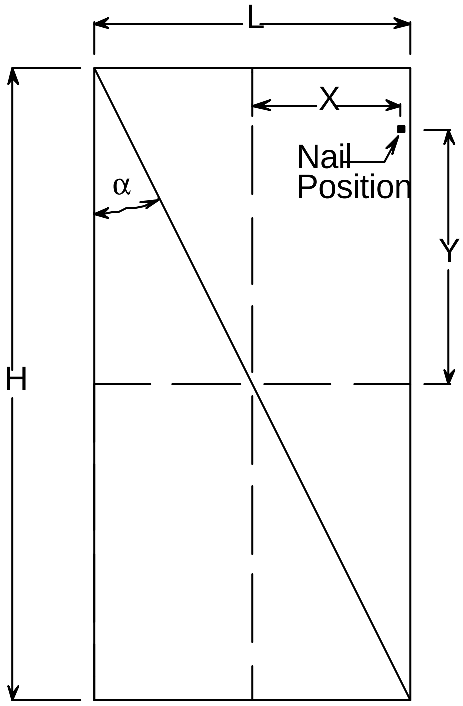


Figure A.1. Panel geometry used to describe fastener placement

The nail load-slip test specimens used by Patton-Mallory and McCutcheon[5] had two layers of Teflon in the slip plane between the framing timber and the plywood because it was stated that standard fastener tests over-estimated the initial stiffness of the fasteners when used in walls. Presumably, this was because the friction between the sheet and frame reduced as the timber shrank and thus the normal ‘clamping’ force reduced. This would imply that the Teflon should not be used in the slip tests for wall load prediction if the wall timber is dry and the testing is begun shortly after wall construction. It would also be expected that over a period of time nail "corrosion" would tend to increase the nail ‘grip’ in the timber, due to a nail surface/timber fibre binding action. Gypsum plasterboard has a paper facing which would reduce friction between sheathing and framing, so the use of Teflon in the slip tests is considered unnecessary.

Various researchers have used finite element computer models incorporating nail (linear and non-linear) slip elements to predict wall racking behaviour. One of the first finite element models was developed by Foschi[9]. The timber frame was modelled with beam elements, plane stress finite elements were used for the sheathing, and non-linear springs were used for the fasteners. To reduce the computational effort, Itani and Cheung[10] altered this model by modelling the connectors as a group. This was somewhat improved by Falk and Itani[11]. Two other effects ignored by the above researchers were included by Dolan[12]. His model allowed slip between framing member connections and also considered the effects of plywood panels touching each other. The influence of these additional effects was found to be small for typical shear walls. Easley and Dodds[13] obtained excellent agreement between results of plywood test walls, using a finite element program POLY-FINITE and a proposed formula. This formula gives similar results to that in Eqns. 1-3 above but is expected to be slightly less accurate as it assumes nail forces between sheathing and studs are parallel to the studs.

Gupta and Kuo[14,15] developed a more general (and complex) analysis method than Eqns 1-3 which was also based on an energy concept and also included the effects of stud and top plate bending but which did not assume a predetermined nail deformation pattern. The method could also incorporate a nonuniform nail load-slip relationship. They obtained a good agreement with the Easley and Dodds[13] results and showed that the effects of stud bending were small. Kuo and Gupta[16] used the same method to compare their theoretical model and experimental full house behaviour reported by others. They found that a good agreement could only be obtained if their model could take account of uplift of studs and bottom plate. Yoon and Gupta[17] developed similar equations based on a static equilibrium analysis and extended this to include panel uplift. A computer program N-HOUSE was developed to analyse three-dimensional buildings which was shown to give a good agreement with published data for a number of tests.

The most widely known model is the CASHEW computer program developed by Foltz and Filiatrault[18] as part of the CUREe project. The sheathing is assumed to rotate around its centroid and the framing members distort into a parallelogram with top and bottom plate remaining horizontal – i.e., rocking of panel is prevented. The program predicts the complete cyclic hysteresis loops for a given wall deflection regime as well as the monotonic loading curve. The authors found good agreement between the CASHEW predictions and experimental results.

A.2 Fastener Slip Tests

Fastener slip tests were performed using the same type of fasteners used to connect the wall lining to the framing in the test house as shown in Figure A.2. The load rate was 10 seconds per cycle. Fastener slip tests were done using nail fasteners with washers into BRL bracing-grade gypsum plasterboard and nailed and screwed fasteners into PLB gypsum plasterboard. Six tests were done for each fastener type and for the loading direction both parallel to the papered edge and parallel to the non-papered edge. Note that BRL bracing grade gypsum plasterboard contains glass fibre in the plasterboard matrix, whereas PLB gypsum plasterboard does not.

A typical fastener slip plot is shown in Figure A.3. Peak points for the first and third cycles were extracted from the plot (shown with "+" symbols in Figure A.3). Regressional analysis was used to determine the coefficients, A, B and C to give best fit to Eqn (A.1) using the peaks. Results are given in Table A.1. In addition, the CASHEW parameters were also determined from the fastener slip results. This data are shown in Table A.2.

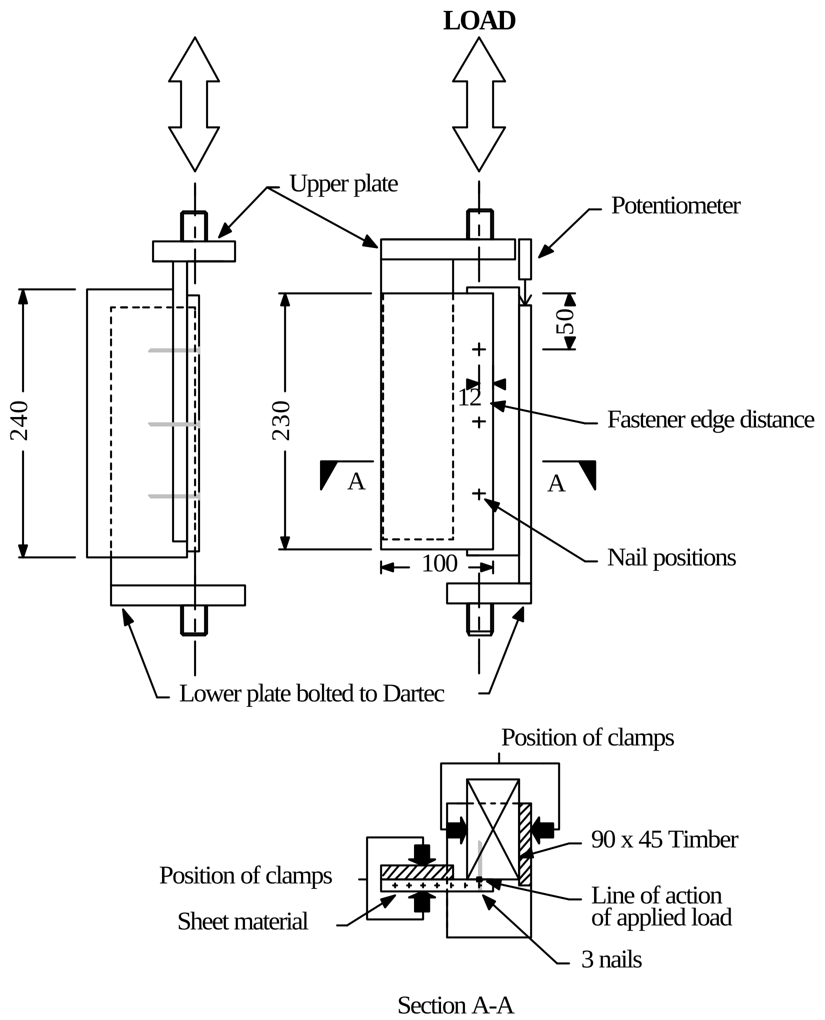


Figure A.2. Test setup for fastener slip test

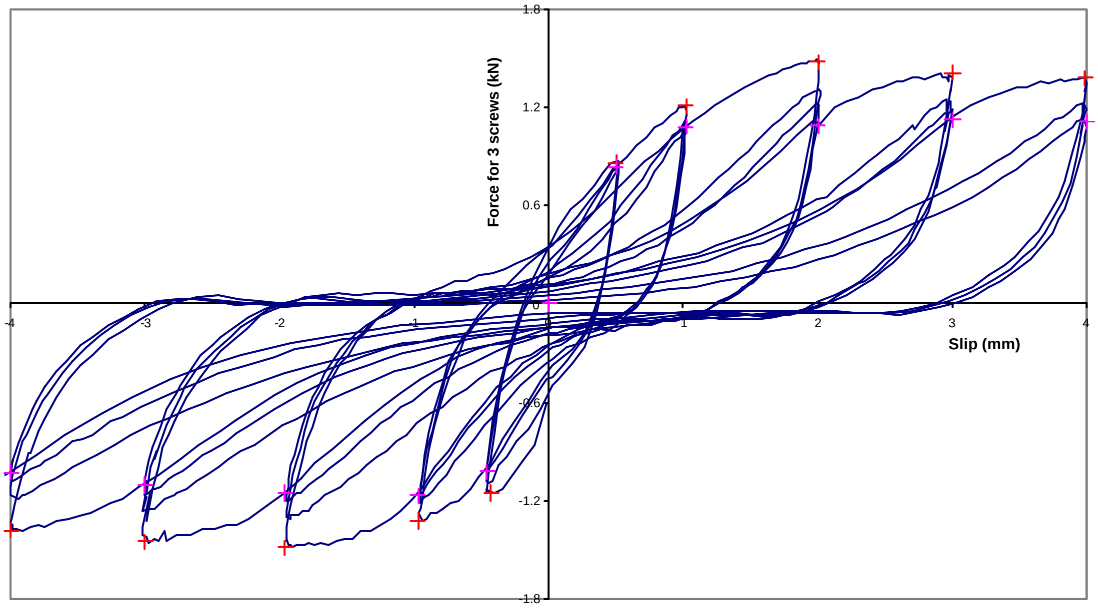


Figure A.3 Typical fastener slip test result

Table A.1. Fastener A, B and C parameters to fit Eqn A.1

		With paper edge		Without paper edge	
		1st Cycle	3rd Cycle	1st Cycle	3rd Cycle
BRL bracing grade gypsum plasterboard Nails with Washers	A	0.803	0.676	1.123	0.968
	B	0.38	0.349	1.026	0.968
	C	1	1.108	1.173	1.345
PLB gypsum plasterboard Nails without Washers	A	0.488	0.467	0.370	0.355
	B	0.265	0.349	0.305	0.371
	C	1.016	1.108	1.020	1.115
PLB gypsum plasterboard Screws	A	0.738	0.578	0.448	0.381
	B	0.638	0.532	0.380	0.407
	C	1.275	1.371	1.149	1.225

Notes to Table A.1

1. All the plasterboard used was donated and manufactured by Winstone Wallboards.
2. Nails were 35 x 2.5 mm galvanised clouts.
3. Washers were 15 mm diameter.
4. Screws were type Atlas 6 x 25 Gypsum screws (tapered, zinc chromate coated with 8mm diameter head).
5. Equations apply to a maximum slip of 2 mm for the screws, 3 mm for the nails into PLB gypsum plasterboard without washers and 4mm for nails with washers into BRL bracing grade gypsum plasterboard.

Table A.2. Fastener CASHEW Parameters

	F0	FI	DU	S0	R1	R2	R3	R4	Alpha	Beta
BRL bracing grade gypsum plasterboard Nail with washer	0.500	0.082	4.0	1.22	0.055	–0.02	1.5	0.1	0.8	1.1
PLB gypsum plasterboard, Nail only	0.360	0.04	3.0	0.80	0.028	–0.02	1.5	0.1	0.8	1.1
PLB gypsum plasterboard, Screw	0.260	0.033	2.0	1.05	0.064	–0.02	1.5	0.1	0.8	1.1

A.3. Prediction of Racking Backbone Curve

The backbone curve of cyclic tests tends to be very close to the monotonic curve, provided that the test regime is designed so that when cycling to one limit ($?_i$) is complete the next cyclic limit ($?_j$) is selected so that $?_j - ?_i > ?_{step}$ where $?_{step}$ varies depending on lining, configuration etc, but the writer has found it to be approximately 5 mm for plasterboard.

The backbone curve for 1.2 m and 2.4 m wide bracing walls using BRL bracing grade gypsum plasterboard (nailed with washers) and PLB gypsum plasterboard (both nailed and screwed) was calculated using both the McCutcheon[6] theory and the CASHEW computer program. Because both methods assume the sheet is in uniform shear, only fasteners around the perimeter of the sheet were included in the analysis. (Unreported tests at BRANZ found that omitting fasteners from the body of the sheet had little effect on the strength of the bracing wall.) The fastener pattern was a single nail in each corner and fasteners at 150 mm centres around the rest of the bracing panel perimeter.

The analysis for the McCutcheon[6] theory was programmed by the writer into a Visual Basic computer program called WallRack.vbp. The input data, which included the locations of all fasteners, was generated in an Excel spreadsheet which was saved as a .txt file. The output data was a force-versus-deflection relationship due to fastener slip alone. The shear deformation deflection of the panel assuming the modulus of rigidity of the panel was 1.5 GPa and the rocking deflection expected for both “three-nail” and “six-nail” end restraints. The method of calculating the rocking deflection is detailed in Appendix B.

Problems were encountered using the CASHEW computer program. The program often terminated execution without identifying a reason. Communication with the writers of the software revealed that it was written for plywood-sheathed walls and could not handle the stiffer and less-ductile plasterboard parameters. Hence, the fastener initial stiffness parameter, S0, was decreased in value. Some runs were successful and the full hysteretic response of the shear wall was obtained. The agreement with P21 test results in these instances was good. However, only the monotonic prediction from CASHEW is compared with test results in this Appendix. A successful run could not be obtained for the 2.4 m wide screwed panel even when the dataset used was the successful 2.4 wide nailed panel with the nailing parameters replaced by the successful 1.2 m wide screwed panel. It was concluded that CASHEW is currently not suitable to be used for plasterboard walls. However, the good agreement between CASHEW, McCutcheon[6] theory and tests gave confidence in continuing to use the McCutcheon theory.

```
'Data now read in. Start doing the sums.
Def = 0#
DefInc = 0.05
Do Until Def > 40#
    Force = 0#
    Def = Def + DefInc
    DefInc = DefInc * 1.125
    ' Determine force in wall to generate this deflection.
    For I = 1 To NoLinings
        For J = 1 To NoFastenersTypes(I)
            For L = 1 To NoOfFasteners(I, J)
                Force = Force + A(I, J) * K(I, J, L) ^ 2 * Def / (B(I, J) + (K(I, J, L) * Def) ^ C(I, J))
            Next L
        Next J
    Next I

    ' Add on deflection due to rocking motion
    For M = 1 To 35
        If Force < F3(M) Then
            Def3 = D3(M - 1) + (D3(M) - D3(M - 1)) * (Force - F3(M - 1)) / (F3(M) - F3(M - 1))
            GoTo P21_6Nail
        End If
    Next M
    Def3 = 100#
P21_6Nail:
```

Explanation of Program WallRack

A, B and C are arrays of the fastener data from Table A.1.

K is the K parameter for each fastener from Eqn A.2 and was calculated from the geometry as each fastener location was read in from:

```
    ' Calculate the coefficient K for this X and Y geometry
    K(I, J, KT) = Sin2 * Sqr((X(I, J, KT) * Cos1) ^ 2 + (Y(I, J, KT) * Sin1) ^ 2)
```

The deflection versus force relationship for panel rocking motion was determined as described in Appendix B and stored in D3 and F3 arrays for the “three nail” restraint. The coding above is just a linear interpolation for the rocking deflection at the panel force at that step.

Figure A.4 Extract from program WallRack

A.4 Comparison Theory and Test

Figures A.5-A.6 present a comparison between the McCutcheon[6] theory and the CASHEW computer program and test results on BRL bracing grade gypsum plasterboard walls with the same fastener pattern as analysed. Figures A.7-A.10 are similar graphs but for PLB gypsum plasterboard walls. The 2.4 m long walls in these tests had fully plastered joints between sheets and this joint remained intact for the entire test and was considered to be infinitely strong in the theoretical analyses. Figure A.11 presents similar graphs but for a 2.4 m long wall without plastered joints and was considered to be two separate sheets for the analyses. The theoretical curves for Figures A.5-A.8 include the predicted P21 uplift induced deflection calculated as described in Appendix B. Uplift was effectively prevented in the tests in Figures A.9-A.11 by nailing the P21 restraint with 12 nails and thus the theoretical curves are for full uplift restraint.

Generally there is a good agreement between the initial stiffness and measured peaks with the theoretical curves. Where S0 needed to be reduced to make the CASHEW program work (Figure A.7-A.9) the CASHEW initial stiffness prediction of wall behaviour is on the low side.

At larger deflections the test peaks often deviated from the predictions. This is attributed to fasteners breaking out of the sheet bottom plate perpendicular to the sheet edge due to uplift forces being transferred from the sheet to the bottom plate. If the stud had been prevented from uplifting then the theory suggests that the sheet is in pure shear and this force transfer would not occur. Thus, with stud uplift prevented, breakout is expected to be less significant and the agreement between theory and test at the larger deflections is expected to be better.

In summary, agreement between theoretical predictions of the backbone curve based on fastener load/slip data and racking test results was good.

A.5 Conclusions

This Appendix presents a method of deriving the backbone curve for an uplift-restrained racking test using a modified form of the McCutcheon[6] theory. A computer program for performing this analysis called WallRack is described. The data required for this analysis is the panel fastener slip data and the Modulus of Rigidity of the panel. The latter is used to derive the deflection due to shear deformation of the panel and is of lesser importance because the deflection due to shear deformation of the panel is generally small (less than 1 mm). The method allows inclusion of total body rotation due to the panel being allowed to move vertically at the ends. Formulae are presented in Appendix B for calculating the resulting rocking component of horizontal deflection for "three-nail" and "zix-nail" P21 end restraints.

Good agreement was obtained between racking tests (some using P21 end restraints) and predictions from the modified McCutcheon[6] theory and also the well-known CASHEW[18] computer program provided only the fasteners on the perimeter of the panels was included in the analysis. Limitations on using CASHEW for plasterboard panels were described.

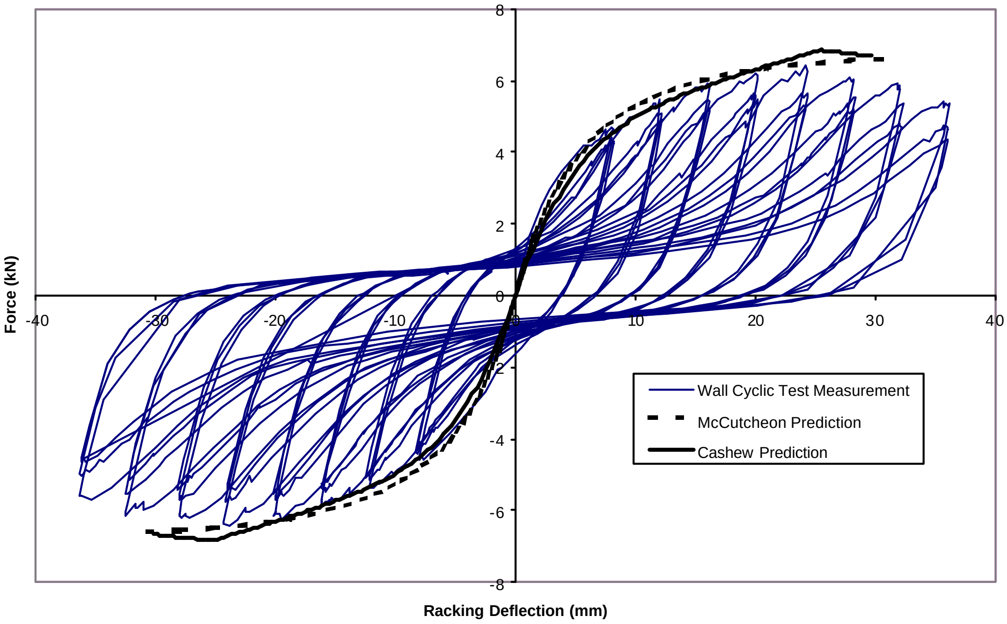


Figure A.5 Comparison of P21 test and theory for a 1.2 m long BRL bracing grade gypsum plasterboard wall

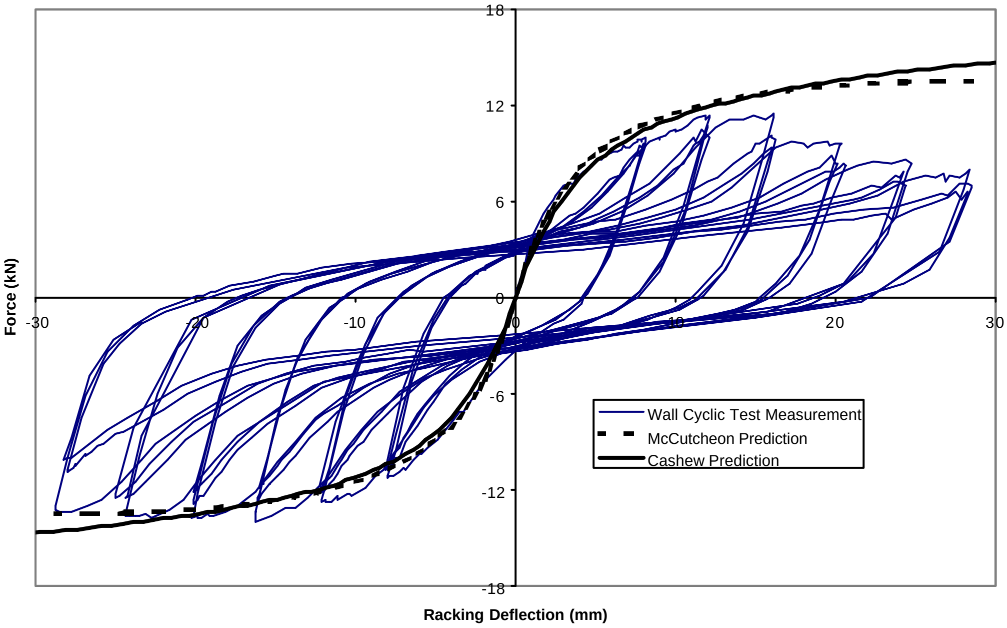


Figure A.6 Comparison of P21 test and theory for a 2.4 m long BRL bracing grade gypsum plasterboard wall

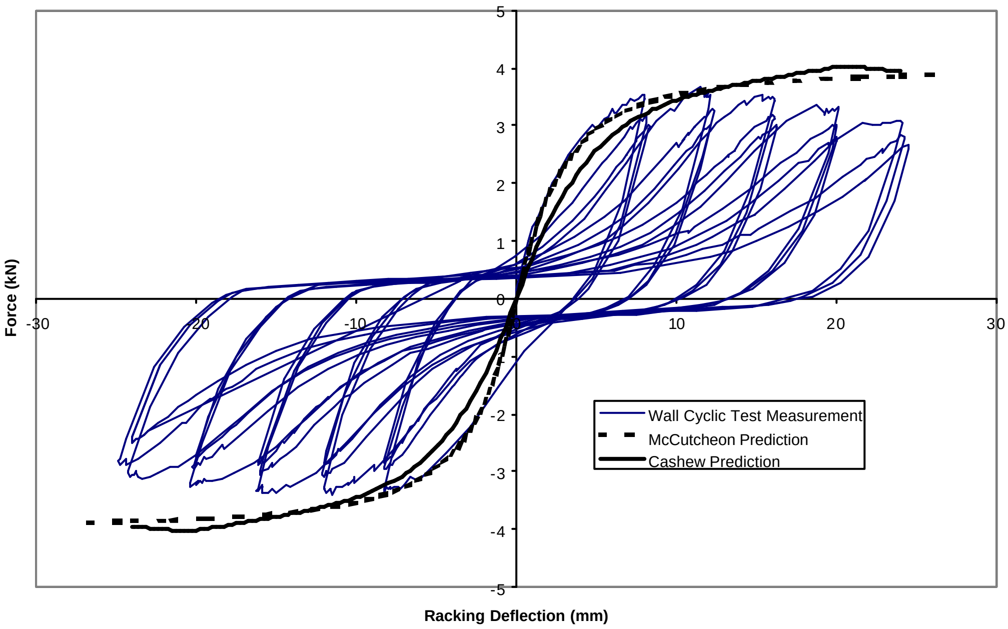


Figure A.7 Comparison of P21 test and theory for a 1.2 m long nailed wall

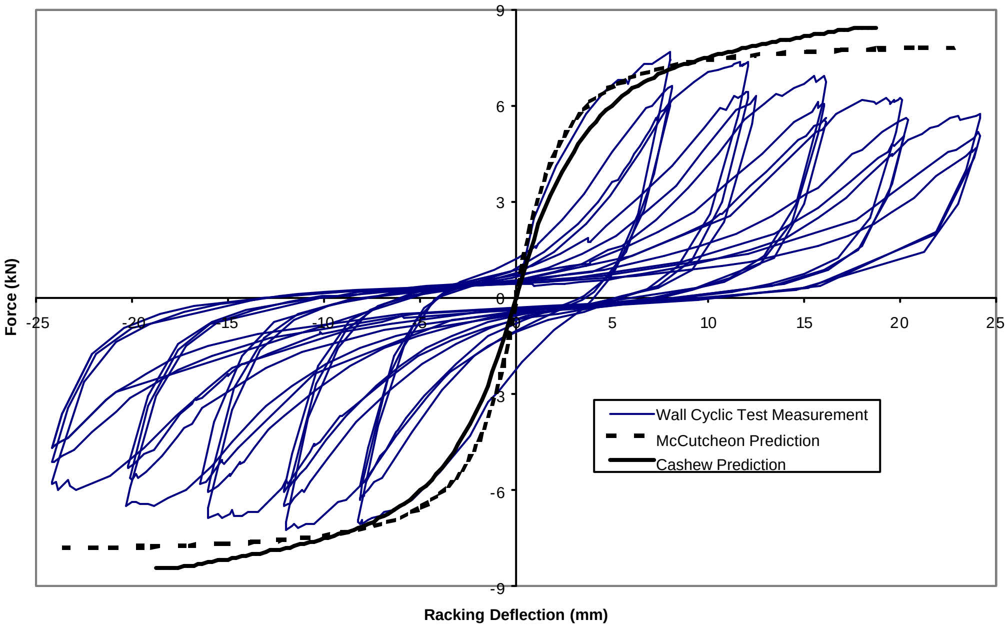


Figure A.8 Comparison of P21 test and theory for a 2.4 m long nailed wall

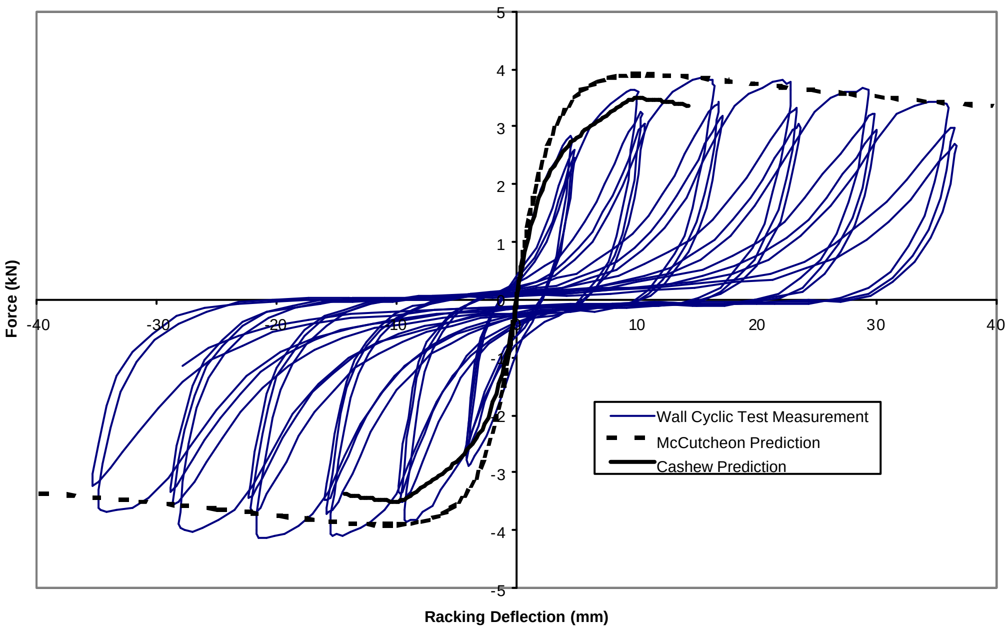


Figure A.9 Comparison of P21 test and theory for a 1.2 m long screwed wall

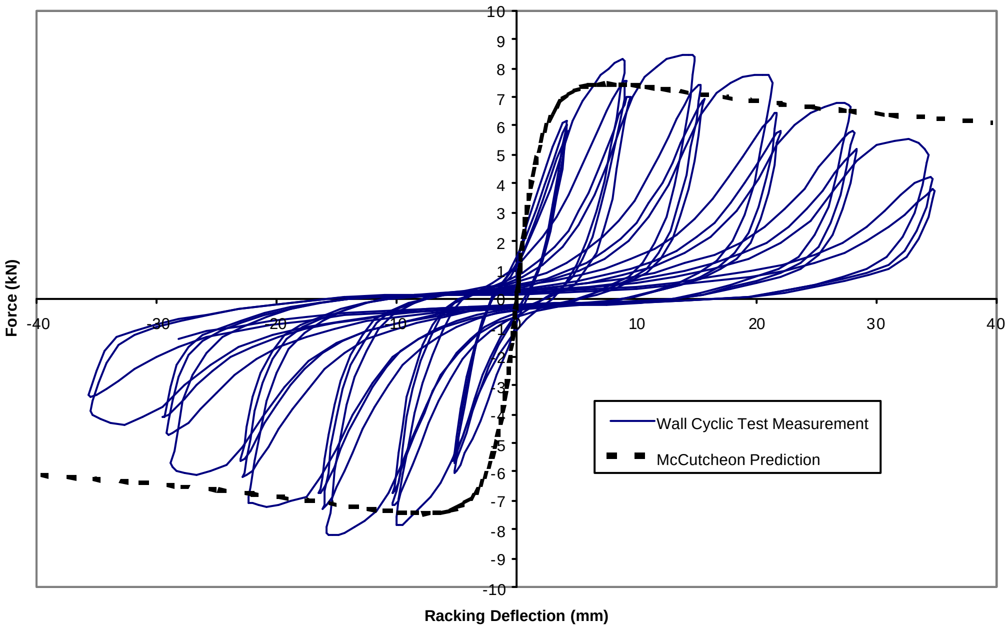


Figure A.10 Comparison of P21 test and theory for a 2.4 m long screwed wall

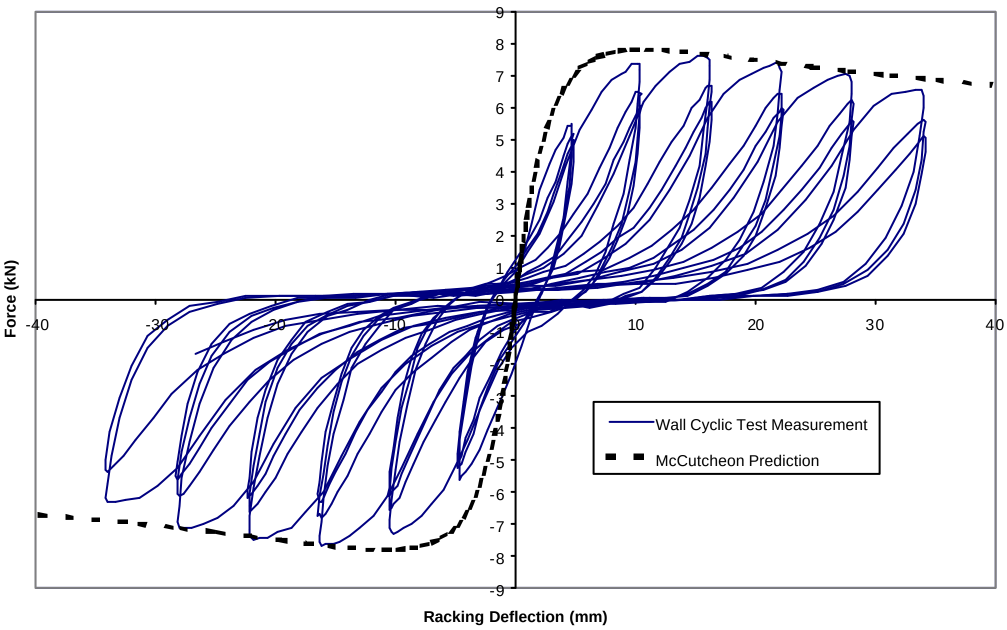


Figure A.11 Comparison of P21 test and theory for a 2.42 m long screwed but not plastered wall

A.6 References

1. Walker, G. 1979 G. R. Racking strength of sheet clad wall panels. Proceedings, Diamond Jubilee Conference, Institute of Engineers, Australia, Perth, April 1979.

2. Kuenzi, E.W. (-). Some theoretical considerations relating to design criteria for racking resistance of walls. U.S. Forest Products Laboratory (unpublished).

3. Burgess, J.H. 1976a. Derivation of the wall racking formulae in TRADA'a design guide for timber frame housing. Timber Research and Development Association, England.

4. Burgess, J.H. 1976b. Comparison of the TRADA wall racking formulae with those of Kuenzi . Timber Research and Development Association, England.

5. Patton-Mallory, M. and McCutcheon, W.J. 1987. Predicting racking performance of walls sheathed on both sides. Forest Products Journal, Vol 37 27-32, No 9, Sept 1987.

6. McCutcheon, W.J. 1985. Racking deformations in wood shear walls. J. Strc. Eng., Am. Soc. Civil Eng. 111(2):257-269.

7. Patton-Mallory, M., Gutkowski, R.M. and Soltis. 1984. Racking Performance of Light-Frame Walls Sheathed on Two Sides. US Department of Agriculture Forest Service, Forest Products Laboratory, Sept 1984.

8. Thurston S.J. 1993. Racking resistance of long sheathed timber framed walls with openings. Building Research Association of New Zealand. Study Report SR54., Judgeford, Wellington.

9. Foschi, R.O. 1977. Analysis of wood diaphragms and trusses. Part 1: Diaphragms. Canadian Journal of Civil Engineering. Vol 4, No 3, pp 345-352, Sept 1977.

10. Itani, R.Y. and Cheung, C.K. 1984. Non-linear analysis of sheathed wood diaphragms. ASCE Journal of the Structural Division, Vol 110, No 2, pp. 2137-2147.

11. Falk, R.H. and Itani, R.Y. 1989. Finite element modelling of wood diaphragms. ASCE Journal of the Structural Division, Vol 115, No 3, pp. 543-559.

12. Dolan, J.D. 1989. The dynamic response of timber shear walls. Ph.D. Thesis, The University of British Colombia, Vancouver, Canada.

13. Easley, J.T. and Dodds, R.H. 1982. Formulas for wood shear walls. ASCE, Journal of the Structural Division, Vol 108, No. ST11, Nov 1982.

14. Gupta, A.K. and Kuo, G.P. 1985. Behaviour of wood-framed shear walls. J. Struct. Eng., Vol 111, No 8, ASCE Pap 19946.

15. Gupta, A.K. and Kuo, G.P. (1987) Wood-framed shear walls with uplifting, Journal of Structural Engineering, 1113(2): 241-259.

16. Kuo, G.P. and Gupta, A.K. 1989. A three dimensional macro-element model for wood-framed buildings. Dept. of Civil Eng. North Carolina State University, Raleigh, North Carolina, USA.

17. Yoon, T. and Gupta, A.K. 1991 Behaviour and failure modes of low rise wood-framed buildings subjected to seismic and wind forces. Dept. of Civil Eng. North Carolina State University, Raleigh, North Carolina, USA.

18. Foltz, B and Filiatrault, A. 2000. A computer program for cyclic analysis of wood shear walls.

Appendix B: Deflection due to rocking in P21 racking tests

B.1 Prediction from historic P21 tests

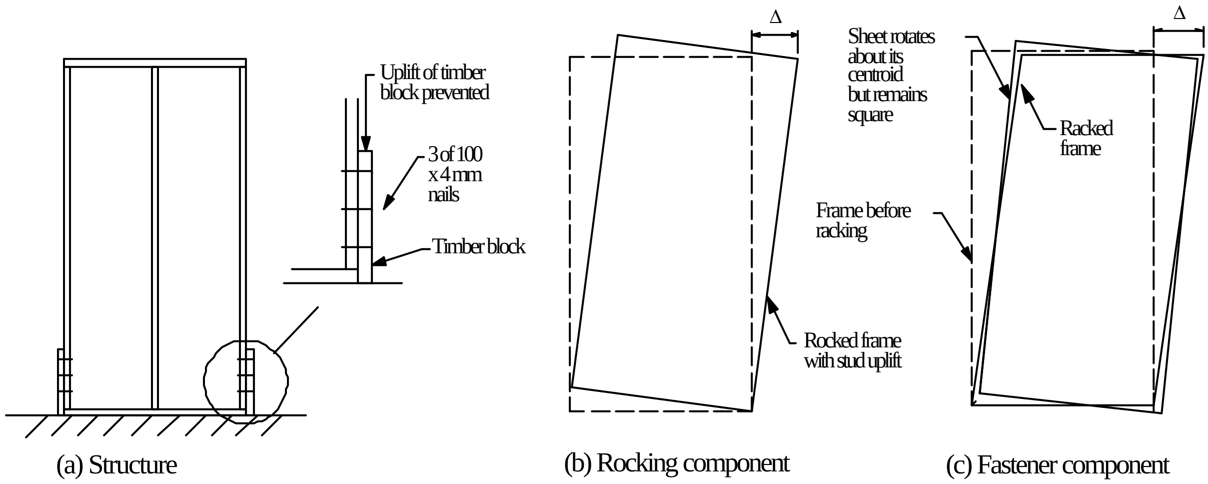


Figure B.1. Components of P21 test racking deflection

The racking deflection of a bracing wall is largely the sum of three components (McCutcheon 1985); namely the deflection due to fastener slip alone (referred to as the Fastener Component), the shear deformation of the panel and the rocking deflection due to panel vertical movement at the panel ends (referred to as the Rocking Component). The Rocking and Fastener Components are sketched in Figure B.1. The deflection due to the shear deformation of the panel is relatively small compared to the Fastener Component. Appendix A provides methodology for calculating the deflection of a bracing wall due to fastener slip and due to shear deformation of the panel. This appendix focuses on determining the relationship between the racking force and rocking component of deflection in a P21 test which will then allow the total wall response from a P21 test to be determined.

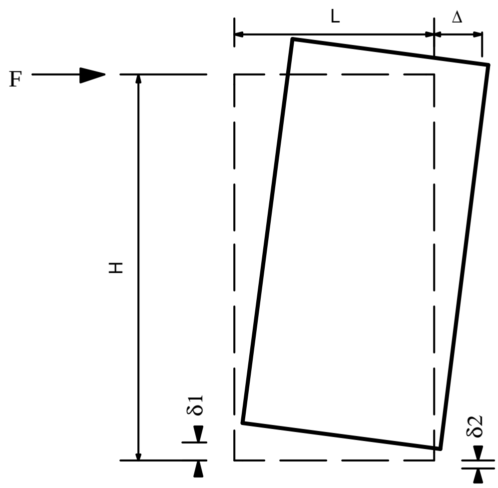


Figure B.2 Nomenclature for rocking component

Using the nomenclature of Figure B.2 the predicted Rocking Component of deflection, Δ, can be calculated as:

$$\Delta = H/L \,.\, (d1 + d2) \qquad \ldots\ldots\ (B.1)$$

Note that the panel not only lifts at the tension stud but sinks at the compression stud due to embedment of the stud into the bottom plate.

The applied moment on the panel is the racking force, F, factored by the panel height H. This is resisted by:

(a) force in end uplift restraint, F_u, multiplied by the distance from uplift restraint to the centre of compression zone – taken as L – i.e. = F_u . L.
(b) total vertical load x distance from the centre of vertical load to the centre of compression zone. i.e., if the vertical load is a UDL of magnitude w then the moment = $wL^2/2$.
(c) The weaker of the fixing of bottom plate to foundation and the fixing of sheet to bottom plate. This is taken to = $fL^2/2$ where f is the force per unit length.

Thus: $$F.H = F_u \,.\, L + (f+w)L^2/2 \qquad \ldots\ldots\ (B.2)$$

As stated above, the purpose of this appendix is to determine the relationship between the racking force and Rocking Component of deflection in a P21 test – i.e., a relationship between F and Δ. From equations (B.1) and (B.2) it can be seen that for a given wall height H and "three-nail" end restraint detail as per Figure B.1, a separate relationship needs to be determined for each wall length and sheathing material as sheathing material may affect both f and w. Many short bracing walls in New Zealand use steel end straps with 6 nails to both studs and foundation beam to connect studs to foundation beam which will provide additional rocking restraint. Walls with these straps are also considered below.

BRANZ performs many racking tests each year where the relationship between racking force F and end stud vertical movement ($\delta 1$ and $\delta 2$ of Eqn B.1) is generated as a by-product of the test. The bottom plate of these walls is nailed to the foundation beam with pairs of 100 x 4 mm flat head nails at 600 mm centres. It was found that a good fit to experimental data was given by Equation (B.3).

$$F = \frac{A.\Delta}{B + \Delta^C} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ (B.3)$$

where:

F = wall racking load (kN) at rocking component deflection Δ (mm) calculated from Eqn (B.1).
A, B and C are constants determined by regressional analysis.

By examining many test results Thurston (1994) found the constants A, B and C of Eqn. (B.3) for the various wall lengths and end conditions. Thurston presents plots which showed good agreement between the measured rocking component deflection from many tests and the best fit relationship of Eqn. (B.3). He found the dependence on wall lining for the plasterboard, fibre cement and plywood walls in the relationship of Eqn (B.3) was small. Thurston's results were checked using a different set of test data for this project for all except the 3.0 m long walls and were found to only need modification (and this was relatively small) for a 2.4 m long wall without end straps and a 1.2 m long wall with end straps. Plots comparing test and best fit curves for these two walls are given in Figures B.3 and B.4. These new equations and the remaining equations from Thurston (1994) are given as Eqns B.4-B.9. The equations determined are considered applicable to all three generic linings and are given for walls both with and without end straps for completeness. Only the results for walls without end straps are applicable to this report as the test house walls did not use end straps. It should also be noted that the bottom plate of the house walls was not nailed as securely as the P21 test specimens (see Section 2).

1.2 m long wall (no end straps): $F = (5.6 \times \Delta)/(3.0 + \Delta^{0.95})$ ..... (B.4)

1.8 m long wall (no end straps): $F = (8.7 \times \Delta)/(1.3 + \Delta^{0.95})$ ..... (B.5)

2.4 m long wall (no end straps): $F = (14 \times \Delta)/(1.2 + \Delta^{0.95})$ .... (B.6)

3.0 m long wall (no end straps): $F = (20.0 \times \Delta)/(0.4 + \Delta^{0.95})$ .... (B.7)

1.2 m long wall (end straps): $F = (1 \times \Delta)/(4.0 + \Delta^{1.05})$ .... (B.8)

0.9 m long wall (end straps): $P_W = (7.1 \times \Delta)/(6.6 + \Delta^{0.95})$ .... (B.9)

Figure B.5 compares the "normalised" form of Eqns. B.4-B.7 (walls without end straps). The equations were normalised by dividing the bracing force, F, by the wall length and multiplying the rocking component deflection, ? , by the wall length. By examining Eqns. B.1-B.2 with "w" and "f" set equal to zero it may be deduced that the normalised curves would be expected to coincide for all wall lengths. The normalised curves for longer walls do, in fact, lie above those for shorter walls, as expected from Eqn. B.2, as "w" and "f" are not zero in practice. However, the close agreement between the curves does give some confidence in the derived equations.

This project is concerned with comparing the measured strength of the test house with the strength from summing the predicted P21 test strength of all walls aligned in the direction of the applied force. To determine the predicted strength the rocking component of the deflection needs to be calculated for all lengths of wall used in the test house. For walls of lengths between 1.2 m and 3 m the Rocking Component deflection was interpolated from the Eqns. B.4-B.9. The rocking component deflection for walls of length L (where L is less than 1.2 m), was based on the "normalised" form of Eqn. B.4 using the following procedure:

- The racking force deflection F was calculated for each racking wall deflection ignoring rocking as described in Appendix A.
- F was factored by L/1.2 and this force was considered applicable to a rocking component deflection, $?_L$ = to the ? used in Eqn. B.4 factored by 1.2/L.

The results of this process are illustrated in Figure B.6.

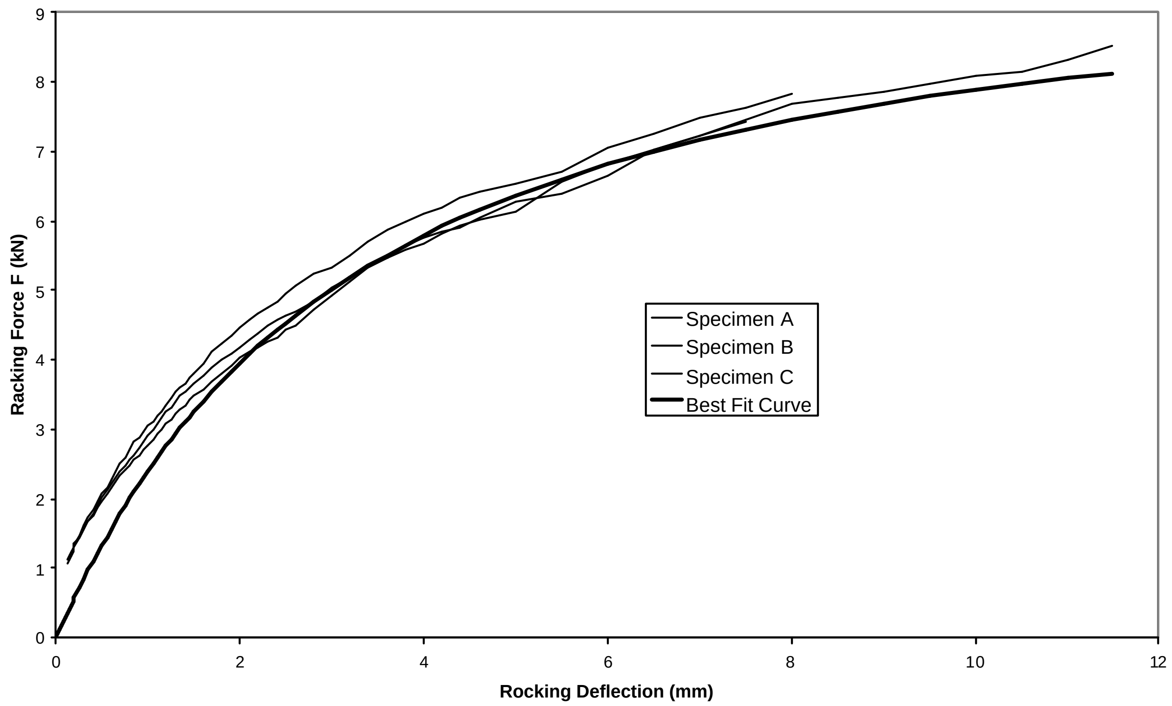


Figure B.3. Comparison of best fit curve and test data for rocking component of deflection for 1.2 m long wall – with end straps

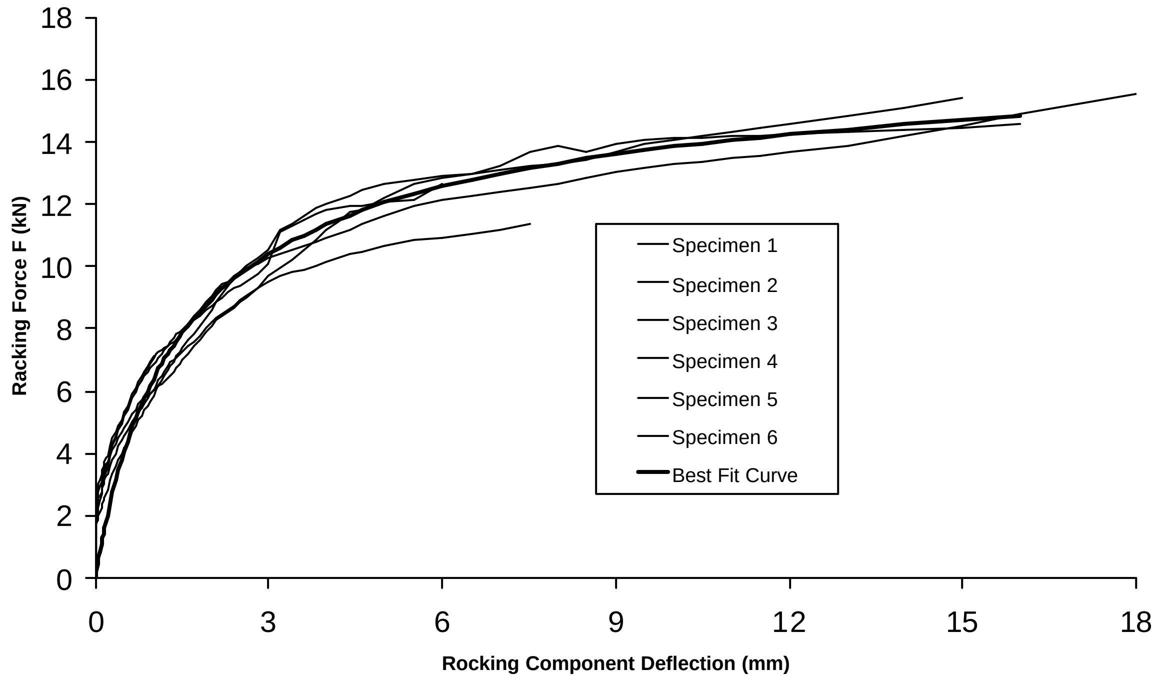


Figure B.4. Comparison of best fit curve and test data for rocking component of deflection for 2.4 m long wall – without end straps

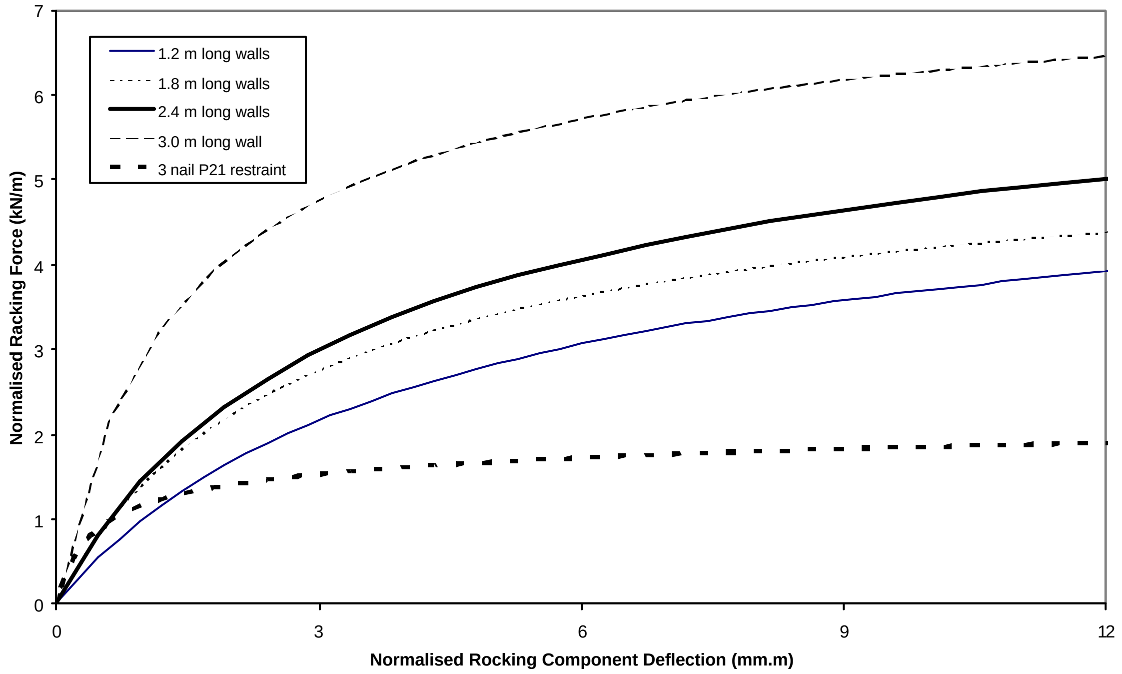


Figure B.5. Normalised rocking component response of P21 bracing walls

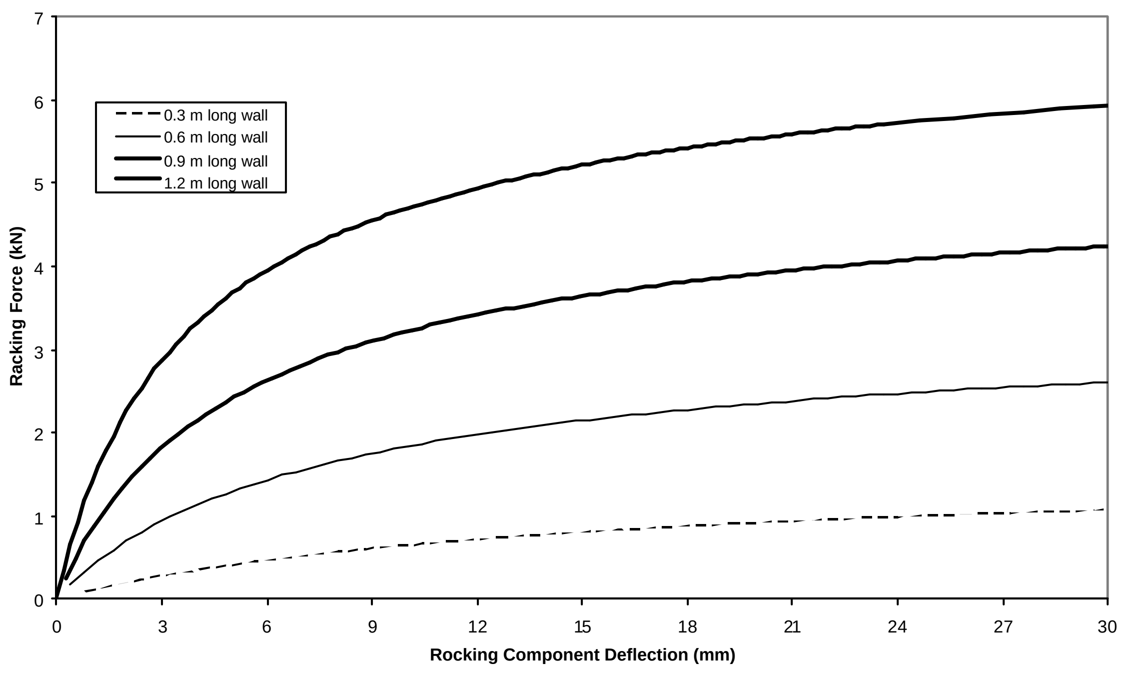


Figure B.6. Rocking component response of short P21 bracing walls

B.2 Measurement of Strength of P21 Uplift Restraint

The force versus deflection relationship of the P21 uplift restraint was simulated by the test rig shown in Figure B.7. The test regime imposed is illustrated in Figure B.8. The test was effectively four nails in shear in kiln-dried radiata pine timber. Five tests were performed for each of the following configurations.

1. 35 mm thick timber with 100 x 4 mm hand-driven nails
2. 35 mm thick timber with 90 x 3.15 mm Paslode JDN gun nails
3. 45 mm thick timber with 100 x 4 mm hand-driven nails
4. 45 mm thick timber with 90 x 3.15 mm Paslode JDN gun nails

Note that with 35 mm timber the nails had penetrated right through the timber, whereas with 45 mm thick timber the nails were almost fully embedded into the timber.

The backbone curve was extracted from each test – illustrated as a thick line in Figure B.8 and was averaged for each of the four test types listed above (example shown in Figure B.9). A best fit curve was fitted to the average curve for each test type (Figure B.10). The rocking component force versus deflection relationship for a P21 end restraint was then derived for a 1.2 m long wall using Eqns. B.1 and B.2 with “f” and “w” put equal to zero. This was then normalised as described above and plotted in Figure B.5. It is interesting to compare this plot with that based on P21 racking tests for a 1.2 m long wall. The racking force for a given rocking component deflection is approximately half that of the 1.2 m long wall (for deflections greater than 6 mm). This illustrates that approximately half the overturning force is resisted by the “f” and “w” mechanisms. At low deflections the prediction based on the P21 restraint alone appears to be too stiff. This is a little difficult to explain except that other sources of rocking deflection may have been influential at these low displacements in a P21 test – such as compression indentation of the stud into the bottom plate, bedding in of the bottom plate and P21 restraint.

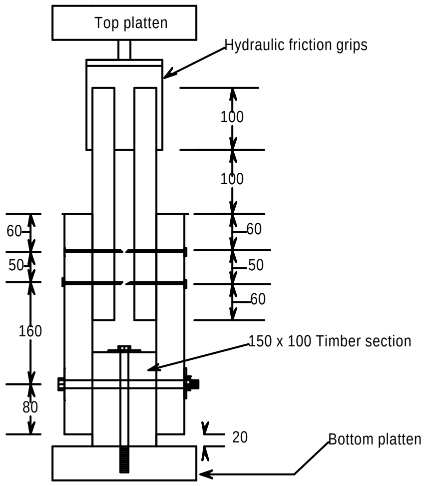


Figure B.7. Test setup for cycling loading of P21 end restraints

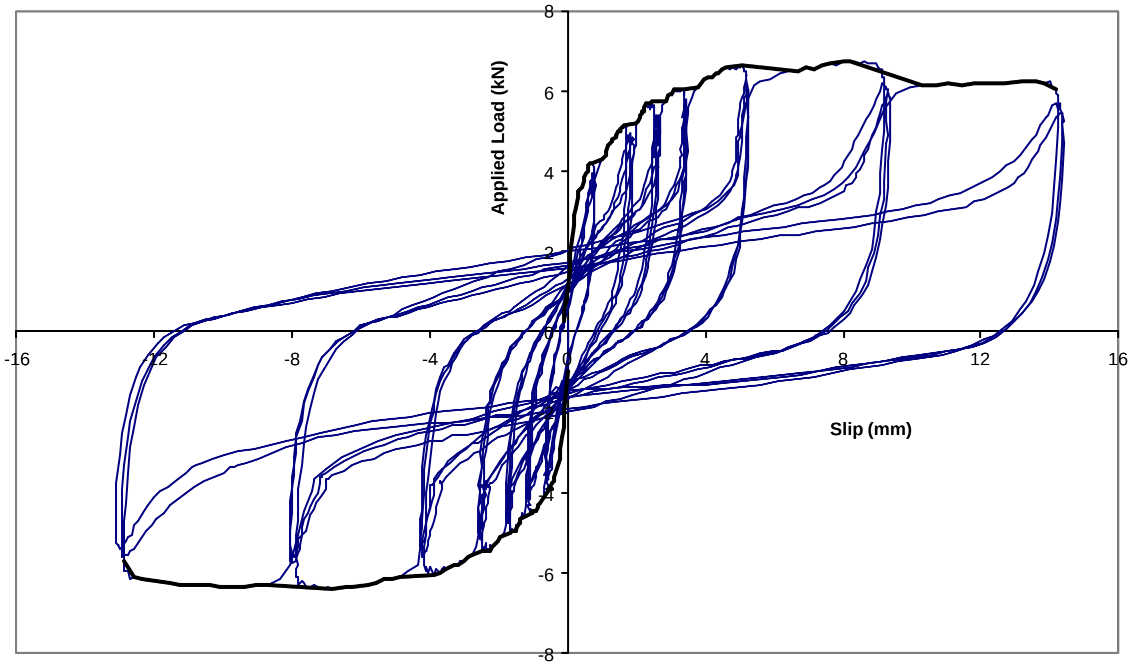


Figure B.8. P21 end restraint cyclic test

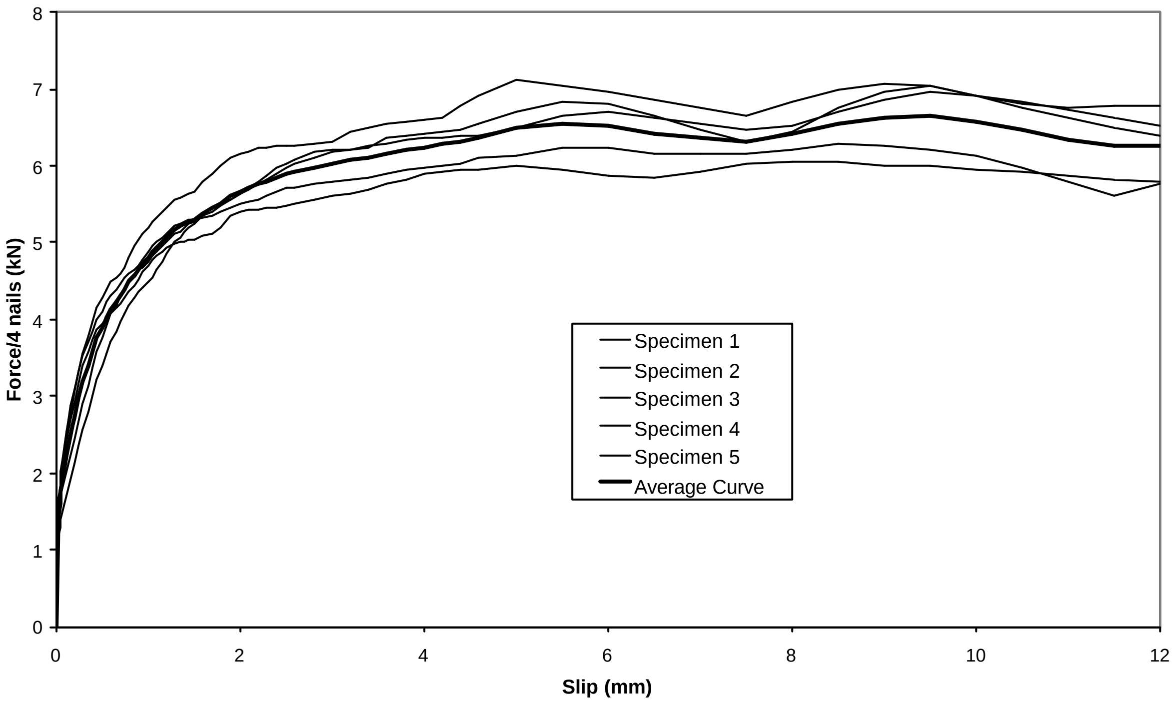


Figure B.9. Backbone curves for hand driven nails through 45 mm-thick timber

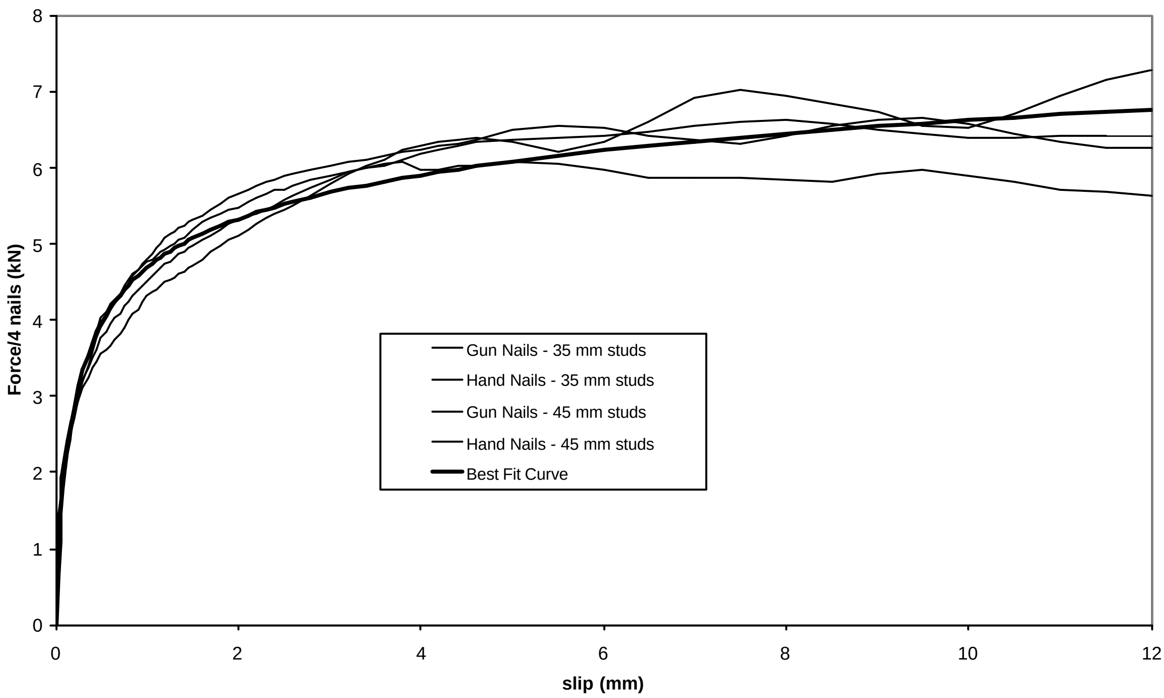


Figure B.10. Best fit curve to average curves for all tested conditions

Appendix C: Building Drawings and Fastenings on Test Walls

C.1 Introduction

Details of the plasterboard used on the walls is given in Appendix E.

All the drawings in this appendix are drawn to scale and dimensions not shown can be determined by ratio from the dimensions shown. Figure C.1 shows a plan view of the test house and Figure C.2 is a sectional drawing of both house and test rig. The next five drawings are details at locations marked on drawing C.2. Note, Detail 5 is shown at cross sections A-A and B-B as located on Figure C.1. Figure C.1 also shows the compass directions rotated to align with the main house axis.

The next two pages of drawings show the geometry of the four wall lines running in the E-W direction. This is the two internal views of the exterior walls and the two sides of wall lines A and B. (Wall lines are defined in Figure C.1)

Finally, elevations of all interior wall surfaces in the N-S direction are shown. These are defined as Wall1-Wall10, as depicted in Figure C.1, and show the locations of each and every fastener used on these walls and sheet vertical joints. This detail is necessary if computer models are develop to predict wall behaviour from fastener properties. Details of fastener types are given in Table C.1.

The walls and ceiling were lined with 9.5 mm plasterboard as shown in Table C.1. All plasterboard joints were fully plastered and reinforced with paper tape. No skirting was used at the base of the walls and no ceiling coves at the top of the walls. Timber ceiling battens of size 68 x 32 mm ran perpendicular to trusses at 400 mm centres. Ceilings were screwed to these battens using PLB gypsum plasterboard fixing details as detailed in the manufacturer's fixing and jointing instruction booklet. This was screwed at 200 mm centres along each batten. At exterior walls the ceilings were fixed to double top plates. At interior walls the ceilings were not directly fixed to the wall plates but rested on top of, and were plastered to, the wall lining at the edges.

Table C.1 Wall fasteners and lining type

Wall No.	Fastener Type	Fastener Pattern	Lining Type
1A	3	4	1
1B	2	2	2
1C	3	4	1
2A-East	2	2	2
2-West	1	1	3
3	1	1	3
4-East	1	1	3
4-West	3	3	1
5	3	4	1
6	3	3	1
7-West	3	4	1
7-East	1	1	3
8	1	1	3
9-West	1	1	3
9-East	3	4	1
10	3	4	1

C.2 Legend for Table C.1

Fastener Type

1 – 30 x 2.5 mm galvanised flat-head clout nails (no washers).
2 – 30 x 2.5 mm galvanised flat-head clout nails (with washers).
3 – 32 mm x 6g bugle head gypsum dry wall screw.

Fastener Patterns – edge distance to fasteners = 12 mm.

1 – Nails were placed at 300 mm centres around each sheet perimeter commencing at each corner with pairs of nails (60 mm apart) being placed at 600 mm centres to intermediate studs and at mid-length of each nog. (Note that the studs are generally at 600 mm centres and the nogs were at 800 centres).

2 – Nails with washers were placed at 150 mm centres on the perimeter of the bracing element commencing 50 mm from each corner in both the horizontal and vertical directions. Nails along sheet joints were at 300 mm centres and did not use washers. Intermediate studs were not nailed.

3 – Screws were placed at 150 mm centres on the perimeter of bracing element commencing at each corner. Screws along sheet joints were at 300 mm centres. Intermediate studs were not fastened.

4 – Screws were placed at 300 mm centres on the perimeter of each sheet commencing at each corner. Intermediate studs were not fastened.

Lining Type

1 – New PLB gypsum plasterboard. Screwed into place.
2 – New BRL bracing grade gypsum plasterboard. Nailed with washers.
3 – Existing PLB gypsum plasterboard (already nailed in place).

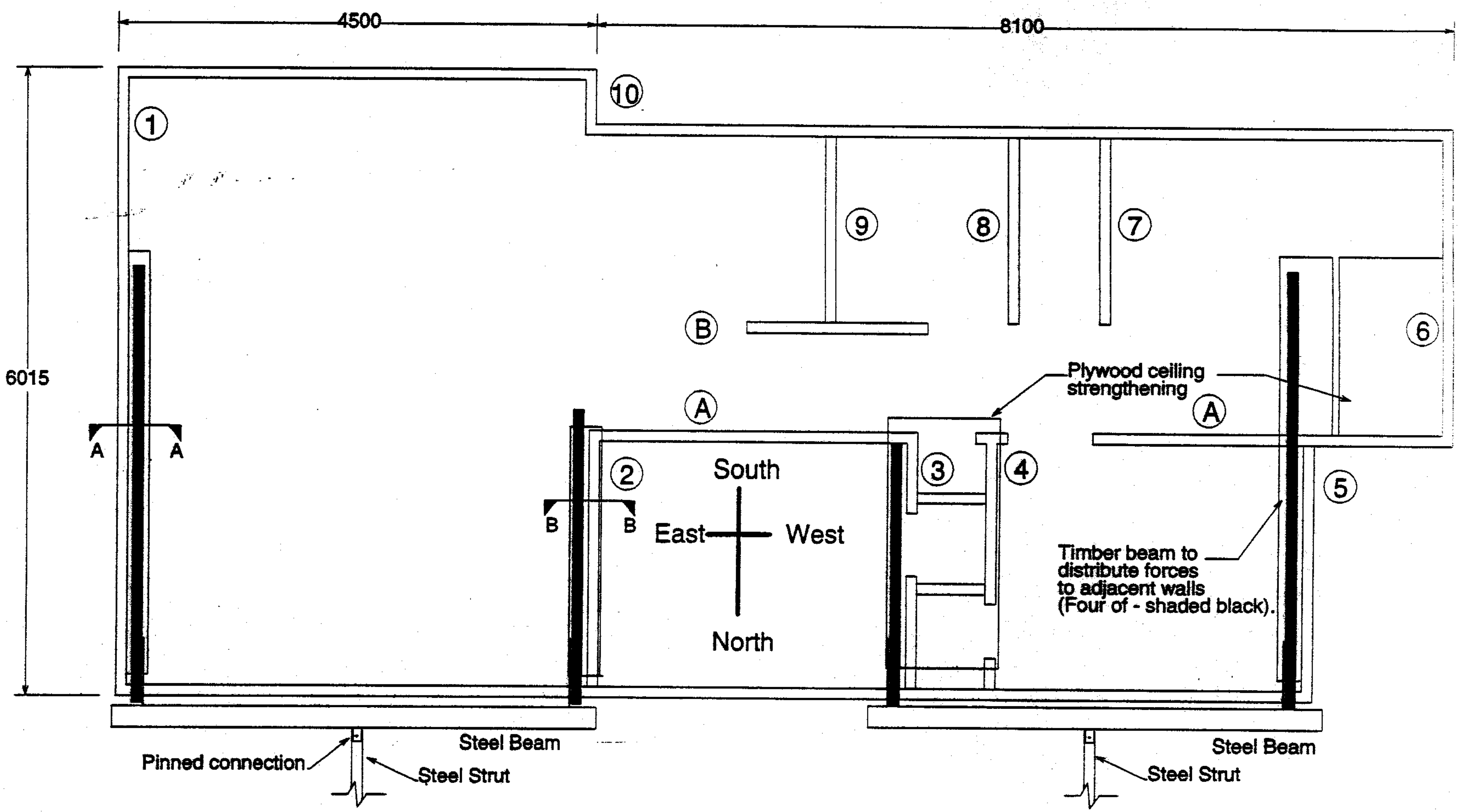


Figure C.1 Plan of Test House

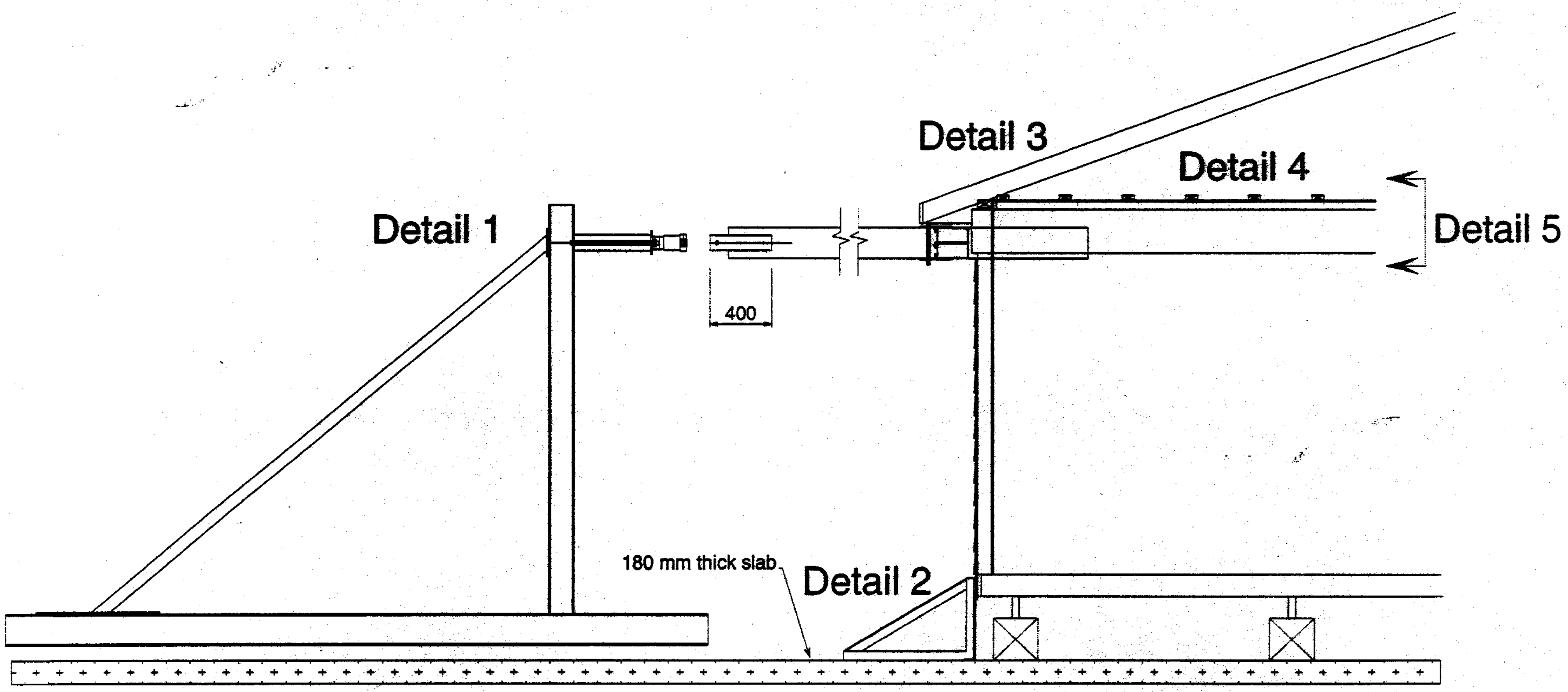
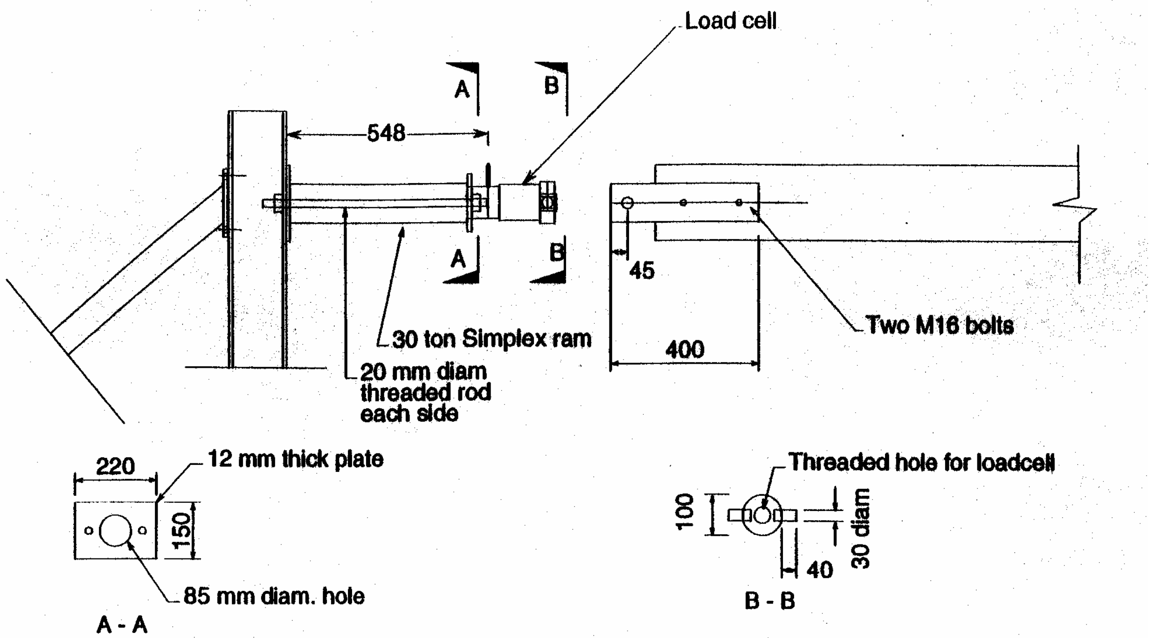
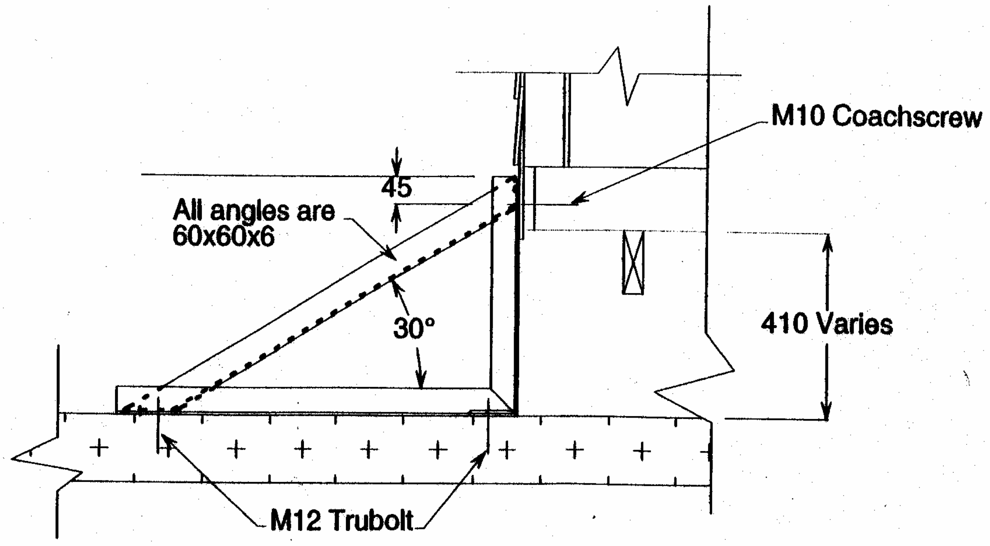


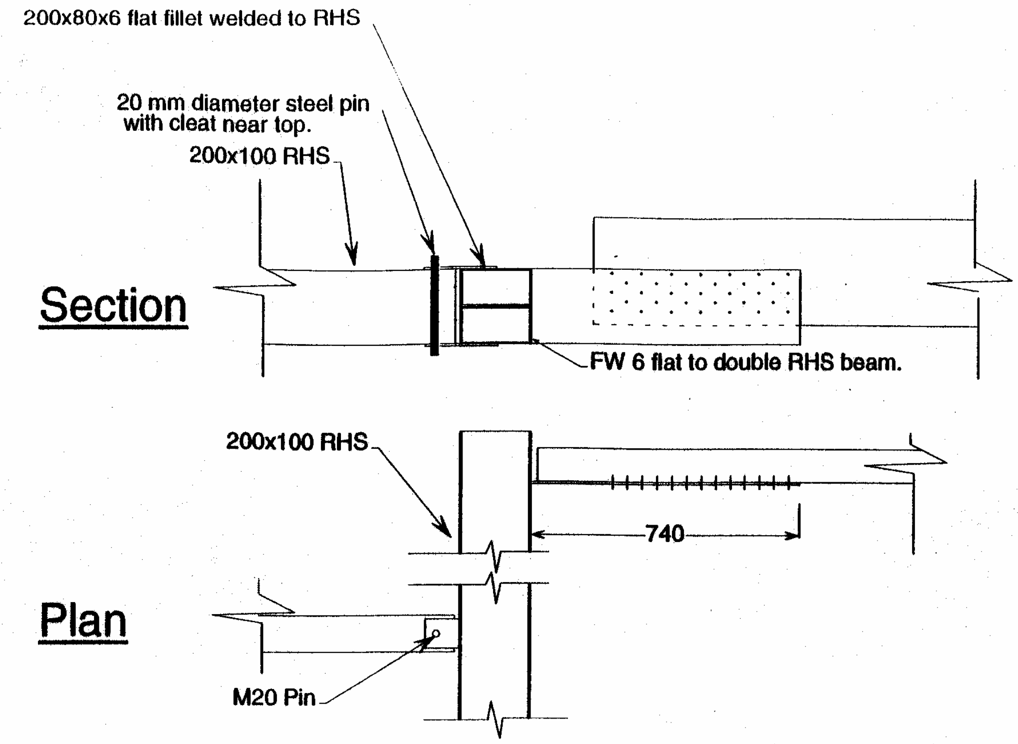
Figure C.2 Cross Section of Test House and Test Rig



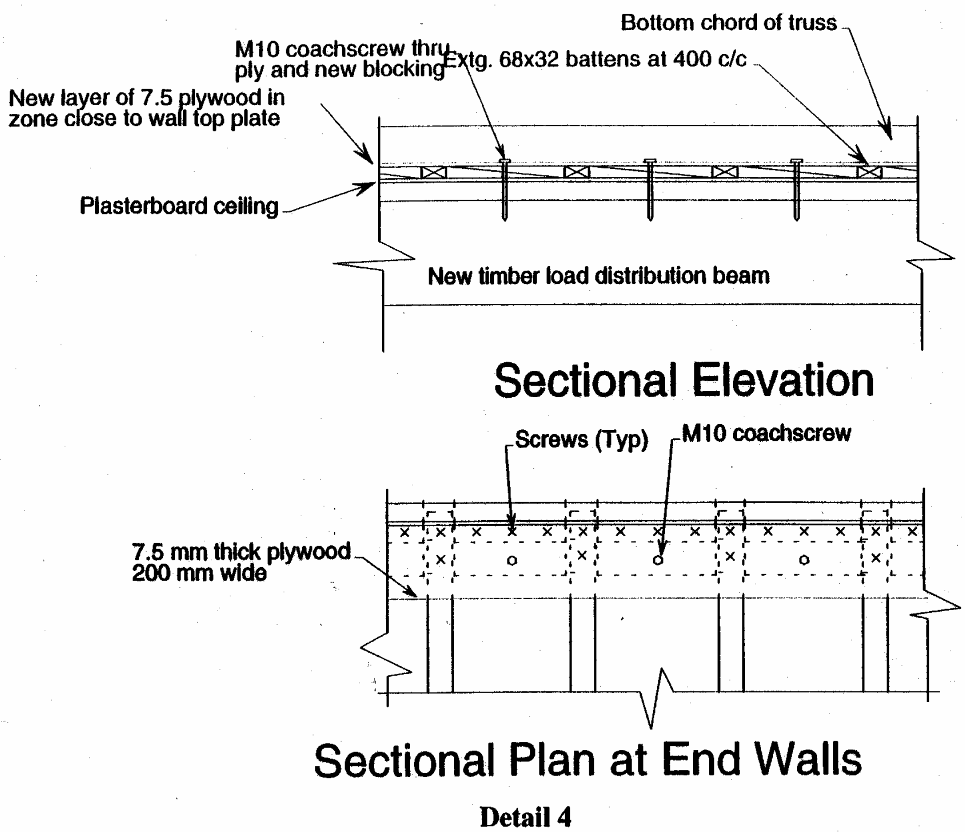
Detail 1



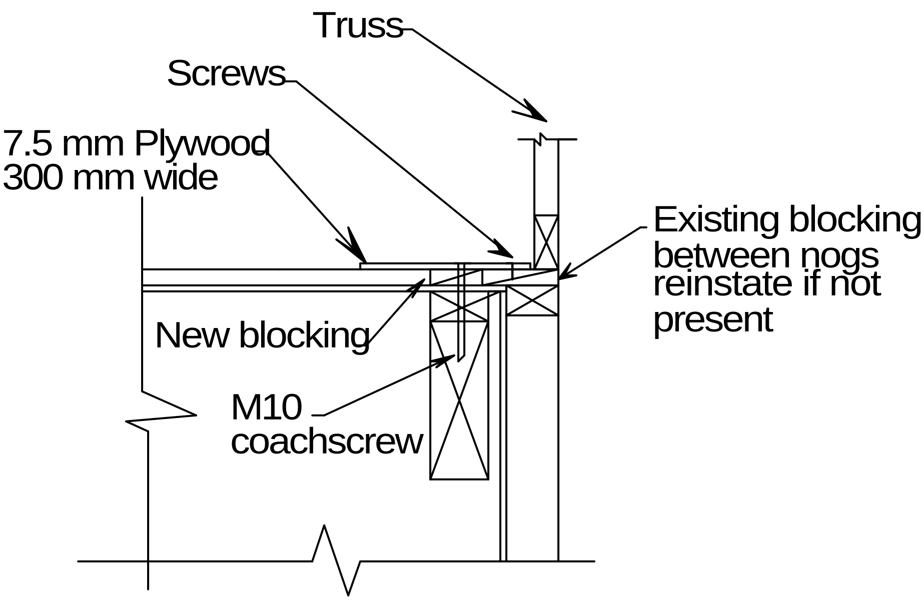
Detail 2



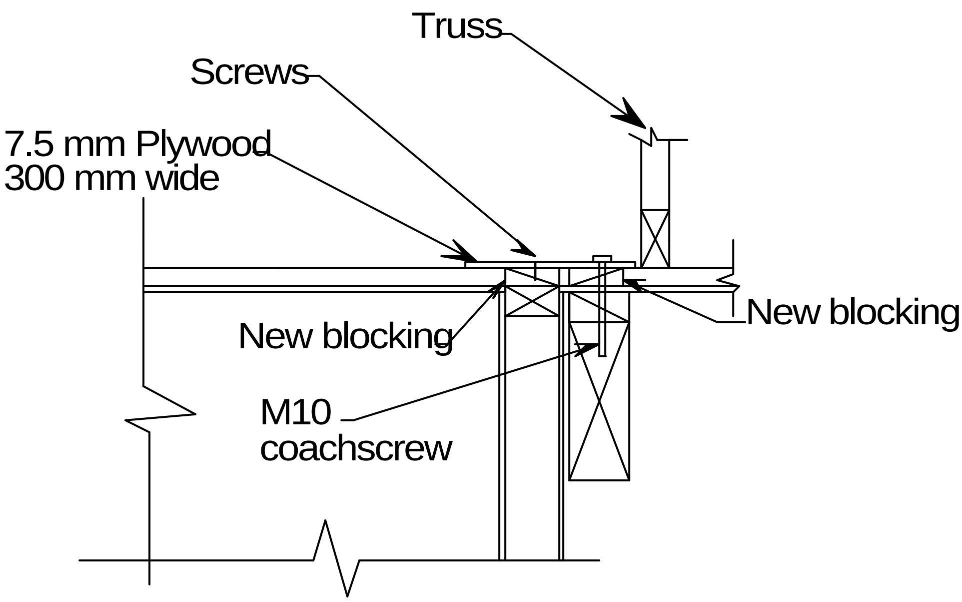
Detail 3



Detail 4

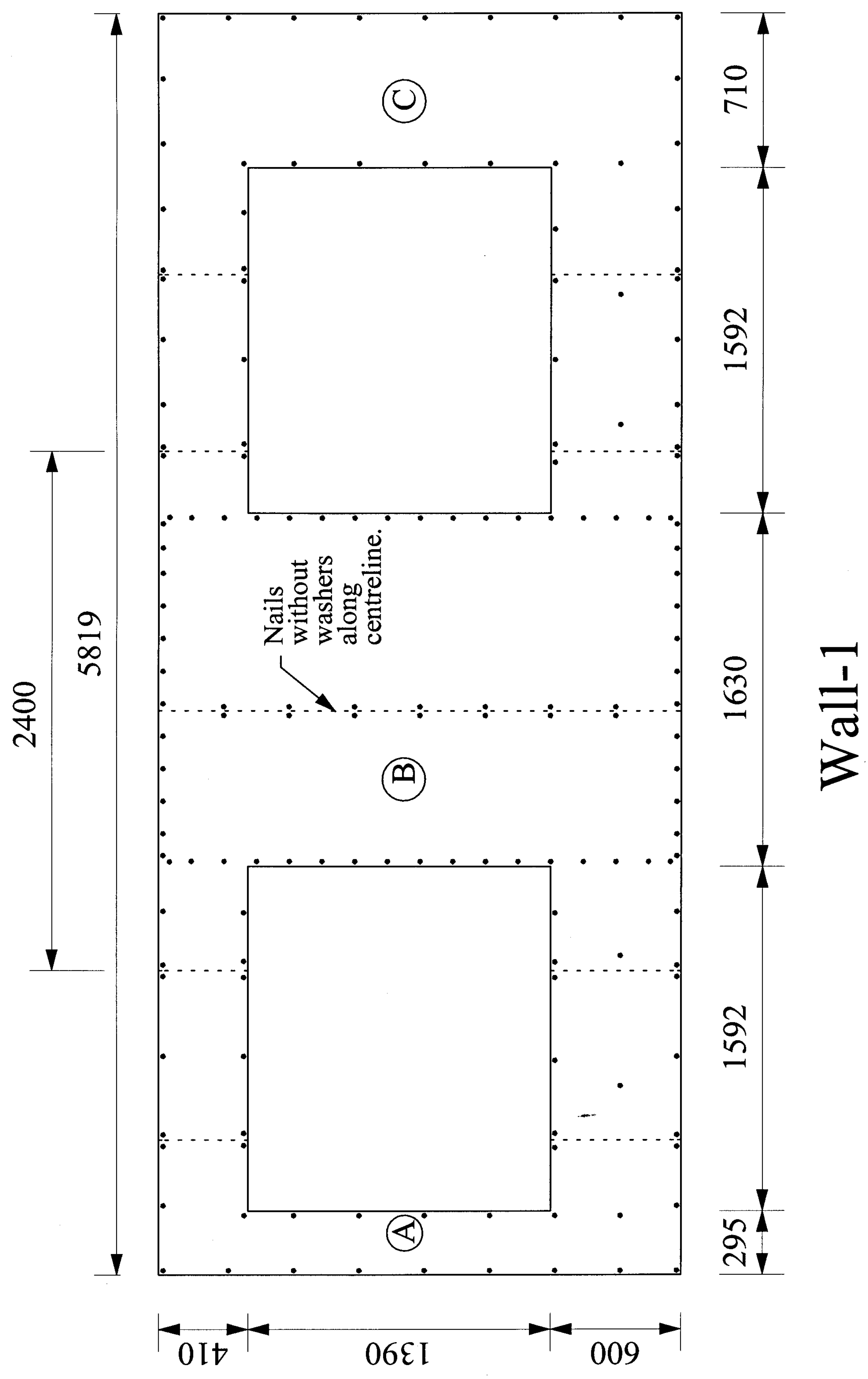


Section A-A

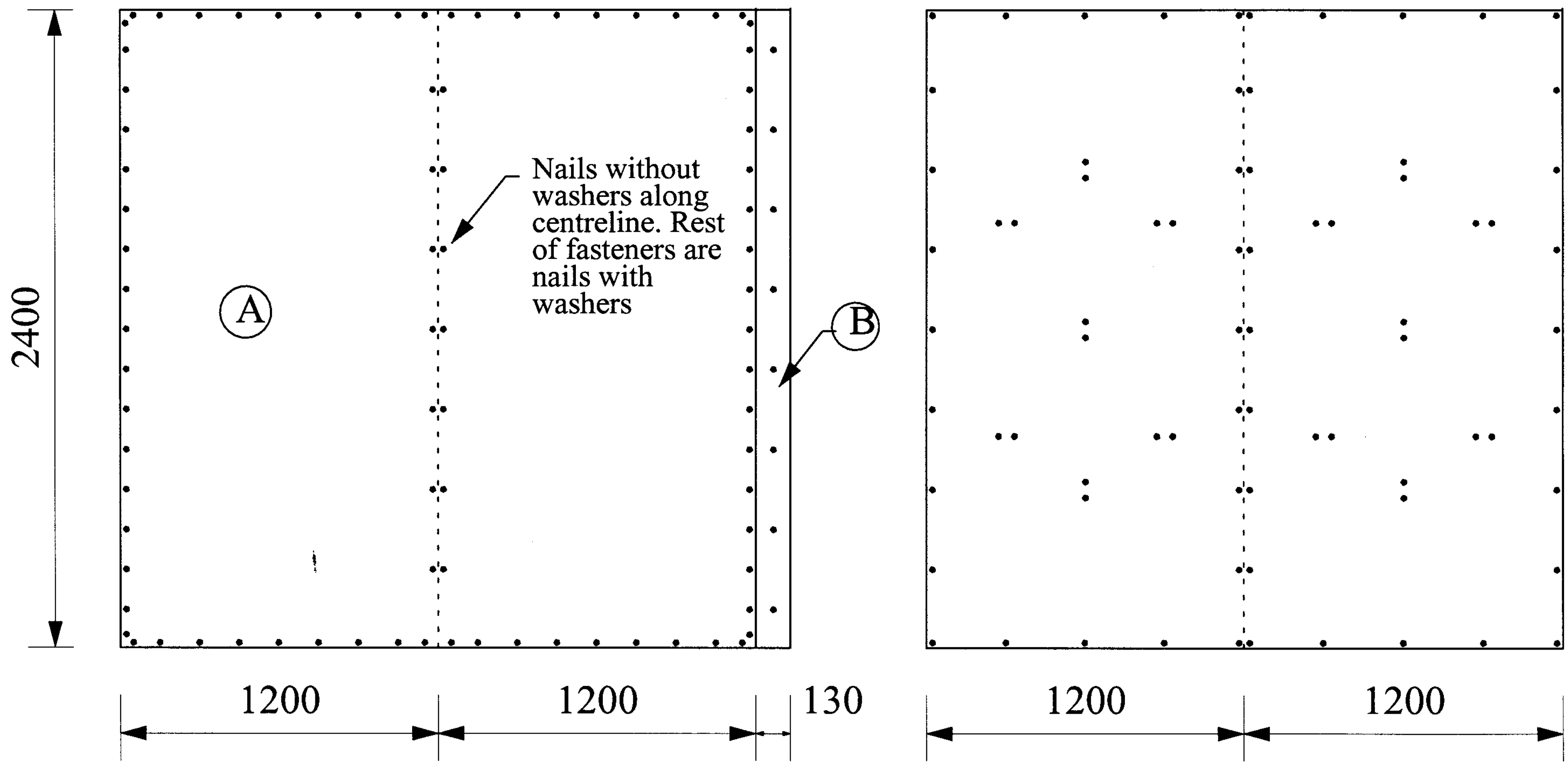


Section B-B

Detail 5
(See Figure C.1 for locations of sections)



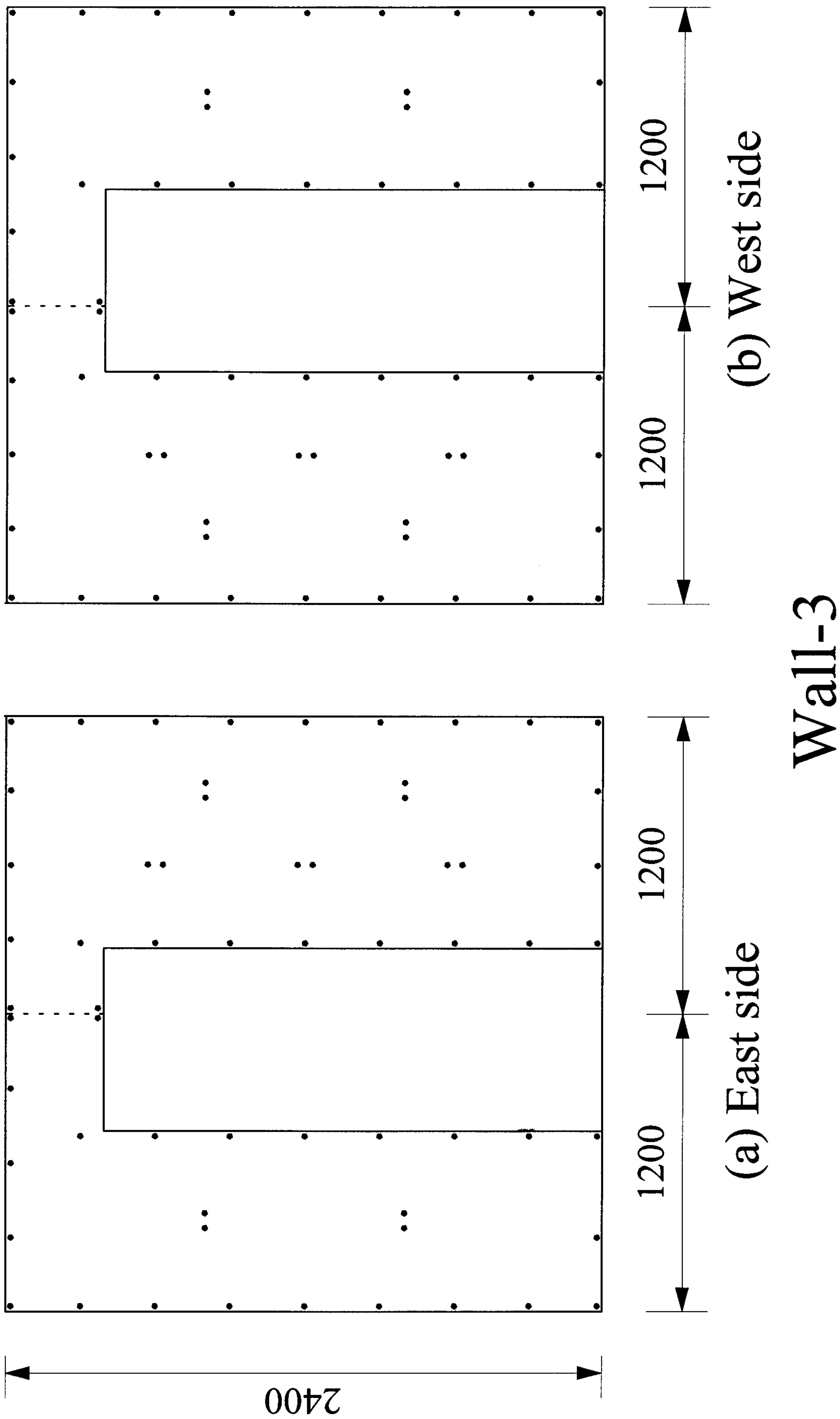
2400
5819
C
B
A
Nails without washers along centreline.
710
1592
1630
1592
295
410
1390
600
Wall-1



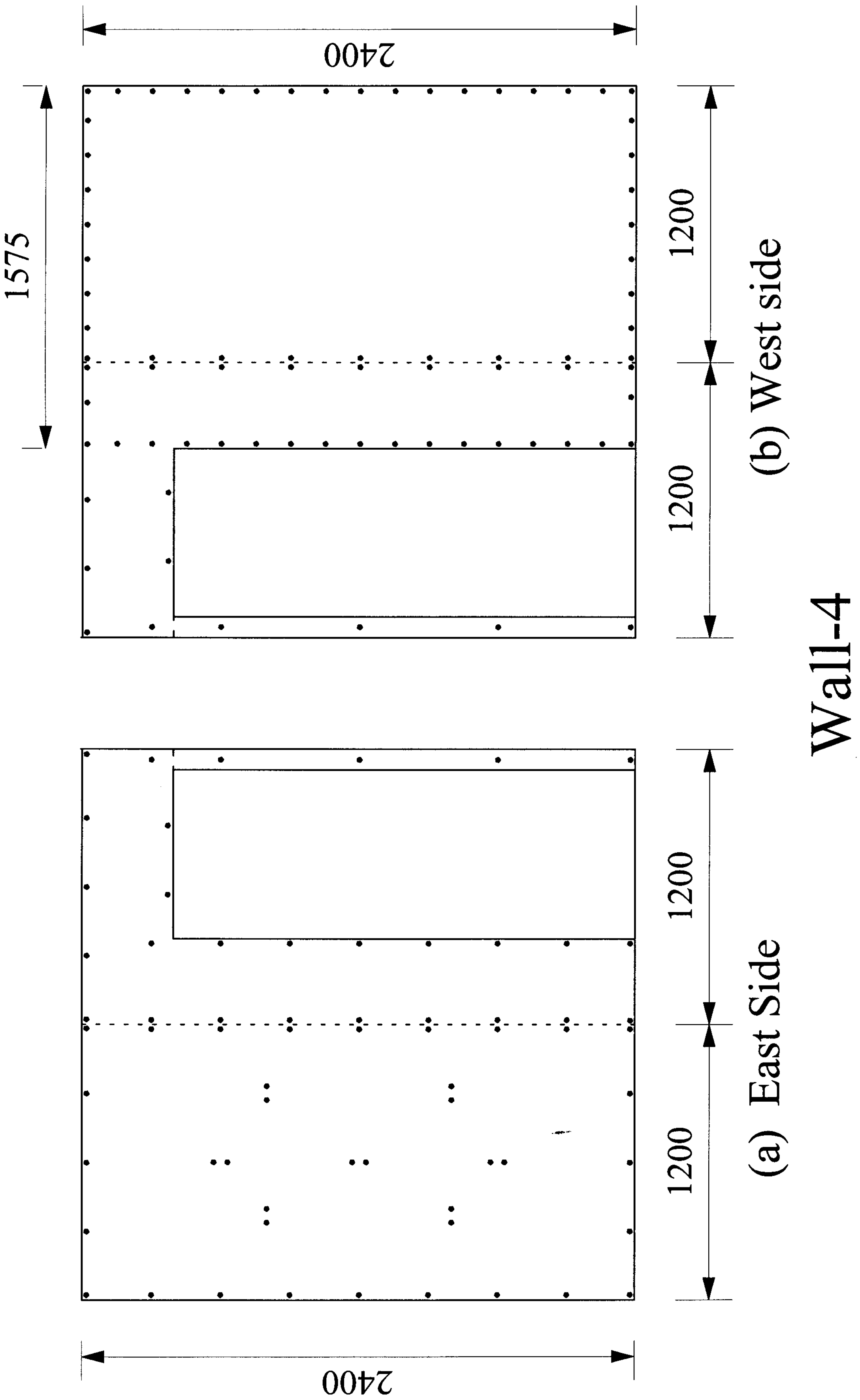
(a) East Side

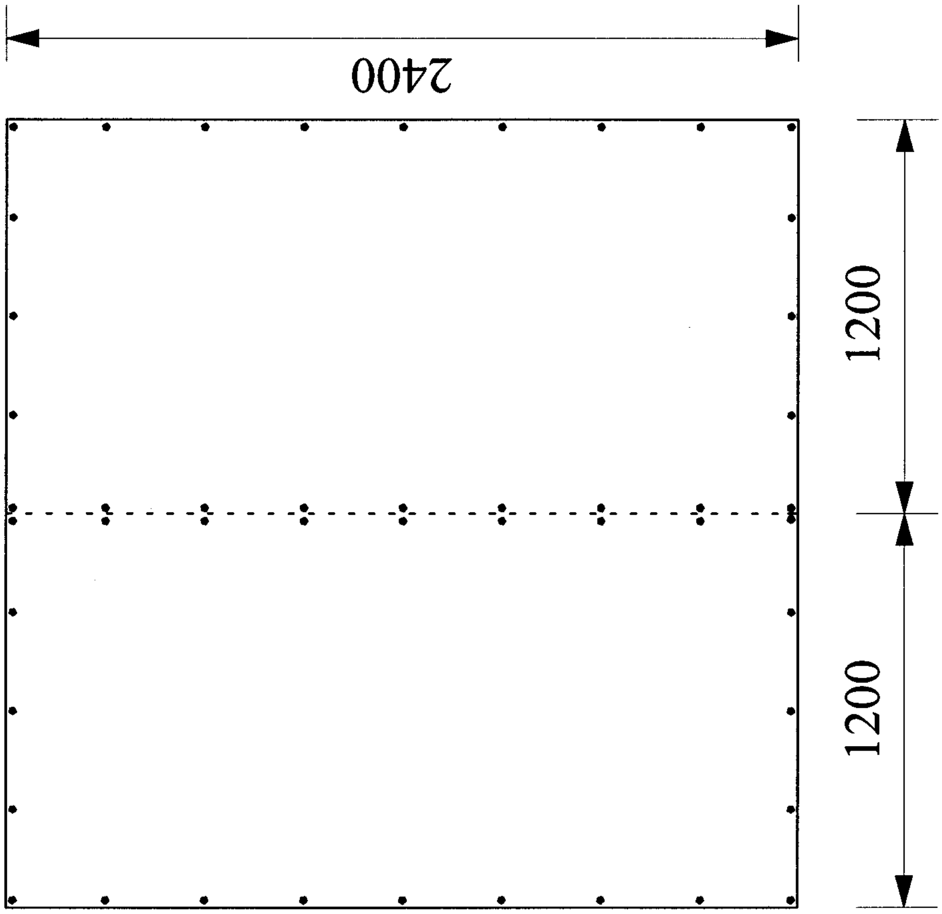
(b) West Side

Wall-2

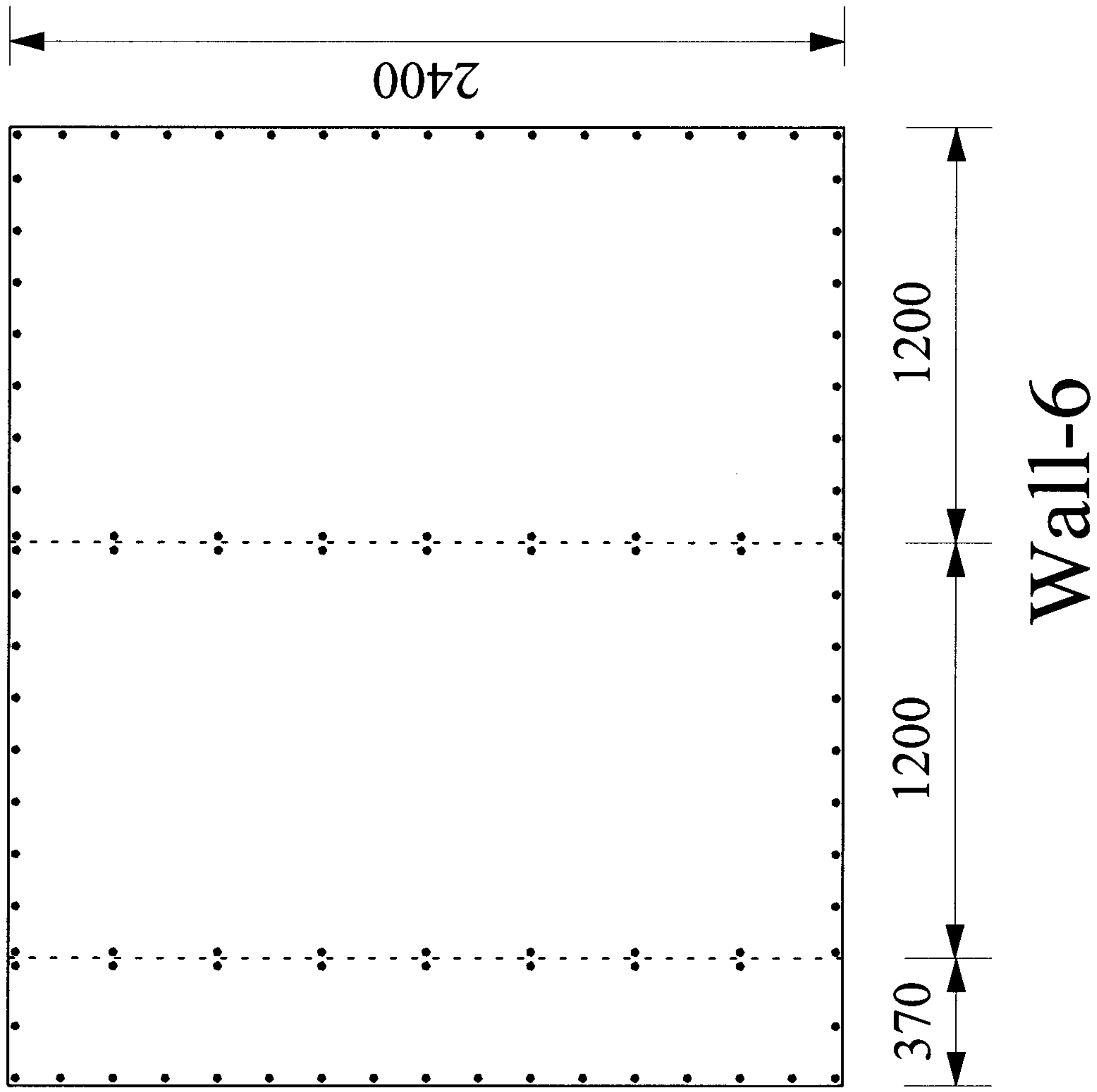


Wall-3
(a) East side
(b) West side
1200
1200
1200
1200
2400

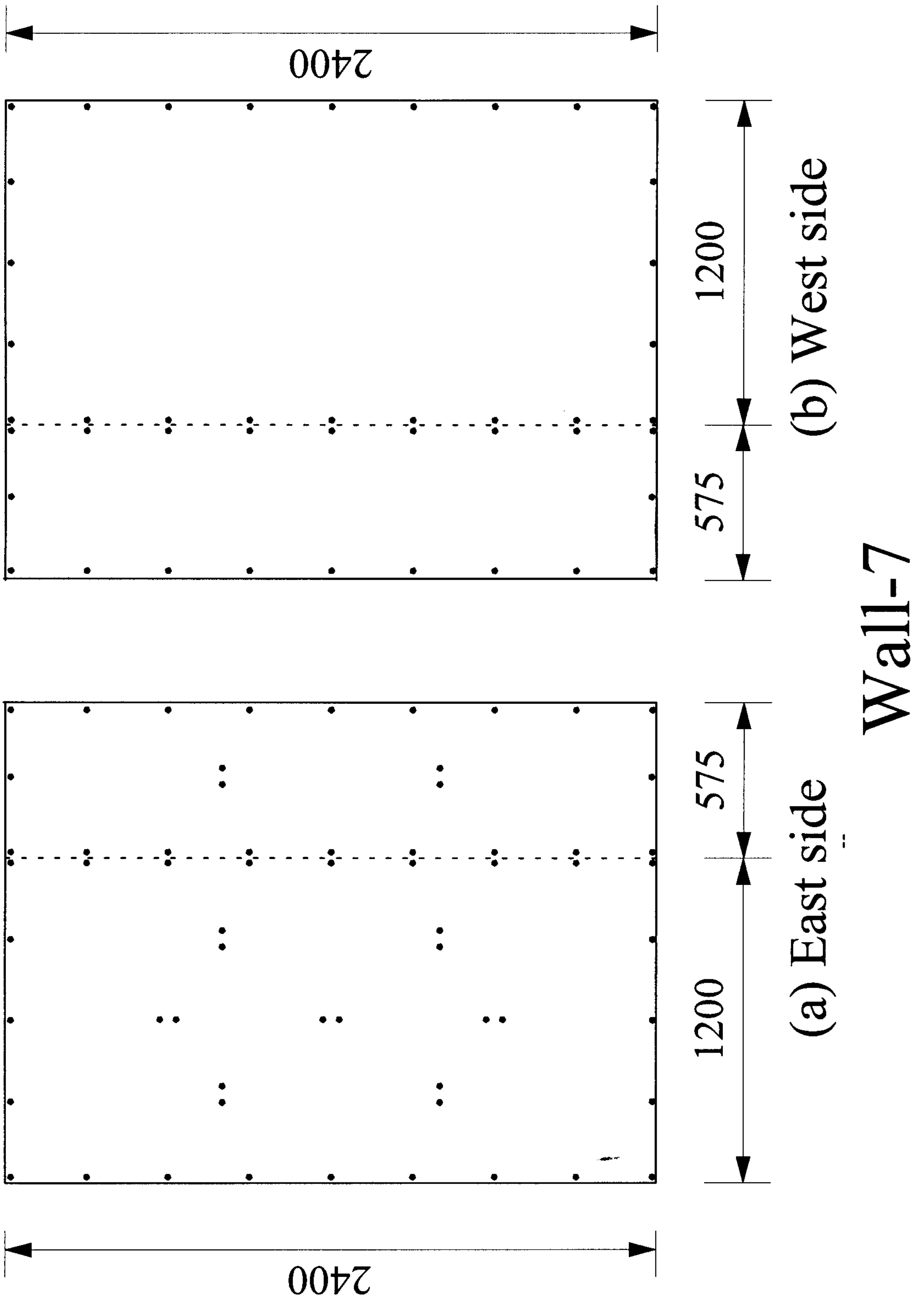

2400
1575
1200
1200
(b) West side
Wall-4
1200
1200
(a) East Side
2400

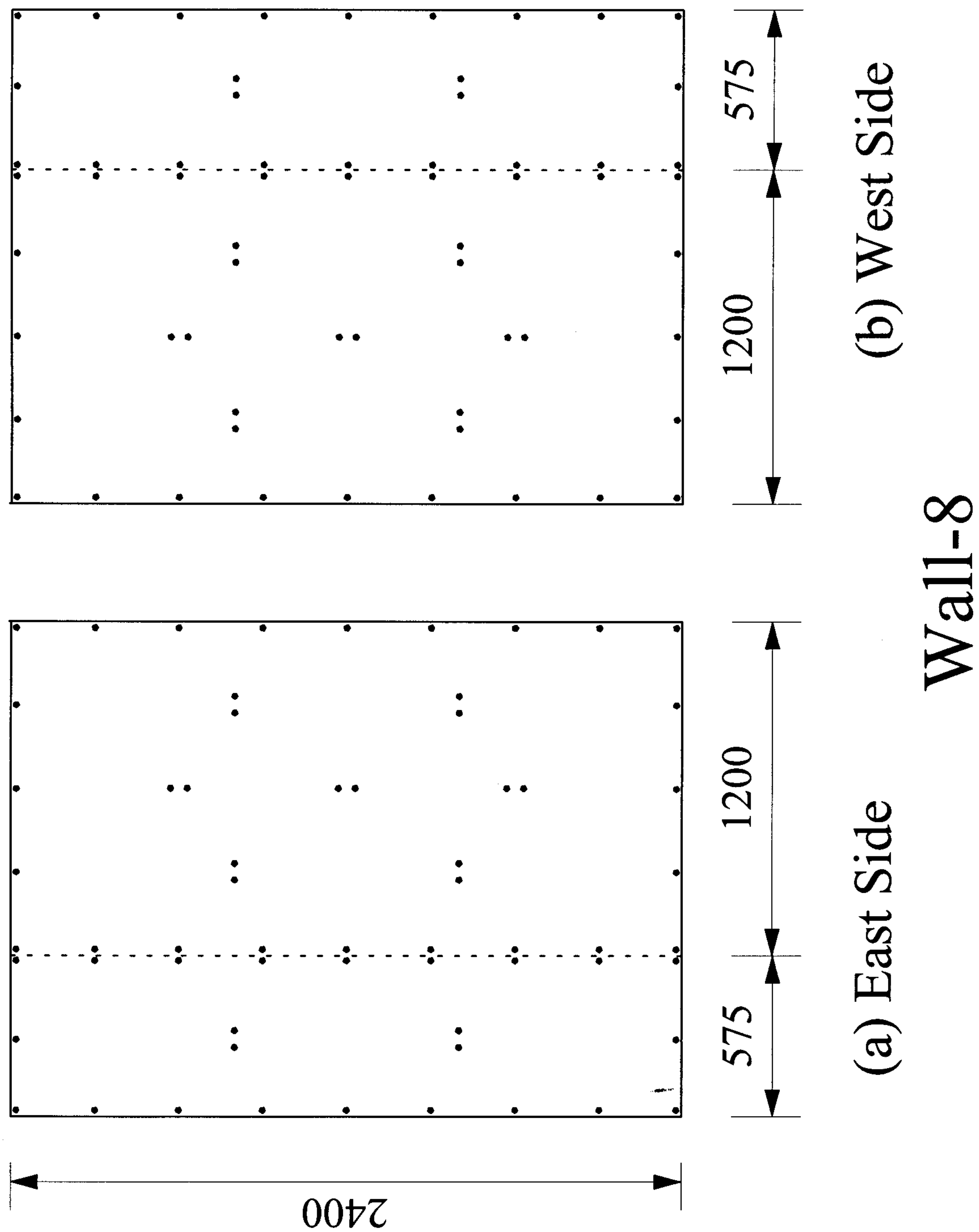


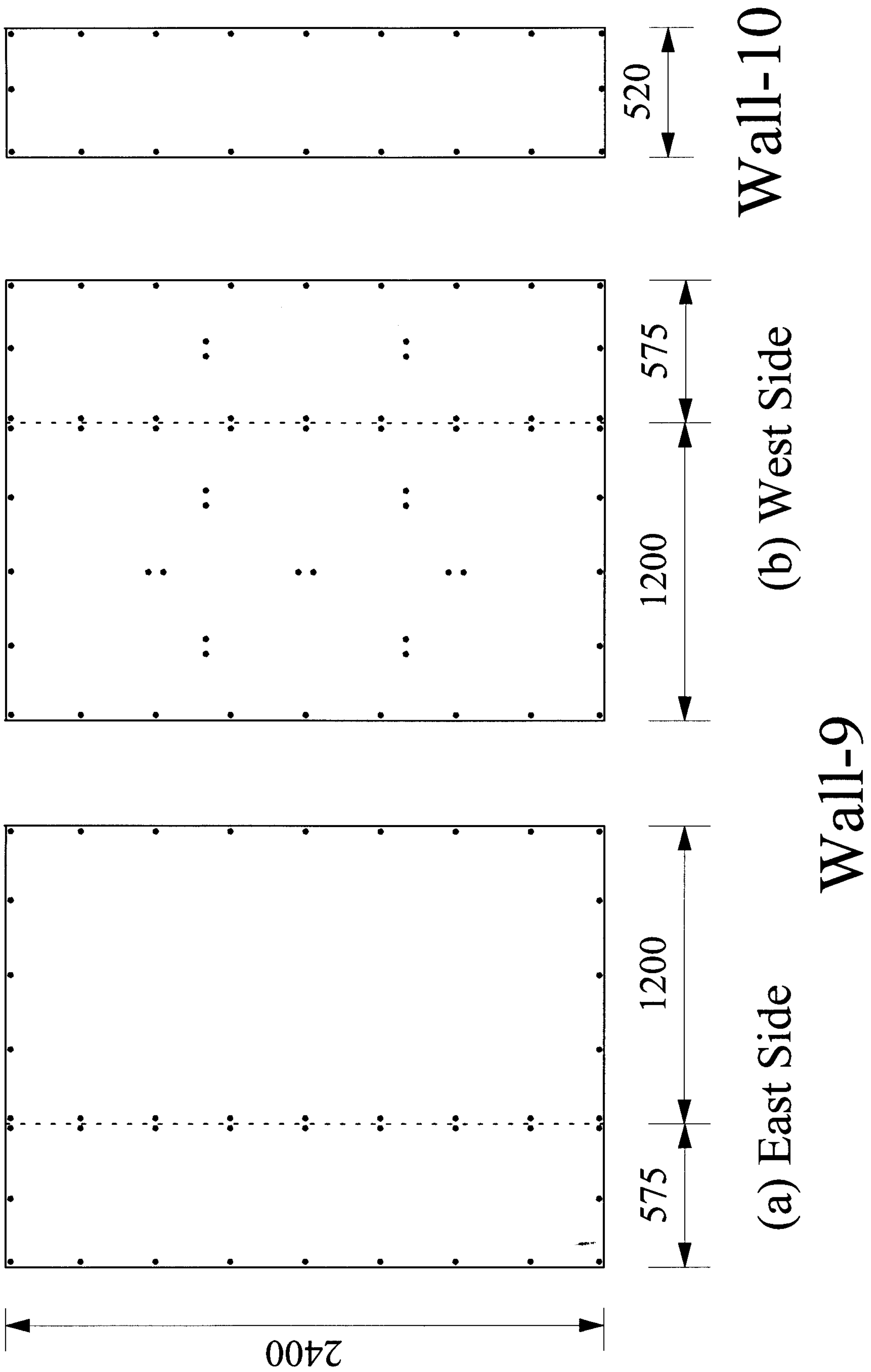
Wall-5



2400
1200
1200
370
Wall-6


2400
1200
575
(b) West side
575
1200
(a) East side
2400
Wall-7


Wall-8
(a) East Side
(b) West Side
575
1200
1200
575
2400


Wall-10
520
Wall-9
(b) West Side
575
1200
(a) East Side
1200
575
2400

Appendix D: Tentative Proposal to Change to NZS 3604 Bracing Distribution

D.1 Current requirements of NZS 3604

This appendix does not consider dragon ties as no changes are envisaged in this regard.

Section 5.5.6 of NZS 3604:1999 requires external walls to have a bracing rating of at least 10 BU/m length. Parallel external walls offset by not more that 2 m may be included together for this calculation.

Internal bracing line spacing must not exceed 6 m. (For low- density ceilings and construction without a double top plate, this is reduced to 5 m.) Section 5.5.5.4 of NZS 3604:1999 requires internal bracing lines to have a bracing rating of at least 70 BU. Parallel internal walls offset by not more that 2 m may be included in this calculation.

D.2 Inferences drawn from test results on the house tested herein

The test house was somewhat stronger than the sum of the component parts – even including the so-called non-structural walls. These non-structural walls are expected to have made a significant contribution to the house strength.

When the east end external wall (ie wall with windows) was loaded beyond its capacity, it had limited ability to transfer load to internal walls, even though the ceiling was fully plastered and exhibited little damage. It was concluded that the ceiling diaphragm was strong but soft. For this situation, where the diaphragm was acting like a cantilever, rather than a beam, load distribution may perhaps best be considered by tributary areas rather than by assuming a rigid diaphragm.

Despite special plywood ceiling diaphragm overlay, the end west wall only deflected about 5mm, which was short of its deflection at ultimate capacity, when the remainder of the house walls had reached near ultimate load. (See Figure 8 of the main text)

D.3 Suggested philosophy for house bracing design.

The author believes that houses designed to either the current NZS 3604:1999 or the draft EM3 will generally have sufficient total wall bracing capacity to resist design wind and earthquake loads. Problems may arise due to the distribution of the bracing walls - not so much due to torsion effects but more due to the end walls being inadequate in some instances. It should be noted that there is a differentiation made between the end walls for the particular direction being loaded and other external walls.

The author considers that a problem with the current NZS 3604:1999 bracing rating distributions is that the magnitudes stipulated as minimums are independent of the actual house bracing demand per metre width. As an extreme case, consider a pair of rooms built above another storey and basement with rooms being 5 m wide and 4 m deep (See Figure D.1 below). In a very high wind zone the bracing demand from Table 5.6 of NZS 3604:1999 is 340 BU/m width. Hence, the bracing demand on the storey is 340x(5+5) = 3400 BU. However, the bracing in the end walls need only be 10x4 = 40 BU. So the internal wall may effectively carry the entire bracing load. This is neither realistic nor good for torsional effects.

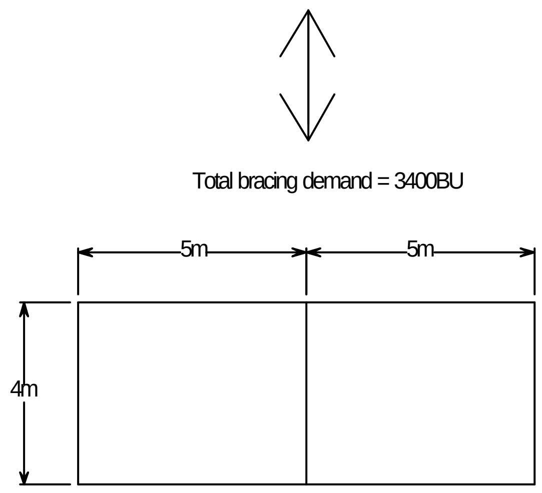


Figure D.1. Wall bracing demand of first-storey rooms

D.4 Proposed on house bracing design

For each floor level the designer should calculate total floor bracing demand for both earthquake and wind loading. The maximum of these values, F divided by building width, W, for the direction under consideration is determined, = F/W.

The proposal shown in Figure D.2 is for structures with a plastered ceiling or equivalent. For structures without such ceilings a tributary area approach is recommended.

D.4.1 End and exterior walls

The writer considers that the end walls should be able to carry at least 2 m width of the face load – ie a force of 2x F/W. (See Figure 21 of main text.). Other exterior walls are not so important and should be able to carry at least 1 m width of the face load. Hence, in the example given in Figure D.1, the end walls would need to each carry 340x2 = 680 BU. Parallel (internal and external) walls offset not more that 2 m may be included together for this calculation. (e.g. wardrobe internal walls could be used.)

D.4.2 Interior bracing lines

The only change recommended is that the bracing lines should be able to carry at least 2 m width of the face load – i.e., 1 m either side of bracing line. Parallel internal and external walls offset by not more that 2 m may be included for this calculation.

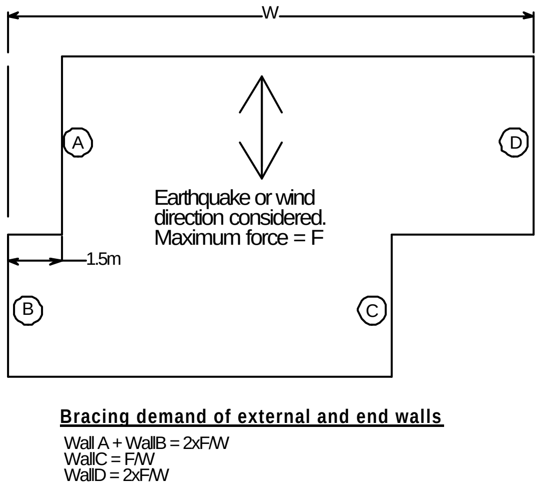


Figure D.2. External and internal wall bracing demand

D.4.3 Nominal walls

Where test data is not available, walls sheathed on only one side with sheet material of plasterboard with plastered joints, fibre-cement sheet or timber-based products at least 4 mm thick (this does not include horizontal plank construction) may be rated at 20 BU/m width provided fastenings are at not more than 300 mm centres. Where walls are sheathed on both sides they may be rated at 40 BU/m width. (These values are likely to change based on future tests on “nominal” walls – but at this stage are taken as approximately 1/3 and 2/3 respectively, typical published bracing ratings of house walls lined on one side only with plasterboard without any core reinforcing and which are nailed at 150 mm centres.

Appendix E: Proprietary Products Used

Three proprietary sheathings were used in the experimental programme described in this report and referred to as Type PLB or BRL. These products are defined below:

Type PLB was nominal 10 mm standard Gib® Gibraltar Board manufactured by Winstone Wallboards. This was an off-white paper-faced gypsum plaster based board.

Type BRL was nominal 10 mm Gib Braceline® Gibraltar Board with fibreglass reinforcement in the core manufactured by Winstone Wallboards. This was a blue paper faced gypsum plaster based board.

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 nor imply that it is better or worse than any other available product of its type. A Laboratory test may not be exactly representative of the performance of the item in general use.